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16. Abstract

A major national field and analytical study has been conducted into the effect of various design features on the performance of jointed concrete pavements. Extensive design, construction, traffic, and performance data were obtained from numerous experimental and other concrete pavement sections throughout the country. Field data collected and analyzed included distress, drainage, roughness, present serviceability rating (PSR), deflection, destructive testing (coring and boring), and weigh-in-motion (WIM) on selected sites. This information was compiled into a comprehensive microcomputer database. Projects were evaluated on an individual basis and then compared at a national level to identify performance trends. The performance data was used to evaluate and modify several concrete pavement design procedures and analysis models.

This volume provides a brief introduction to the data collection activities and presents a description and performance evaluation of the 95 pavement sections included in the study. Documentation is presented on the effects of the following design features on concrete pavement performance: slab thickness, base type, joint spacing, reinforcement, joint orientation, load transfer, dowel bar coatings, longitudinal joint design, joint sealant, tied shoulders, and subdrainage.

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

FHWA No.	Vol. No.	Short Title
FHWA-RD-89-137	II	Evaluation and Modification of Concrete Pavement Design and Analysis Models
FHWA-RD-89-138	III	Summary of Research Findings
FHWA-RD-89-139	IV	Appendix A Project Summary Reports and Summary Tables
FHWA-RD-89-140	V	Appendix B Data Collection and Analysis Procedures
FHWA-RD-89-141	VI	Appendix C Synthesis of Concrete Pavement Design Methods and Analysis Models Appendix D Summary of Analysis Data for the Evaluation of Predictive Models

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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

LENGTH

Symbol	When You Know	Multiply By	To Find	Symbol
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

Symbol	When You Know	Multiply By	To Find	Symbol
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

Symbol	When You Know	Multiply By	To Find	Symbol
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.

MASS

Symbol	When You Know	Multiply By	To Find	Symbol
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams	Mg

TEMPERATURE (exact)

Symbol	When You Know	Multiply By	To Find	Symbol
°F	Fahrenheit temperature	5(F-32)/9	Celcius temperature	°C

APPROXIMATE CONVERSIONS FROM SI UNITS

LENGTH

Symbol	When You Know	Multiply By	To Find	Symbol
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

Symbol	When You Know	Multiply By	To Find	Symbol
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

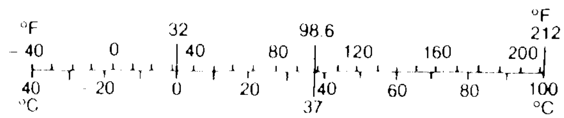
Symbol	When You Know	Multiply By	To Find	Symbol
mL	millilitres	0.034	fluid ounces	fl oz
L	litres	0.264	gallons	gal
m^3	metres cubed	35.315	cubic feet	ft^3
m^3	metres cubed	1.308	cubic yards	yd^3

MASS

Symbol	When You Know	Multiply By	To Find	Symbol
g	grams	0.035	ounces	oz
kg	kilograms	2.205	pounds	lb
Mg	megagrams	1.102	short tons (2000 lb)	T

TEMPERATURE (exact)

Symbol	When You Know	Multiply By	To Find	Symbol
°C	Celcius temperature	1.8C + 32	Fahrenheit temperature	°F



* SI is the symbol for the International System of Measurement

(Revised April 1989)

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CHAPTER 1 INTRODUCTION

A major national field and analytical study has been conducted into the effect of various design features on the performance of jointed concrete pavements, and also into selected structural overlay rehabilitation techniques. This first report from the study documents the field performance of 95 concrete pavement sections. In addition, observations are made regarding the effects of various design features on concrete pavement performance.

1. OBJECTIVES

The overall objectives for this research study are to:

- Evaluate the performance of different jointed concrete pavement design features on inplace pavement sections under similar environmental and traffic loading conditions. Relate the observed distress to the probable cause to allow valid analysis of the data.

- Determine the adequacy of available models and design procedures to predict the performance of inplace pavement sections. Estimate the expected performance periods of recently constructed projects incorporating improved design features that provide improved drainage and reduced deflections. Determine the cost-effectiveness of these features.

- Improve the analysis and design procedures and guidance for the design of jointed concrete pavements to reflect the effects of sealing, drainage, and deflection on pavement performance.

- Develop improved design and construction procedures for the following structural overlay techniques: (a) thin bonded portland cement concrete (PCC) overlays, (b) crack and seat and asphalt concrete (AC) overlay, and (c) sawing and sealing joints in AC overlays over existing PCC joints.

- Develop guidance on how to determine the most appropriate structural overlay technique(s) so the cost-effectiveness can be compared with other strategies (i.e., concrete pavement restoration, unbonded overlays, or reconstruct/recycle).

It is noted that the overall project objectives cover not only the evaluation of concrete pavement performance, but also specific structural overlay techniques for the rehabilitation of concrete pavements.

This report addresses a major portion of the first objective of the study. Numerous observations and preliminary conclusions regarding the effects of design features on concrete pavements are presented in this report. Volume II in this series of reports presents extensive analytical studies that have been conducted to further determine detailed causes of distress and to evaluate and improve predictive distress models.

2. BACKGROUND AND RESEARCH APPROACH

There have been numerous experimental concrete pavements constructed in the U.S. since the 1960's that are of sufficient age and level of traffic to provide some very significant insights as to their performance. Many of these pavements were constructed to evaluate the effect of one or several design features on performance. It was believed that there existed valuable inservice pavement information from these sections that could be used for improving concrete pavement design and construction procedures.

To fulfill the research objectives, the first aspect of the study involved the identification of candidate concrete sections. Once identified, those sections most likely to provide valuable insight into concrete pavement performance were selected. Sixteen experimental projects totaling 80 sections were ultimately selected, along with 15 other sections not part of any experiment for a total of 95 concrete pavement sections. These sections included both jointed reinforced and jointed plain designs and were the subject of comprehensive condition surveys in 1987. Fifty-five rehabilitated concrete sections were also selected and surveyed in 1988. The following types of information were collected for all pavement sections in the study:

- Visible distress (cracking, faulting, joint spalling, etc.).
- Drainage survey.
- Roughness (Mays Meter) and Present Serviceability Rating (PSR).
- Deflections (Falling Weight Deflectometer).
- Photographs.
- Cores of concrete slab and base and borings of subgrade.
- Laboratory tests of concrete strength and base and subgrade materials.
- Traffic (volumes, axle weights from weigh-in-motion and W-4 tables).
- Climatic information.
- Past maintenance and rehabilitation data.

This data was entered into a comprehensive microcomputer database that was developed exclusively for this study. The database was carefully reviewed, verified and cleaned of data errors. Information can be easily extracted from this database to analyze many aspects of concrete pavement performance.

After the field data collection activities, the performance data of each section and of each group of experimental sections was documented and carefully analyzed. Many interesting and significant results were obtained from the initial performance analysis presented in this report. These include the documentation of the effects of the following design variables on performance: slab thickness, base type, subdrainage, traffic loadings, climate, joint spacing, dowels and other mechanical devices, joint skewing and shoulders. The analytical evaluations presented in volume II has utilized these initial results and expanded upon them.

3. SEQUENCE OF REPORT

Chapter 2 provides a brief description of the pavement sections in the database and a general overview of the work effort performed on each section. Chapter 3 gives a summary evaluation of the performance of each of the major experimental projects and selected other single sections. An overall evaluation of performance results is given in chapter 4, followed by conclusions and recommendations in chapter 5. Chapter 6 provides a listing of various research reports that discuss, in much greater detail, the design, construction, and performance of the pavement sections included in this study.

Information provided in separate volumes are cited frequently throughout this report. Volume IV provides detailed summary reports documenting the design and performance of the experimental and selected single sections. Also included in this volume are a collection of summary tables providing a listing of all pavement sections with pertinent design and performance data in tabular format.

Volume V contains documentation of data collection and analysis procedures. Included in this appendix are discussions of field data collection procedures, weigh-in-motion (WIM) data collection activities, traffic loading (18-kip [80 kN] equivalent single-axle load [ESAL]) estimation procedures, the drainage evaluation, the backcalculation methodology, and a description of the database. An annotated bibliography of key references for jointed concrete pavements is also provided.

CHAPTER 2 DESCRIPTION OF PROJECTS IN STUDY

1. INTRODUCTION

In order to perform an evaluation of concrete pavement design features, every attempt was made to select experimental projects for inclusion in this study. These are projects, located all over the country, in which State DOT's have made a concerted effort to investigate the performance of design features not typical of their concrete pavement designs of the time. These experimental projects, constructed over the past 20 years, have allowed investigators to study the effect of a feature or a set of features on concrete pavement performance. This task of feature evaluation is made easier because at each project location these experimental projects have two key variables, traffic and environment, that may be treated as constants. Figure 1 shows the States and provinces that participated in the study.

While the overall emphasis was to include experimental concrete pavement sections in the study, there were many single pavement sections that were also included. Some of these sections were relatively new and were included because they incorporated a unique design feature, such as widened lanes or permeable base courses. Other single pavement sections were included because they were exhibiting either very good or very poor performance and it was felt that much could be learned from their different performance levels.

This chapter consists of two parts. Presented first are descriptions of the projects included in this study. The information presented is intended to provide a general overview of the different pavement sections which were evaluated. The sections are grouped by climatic zone, which is the overall level of differentiation among projects. The results of the evaluation of these sections is presented in chapter 3. A much more detailed presentation of the design and analysis of each individual project is found in volume IV.

The data included here for each project is background information on the experimental designs and some of the older sections. It includes the location and date of construction of the sections, the variables which were evaluated, and those notable design parameters which were held constant for all sections. The most recent two-way average daily traffic (ADT) and percent of heavy truck traffic are also given for all sections. Volume IV provides a complete listing of the design information for each project, as previously indicated.

Also of interest in this section are the design matrices or experimental layouts for each of the experimental projects. This method of presenting the data clearly shows which variables' effects can be isolated and directly studied and which variables are confounded by the effects of other changes. For any variable's effect to be directly studied, all other variables must be held constant while the variable is changed over its range. A simple example would be comparing the effect on performance of dowels by constructing a doweled and a nondoweled pavement section. However, this effect would be confounded if, for example, all doweled joints were perpendicular and all nondoweled joints were skewed.

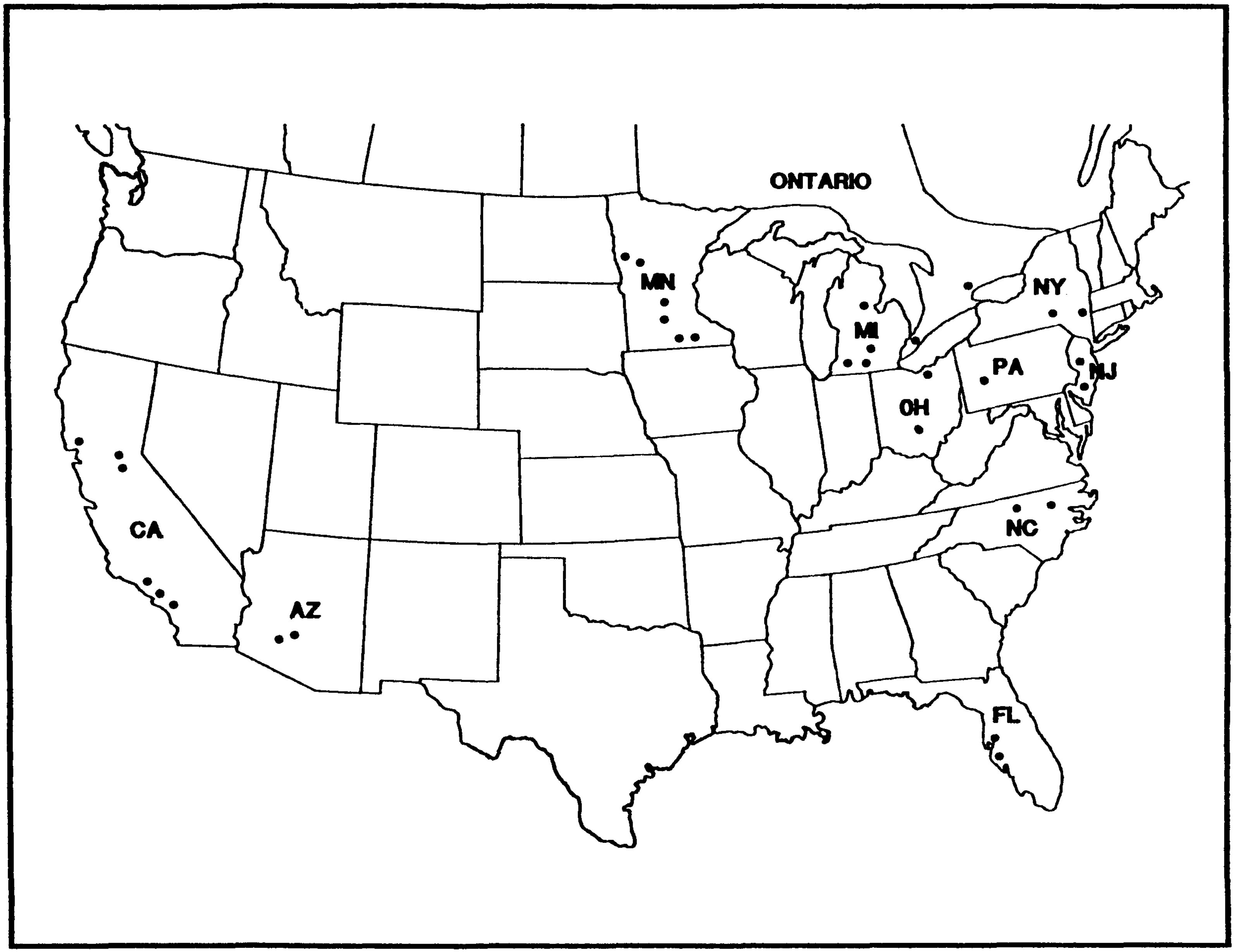

Figure 1. States and Provinces participating in study.

The second part of this chapter describes the general work effort performed on each section. The data collection was an enormous effort involving close cooperation between personnel from State agencies, other researchers, the FHWA, and investigators on this project. Volume V presents a complete and detailed description of the field data collection activities.

2. DESCRIPTION OF EXPERIMENTAL AND SINGLE PAVEMENT SECTIONS

A description of the pavement sections included in the study follows. As previously mentioned, this information is presented by environmental zone.

Dry-Freeze Environmental Region

Minnesota 1

The experimental project at I-94 near Rothsay, Minnesota was constructed in 1970 to evaluate the effect of base type, slab thickness and load transfer on concrete pavement performance. The variables included aggregate (AGG), asphalt-treated (ATB), and cement-treated (CTB) bases; 8- and 9-in (203 and 229 mm) slabs; and doweled and nondoweled joints. This is shown in the full factorial design matrix in figure 2. As part of this project, MN 5, a nearby pavement typical of Minnesota's design in the 1960's, was also included.

Common to all of the designs is a JRCP slab and an A-6 subgrade. Sections 1-1, 1-3, 1-9, and 1-11 had tied and doweled concrete shoulders added in 1984, In 1986, the ADT was approximately 5000 vehicles per day, with 21 percent trucks.

Minnesota 2

A second experimental project in Minnesota was located on I-90, near Albert Lea. These sections were constructed in 1977 in order to evaluate the effects of pavement type, tied concrete shoulders, and slab thickness on pavement performance. All sections were constructed on an aggregate base course and an AASHTO A-6 subgrade soil; the sections were all doweled except the inner lanes of 2-1 and 2-2. The 1986 ADT was approximately 3900, of which 20 percent were trucks. The setup of the experiment is shown figure 3.

Minnesota 3

This single pavement section, constructed in 1984, is located on I-90 in Austin. It was selected for inclusion in the study because it was constructed with widened outside lanes. The pavement slab is a 9-in (229 mm) JRCP with 27-ft (8.2 m) skewed joints that rests on an aggregate base and an AASHTO A-4 subgrade. The 1987 two-way ADT was approximately 10,600, including 15 percent trucks.

Minnesota 4

Minnesota 4 is a single pavement section located on Trunk Highway 15 near New Ulm that was built in 1986. This project was included in the study because it was constructed with widened outside lanes. The pavement is a 7.5-in (191 mm) JPCP constructed over an aggregate base course. The subgrade

		8 in JRCP		9 in JRCP	
		No Load Transfer	Load Transfer	No Load Transfer	Load Transfer
27 ft Joints	AGG	MN 1-3	MN 1-4	MN 1-1	MN 1-2
	ATB	MN 1-5	MN 1-6	MN 1-7	MN 1-8
	CTB	MN 1-11	MN 1-12	MN 1-9	MN 1-10
40 ft Joints	AGG				MN 5

Figure 2. Experimental design matrix for Minnesota 1.

	8 in Slab	9 in Slab	
	PCC Shoulder	AC Shoulder	PCC Shoulder
JPCP 13-16-14-19 ft Joints	MN 2-2		MN 2-1
JRCP 27 ft Joints		MN 2-3 MN 2-4	

Note: All sections constructed on an aggregate base. All sections are doweled except for the inner lanes of 2-1 and 2-2.

Figure 3. Experimental design matrix for Minnesota 2.

for the project is an AASHTO A-2-6 material. The 1987 two-way ADT was approximately 2900, which includes 13.5 percent trucks.

Minnesota 6

Minnesota 6 is a single pavement section located on Trunk Highway 15 near Truman. Built in 1983, this project was included in the study because it contains a permeable asphalt-treated base (PATB) course and it was constructed with widened outside lanes. The pavement is an 8-in (203 mm) JRCP constructed on an AASHTO A-2-4 subgrade. The 1987 two-way ADT was approximately 3900 vehicles per day, which includes 17 percent trucks.

Dry-Nonfreeze Environmental Region

Arizona 1

Experimental pavement sections were constructed over a number of years in the 1970's on State Route 360 in Phoenix, Arizona. These different sections incorporated a number of features and are summarized in figure 4.

All of the sections had nondoweled, random, and skewed transverse joints. The subgrade varied from an AASHTO A-4 to an A-6. However, these sections were not only constructed at different times, they are also located a considerable distance from each other. The ADT for these sections ranges from about 75,300 to 118,600 vehicles per day, with about 3 percent trucks.

Arizona 2

This single pavement section is located on I-10 near Phoenix. Built in 1983, it was included in the study because it represents one of the few JPCP sections in the dry climate that incorporates dowel bars in its design. The pavement consists of 10 in (254 mm) slabs over a lean concrete base (LCB) course; the subgrade is an AASHTO A-6 material. The 1987 ADT for this section is about 50,000 vehicles per day, of which 9 percent are trucks.

California 1

A set of experimental sections was constructed on I-5 near Tracy, California in 1971. Four different designs were constructed to study the effect of slab thickness, joint spacing, and base type on pavement performance. One section was constructed with high-strength concrete. All of the sections had JPCP slabs and nondoweled, skewed and nonsealed transverse joints. The subgrade soils ranged from an AASHTO A-1-a to an A-2-4. The ADT in 1987 was 12,000 vehicles per day, including 19 percent trucks. The experimental matrix for this project is shown in figure 5.

California 2

In 1980, California constructed two different designs on I-210 near Los Angeles. The only variable considered was base type and permeability; one section contained a permeable cement-treated base (PCTB), while the other section had a nonpermeable cement-treated base. However, for the permeable base section, it should be pointed out that a thin layer of asphalt concrete was placed between the slab and the base, thus altering its drainage characteristics.

		No Edge Drains		Edge Drains
		AC Shoulder	PCC Shoulder	PCC Shoulder
9 in JPCP	CTB	AZ 1-1		
	LCB		AZ 1-6	AZ 1-7
11 in JPCP	No Base		AZ 1-5	
13 in JPCP	No Base		AZ 1-2 AZ 1-4	

Note: All sections had skewed, nondoweled joints spaced at 13-15-17-15 ft intervals.

Figure 4. Experimental design matrix for Arizona 1.

		12-13-19-18 ft Joints		5-8-11-7 ft Jts
		8.4 in JPCP	11.4 in JPCP	8.4 in JPCP
Normal Strength Concrete	CTB	CA 1-3	CA 1-5	CA 1-1
	LCB	CA 1-7		
High Strength Concrete	LCB	CA 1-9		

Figure 5. Experimental design matrix for California 1.

	BASE TYPE	
	Dense AC/ PCTB	CTB
8.4 in JPCP	CA 2-2	CA 2-3

Note: All sections are nondoweled and have 12-13-19-18 ft joint spacing.

Figure 6. Experimental design matrix for California 2.

Common to both sections were an 8.4-in (213 mm) JPCP slab and random, skewed, and nondoweled transverse joints. The subgrade soil was an AASHTO A-4. The ADT in 1985 was approximately 42,000 vehicles per day, with 9 percent trucks. The design matrix for the project is shown in figure 6. Direct comparisons are possible for the two different base types.

California 6

Constructed in 1980, this single section is located on Route 14 in the greater Los Angeles area. This section was originally included because it was constructed with a permeable asphalt-treated base course. However, on-site coring and boring revealed that it actually had a lean concrete base. Nevertheless, this 9-in (229 mm) JPCP section was retained in the study. The 1987 two-way ADT for this section is 46,000 vehicles per day, including 9 percent trucks.

California 7

This single section is included because it is one of the earliest projects utilizing California's new drainage design. It was constructed in 1979 on I-5 near Sacramento. This design included a 10.2-in (259 mm) JPCP slab placed on a cement-treated base. The joints were randomly-spaced and skewed, and contain no dowels. The AASHTO A-7 subgrade was lime-stabilized to a depth of 5.4 in (137 mm). Traffic records for the section show a 1985 ADT of 20,000 vehicles per day, including 22 percent trucks.

California 8

This 10.2-in (259 mm) JPCP section was included in the study because it was constructed with widened outside lanes. Built in 1983 on U.S. 101 near Los Angeles, the slab rests on an asphalt-treated base course and an AASHTO A-7 subgrade material. The pavement has skewed, random joints and contains longitudinal edge drains. Traffic records for the section show a 1987 ADT of 135,000 vehicles per day, including 7 percent trucks.

Wet-Freeze Environmental Region

Michigan 1

An experimental project was constructed on U.S. 10 at Clare, MI in 1975. It includes the following variables: jointed plain and jointed reinforced concrete pavements 9 in (229 mm) thick; random joint spacing and long-jointed design; three different base types; skewed and nonskewed joints; doweled and nondoweled sections; and drained and nondrained sections. The drains were actually French drains retrofitted in 1981, but they still allow for a comparison of drained and nondrained sections. The subgrade for all sections was A-2-4. The average daily traffic on this section in 1987 was 5100 vehicles, with 8 percent trucks.

Twenty-five sections were constructed, representing 8 different designs. The selected sections are shown in the design matrix in figure 7. This design matrix represents an unbalanced experimental layout, in which only the effect of drainage is isolated. However, comparisons can be made between different design types.

		Drained		Nondrained	
		Skewed Joints	Nonskewed Joints	Skewed Joints	Nonskewed Joints
		No Load Transfer	Load Transfer	No Load Transfer	Load Transfer
12-13-17-16 ft Joints JPCP	AGG		MI 1-7a		MI 1-7b
12-13-19-18 ft Joints JPCP	PATB	MI 1-4a			
12-13-19-18 ft Joints JPCP	ATB	MI 1-10a		MI 1-10b MI 1-25	
71.2 ft Joints JRCP	AGG		MI 1-1a		MI 1-1b

Note: All sections have 9 in slabs, full-depth AC shoulders, and include a 10 in sand subbase.

Figure 7. Experimental design matrix for Michigan 1.

	PCC Shoulder	AC Shoulder
9 in JRCP 71 ft Joints	MI 4-1	MI 4-2

Figure 8. Experimental design matrix for Michigan 4.

Michigan 3

This single section, located on I-94 near Marshall, was constructed in 1986 using recycled aggregate. It was included in the study because of its permeable aggregate base course and its tied, reinforced concrete shoulders. The pavement is a 10-in (254 mm) JPCP with 41-ft (12.5 m) joints. The pavement was constructed on an AASHTO A-2-4 subgrade material. The 1986 two-way ADT was 31,300 vehicles per day, which includes 22 percent trucks.

Michigan 4

A project located on I-69 near Charlotte, Michigan was constructed in 1971 to study the effect of different asphalt and concrete shoulder designs. Common to both sections was the doweled 9-in (229 mm) JRCP slab with 71.2-ft (21.7 m) joint spacing. The base was an aggregate layer over a 10-in (254 mm) sand subbase and an AASHTO A-4 subgrade. The 1987 ADT and percent trucks were 13,700 and 11 percent, respectively. Direct comparisons can be made for the shoulder type, as shown in figure 8.

Michigan 5

This single section was constructed in 1984 using recycled aggregate. Located on I-94 near Paw Paw, this section was selected because of its permeable aggregate base course and tied, nonreinforced concrete shoulders. The JRCP slab is 10 in (254 mm) thick with 41 ft (12.5 m) joint spacing. The subgrade for the project was an AASHTO A-2-4 material. The 1986 two-way ADT was 19,300 vehicles per day, including 20 percent trucks.

New York 1

Route 23 between Catskill and Cairo, New York, is the site of an experimental project constructed in 1968. The variables included in this project are load transfer, skewed and nonskewed joints, base type, two joint spacings, and JPCP and JRCP. All slabs were 9 in (229 mm) thick and the load transfer devices were ACME two-part malleable iron load transfer devices. The subgrade varied from an A-1-a to an A-2-4. The 1987 ADT was 7250 vehicles and 10 percent trucks.

Eight different designs were constructed and, with replicates, there were a total of 30 sections. The sections selected for this project are shown in figure 9. As can be seen, this is not a complete factorial design of this experiment. However, several direct comparisons can be made, following along the filled boxes either vertically or horizontally.

New York 2

In 1975, an experimental project was constructed on I-88 near Otego. The design variables include joint spacing, shoulder type, and slab type. Also, portions of the longitudinal lane-shoulder joint were not sealed to determine the effect of joint sealing on pavement performance. All slabs are 9 in (229 mm) thick and have epoxy-coated I-beams for load transfer. Aggregate bases are common to all sections, as is the A-1-a subgrade. The 1987 ADT was 8,500 vehicles, with 10 percent trucks. The experimental layout of this project is shown in figure 10.

		Nonskewed Joints		Skewed Joints
		Load Transfer	No Load Transfer	No Load Transfer
20 ft Joints JPCP	AGG	NY 1-8		
	ATB	NY 1-1	NY 1-8a	NY 1-8b
61 ft Joints JRCP	AGG	NY 1-4		
	ATB	NY 1-3		

Note: All sections have 9 in slabs.

Figure 9. Experimental design matrix for New York 1.

	PCC Shoulder	AC Shoulder
20 ft Joints Sealed JPCP	NY 2-3	
20 ft Joints Nonsealed JPCP	NY 2-9	
26.7 ft Joints JPCP	NY 2-11	
63.5 ft Joints JRCP		NY 2-15

Note: All sections have 9 in slabs.

Figure 10. Experimental design matrix for New York 2.

Ohio 1

An experimental project was constructed on U.S. 23, near Chillicothe, Ohio, in 1973. Two base types, two joint spacings, and coated and noncoated dowel bars were evaluated in this project. All slabs were 9 in (229 mm) thick. The subgrade ranged from an A-4 to an A-6. The 1986 ADT was 12,300 vehicles, of which 12 percent were trucks.

The experimental design is shown in figure 11. Several effects are isolated in this design, including the effect of base type, joint spacing, and dowel coating.

Ohio 2

In 1974, an experimental project was constructed in northern Ohio, on State Route 2 near Vermilion, in order to study the factors that influence the development of "D" cracking. For this study, two sections were selected. They were both 15-in (381 mm) nondoweled JRCP slabs with 20-ft (6.1 m) joint spacing constructed on an A-4 subgrade. The 1984 average ADT at this site was 12,700 vehicles per day, with 16 percent trucks. The reduced design matrix for this design is shown in figure 12.

Ontario 1

An experimental project was constructed on Highway 3N near Windsor, Ontario, in 1982. Each lane of this two-lane highway was surveyed. This project includes variations in base type, slab thickness, shoulder type, and surface textures on pavement performance. Four sections containing these variations were selected for this project. Each of the sections had random, skewed joint spacing, subdrainage, and an AASHTO A-7-6 subgrade. The 1987 ADT was 5400 vehicles per day, including 13 percent trucks.

The experimental design matrix for this project is shown in figure 13. As can be seen, a comparison of design features is not readily made. However, design types may be compared.

Ontario 2

This single section, located on Highway 427 in Toronto, was constructed in 1971. Located in a heavily-trafficked part of the city, the highway has 11 total lanes with an ADT of approximately 228,400 vehicles per day, including 10 percent trucks. This older section is included as an example of a high volume pavement incorporating several different design features. The 9-in (229 mm) JPCP slab is constructed on a 6-in (152 mm) cement-treated base. The transverse joints are randomly-spaced, skewed, and contain dowels. Longitudinal edge drains were added in 1982.

Pennsylvania 1

An experimental project consisting of bases of varying permeabilities was constructed on Routes 66 and 422 near Kittanning, Pennsylvania. These sections, built in 1980, were placed in both directions of a divided roadway. A control section utilizing Pennsylvania's conventional aggregate base design, was constructed in four different locations.

		Aggregate Base	Bituminous Base
21 ft Joints JRCP	Standard Dowels	OH 1-10	OH 1-3
	Plastic-Coated Dowels	OH 1-6	
40 ft Joints JRCP	Standard Dowels	OH 1-1 OH 1-9	OH 1-4
	Plastic-Coated Dowels	OH 1-7	

Note: All sections contain 9 in slabs.

Figure 11. Experimental design matrix for Ohio 1.

	AC Shoulder	PCC Shoulder
20 ft Joints No Dowels 15 in JPCP	OH 2-33b	OH 2-33a

Figure 12. Experimental design matrix for Ohio 2.

	AC Shoulder		PCC Shoulder	
	8 in JPCP	12 in JPCP	7 in JPCP	8 in JPCP
PATB	ONT 1-2			
LCB			ONT 1-4	ONT 1-3
No Base		ONT 1-1		

Note: All sections have 12-13-19-18 ft skewed joints.

Figure 13. Experimental design matrix for Ontario 1.

Typical of all the sections were 10-in (254 mm) thick JRCP slabs with 46.5-ft (14.2 m) joint spacing and epoxy-coated dowels. The 1987 ADT was approximately 10,000 vehicles per day, including 4 percent trucks.

The experiment is represented by the matrix in figure 14. This experimental design allows for direct comparison among the different base types.

New Jersey 2

A part of Route 130 near Yardville, New Jersey is the oldest section included in this study. Once a major access route to New York City, it was constructed in 1951 and is typical of what is now New Jersey's standard concrete pavement design. The pavement is JRCP, 10 in (254 mm) thick, resting on an aggregate base and subbase material. The subgrade is an AASHTO A-4. The slabs are 78.5 ft (24 m) long and are constructed with expansion joints at that interval. Load transfer is provided by stainless steel clad dowel bars. The two-way ADT is approximately 11,900 vehicles per day, including 21 percent trucks.

New Jersey 3

The last experimental project in the wet-freeze environmental zone is located on I-676 near Camden, New Jersey. Built in 1979, the project is a drainage study, consisting of open-graded aggregate bases and bituminous-stabilized open-graded base layers. Two sections were placed with 9-in (229 mm) JRCP slabs, 78.5-ft (24 m) joint spacing, and 1.25-in (32 mm) diameter stainless-steel clad dowel bars. There is a filter fabric placed full-width beneath both of the open-graded layers. The 1986 ADT was approximately 77,000 vehicles per day, including 5 percent heavy trucks.

The simplified design matrix is presented in figure 15. It is observed that a comparison can be made between the performance of the two base types.

Wet-Nonfreeze Environmental Region

California 3

California constructed an experimental project on U.S. 101 near Geyserville, California in 1975 to study the effects of shoulder type on pavement performance. Seven different sections were constructed, including tied and nontied PCC shoulders, and various asphalt concrete shoulders. Some of these sections had sealed transverse joints, contrary to the usual practice of not sealing joints in California. Common to these pavements is a 9-in (229 mm) JPCP slab over a 5.4-in (137 mm) cement-treated base and a 6-in (152 mm) aggregate subbase. The subgrade soil classification varied from an AASHTO A-4 to an AASHTO A-6. The 1985 ADT was 15,000 vehicles per day, with 14 percent trucks.

Variables in the study are shown in the matrix in figure 16. The effects of sealed and nonsealed transverse joints and tied and nontied concrete shoulders can be directly studied.

BASE TYPE	46.5 ft Joints 10 in JRCP
CTB	PA 1-1
PATB	PA 1-2
Uniform-Graded AGG	PA 1-3
Well-Graded AGG	PA 1-4
Dense-Graded AGG	PA 1-5

Figure 14. Experimental design matrix for Pennsylvania 1.

	Permeable Aggregate	Permeable ATB
78.5 ft Expansion Joints 9 in JRCP	NJ 3-1	NJ 3-2

Figure 15. Experimental design matrix for New Jersey 3.

	9 in Nondoweled JPCP 12-13-19-18 ft Joints	
	Tied PCC Shoulder	Nontied PCC Shoulder
Sealed Transverse Joints	CA 3-1	
Nonsealed Transverse Joints	CA 3-2	CA 3-5

Figure 16. Experimental design matrix for California 3.

North Carolina 1

Experimental pavement sections were constructed on I-95 near Rocky Mount, North Carolina, in 1967. Design variables in the project include base type, jointed reinforced and jointed plain concrete pavements, joint spacing, slab thickness, skewed and nonskewed joints, and doweled and nondoweled joints. The subgrade soil for these sections varied from an AASHTO A-2-4 to an AASHTO A-4. Traffic data from 1987 showed an ADT of 19,100 vehicles per day, including 9 percent trucks.

This design matrix is depicted in figure 17. As the design matrix shows, only eight total sections were constructed, which results in the confounding of several of the design variables.

North Carolina 2

This single section is located on I-85 near Greensboro, North Carolina. Constructed in 1982, this pavement section was included in the study because of its doweled, JPCP design and its tied concrete shoulders. The slabs are 11 in (279 mm) thick, the base course is lean concrete, and the subgrade is an AASHTO A-4 material. Traffic data from 1987 showed a two-way ADT of 26,000 vehicles per day, including 17 percent trucks.

Florida 2

Constructed in 1986, this single section is a six-lane Interstate located on I-75 in Hillsborough County near Tampa. This section was selected for inclusion in the study due to its thick slab design (13 in [330 mm]). The pavement is a JPCP that rests on a sand base course. The subgrade was classified as an AASHTO A-3 material. The 1987 two-way ADT was approximately 28,700 vehicles per day, including 20 percent trucks.

Florida 3

This single section, constructed in 1982, is a six-lane Interstate located on I-75 south of Tampa in Manatee County. The section has demonstrated poor performance since its construction and it was included in the study in an effort to learn why the pavement performed so poorly.

The section is a 9-in (229 mm) JPCP with random, skewed joint spacing. The transverse joints are doweled and sealed with a silicone sealant. The base consists of 6 in (152 mm) of lean concrete and the subgrade is an AASHTO A-3 material. This section has tied, lean concrete shoulders. The 1987 ADT was approximately 32,700 vehicles per day, including 20 percent trucks.

3. GENERAL DATA COLLECTION EFFORTS

The data collection activities represented a lengthy and multi-faceted work effort. Although every attempt was made to collect information compatible with the Strategic Highway Research Program (SHRP) Long-Term Pavement Performance (LTPP) database, some data elements could not be obtained or determined. Nonetheless, as the following description will show, an exhaustive effort was made to create as complete a data record for each pavement section in this project as was possible.

		JPCP 30 ft Joints			JRCP 60 ft Joints
		Skewed Joints	Perpendicular Joints		Perpendicular Joints
		No Load Transfer	Load Transfer	No Load Transfer	Load Transfer
8 in PCC	AGG				NC 1-7
9 in PCC	AGG	NC 1-1	NC 1-4	NC 1-8	
	Soil Cement (SC)		NC 1-2	NC 1-3	
	CTB			NC 1-5	
	ATB			NC 1-6	

Figure 17. Experimental design matrix for North Carolina 1.

The first step was to select the pavement sections to be used for this project. An extensive literature search was completed to identify experimental and research projects and regular construction pavement sections for which design, construction, and performance data were available. Over 300 candidate sections were identified in this manner. This list was then narrowed considerably by eliminating those projects that were of extremely limited scope, poor experimental design, or turned out to include variables outside the range of this study. Further cuts were made until a total of 95 sections were selected. Some of these sections represented recently constructed sections incorporating new or innovative design features. However, 80 of the sections were parts of experimental projects. One of the major criteria in the selection of these sections was their ability to fill either site specific or regional feature matrices.

Extensive surveys were initiated on the project sections starting in the spring of 1987. Each survey consisted of identification and marking of the pavement sections, followed by a distress survey. Key distresses for each section were mapped on survey sheets designed for this project. In addition to the mapping of all cracking and other surface conditions, each transverse joint was evaluated for sealant condition and joint condition. The longitudinal joints were also evaluated. Faulting measurements were taken in the outer wheelpath at each joint, as were joint width, lane-shoulder separation, and lane-shoulder dropoff measurements.

Part of the distress survey included a drainage evaluation of the section. This consisted both of a subjective evaluation of several general indicators of the drainage condition of the section and of actual measurements of slopes. These inputs were used with the materials information to calculate the drainability of the pavement sections. This is described in volume V.

A Mays Meter was used to obtain a Roughness Index for each lane in the section, and an average PSR was also calculated for each section. Deflection testing with a Falling Weight Deflectometer was performed on all but three of the projects. The deflection data was used to backcalculate material properties, to identify voids under the concrete slabs, and to determine the load transfer efficiencies at joints.

At the same time as the nondestructive FWD testing took place, coring and boring was performed. Concrete cores, 6 in (152 mm) in diameter, were obtained from slab centers and most joints. The center slab cores were subjected to a split tensile test in order to calculate a modulus of rupture of the concrete. Split tensile strengths obtained from the testing of 6-in (152 mm) cores have been shown to provide better estimates of the modulus of rupture than split tensile strengths of 4-in (102 mm) cores.

The joint cores were examined to evaluate the subsurface condition of the concrete at the joints, where moisture related distresses first start. Any signs of deterioration or material durability problems were noted. Material from any layers present below the pavement slab was collected and gradations and moisture contents for these layers was calculated. Thicknesses of all subsurface layers were verified.

The field data collection efforts were completed in December 1987. All available data was then entered both onto data sheets and into the database. This helped to identify data elements which were missing and for which further collection efforts needed to be made. Information was solicited from the States involved, other researchers who had studied these sections, and FHWA personnel in several key categories. In addition to providing missing design information, data was received on the maintenance and rehabilitation efforts undertaken on the section since construction. Traffic data formed another set of information which needed to be solicited from the States and provinces. Both traffic and truck volumes were requested for every year since the pavement was opened to traffic.

Finally, additional performance and/or testing results were available for several of the sections from previous research efforts. Much of this information was available in published reports. However, many unpublished testing reports and other information were provided from all sources.

CHAPTER 3 PERFORMANCE EVALUATION OF PROJECTS

1. INTRODUCTION

The projects included in the study, briefly described and laid out in chapter 2, were evaluated quite extensively during a 7-month period of 1987. This evaluation consisted of a detailed condition survey, a drainage survey, deflection testing with a Falling Weight Deflectometer, and roughness and serviceability evaluations. At twelve of the project sites, Weigh-in-Motion (WIM) studies were performed. For all locations, design and construction information was solicited and received from the States. Historical traffic information, up to the most recent ADT and percent trucks, was also obtained.

For each project, all of this information was synthesized into a summary report. These reports present the original design and construction information, climatic data, traffic data, physical testing information, drainage information, and performance data for the sections in each project. Probable causes for the observed distresses are suggested and overall performance trends discussed. The summary reports, assembled by environmental region, are presented in volume IV.

In this chapter, the highlights of the summary reports are presented. General climatic information for each region is given, followed by a brief introduction to each project and background traffic and materials testing data. Then, within each project, a discussion of the observed performance of each section is given, organized by the different variables evaluated in each project. Relative performance comparisons within a project are made and some probable causes of the observed distress or poor performance are noted where appropriate. Conclusions concerning the performance of the different design features are discussed, in terms of both the observed distresses and other available information.

Table 1 provides a listing of critical distress values for jointed plain concrete pavements (JPCP) and jointed reinforced concrete pavements (JRCP). It is at these values that a pavement section should be considered a candidate for some sort of rehabilitation. These values are provided only as general guidelines to provide some significance to the distress data often cited throughout the report.

2. DRY-FREEZE ENVIRONMENTAL REGION

Only one State in the dry-freeze climate, Minnesota, was included in the study. Minnesota is actually located in a transition area between wet-freeze and dry-freeze, but an examination of the Thornthwaite Moisture Indices shows values that are much drier than States located in the wet-freeze environmental region, such as Michigan or New York, for example.

Climatic data for the sections in this environmental region includes a Corps of Engineers Freezing Index range of 1688 to 2188, a Thornthwaite Moisture Index range of 0 to 10, and an annual precipitation range of 23 to 30 in (584 to 762 mm). The highest average monthly maximum temperature averaged about 84 °F (29 °C), while the lowest average monthly minimum temperatures ranged from -3 °F (-19 °C) to 6 °F (-14 °C).

Table 1. Listing of critical distress values by pavement type.

Performance Indicator	JPCP	JRCP
Joint Spalling, %	15-20	20-30
Joint Faulting, in	0.13	0.26
Transverse Cracks/mi	67	70
Longitudinal Cracking, ft/mi	500	500
PSR	3.0	3.0

Notes:

Joint spalling represents medium- and high severity levels for both pavement types.

All transverse cracks are included for JPCP, but only deteriorated (medium- and high-severity) cracks are counted for JRCP.

Longitudinal cracking includes all severity levels.

Critical level for PSR assumes Interstate-type pavement.

Minnesota 1

This roadway is a four-lane, divided highway (two lanes in each direction). From 1970 through 1987, the pavement sustained approximately 5.5 million 18-kip (80 kN) Equivalent Single-Axle Load (ESAL) applications in the outer traffic lane and nearly 0.6 million ESAL's in the inner lane.

Slab modulus values and composite k-values (on top of the base) for each section are reported in table 2. These values were determined from deflection testing. Table 3 provides additional results from deflection testing as well as drainage coefficients for each section. The performance data obtained from the condition survey is summarized in figure 18 for the outer lane only.

Observations

Low severity "D" cracking was observed in every section. However, it was much more prevalent on the asphalt-treated base course sections, where it was primarily located along the many longitudinal cracks occurring in those sections. Low severity pumping was also observed on most of the sections, and medium severity pumping was reported for one asphalt-treated base course section.

The asphalt-treated base course sections were the poorest performers of the three different base types. This assessment is primarily based on the amount of transverse and longitudinal cracking present in those sections. The aggregate base course sections performed the best, as they had less faulting and longitudinal cracking.

The sections without load transfer devices faulted much worse than those with load transfer devices. These sections were much rougher and required rehabilitation in 1984. However, it should be noted that the load transfer was not particularly good for the doweled sections, probably because the 1-in (25 mm) devices were of insufficient size for the traffic loading. Also, the sections without load transfer devices actually had less joint spalling and less deteriorated transverse cracking than the doweled sections.

The 9-in (229 mm) pavement sections performed better than their 8-in (203 mm) counterparts in every distress category. The most apparent advantage of the 9-in (229 mm) sections is the smaller amounts of transverse and longitudinal cracking as compared to the 8-in (203 mm) sections.

A direct comparison of joint spacing can be made if the 40-ft (12.2 m) control section is included. Based on this information, the sections with 27-ft (8.2 m) joint spacing are believed to be performing slightly better than the control section, based on the amount of cracking, spalling, and faulting.

Conclusions

The most apparent conclusion that can be drawn from the experiment is that the sections with asphalt- or cement-treated base courses did not perform nearly as well as the sections constructed with the traditional

Table 2. Slab modulus and composite k-values for Minnesota 1.

Section	E-Value, psi	k-Value, pci
MN 1-1	7,090,000	191
MN 1-2	7,775,000	172
MN 1-3	6,666,000	217
MN 1-4	6,920,000	222
MN 1-5	9,130,000	304
MN 1-6	9,360,000	314
MN 1-7	8,300,000	287
MN 1-8	7,880,000	278
MN 1-9	6,670,000	291
MN 1-10	6,740,000	285
MN 1-11	8,030,000	245
MN 1-12	7,790,000	239
MN 5	7,560,000	156

Table 3. Deflection testing results and drainage coefficients for Minnesota 1.

Section	Load Transfer Efficiency %	Corner Deflection, mils	% Corners Exhibiting Voids	Drainage Coefficient
MN 1-1	28	15.5	0	1.05
MN 1-2	48	12.3	0	1.05
MN 1-3	47	11.5	7	1.05
MN 1-4	56	12.9	0	1.05
MN 1-5	51	9.8	0	0.85
MN 1-6	54	8.5	0	0.85
MN 1-7	42	9.3	0	0.85
MN 1-8	87	7.3	0	0.85
MN 1-9	45	14.4	14	0.80
MN 1-10	65	11.9	0	0.80
MN 1-11	62	15.5	29	0.80
MN 1-12	75	11.8	0	0.80
MN 5	62	10.6	0	0.95

MINNESOTA 1 AND MINNESOTA 5
I-94 ROTHSAY

OUTER LANE PERFORMANCE DATA

			8 in JRCP		9 in JRCP		
			NO LOAD TRANSFER	LOAD TRANSFER	NO LOAD TRANSFER	LOAD TRANSFER	
27 ft JT. SPACING	AGG BASE		MN 1-3	MN 1-4	MN 1-1	MN 1-2	AVG.
		FAULTING, in	0.31	0.06	0.31	0.06	0.19
		T. CKS./mi	33	47	0	23	26
		LONG. CKS., ft/mi	301	0	0	0	75
		% JT. SPALL.	8	15	4	14	10
	ATB		MN 1-5	MN 1-6	MN 1-7	MN 1-8	
		FAULTING, in	0.37	0	0.31	0.06	0.19
		T. CKS./mi	41	0	0	102	36
		LONG. CKS., ft/mi	1776	2073	1768	2589	2051
		% JT. SPALL.	24	31	15	69	35
	CTB		MN 1-11*	MN 1-12*	MN 1-9	MN 1-10*	
		FAULTING, in	0.50	0.06	0.37	0.13	0.27
		T. CKS./mi	0	48	0	39	44
		LONG. CKS., ft/mi	1386	176	0	0	391
		% JT. SPALL.	37	29	4	6	19
40 ft JT. SPACING	AGG BASE					MN 5	
		FAULTING, in				0.09	0.09
		T. CKS./mi				53	53
		LONG. CKS., ft/mi				1261	1261
		% JT. SPALL.				36	36

AVG.		8 in NO LOAD TRANSFER	8 in LOAD TRANSFER	9 in NO LOAD TRANSFER	9 in LOAD TRANSFER WITHOUT MN 5	9 in LOAD TRANSFER WITH MN 5
	FAULTING, in	0.39	0.04	0.33	0.08	0.09
	T. CKS./mi	25	47	0	55	54
	LONG. CKS., ft/mi	1154	750	589	863	963
	% JT. SPALL.	23	25	8	30	31

ALL DATA FROM 1987 CONDITION SURVEY, WITH THE EXCEPTION OF JOINT FAULTING FOR MN 1. THIS DATA REPRESENTS 1984 PRE-REHABILITATION AVERAGE JOINT FAULTING.

* THESE SECTIONS HAD SIGNIFICANT TRANSVERSE CRACKING IMMEDIATELY AFTER CONSTRUCTION.

Figure 18. Outer lane performance data for Minnesota 1 (Age = 17 years, ESAL's = 5.5 million).

aggregate base course. Because they did not improve performance, MN/DOT determined that the use of those treated base courses may not be justified due to the higher cost associated with their construction. The performance of the aggregate base course sections may have been enhanced by the better overall drainage characteristics of those sections.

The cause of the extensive amount of longitudinal cracking that occurred on the project must be related to the amount of friction between the base and slab. Asphalt- and cement-treated base courses have much higher friction coefficients than aggregate base courses, thus restraining the slab from moving and resulting in cracking of the slab.

In addition, the longitudinal joint for all sections was formed by placing a plastic insert to a depth of 2.75 in (70 mm). This depth is 34 percent of the slab thickness for the 8-in (203 mm) sections and 31 percent of the slab thickness for the 9-in (229 mm) sections. The plastic insert may not have produced a deep enough weakened plane for the treated bases to allow for the slab to crack at the appropriate location. The MN/DOT has taken cores of the longitudinal joint and determined that the crack did not always occur. The plastic inserts were placed approximately at the suggested depth of one-third the slab thickness, but it is believed that improved longitudinal joint forming practices are needed for pavement slabs constructed over treated base courses.

It is also known that the asphalt-treated base courses were not "whitewashed" prior to the placement of the concrete slab. This generated higher curing temperatures for the concrete and could increase the probability of cracking upon cooling and shrinking. Greater layer-to-layer friction may also be a result of not whitewashing the asphalt-treated base. These two factors could contribute to the large amount of cracking occurring on the asphalt-treated base course sections.

Other possible explanations for the substantial amount of longitudinal cracking which occurred on the project include late sawing, "skip-joint" sawing (sawing every other joint), and the temperature ranges at the time of paving. The paving was done in July and temperatures ranged from a high of 85 ^{o}F (29 ^{o}C) during the day to a low of 30 ^{o}F (-1 ^{o}C) at night. This large swing in temperature may have contributed to the large amount of cracking which occurred on the project.

This project also indicates the benefit of doweling 27-ft (8.2 m) transverse joints in a cold climate. The faulting of the doweled joints was minimal and far less than that of the nondoweled joints. However, the load transfer of the doweled joints was not extremely high, indicating perhaps that 1-in (25 mm) diameter dowels are not of sufficient size to provide reliable load transfer.

The nondoweled pavement sections had less spalling and transverse cracking than the doweled pavement sections. Since the dowel bars had no treatment to inhibit corrosion, the dowel bars have probably corroded, resulting in spalling at the joint above the dowel bar and also causing any transverse cracks in the adjacent panel to open. The transverse cracks could easily open because the amount of reinforcing steel present in the slab was so small (0.08 percent) that it could not adequately hold the transverse cracks tight. A greater amount of reinforcement and the use of coated dowel bars is recommended to inhibit corrosion of the devices.

Another conclusion that can be drawn from this project is the benefit of increased slab thickness on performance. The thicker pavement sections outperformed the thinner pavement sections in every distress category.

Finally, it can be concluded that pavement sections with shorter 27-ft (8.2 m) joint spacings perform better than the sections with the longer 40-ft (12.2 m) joint spacings. Again, the performance of these reinforced sections was perhaps adversely influenced by the small amount of reinforcing steel in the pavement (0.08 percent). This amount of steel is probably insufficient for the temperature extremes experienced by a pavement in this climate.

Minnesota 2

Four experimental concrete pavement sections were constructed in 1977 on I-90 between Albert Lea and Fairmont. Interstate 90 is a four-lane divided highway with two lanes in each direction. The roadway's functional classification is Rural Interstate. From 1977 through 1987, the pavement sustained approximately 2.8 million 18-kip (80 kN) ESAL applications in the outer lane and over 0.2 million 18-kip (80 kN) ESAL's in the inner lane.

Table 4 provides slab modulus values and composite k-values for each section within this experiment. Additional results from deflection testing and drainage coefficients are presented in table 5. Longitudinal lane-shoulder joint load transfer efficiencies are listed in table 6 for those sections with tied concrete shoulders, while performance data, for the outer lane only, is given in figure 19.

Observations

Based on the performance data, the JRCP sections demonstrated slightly better performance than the JPCP sections. Although the JRCP sections exhibited some transverse cracking and joint spalling, the amounts were insignificant. Both sections, which were doweled, exhibited about the same amount of joint faulting, but the JPCP sections did display a substantial amount of longitudinal cracking.

It should be noted that the performance of these different pavement types is confounded by joint spacing and shoulder type. The JPCP had 13-16-14-19-ft (4.0-4.9-4.3-5.8 m) skewed joints with tied PCC shoulders, whereas the JRCP had 27-ft (8.2 m) skewed joints and AC shoulders. Overall, both pavement types were performing very well at the time of the survey.

A direct comparison of slab thickness can be made between sections MN 2-1 and MN 2-2. MN 2-1 (9-in [229 mm] slab) performed slightly better than MN 2-2 (8-in [203 mm] slab) due to the longitudinal cracking which occurred on the thinner section. However, as previously discussed, this is not attributed to the slab thickness. In every other distress category, the 8- and 9-in (203 and 229 mm) pavement sections performed about the same.

Table 4. Slab modulus and composite k-values for Minnesota 2.

Section	E-Value, psi	k-Value, pci
MN 2-1	6,780,000	128
MN 2-2	8,010,000	127
MN 2-3	7,320,000	162
MN 2-4	6,620,000	178

Table 5. Deflection testing results and drainage coefficients for Minnesota 2.

Section	Load Transfer Efficiency %	Corner Deflection, mils	% Corners Exhibiting Voids	Drainage Coefficient
MN 2-1	79	11.2	7	0.85
MN 2-2	86	11.1	7	0.75
MN 2-3	99	8.2	0	0.80
MN 2-4	86	6.0	0	0.95

Table 6. Longitudinal lane-shoulder joint load transfer efficiency for Minnesota 2.

Section	Longitudinal Lane-Shoulder Joint Load Transfer Efficiency, %
MN 2-1	100
MN 2-2	96
MN 2-3	N/A
MN 2-4	N/A

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		OUTER LANE PERFORMANCE DATA					
		8 in SLAB		9 in SLAB			
		AC SHOULDER	PCC SHOULDER	AC SHOULDER		PCC SHOULDER	
			MN 2-2			MN 2-1	AVG.
JPCP	PSR		3.9			3.8	3.8
13-16-14-19 ft	ROUGHNESS, in/mi		82			72	77
JT. SPACING	FAULTING, in		0.06			0.06	0.06
DOWELED	T. CKS./mi		0			0	0
JOINTS	LONG. CKS., ft/mi		150			0	75
AGGREGATE BASE	% JT. SPALL.		9			3	6
				MN 2-3	MN 2-4		
JRCP	PSR			4.0	4.0		4.0
27 ft	ROUGHNESS, in/mi			76	96		86
JT. SPACING	FAULTING, in			0.05	0.06		0.06
DOWELED	T. CKS./mi			0	5		2
JOINTS	LONG. CKS., ft/mi			0	0		0
AGGREGATE BASE	% JT. SPALL.			3	8		6

Figure 19. Outer lane performance data for Minnesota 2 (Age = 10 years, ESAL's = 2.8 million).

Sections MN 2-1 and MN 2-2 each contain a tied and keyed outside PCC shoulder and a widened (15 ft [4.6 m]) inner lane. Sections MN 2-3 and MN 2-4 each contain the same widened inner lane, but have an AC outside shoulder. Although direct comparisons of designs are confounded by joint spacing, by pavement type, and by pavement thickness, some interesting observations can still be made. For instance, the outside corner deflection was actually higher for the sections with PCC shoulders than for those with AC shoulders. Also, while the PCC shoulders contained no transverse cracks, there were some locations along the longitudinal lane-shoulder joint exhibiting spalling. There were also two corner breaks at these locations (one on the shoulder). This may be due to the fact that the paint stripe delineating the break between the outer lane and the outer shoulder had at one time been painted on the outer shoulder for all of the sections. This would have resulted in more trucks encroaching on the shoulder and causing large edge stresses and deflections.

Even though the presence of the tied PCC shoulders did not reduce the corner deflections of the pavement, the PCC shoulders are in much better condition than the AC shoulders. The PCC shoulders contained no transverse cracking and were considered to be in excellent condition. The AC shoulders exhibited medium- to high-severity alligator cracking throughout their length and were rated as poor. Additionally, the sealant between the outer traffic lane and the AC shoulders was in very poor condition, and the lane-shoulder dropoff averaged nearly 0.4 in (10 mm). The lane-shoulder joint sealant was in excellent condition for the concrete shoulders.

Conclusions

One conclusion drawn from this experimental project concerns the benefits gained from using short-jointed slabs, whether plain or reinforced. Both JRCP and JPCP sections, which had short joint spacing for their respective pavement type, performed very well. The distress which was exhibited by each was typical of the design. For instance, the JRCP displayed increased levels of transverse cracking, while the JPCP with the tied shoulders exhibited some longitudinal cracking.

The presence of inside widened lanes and tied PCC shoulders appears to be the reason for the longitudinal cracking which occurred on the project. The longitudinal cracking could have been caused by late sawing or inadequate depth of sawing of the longitudinal joint. This factor becomes more critical as the pavement slab becomes wider. In addition, while widened lanes do reduce edge stresses and deflections, they obviously cannot be widened to the point where they induce longitudinal cracking. Based on information from this project, a 13-ft (4 m) widened lane may be the maximum width allowable for this short, random joint spacing.

The effects of the tied concrete shoulders are unclear. Those pavement sections with the tied PCC shoulders actually displayed larger corner deflections than the sections with AC shoulders. Additionally, more distresses, notably longitudinal joint spalling and two corner breaks, were associated with the longitudinal lane-shoulder joint on the sections with tied PCC shoulders. However, the PCC shoulders were in much better overall condition than the AC shoulders.

It is possible that the poor drainability of the sections with PCC shoulders (MN 2-1 and MN 2-2) reduced or eliminated the potential benefits of the tied shoulders. These sections both were constructed over poorly-draining subgrades, which could have detracted from the performance of the tied concrete shoulders. Good drainability is still required for the overall performance of the sections; the addition of tied concrete shoulders is not a substitute for the consideration of drainage in the pavement design.

Since the load transfer across the longitudinal joint was excellent, it is possible that the higher corner deflections for those sections constructed with concrete shoulders may be due to the tiebars not being strategically located at the corners.

Minnesota 3

This single section located on I-90 was included in the study because of the widened outside lanes which were incorporated in the design. Built in 1986, the pavement is a 9-in (229 mm) JRCP with 27-ft (8.2 m) skewed joints. The roadway has two lanes in each direction and has sustained approximately 1.5 million ESAL applications in the outer lane and nearly 0.25 million ESAL applications in the inner lane.

From deflection testing, the modulus of the slab was backcalculated as 8,810,000 psi (60,740 MPa) and the composite k-value on top of the aggregate base was determined to be 256 pci (70 kPa/mm). The average transverse load transfer was 93 percent and voids were detected at 17 percent of the joints. In terms of drainability, this section had an overall drainability rating of poor to fair, corresponding to an AASHTO drainage coefficient of 0.8.

Project design data and outer lane performance data for this section is summarized in table 7. It is observed that this pavement section is not displaying any major distress.

Minnesota 4

This single section located on Trunk Highway 15 near New Ulm was included in the study because it incorporated widened lanes in its design. Built in 1986, the pavement is a 7.5-in (191 mm) doweled JPCP with 13-16-14-17 ft (4.0-4.9-4.3-5.2 m) skewed joints. The roadway has one lane in each direction and has sustained approximately 0.22 million ESAL applications in each lane.

The modulus of the slab was backcalculated to be 6,300,000 psi (43,440 MPa) and the composite k-value on top of the aggregate base was determined to be 222 pci (61 kPa/mm). The average transverse load transfer was 86 percent and voids were not detected at any of the joints. The subgrade and base materials were poorly-drained, but the presence of 4-in (102 mm) diameter edge drains improves the overall drainability of the pavement section to fair, corresponding to an AASHTO drainage coefficient of 0.9.

Table 8 summarizes the design data and the outer lane performance data for this project. It is observed that no major distresses are occurring and that the pavement is in excellent overall condition.

Table 7. Design and performance data for the outer lane of Minnesota 3 (Age = 1 year, ESAL's = 1.5 million).

Design Data	Performance Data	
9.0 in JRCP	Joint Spalling, %	0
4.0 in AGG base	Joint Faulting, in	0.01
10.0 in AGG subbase	Trans. Cracks/mile	0
27 ft joints	Long. Cracks, ft/mile	0
1.00 in dowels	Roughness, in/mi	44
Widened lane	PSR	3.8
A-4 subgrade		

Table 8. Design and performance data for the outer lane of Minnesota 4 (Age = 1 year, ESAL's = 0.22 million).

Design Data	Performance Data	
7.5 in JPCP	Joint Spalling, %	0
5.0 in AGG base	Joint Faulting, in	0.01
No subbase	Trans. Cracks/mile	0
13-16-14-17 ft joints	Long. Cracks, ft/mile	0
1.00 in dowels	Roughness, in/mi	40
Widened lane	PSR	4.7
A-2-6 subgrade		

Table 9. Design and performance data for the outer lane of Minnesota 6 (Age = 4 years, ESAL's = 0.85 million).

Design Data	Performance Data	
8.0 in JRCP	Joint Spalling, %	0
4.0 in PATB	Joint Faulting, in	0.01
4.0 in AGG subbase	Trans. Cracks/mile	0
27 ft joints	Long. Cracks, ft/mile	0
1.00 in dowels	Roughness, in/mi	51
Widened lane	PSR	4.4
A-2-4 subgrade		

Minnesota 6

This single section located on Truck Highway 15 near Truman has widened lanes and a permeable asphalt-treated base. Constructed in 1983, the pavement is an 8-in (203 mm) JRCP with 27-ft (8.2 m) skewed joints. The roadway has one lane in each direction and has sustained approximately 0.85 million ESAL applications in each direction.

The modulus of the slab was backcalculated to be 6,570,000 psi (45,300 MPa) and the composite k-value on top of the permeable asphalt-treated base was determined to be 199 pci (54 kPa/mm). Voids were not detected at any of the slab corners, and the average transverse load transfer was calculated as 80 percent. The overall drainability of the section was rated as good, with an AASHTO drainage coefficient of 1.05.

A summary of the design data and the outer lane performance data for this project is summarized in table 9. There were no indications of pumping, joint spalling, transverse cracking, or longitudinal cracking observed within the pavement section. Faulting is extremely low (0.01 in [3 mm]) after 4 years and roughly 0.85 million ESAL's.

3. DRY-NONFREEZE ENVIRONMENTAL REGION

Two States located in the dry-nonfreeze environmental region were included in the study, California and Arizona. Two experimental projects were located in California and one was located in Arizona. In addition, three single projects were included from California and one from Arizona.

Climatic indicators for this region include a Corps of Engineers Freezing Index of 0, a Thornthwaite Moisture Index range of -10 to -30, and annual precipitation ranging from 8 to 17 in (203 to 432 mm). The highest average monthly maximum temperature ranges from 89 $^{\circ}$F (32 $^{\circ}$C) to 105 $^{\circ}$F (41 $^{\circ}$C), while the lowest average monthly minimum temperatures ranges from 36 $^{\circ}$F (2 $^{\circ}$C) to 41 $^{\circ}$F (5 $^{\circ}$C).

Arizona 1

Experimental pavement sections were built on the Superstition Freeway in Phoenix over a number of years from the mid-1970's through the early 1980's. This roadway's functional classification is Urban Principal Arterial. All of the sections were originally constructed as four-lane divided highway (two lanes in each direction), but a third lane was retrofitted to the inside of several of the sections. Traffic information and the number of 18-kip (80 kN) ESAL applications sustained by each section (through 1987) is shown in table 10.

Tables 11 through 13 provide testing information for this experimental project. Table 11 lists slab modulus values and composite k-values for each section; table 12 reports additional deflection testing data and drainage coefficients for the sections. Table 13 gives longitudinal lane-shoulder joint load transfer efficiencies for those sections with tied concrete shoulders, and figure 20 provides a summary of the performance data for the outer lane only.

Table 10. Traffic information for Arizona 1.

Section	Year Built	2-Way ADT, 1987	Lanes Each Direction	Year Third Lane Added	Outer Lane ESAL's	Lane 2 ESAL's
AZ 1-1	1972	110,380	3	1985	4.0	2.0
AZ 1-2	1975	118,580	3	1984	3.4	1.8
AZ 1-4	1979	93,690	3	1985	2.4	1.2
AZ 1-5	1979	106,440	3	1985	2.8	1.5
AZ 1-6	1981	97,770	2	- -	2.0	0.8
AZ 1-7	1981	75,370	2	- -	1.5	0.5

Table 11. Slab modulus and composite k-values for Arizona 1.

Section	E-Value, psi	k-Value, pci
AZ 1-1	3,140,000	546
AZ 1-2	3,440,000	492
AZ 1-4	3,490,000	344
AZ 1-5	3,290,000	439
AZ 1-6	3,090,000	621
AZ 1-7	3,690,000	584

Table 12. Deflection testing results and drainage coefficients for Arizona 1.

Section	Load Transfer Efficiency %	Corner Deflection, mils	% Corners Exhibiting Voids	Drainage Coefficient
AZ 1-1	94	6.3	30	1.00
AZ 1-2	100	4.0	0	1.10
AZ 1-4	100	5.5	0	1.10
AZ 1-5	100	9.9	36	1.10
AZ 1-6	100	4.4	0	1.10
AZ 1-7	94	5.1	24	1.15

Table 13. Longitudinal lane-shoulder joint load transfer efficiency for Arizona 1.

Section	Longitudinal Lane-Shoulder Joint Load Transfer Efficiency, %
AZ 1-1	N/A
AZ 1-2	86
AZ 1-4	79
AZ 1-5	72
AZ 1-6	100
AZ 1-7	95

ARIZONA 1 RTE. 360 PHOENIX

OUTER LANE PERFORMANCE DATA

			13-15-17-15 ft SKEWED JOINTS				
			NO EDGE DRAINS		EDGE DRAINS		
			AC SHOULDER	PCC SHOULDER	AC SHOULDER	PCC SHOULDER	AVG.
9 in JPCP	CTB		AZ 1-1				
		PSR	3.4				3.4
		ROUGHNESS, in/mi	114				114
		FAULTING, in	0.08				0.08
		T. CKS./mi	0				0
		LONG. CKS., ft/mi	149				149
		% JT. SPALL.	22				22
	LCB			AZ 1-6		AZ 1-7	
		PSR		3.5		3.8	3.6
		ROUGHNESS, in/mi		97		91	94
		FAULTING, in		0.01		0.02	0.02
		T. CKS./mi		0		0	0
		LONG. CKS., ft/mi		0		0	0
		% JT. SPALL.		1		0	1
11 in JPCP	NO BASE			AZ 1-5			
		PSR		3.8			3.8
		ROUGHNESS, in/mi		97			97
		FAULTING, in		0.01			0.01
		T. CKS./mi		0			0
		LONG. CKS., ft/mi		0			0
		% JT. SPALL.		0			0
13 in JPCP	NO BASE			AZ 1-2 AZ 1-4			
		PSR		3.8 3.6			3.7
		ROUGHNESS, in/mi		65 102			88
		FAULTING, in		0.01 0.01			0.01
		T. CKS./mi		0 0			0
		LONG. CKS., ft/mi		0 0			0
		% JT. SPALL.		1 0			1
AVG.		PSR	3.4	3.7		3.8	
		ROUGHNESS, in/mi	114	90		91	
		FAULTING, in	0.08	0.01		0.02	
		T. CKS./mi	0	0		0	
		LONG. CKS., ft/mi	149	0		0	
		% JT. SPALL.	22	1		0	

Figure 20. Outer lane performance data for Arizona 1.

For the following discussion, the sections are compared as if they had the same age and were exposed to the same loadings. This is not a good assumption for deterioration, which accelerates with age and traffic, and that fact should be kept in mind.

Observations

Of the three base types, the best composite k-value (on top of the base) was found on the sections with lean concrete base. The next best was the CTB section, followed by the sections constructed without a base. However, these were all in the range of 400 to 600 pci (108 to 163 kPa/mm), and would be considered uniformly excellent in any location. While no means of differentiating among the performance of the three separate base types is readily apparent, it can be said that the thickened sections constructed without a base are performing at least as well as the other sections constructed on a base. The CTB section does have more faulting than any of the other sections, but it has been subjected to more traffic loading than the other sections.

The use of load transfer devices was not a variable in this project. However, the fact does stand out that in these pavements ranging in age from 5 to 15 years and subject to high traffic levels and loads, load transfer efficiency was uniformly excellent for each section, varying from 94 percent to 100 percent. These would be good values even for pavements with load transfer devices. In informal discussion with those familiar with the Phoenix-area pavements, it has been suggested that in allowing the intrusion of incompressibles into the joint, a situation is created where the slabs are always in compression against each other and the load transfer stays high. The validity of this or other explanations would require further evaluation and field testing before it is accepted.

Slab thickness varied from 9 in (229 mm) to 13 in (330 mm), although the effect of slab thickness is confounded with base type. The 11-in (279 mm) and 13-in (330 mm) slabs constructed over subgrade have performed as well as the 9-in (229 mm) slab constructed on LCB. There are no significant differences between the performance of the 11-in (279 mm) section and the 13-in (330 mm) sections.

PCC shoulders and AC shoulders were both included in this project. It is believed that tied PCC shoulders reduce slab edge deflections and load-related deterioration. However, this feature is not isolated so that its possible benefits can be evaluated. The PCC shoulders were performing quite well, and the shoulder-mainline pavement load transfer was good.

One of the pavement sections, AZ 1-7, was constructed with edge drains. This section shares the same construction date and cross-section as AZ 1-6, so the two sections can be readily compared. There was no cracking or spalling observed on either section and faulting is insignificant on both. The PSR and roughness for each section were comparable.

Conclusions

It is difficult to draw any strong conclusions from this project, due to the limitations of the experimental factorial design and the differences in age and accumulated ESAL's of all of the sections. However, several inferences can be made. The only section that is showing significant deterioration, in the form of faulting and joint spalling, is AZ 1-1. The joint spalling may be caused by infiltration of incompressibles into the joints. The other sections do not show any significant deterioration to date and appear to be equivalent designs. This includes 11- to 13-in (279 to 330 mm) of JPCP placed on the subgrade, and 9-in (229 mm) of JPCP placed over a lean concrete base. The available performance data suggests that these thickened pavements constructed on the subgrade are performing as well as the thinner pavements constructed on the LCB. There is no real evidence to support the beneficial effects of edge drains on performance in this project.

Longitudinal cracking was observed on one section (AZ 1-1). This section happened to be the oldest section in the experiment, but it is not believed that the cracking is due to loading. Rather, it is believed that the insufficient depth of the sawcut used to form the longitudinal lane-lane joint (only 25 percent of the slab thickness) was the cause.

It is a widely-held feeling in the ADOT organization that there are certain factors which help almost any pavement design to perform well there. These include a very low annual rainfall, the absence of freeze-thaw problems, and a source of exceptionally durable aggregate used in their concrete. While these factors may not be sufficient to allow any design to perform indefinitely, they may help to explain why significant distresses are taking so long to develop.

Arizona 2

Located on I-10 just outside of Phoenix, this single section, built in 1983, was included in the study because it is a doweled JPCP. The roadway has three lanes in each direction and has sustained approximately 1.6 million, 0.8 million, and 0.2 million 18-kip (80 kN) ESAL applications in the outer, center, and inner lanes, respectively.

Backcalculation of deflection testing results provided a slab modulus value of 5,560,000 psi and a composite k-value on top of the lean concrete base of 174 pci (47 kPa/mm). An AASHTO drainage coefficient of 1.05 was assigned to this section, indicating an overall drainability of fair-good.

As seen from table 14, this section is in excellent condition. No major distresses were observed on the pavement section. It is believed that the relatively low PSR rating for a 4-year-old pavement may be attributed to roughness built in to the pavement during construction.

California 1

Interstate 5 near Tracy, California, was the site of an experimental project constructed in 1971. The roadway is a four-lane divided highway, with a functional classification of Interstate Rural. Through 1987, the pavement has sustained approximately 7.6 million and 1.1 million 18-kip (80 kN) ESAL applications in the outer and inner lane, respectively. Slab modulus values for the project are reported in table 15. Deflection information and drainage coefficients for the sections are provided in table 16. Figure 21 provides a summary of the performance data for the outer lane only.

Table 14. Design and performance data for the outer lane of Arizona 2 (Age = 4 years, ESAL's = 1.6 million).

Design Data	Performance Data	
10.0 in JPCP	Joint Spalling, %	0
5.0 in LCB	Joint Faulting, in	0.01
No subbase	Trans. Cracks/mile	0
13-15-17-15 ft joints	Long. Cracks, ft/mile	0
1.25 in dowels	Roughness, in/mi	51
PCC shoulder	PSR	4.4
A-6 subgrade		

Table 15. Slab modulus and composite k-values for California 1.

Section	E-Value, psi	k-Value, pci
CA 1-1	6,610,000	232
CA 1-3	5,240,000	349
CA 1-5	5,280,000	335
CA 1-7	6,480,000	433
CA 1-9	6,950,000	298

Table 16. Deflection testing results and drainage coefficients for California 1.

Section	Load Transfer Efficiency %	Corner Deflection, mils	% Corners Exhibiting Voids	Drainage Coefficient
CA 1-1	85	12.1	60	1.10
CA 1-3	87	10.4	50	1.00
CA 1-5	89	10.5	60	1.10
CA 1-7	88	6.1	10	1.15
CA 1-9	86	11.5	55	1.10

CALIFORNIA 1 I-5 TRACY

OUTER LANE PERFORMANCE DATA

			JPCP SKEWED JOINTS			
			5-8-11-7 ft JT. SPACING	12-13-19-18 ft JT. SPACING		
			8.4 in PCC	8.4 in PCC	11.4 in PCC	
			CA 1-1	CA 1-3	CA 1-5	AVG.
NORMAL STRENGTH PCC	CTB	PSR	2.9	3.0	2.7	2.9
		ROUGHNESS, in/mi	102	94	122	106
		FAULTING, in	0.06	0.10	0.11	0.09
		T. CKS./mi	5	30	0	12
		LONG. CKS., ft/mi	0	500	0	167
		% JT. SPALL.	2	3	3	3
				CA 1-7		
	LCB	PSR		2.7		2.7
		ROUGHNESS, in/mi		122		122
		FAULTING, in		0.06		0.06
		T. CKS./mi		75		75
		LONG. CKS., ft/mi		230		230
		% JT. SPALL.		9		9
				CA 1-9		
HIGH STRENGTH PCC	CTB	PSR		2.5		2.5
		ROUGHNESS, in/mi		116		116
		FAULTING, in		0.13		0.13
		T. CKS./mi		190		190
		LONG. CKS., ft/mi		449		449
		% JT. SPALL.		3		3
AVG.		PSR	2.9	2.7	2.7	
		ROUGHNESS, in/mi	102	111	122	
		FAULTING, in	0.06	0.10	0.11	
		T. CKS./mi	5	95	0	
		LONG. CKS., ft/mi	0	341	0	
		% JT. SPALL.	2	5	3	

Figure 21. Outer lane performance data for California 1 (Age = 16 years, ESAL's = 7.6 million).

Observations

All sections showed some signs of pumping, from blowholes to the extensive presence of fines on the shoulders. There is a significant difference between the cement-treated base and lean concrete base in terms of loss of support development. The visual extent of pumping, as evidenced by fines on the surface, was much greater for the lean concrete base section (high severity) than for the cement treated sections (none to medium). However, the extent of fines on the surface is not indicative of the actual loss of support beneath the slab.

The cement-treated base section has performed better in all cases except faulting and longitudinal cracking. The longitudinal cracking present is suspected to be a construction-related problem and not related to the base type. While faulting on this base type is higher, overall roughness and serviceability rating shows the cement-treated base section to have performed better.

Faulting is less for the section with shorter joint spacing. The measured joint opening after construction for the shorter jointed pavements was about one-half that of the other sections. The shorter openings would help to maintain good load transfer and thus result in lower faulting. However, the 11-ft (3.4 m) slabs on the shorter-jointed section required asphalt concrete patching because of interior corner breaks. The section with longer joint spacing has greater transverse cracking, probably due to increased thermal curling of the longer slabs.

The "high" strength section contained 7.5 sacks of cement per cubic yard of concrete, compared to 5.5 sacks of cement for the "normal" concrete mixtures. This resulted in a higher early strength of the concrete in that section. After 16 years, the extremely limited core data showed about the same strength for each section. However, the backcalculated modulus of elasticity showed a much higher value for the "high" strength concrete section. This section with high strength concrete had more deterioration for every distress (transverse cracking, longitudinal cracking and faulting) and increased roughness and decreased PSR. The reason for this poor performance is not clear, although it may be related to higher shrinkage of the concrete as the cracking may result from a higher heat of hydration in the richer mix while curing.

One of the sections had a slab thickness of 11.4 in (290 mm) as compared to 8.4 in (213 mm) for the others. The thicker slab had far less slab cracking, but increased roughness and lower PSR. The reason for increased roughness is not clear, as after construction this section was not rougher than the other sections. Over time and with increased traffic loadings, the level of cracking present in the thinner section will most likely accentuate the difference in performance between these sections.

Conclusions

The most significant conclusions that can be drawn from this experimental project are related to the individual design parameters, rather than to any one given pavement section. They are presented below.

The lean concrete base did not perform significantly better than the cement treated base, except that faulting was lower due to the use of a less erodible material. The thicker slab reduced slab transverse cracking greatly. As traffic increases in the future, this difference will become even greater. The "high" strength concrete slab showed significantly poorer performance than the "normal" strength concrete. The shorter jointed pavement performed similarly to the longer jointed pavement, except for reduced transverse cracking and faulting. However, overall roughness was about the same. Most of the cracking occurred in the 18-19 ft slabs. This suggests that reducing joint spacing may be a very effective way to reduce transverse cracking.

The longitudinal cracking occurring on the project was located very near the centerline joint and is attributed to improper construction of that joint. The longitudinal joint was only sawed to a depth of 24 to 26 percent of the slab thickness, which is less than the accepted minimum depth of 33 percent of the slab thickness.

The much higher traffic loadings on the outer lane resulted in greater cracking, greater roughness and reduced PSR than on the inner lane.

California 2

This experimental project, located on I-210 near Los Angeles, California, was constructed in 1980. The roadway is an eight-lane divided highway (four lanes in each direction), and has a functional classification of Urban Interstate. Through 1987, the pavement sustained approximately 4.4 million 18-kip (80 kN) ESAL applications in the outer lane, 2.1 million ESAL applications in the outer center lane, and roughly 0.3 million ESAL applications in both the inner center lane and the inner lane.

Table 17 gives slab modulus and composite k-values for the project, table 18 indicates additional deflection testing results and AASHTO drainage coefficients, and figure 22 provides a summary of the performance data.

Observations

Medium- and high-severity joint spalling was not observed in either section, although a small amount of low-severity joint spalling was identified. It was noted, however, that reactive aggregate distress was present in the pavement for both sections; this could eventually lead to an increase in joint spalling.

Both sections exhibited a large amount of faulting, as indicated in figure 22. The large amount of faulting is not surprising after examination of the erodibility analysis. The majority of the corners exhibited voids, and since the joints do not contain dowel bars, the faulting exhibited by these sections is not unusual. Generally speaking, the data collected does not indicate a difference in performance of the two sections with respect to faulting.

No transverse cracking occurred on the permeable base/edge drain section (CA 2-2). The control section, however, experienced transverse mid-panel cracking in nearly every long-jointed slab (18- and 19-ft [5.5 and 5.8 m] slabs). The cracks were all of low- or medium-severity.

Table 17. Slab modulus and composite k-values for California 2.

Section	E-Value, psi	k-Value, pci
CA 2-2	7,040,000	1423
CA 2-3	4,980,000	572

Table 18. Deflection testing results and drainage coefficients for California 2.

Section	Load Transfer Efficiency %	Corner Deflection, mils	% Corners Exhibiting Voids	Drainage Coefficient
CA 2-2	10	14.7	65	1.10
CA 2-3	19	24.4	90	0.90

CALIFORNIA 2 I-210 LOS ANGELES

		OUTER LANE PERFORMANCE DATA	
		AC/PERMEABLE CTB	CTB
		CA 2-2	CA 2-3
8.4 in JPCP 13-19-18-12 ft JT. SPACING	PSR	3.8	4.1
	ROUGHNESS, in/mi	107	98
	FAULTING, in	0.11	0.11
	T. CKS. /mi	0	290
	LONG. CKS., ft/mi	0	0
	% JT. SPALL.	0	0

Figure 22. Outer lane performance data for California 2 (Age = 7 years, ESAL's = 4.4 million).

This difference in cracking may be partially attributed to the use of a hot-mix asphalt separation layer (2.4 in [61 mm] thick) placed between the PCTB and the PCC slab on CA 2-2. The PCC slab was placed directly on the nonpermeable CTB for CA 2-3. It is possible that the large amount of friction produced by the CTB at the slab/base interface contributes to the cracking on the longer slabs, since longer slabs experience more movement than shorter slabs due to expansion and contraction. This large amount of friction would be reduced on the permeable CTB section that has the AC separation layer. Another possible explanation is the development of large thermal curling stresses in the longer slabs, acting in combination with a very stiff base.

Conclusions

The design of the permeable CTB appears to facilitate drainage and reduce the erosion/loss of support beneath the slab corners; however, the amount of faulting which occurred was still high. The true effect of the permeable CTB on performance cannot be unmasked because of the inclusion of an AC separation layer over the base course. It appears that the AC separation layer assisted in reducing transverse cracking.

While the construction of a strong permeable CTB can be accomplished (the k-value on top of the base for this section was greater than 1400 pci [380 kPa/mm]), and while the permeable CTB may have reduced the amount of voids beneath the slabs by removing excess moisture, such a base course by itself is not a substitute for providing a mechanical means of load transfer. It is believed that for pavements under heavy truck traffic, positive load transfer devices such as dowel bars are required to keep faulting at an acceptable level.

Finally, longer slab lengths appear to be more vulnerable to transverse cracking than shorter slab lengths. All of the transverse cracking which occurred on section 2-3 was in the 18- and 19-ft (5.5 and 5.8 m) slabs.

California 6

This single section is located on Route 14 near Solemint, in the greater Los Angeles Area. This section, built in 1980, has three lanes in each direction and was included in the study because of its permeable asphalt-treated base course. However, coring and boring operations revealed that it actually had a lean concrete base course.

The slab modulus was backcalculated to be 6,710,000 psi (46,260 MPa), and the composite k-value on top of the lean concrete base was determined to be 294 pci (80 kPa/mm). Transverse joint load transfer was determined to be 79 percent and no voids were detected beneath the slab corners. This pavement section was assigned an AASHTO drainage coefficient of 1.0, indicating an overall drainability of fair-good.

A summary of the design data and the outer lane performance data for this project is provided in table 19. It is observed that the pavement is displaying a substantial amount of faulting, which is also reflected in the roughness and serviceability. However, little spalling or cracking has occurred. The large amount of faulting is somewhat surprising given the fact that no voids were detected beneath slab corners and that no pumping was observed.

Table 19. Design and performance data for the outer lane of California 6 (Age = 7 years, ESAL's = 4.4 million).

Design Data	Performance Data	
9.0 in JPCP	Joint Spalling, %	3
4.2 LCB	Joint Faulting, in	0.15
1.8 in AC subbase	Trans. Cracks/mile	10
12-13-15-14 ft joints	Long. Cracks, ft/mile	0
No dowels	Roughness, in/mi	158
AC shoulder	PSR	3.4
A-2-4 subgrade		

Table 20. Design and performance data for the outer lane of California 7 (Age = 8 years, ESAL's = 10.5 million).

Design Data	Performance Data	
10.2 in JPCP	Joint Spalling, %	0
5.4 in CTB	Joint Faulting, in	0.06
5.4 in lime-treated subbase	Trans. Cracks/mile	119
	Long. Cracks, ft/mile	598
12-13-19-18 ft joints	Roughness, in/mi	87
No dowels	PSR	3.8
AC shoulders		
A-2-4 subgrade		

Table 21. Design and performance data for the outer lane of California 8 (Age = 4 years, ESAL's = 5.3 million).

Design Data	Performance Data	
10.2 in JPCP	Joint Spalling, %	0
5.4 in AC	Joint Faulting, in	0.04
9.0 in AGG subbase	Trans. Cracks/mile	0
12-13-15-14 ft joints	Long. Cracks, ft/mile	0
No dowels	Roughness, in/mi	92
AC shoulder	PSR	3.8
A-7 subgrade		

California 7

One pavement section, located on I-5 near Sacramento and built in 1979, was included in the study because it was one of the older projects utilizing the California Department of Transportation's current drainage design. The section was constructed with longitudinal drains placed at a depth of 15.6 in (396 mm) below the surface of the pavement. The drainpipes were 1.5 in (38 mm) in diameter and the lateral outlets were spaced every 250 ft (76 m) along the project. Drainage fabric was used at the interface between the permeable asphalt material and the subgrade to guard against infiltration of fines into the permeable material.

The roadway is a four-lane divided highway. Through 1987, this pavement section has sustained approximately 10.5 million 18-kip (80 kN) ESAL applications in the outer lane and nearly 2.4 million ESAL applications in the inner lane.

From deflection testing, the modulus of the surface was calculated as 6,260,000 psi (43,160 MPa) and the composite k-value on top of the base was determined to be 326 pci (88.5 kPa/mm). The average load transfer efficiency for the transverse joints was calculated as 35 percent. In addition, 23 percent of the slab corners exhibited voids or loss of support beneath the corners. In spite of the edge drains provided to the section, the poor drainability of the subgrade results in an overall drainability rating of fair and an AASHTO drainage coefficient of 0.85.

Observations

Design and performance data is provided in table 20. The section exhibited a large amount of transverse and longitudinal cracking. This cracking was confined to the first 500 ft (152 m) of the condition survey to a point where a construction joint was located. After the construction joint, no longitudinal or transverse cracking was observed. Thus, the cracking may be construction-related, as the cracked section evidently was at the end of a day's paving. The concrete might have been unable to cure properly or perhaps other construction problems are to blame, such as late sawing of the joints.

No joint spalling or pumping was observed for either lane at the time of the survey. The average transverse joint faulting for the outer lane of this project was 0.06 in (1.5 mm). Joint faulting was not measured in the inner lane due to high traffic volumes.

Conclusions

The longitudinal and transverse cracking exhibited by both the inner and outer lane may be attributed to construction problems, to late sawing of the joints, or to improper depth of saw cut of the joints. The latter appears to be the most logical conclusion, in that all of the cracking occurred before placement of a construction joint which was located in the middle of the section. Apart from the longitudinal cracking, the section is performing quite well in terms of spalling and roughness/serviceability. However, the 0.06 in (1.5 mm) level of faulting after 8 years of traffic coupled with a load transfer efficiency of 35 percent raise concerns for the long-term performance capabilities of this pavement.

California 8

This single section on U.S. 101 near Thousand Oaks has four lanes in each direction, including a full-time exit lane. This exit lane was a widened lane and was the reason for the inclusion of this section into the study. This project was constructed and opened to traffic in 1983 and has sustained approximately 5.3 million 18-kip (80 kN) ESAL applications in the outer (exit) lane.

The slab modulus was backcalculated to be 6,420,000 psi (44,260) and the composite k-value of the asphalt-treated base was determined to be 339 pci (92 kPa). Deflection testing indicated that no voids were present beneath any slab corners and that the transverse joint load transfer efficiency was 92 percent. This section was assigned an AASHTO drainage coefficient of 0.95, corresponding with an overall drainability of fair.

Project design data and outer (exit) lane performance data is summarized in table 21. It is seen from this table that the pavement is in pretty good condition. However, faulting, is beginning to develop and may soon increase and result in pavement roughness.

4. WET-FREEZE ENVIRONMENTAL REGION

In the wet-freeze environmental region, experimental concrete pavement sections were included from five States and one province: Michigan, New York, Ohio, Pennsylvania, New Jersey, and Ontario. Of all the environmental regions, this region contains the most projects and is the best represented.

Climatic indices for the region include a range of 25 to 1000 for the Corps of Engineers Freezing Index, a range of 30 to 60 for the Thornthwaite Moisture Index, and an annual precipitation range of 30 to 43 in (762 to 1092 mm). The highest average monthly maximum temperature ranges from 80 ^{o}F (27 ^{o}C) to 86 ^{o}F (30 ^{o}C). The lowest average monthly minimum temperature ranges from 10 ^{o}F (-12 ^{o}C) to 25 ^{o}F (-4 ^{o}C).

Michigan 1

This experimental project, constructed in 1975, is located on U.S. 10 near Clare, Michigan. U.S. 10 is a four-lane divided highway (two lanes in each direction). The roadway's functional classification is Rural Principal Arterial. From 1975 through 1987, the pavement experienced an estimated 0.89 million cumulative 18-kip (80 kN) ESAL applications in the outer lane and 0.08 million ESAL's in the inner lane.

Backcalculated E and k-values for the pavement sections are given in table 22. Table 23 provides additional deflection testing results and drainage coefficients, while figure 23 presents a summary of performance data for the outer lane of each pavement section.

It should be noted that a 10-in (254 mm) sand subbase was located beneath each of the various base types, and that the subgrade soil is classified as an AASHTO A-2-4 material. In addition, all sections contained full-depth asphalt concrete shoulders, which resulted in bath-tub type cross-sectional designs.

Table 22. Slab modulus and composite k-values for Michigan 1.

Section	E-Value, psi	k-Value, pci
MI 1-1a	5,450,000	353
MI 1-1b	5,790,000	300
MI 1-4a	5,880,000	468
MI 1-7a	6,340,000	292
MI 1-7b	6,090,000	269
MI 1-10a	6,230,000	436
MI 1-10b	5,280,000	502
MI 1-25	N/A	N/A

Table 23. Deflection testing results and drainage coefficients for Michigan 1.

Section	Load Transfer Efficiency %	Corner Deflection, mils	% Corners Exhibiting Voids	Drainage Coefficient
MI 1-1a	98	6.7	0	1.00
MI 1-1b	72	10.3	15	0.90
MI 1-4a	19	21.8	75	1.10
MI 1-7a	100	26.3	94	0.90
MI 1-7b	100	31.8	100	0.80
MI 1-10a	41	13.7	70	0.85
MI 1-10b	42	20.4	100	0.85
MI 1-25	N/A	N/A	N/A	0.70

MICHIGAN 1 U.S. 10 CLARE

OUTER LANE PERFORMANCE DATA

			DRAINED		NONDRAINED		
			SKEWED JOINTS	PERPEND. JOINTS	SKEWED JOINTS	PERPEND. JOINTS	
			NO DOWELS	DOWELS	NO DOWELS	DOWELS	
9 in JPCP 13-17-16-12 ft JT. SPACING	AGG			MI 1-7a		MI 1-7b	AVG.
		PSR		3.6		3.7	3.7
		ROUGHNESS, in/mi		58		37	48
		FAULTING, in		0.04		0.04	0.04
		T. CKS./mi		0		0	0
		LONG. CKS., ft/mi		0		0	0
		% JT. SPALL.		12		11	12
9 in JPCP 13-19-18-12 ft JT. SPACING	PERM ATB		MI 1-4a				
		PSR	3.9				3.9
		ROUGHNESS, in/mi	47				47
		FAULTING, in	0.03				0.03
		T. CKS./mi	0				0
		LONG. CKS., ft/mi	0				0
		% JT. SPALL.	9				9
	ATB		MI 1-10a		MI 1-10b MI 1-25		
		PSR	2.9		2.8 2.9		2.9
		ROUGHNESS, in/mi	98		104 159		120
		FAULTING, in	0.14		0.19 0.20		0.18
		T. CKS./mi	0		18 29		16
		LONG. CKS., ft/mi	0		0 0		0
		% JT. SPALL.	41		63 75		60
9 in JRCP 71 ft JT. SPACING	AGG			MI 1-1a		MI 1-1b	
		PSR		3.6		3.3	3.4
		ROUGHNESS, in/mi		96		91	94
		FAULTING, in		0.05		0.08	0.07
		T. CKS./mi		0		5	3
		LONG. CKS., ft/mi		0		0	0
		% JT. SPALL.		0		0	0
AVG.		PSR	3.4	3.6	2.8	3.5	
		ROUGHNESS, in/mi	72	77	132	64	
		FAULTING, in	0.08	0.04	0.20	0.06	
		T. CKS./mi	0	0	24	3	
		LONG. CKS., ft/mi	0	0	0	0	
		% JT. SPALL.	25	6	69	6	

Figure 23. Outer lane performance data for Michigan 1 (Age = 12 years, ESAL's = 0.89 million).

Observations

The dense-graded asphalt-treated base course sections were the poorest performers of all base types. This is based on the amount of transverse cracking, spalled joints and joint faulting, all of which caused significant roughness in the pavements. The asphalt-treated permeable base sections performed the best, showing very little faulting or joint spalling, and no cracking. The aggregate base course sections performed well, as they had only slightly more deterioration than the permeable base sections.

Those sections with dense asphalt-treated bases were bathtub designs from which water does not drain. This results in continual saturation of the PCC slab when excess moisture is available which causes acceleration of the deterioration of both the transverse and longitudinal joints.

Very little transverse cracking occurred on the experimental sections. The JRCP had practically no deteriorated transverse cracks although cracking is expected on these designs. A possible explanation is that the dowels were coated with epoxy to prevent corrosion and corresponding joint lockup.

Sections with dowels had much higher measured deflection load transfer. In addition, faulting is clearly affected by not only the presence of dowel bars, but also by the drainability of the base course. For the specific traffic, subgrade, and climate of this project, the JPCP with permeable base did not need dowels to prevent faulting. Dowels used with the dense-graded aggregate base, however, did prevent faulting.

French drains were retrofitted to several pavement sections in 1981 allowing for a comparison of sections with and without drainage features. The drained section displayed approximately 33 percent less faulting compared to its nondrained counterpart. The drained section also showed about 15 percent fewer corners with voids, or loss of support. The effect of positive drainage on any other distresses is not apparent, except that for the dense asphalt-treated bases, there was less joint spalling on the drained section (47 percent of the joints) than on the nondrained section (65 to 87 percent).

A direct comparison of joint spacing can be made between the 71.2-ft (21.7 m) JRCP and the 12-17 ft (3.7-5.2 m) JPCP. These sections all contain dowel bars and were constructed over aggregate base courses. The overall performance of these sections is roughly comparable. The only significant difference is the greater roughness and lower PSR for the JRCP section. It is interesting to note that the drainage coefficient for the long-jointed sections is higher than for the short slabs. The excellent drainage characteristics of these sections may have mitigated some of the distresses often associated with long-jointed pavements.

One short section included in the study had a concrete acceleration ramp tied to the outer traffic lane. This section also had a dense-graded, asphalt-treated base course, no subdrainage, and no dowel bars. There was no joint sealant between the lane and acceleration ramp which allowed moisture to freely enter the pavement structure. Also, the MDOT indicated that the tiebars across the lane and ramp failed shortly after construction. The tied acceleration ramp did not improve the performance of this section as it performed as poorly as the two other sections of similar design.

Any evaluation of joint spalling must consider the fact that the large aggregate in the concrete slab was susceptible to freeze-thaw damage, or "D" cracking. Therefore, the more water available to saturate the slab over long periods of time the greater the extent of "D" cracking. This was observed during the field survey and in the joint cores retrieved from transverse joints. Deterioration due to "D" cracking along the longitudinal joint had already led to maintenance patching of this joint in many areas of the project at the time of the survey.

Conclusions

The most significant conclusion that can be drawn from the experiment is that the section with the permeable asphalt-treated base course performed better than any other sections. This section had short joint spacing (12-19 ft [3.7-5.8 m]) and no dowels. The design of this section facilitated drainage of water entering the pavement section. The rapid removal of water meant that moisture was not present long enough to erode the base or saturate the concrete slab, as occurred in other sections. The effect on the pavement's performance was that both joint faulting and spalling were greatly reduced.

The worst performing sections included those having a dense-graded asphalt-treated base course. These sections trapped free water for long periods of time. This led to serious erosion of the base and slab, resulting in faulting, saturation of the concrete, and freeze-thaw damage and spalling at the transverse and longitudinal joints. These sections also had a significant amount of transverse cracking.

The aggregate base sections with dowels performed much better than the asphalt base pavements. Additionally, the aggregate base courses provided some vertical drainage to the sand subbase layer. The drainage afforded by this layer was evidently sufficient to reduce saturation of the concrete slab. Dowels in these pavements also helped to prevent faulting of the joints. Also, french drains placed along some of the sections reduced joint faulting by about 33 percent, with evidence available to show that joint spalling was also reduced.

The joint performance of the 71.2-ft (21.7 m) JRCP was excellent in terms of spalling. This may be attributed to the good performance of the preformed compression seals that apparently kept out incompressibles, and the aggregate base that provided some subdrainage so that significant "D" cracking did not develop.

The drainage coefficients calculated for these sections demonstrates a trend that is followed in the performance of the sections. The sections with the best drainage characteristics had the best performance and the sections with the worst drainage characteristics had the worst performance.

Finally, the doweled pavement sections of this project represent one of the earliest applications of epoxy-coated dowel bars. They apparently have performed well, particularly in the long-jointed slabs which have very few transverse cracks.

Michigan 3

This single section on I-94 near Marshall had a permeable aggregate base course and was constructed in 1986 using recycled concrete. A sand subbase was located beneath the permeable base. The pavement includes tied reinforced concrete shoulders whose 41-ft (12.5 m) joint spacing matches that of the mainline pavement. Through 1987, the pavement had sustained 2.8 and 0.83 million ESAL applications in the outer and inner lanes, respectively.

Backcalculation procedures using deflection data provided an elastic modulus of 4,380,000 psi (30,200 MPa) and a composite k-value of 186 pci (50 kPa/mm). The transverse joint load transfer efficiency was determined to be 88 percent and no voids were detected beneath the slab. Due to the permeable aggregate base course, this section was assigned an AASHTO drainage coefficient of 1.10, corresponding to an overall drainability rating of good.

Table 24 provides a summary of the project design data and the outer lane performance data. Overall, the pavement section is in excellent condition. In only 1 year of service, the pavement has sustained nearly 3 million ESAL applications but is showing little, if any, distress.

Michigan 4

This experimental project on Interstate 69 near Charlotte, constructed in 1970, was intended to compare a tied, plain concrete shoulder with the traditional an AC shoulder. The roadway is a four-lane divided highway with a functional classification of Rural Interstate. Through 1987, the pavement sustained approximately 4.4 million 18-kip (80 kN) ESAL applications in the outer lane and roughly 0.75 million ESAL applications in the inner lane.

Table 25 gives backcalculated E and k-values for each section. Additional deflection information and AASHTO drainage coefficients are provided in table 26. Table 27 gives longitudinal lane-shoulder joint load transfer efficiencies for each section, while figure 24 presents a summary of the performance data of each section.

<u>Observations</u>

Joint corner deflections for the section with AC shoulders were almost double those of the PCC shoulder section. However, adjacent load transfer efficiencies were higher for the section with AC-surfaced shoulders. Neither section had particularly good load transfer efficiency. The load transfer efficiency across the lane-shoulder joint was calculated to be 35 percent for the section with PCC shoulders, which is very low. This indicates that the tied shoulders are providing very little structural support to the mainline pavement.

The transverse joints were in excellent condition on both sections. The section with AC shoulders had 6 percent of the joints spalled and the PCC shoulder section had no transverse joint spalling. The transverse joint sealant for both sections was a preformed compression sealant. It was in fair condition on MI 4-1 and good to fair condition on MI 4-2.

Table 24. Design and performance data for the outer lane of Michigan 3 (Age = 1 year, ESAL's = 2.8 million).

Design Data	Performance Data	
10.0 in JRCP	Joint Spalling, %	0
4.0 in PAGG base	Joint Faulting, in	0.02
3.0 in AGG subbase	Trans. Cracks/mile	0
41 ft joints	Long. Cracks, ft/mile	0
1.25 in dowels	Roughness, in/mi	37
PCC shoulder	PSR	4.8
A-2-4 subgrade		

Table 25. Slab modulus and composite k-values for Michigan 4.

Section	E-Value, psi	k-Value, pci
MI 4-1	4,830,000	283
MI 4-2	4,530,000	189

Table 26. Deflection testing results and drainage coefficients for Michigan 4.

Section	Load Transfer Efficiency %	Corner Deflection, mils	% Corners Exhibiting Voids	Drainage Coefficient
MI 4-1	51	15.6	22	0.75
MI 4-2	67	27.1	95	0.70

Table 27. Longitudinal lane-shoulder joint load transfer efficiency for Michigan 4.

Section	Longitudinal Lane-Shoulder Joint Load Transfer Efficiency, %
MI 4-1	35
MI 4-2	N/A

MICHIGAN 4 I-69 CHARLOTTE

		OUTER LANE PERFORMANCE DATA	
		PCC SHOULDER	AC SHOULDER
		MI 4-1	MI 4-2
9 in JRCP 71.2 ft JT. SPACING	PSR	2.4	2.4
	ROUGHNESS, in/mi	132	112
	FAULTING, in	0.12	0.06
	T. CKS./mi	227	183
	LONG. CKS., ft/mi	40	0
	% JT. SPALL.	0	6

Figure 24. Outer lane performance data for Michigan 4 (Age = 17 years, ESAl's = 4.4 million).

Joint faulting for the tied PCC shoulder section was twice that of the AC shoulder section. However, the faulting is not very large on either section.

The system used to tie the shoulders does not appear to have provided any support at the time the pavement was surveyed in 1987. The spacing of the hook bolts at 40-in (1016 mm) intervals may be too great to provide effective load transfer. However, information furnished by the MDOT indicated that the tiebars were not intended to provide load transfer, only to tie the shoulders to the lane.

With the lower corner deflections on the concrete shoulder section and the greater number of corner voids on the section with AC shoulders, theory suggests that there should be more faulting on the section with AC shoulders. However, the permeability of the base and subgrade on the AC shoulder section were much higher than found on the PCC shoulder section. Perhaps the improved permeability of the AC shoulder section dominates the erosion characteristics of the sections.

Medium- and high-severity transverse cracks were prevalent in both sections and were the major distress observed on this experimental project. A total of 227 deteriorated transverse cracks/mile were recorded on the PCC shoulder section; there were 183 transverse cracks/mile on the AC shoulder section. Thus the section with AC shoulders performed better than the section with PCC shoulders in terms of cracking, although both levels of cracking are very high.

Given the number of spalled and deteriorated transverse cracks, it is not surprising that there were hardly any deteriorated transverse joints. This is probably due to the fact that the dowels at the transverse joints have corroded and locked up. Further support for the fact that the joints are locked and that the cracks are working is the presence of "D" cracking at most of the cracks and its absence at the joints.

Since there was extensive deteriorated transverse cracking, and few deteriorated transverse joints, it is interesting to consider the amount of transverse crack faulting present on these sections. Transverse crack faulting was measured on all transverse cracks where present. Of the 30 transverse cracks exhibiting faulting in the section with PCC shoulders, the average crack faulting was 0.20 in (5 mm). In the section with AC shoulders, there were 11 faulted cracks and the average faulting was 0.31 in (8 mm). While the average was higher in the section with AC shoulders, the total amount of faulting was almost double in the section with PCC shoulders.

An average of 40 linear ft/mile of longitudinal cracking was observed on the section with tied PCC shoulders. No longitudinal cracking was recorded on the pavement section with AC shoulders.

The PSR was 2.4 for both sections. For Interstate pavements such as I-69, a rating of 3.0 is often considered a terminal serviceability point at which major rehabilitation is required. The Mays Roughness Index (RI) was 132 in/mi for the PCC shoulder section and 112 in/mi for the section with AC shoulders. Both the PSR and the RI values are consistent with the amount of joint faulting, transverse cracking, and crack faulting discussed above.

The section with PCC shoulders averaged five transverse cracks/mile on the shoulder. The joint spacing in the nonreinforced shoulders was 18-18-18-17.2 ft (5.5-5.5-5.5-5.2 m). The condition of the lane-shoulder joint sealant was fair and the joint was tight. Spalls were observed on the shoulder adjacent to the mainline pavement transverse joint at 13 percent of those joints. The PCC shoulder averaged 0.2 in (5 mm) of heave above the mainline pavement surface. The overall rating of the outer shoulder on this section was fair to good.

Section 4-2, with AC shoulders, had extensive linear cracking and alligator cracking of the shoulder surface. There was an average dropoff of 0.9 in (23 mm) from the pavement to the shoulder and an average separation of 0.3 in (8 mm). No sealant was present in the lane-shoulder joint.

Conclusions

There was only one design variable, shoulder type, incorporated in this project. This experimental project was designed to evaluate the design and performance of several shoulder types and not particularly to improve pavement performance. If the premise that tied concrete shoulders improve mainline pavement performance is accepted, the results of this project may appear contradictory. However, all factors must be considered. Recall that joint corner deflections were higher for the AC section, with the reduced edge support. Also, the condition of the two shoulders, and especially the condition at the lane shoulder joint, shows that the shoulders are performing as would be expected. The PCC shoulder is structurally superior to the AC shoulder, showing very little deterioration. It can be assumed that they have handled about the same amount of encroaching and parked traffic. With the tighter joint, the presence of joint sealant, and the comparatively distress-free shoulder of the PCC section, the mainline pavement of this section should perform better. The fact that it does not suggests that other mechanisms are controlling the performance of the pavement in this case.

Several factors may help explain this difference in performance. The drainability of the base and subbase under the AC shoulder section was better than that of the PCC shoulder section. Layer permeabilities were better in the AC shoulder section, although their overall drainability ratings were the same. This difference in permeabilities may have had an affect on the performance of the two sections. Other factors that may account for these results is the fact that the spacing of the shoulder ties of the PCC shoulder section was quite large and that the shoulder joint face was finished very smooth to facilitate differential movement between the lane and shoulder. Thus, adequate support was evidently not provided to the mainline pavement and the expected benefits of the tied shoulders were not realized.

The conclusion suggested by these results then, is not that AC shoulders have a more beneficial effect on pavement performance than tied PCC shoulders. The design of the tied shoulder has a very significant effect on the ability of the shoulder to improve pavement performance. Joint spacing in the shoulder and the spacing of the tiebars must be carefully designed to ensure that the desired benefits are achieved. Also, overall pavement drainability has an enormous effect on pavement performance that can overcome other significant design features.

Michigan 5

This single section, located on I-94 near Paw Paw, was built in 1984. The pavement was constructed using recycled concrete and included a permeable aggregate base course and plain concrete shoulders. Through 1987, the pavement had sustained 3.1 million ESAL applications in the outer lane and 0.75 million ESAL applications in the inner lane.

Backcalculated material properties include a slab modulus of 4,490,000 psi (30,960 MPa) and a composite k-value of 233 pci (63 kPa/mm). Load transfer efficiency was calculated to be 94 percent, but the corner deflections were measured to be 26.9 mils. In addition, voids were detected underneath 95 percent of the slab corners. Because of the permeable base course, this pavement was assigned an AASHTO drainage coefficient of 1.05, corresponding to a overall drainability of good.

Observations

Project design data and outer lane performance data for this section is summarized in table 28. There was a tremendous amount of transverse cracking that occurred in this pavement section. It was observed that these cracks in the mainline pavement coincided with the location of joints in the plain concrete shoulders. The joints in the shoulders evidently created or forced open tight cracks in the mainline pavement. Once the cracks opened, they rapidly deteriorated under the heavy traffic loading since the small top size of the recycled coarse aggregate (1 in [25 mm]) was not sufficient to provide adequate aggregate interlock load transfer. More deteriorated transverse cracks were observed in the outer lane, probably due to its higher traffic loading and proximity to the concrete shoulder.

Conclusions

It is clear that the plain concrete shoulders with offset joint spacing had an adverse effect on the performance of this section. However, there are several other factors that also raise serious concerns about this pavement's potential for long-term performance. These factors include the very high corner deflections and the large number of voids detected at slab corners. While these factors have not yet adversely affected the performance of the pavement, it is believed that they will begin to make their presence known in a few years in the form of joint faulting, pumping, and corner breaks. In fact, faulting has already begun.

The voids detected at slab corners could be a result of the permeable base being clogged and poor support conditions. A sand subbase material was located beneath the permeable base which, as indicated from limited testing by Michigan DOT, has contaminated the base layer and reduced its permeability. In addition, there has been concern raised over the percent crushing and gradation specifications of the permeable base material. There was a fairly low percentage of crushed material which, coupled with an improper gradation, may have resulted in an unstable base course.

Another factor contributing to the poor performance of the section is the improper functioning of the epoxy-coated dowel bars. Testing by the Michigan DOT indicated that the bondbreaker placed on the dowel was

Table 28. Design and performance data for the outer lane of Michigan 8 (Age = 3 years, ESAL's = 3.1 million).

Design Data	Performance Data	
10.0 in JRCP	Joint Spalling, %	0
4.0 in PAGG base	Joint Faulting, in	0.05
21.0 in AGG subbase	Trans. Cracks/mile	144
41 ft joints	Long. Cracks, ft/mile	0
1.25 in dowels	Roughness, in/mi	36
PCC shoulder	PSR	4.2
A-2-4 subgrade		

Table 29. Slab modulus and composite k-values for New York 1.

Section	E-Value, psi	k-Value, pci
NY 1-1	3,810,000	549
NY 1-3	3,890,000	503
NY 1-4	3,892,000	534
NY 1-6	4,020,000	619
NY 1-8a	4,100,000	638
NY 1-8b	3,880,000	548

Table 30. Deflection testing results and drainage coefficients for New York 1.

Section	Load Transfer Efficiency %	Corner Deflection, mils	% Corners Exhibiting Voids	Drainage Coefficient
NY 1-1	100	16.4	55	0.90
NY 1-3	47	10.0	20	0.90
NY 1-4	94	11.1	25	0.80
NY 1-6	100	16.5	30	0.75
NY 1-8a	100	11.1	20	1.00
NY 1-8b	100	10.6	25	1.05

ineffective at preventing the dowel from bonding to the concrete. This, and the fact that the dowel basket tie wires were not cut, apparently served to freeze up the joint and open adjacent transverse cracks.

New York 1

This experimental project on Route 23 between Catskill and Cairo was constructed and opened to traffic in 1968. The roadway is a four-lane divided highway, with a functional classification of Rural Collector. Through 1987, the pavement sustained 3.1 million cumulative 18-kip (80 kN) ESAL applications in the outer lane. The inner lane experienced nearly 0.4 million cumulative ESAL's. Table 29 provides the backcalculated E and k-values for each section, while table 30 presents additional deflection information and drainage coefficients. Figure 25 is a summary of performance data for the outer lane of each section.

Observations

The load transfer efficiencies were excellent for the short-jointed slab sections (100 percent), both with and without load transfer devices. The 60.8-ft (18.5 m) pavement sections had lower load transfer efficiencies, varying from 47 percent on ATB to 94 percent on the aggregate base. In addition, voids were detected in all of the pavement sections.

The sections without the two-part malleable iron load transfer devices spalled much less than those sections with the load transfer devices. However, it is not apparent from the data that the devices had any effect on reducing joint faulting. It should be noted that the two-part malleable iron load transfer devices are no longer used by the NYSDOT as they did not reduce faulting and were the cause of many joint-related performance problems.

Joint spacing had a significant effect on pavement performance. The longer JRCP sections had the lowest PSR's and the highest roughness values, but they had the least amount of transverse and longitudinal cracking. However, these sections also had higher incidences of faulting and spalling.

The performance of skewed versus perpendicular joints could not be readily determined, as only one section with skewed joints was included. In general, its faulting was similar to that of the identical design with perpendicular joints, with more transverse cracking recorded in the section with perpendicular joints.

Conclusions

The pavements with short slabs performed better than the long-jointed sections in terms of joint faulting, roughness and PSR. In fact, the long-jointed sections had roughly six times as much faulting as the short-jointed sections. Surprisingly, the short-jointed sections exhibited more transverse cracking. Generally speaking, the sections constructed on the asphalt-treated base performed better than the sections on aggregate base. This is based primarily on the PSR and roughness measurements. Finally, the sections constructed with load transfer devices spalled much worse than those without the devices. In addition, the load transfer devices used in this project were not effective in reducing faulting.

NEW YORK 1 RTE. 23 CATSKILL

OUTER LANE PERFORMANCE DATA

			SKEWED JOINTS		PERPENDICULAR JOINTS		
			NO LOAD TRANSFER	LOAD TRANSFER	NO LOAD TRANSFER	LOAD TRANSFER	AVG.
9 in JPCP 20 ft JT. SPACING	AGG BASE					NY 1-6	
		PSR				3.9	3.9
		ROUGHNESS, in/mi				82	82
		FAULTING, in				0.03	0.03
		T. CKS./mi				35	35
		LONG. CKS., ft/mi				0	0
		% JT. SPALL.				13	13
	ATB		NY 1-8b		NY 1-8a	NY 1-1	
		PSR	3.8		4.1	4.0	4.0
		ROUGHNESS, in/mi	65		64	56	62
		FAULTING, in	0.03		0.01	0.02	0.02
		T. CKS./mi	0		9	0	3
		LONG. CKS., ft/mi	0		0	0	0
		% JT. SPALL.	0		0	6	2
9 in JRCP 60.8 ft JT. SPACING	AGG BASE					NY 1-4	
		PSR				3.4	3.4
		ROUGHNESS, in/mi				90	90
		FAULTING, in				0.09	0.09
		T. CKS./mi				0	0
		LONG. CKS., ft/mi				0	0
		% JT. SPALL.				9	9
	ATB					NY 1-3	
		PSR				3.6	3.6
		ROUGHNESS, in/mi				79	79
		FAULTING, in				0.14	0.14
		T. CKS./mi				0	0
		LONG. CKS., ft/mi				0	0
		% JT. SPALL.				73	73

LOAD TRANSFER DEVICES ARE ACME DEVICES.

AVG.		SKEWED NO LOAD TRANSFER	SKEWED LOAD TRANSFER	PERPENDICULAR NO LOAD TRANSFER	20 ft JOINTS	60.8 ft JOINTS
	PSR	3.8		4.1	3.9	3.5
	ROUGHNESS, in/mi	65		64	69	85
	FAULTING, in	0.03		0.01	0.03	0.11
	T. CKS./mi	0		9	17	0
	LONG. CKS., ft/mi	0		0	0	0
	% JT. SPALL.	0		0	9	41

Figure 25. Outer lane performance data for New York 1 (Age = 19 years, ESAL's = 3.1 million).

New York 2

Constructed in 1975 on Interstate 88 near Otego, the design variables on this project include joint spacing, shoulder type, and pavement type. This roadway's functional classification is Rural Interstate. It is a four-lane divided highway with two through lanes in each direction. Through 1987, the outer lanes of this project sustained 1.4 million 18-kip (80 kN) ESAL applications, while the inner lanes were subjected to approximately 0.14 million ESAL applications.

Table 31 provides backcalculated E and k-values for the sections included in the study, table 32 presents additional results from deflection testing and drainage coefficients, and table 33 provides lane-shoulder joint load transfer efficiencies for the project. Figure 26 summarizes the major performance indicators for the outer lane of each section.

Observations

No joint spalling was recorded for any of these sections which, as noted earlier, all have epoxy-coated I-beams across the joints. The transverse joints on all of the sections are sealed with a rubberized asphalt sealant and the sealant is in excellent condition throughout the project.

Joint faulting on this project was very low, ranging from an average of 0.01 to 0.02 in (0.25 to 0.51 mm). There was no pumping, despite the identification of voids beneath three out of four of the sections. Typically, those sections with a higher percentage of voids would be expected to show greater faulting, although that did not occur here.

The three JPCP sections with tied PCC shoulders showed roughly the same amount of transverse cracking; none was recorded on the JRCP section with AC shoulders. Overall, transverse cracking was a fairly widespread problem on the I-88 sections. It was observed on some parts of the project as soon as 1 year after construction and continued to develop every year thereafter. At the time that transverse cracking first occurred, it caused enough concern to warrant further study. It was determined that the cracking was greater on certain areas of fill and on certain geometric elements for the three different projects. This is unfortunate, because the mechanisms proposed for the occurrence of the distresses is more construction-related than traffic-related. While some transverse cracks may have resulted from load-related causes, it is not possible to isolate those.

One interesting result is the absence of transverse cracking on the JRCP control section. Long-jointed JRCP are expected to crack at intervals determined by their joint spacing, but this section did not even exhibit any low severity transverse cracks. Theory and observations on other projects would suggest that any of the slabs constructed over 20 ft (6.1 m) long would have higher transverse cracking as a result of the increased curling and warping stresses. This effect may be mitigated by the presence of a fairly soft base material. This may also be an indication of the effectiveness of the epoxy-coated I-beams.

Table 31. Slab modulus and composite k-values for New York 2.

Section	E-Value, psi	k-Value, pci
NY 2-3	5,100,000	341
NY 2-9	5,090,000	471
NY 2-11	6,129,000	296
NY 2-15	5,520,000	273

Table 32. Deflection testing results and drainage coefficients for New York 2.

Section	Load Transfer Efficiency %	Corner Deflection, mils	% Corners Exhibiting Voids	Drainage Coefficient
NY 2-3	100	28.7	90	1.00
NY 2-9	97	21.4	50	0.85
NY 2-11	81	11.0	5	0.80
NY 2-15	99	8.4	0	0.85

Table 33. Longitudinal lane-shoulder joint load transfer efficiency for New York 2.

Section	Longitudinal Lane-Shoulder Joint Load Transfer Efficiency, %
NY 2-3	65
NY 2-9	64
NY 2-11	69
NY 2-15	N/A

NEW YORK 2 I-88 OTEGO

OUTER LANE PERFORMANCE DATA

			PCC SHOULDER	AC SHOULDER
9 in JPCP	20 ft JT. SPACING SEALED LANE-SHLDR. JOINT		NY 2-3	
		PSR	4.2	
		ROUGHNESS, in/mi	61	
		FAULTING, in	0.01	
		T. CKS./mi	35	
		LONG. CKS., ft/mi	0	
		% JT. SPALL.	0	
9 in JPCP	20 ft JT. SPACING UNSEAL. LANE-SHLDR. JOINT		NY 2-9	
		PSR	4.0	
		ROUGHNESS, in/mi	66	
		FAULTING, in	0.02	
		T. CKS./mi	25	
		LONG. CKS., ft/mi	543	
		% JT. SPALL.	0	
9 in JPCP	26.6 ft JT. SPACING		NY 2-11	
		PSR	4.1	
		ROUGHNESS, in/mi	76	
		FAULTING, in	0.01	
		T. CKS./mi	26	
		LONG. CKS., ft/mi	0	
		% JT. SPALL.	0	
9 in JRCP	63.5 ft JT. SPACING			NY 2-15
		PSR		4.0
		ROUGHNESS, in/mi		63
		FAULTING, in		0.02
		T. CKS./mi		0
		LONG. CKS., ft/mi		73
		% JT. SPALL.		0

ALL SECTIONS HAVE AGGREGATE BASE COURSES AND I-BEAM LOAD TRANSFER DEVICES.

		PCC SHOULDER	AC SHOULDER
AVG.	PSR	4.1	4.0
	ROUGHNESS, in/mi	68	63
	FAULTING, in	0.01	0.02
	T. CKS./mi	29	0
	LONG. CKS., ft/mi	181	73
	% JT. SPALL.	0	0

Figure 26. Outer lane performance data for New York 2 (Age = 12 years, ESAL's = 1.4 million).

Longitudinal cracking was fairly prevalent on this project. The section with the most longitudinal cracking was the short-jointed section with 20-ft (6.1 m) joint spacing and an unsealed lane-shoulder joint. There was no longitudinal cracking recorded on either the section with 26.7-ft (8.1 m) joint spacing or on the section with 20-ft (6.1 m) joint spacing and a preformed neoprene sealed shoulder. The control section had 73 ft/mi of longitudinal cracking. New York reported that the concrete strength was adequate and that a longitudinal joint did form under the sawcut. They reasoned that the excessive cracking occurred in pavements constructed over base materials with a high frost susceptibility and a ready source of moisture. The depth of the longitudinal sawcut was 2.75 in (70 mm), or 31 percent of the slab thickness. The concrete shoulders also experienced a significant amount of distress in the form of extensive longitudinal cracking and some transverse cracking.

The measured roughness was universally low and the PSR quite high for these 12-year-old sections. The range in PSR for the four sections was from 4.0 to 4.2, which indicates a pavement in excellent condition. The Mays Roughness Index varied from 61 to 76 in/mi, which again is a small range. These values are lower than one might expect for the amount of cracking present on these sections, but are not surprising in light of the low measured faulting on each section. Of particular note is the excellent rideability of the long-jointed section, NY 2-15.

The effect of the various design features on the performance of the different sections can not be easily evaluated. As the performance summary matrix shows, this project was not set up to isolate the design features; its purpose was more to evaluate the long-term performance of several alternative designs to New York's traditional JRCP design. A head-to-head comparison can be made for the two sections with 20-ft (6.1 m) joint spacing, as their designs were identical with the exception of the lane-shoulder joint sealant. In a direct comparison, the section with the sealed joint performed better in most respects. The performance of the other sections can not be readily compared because they had very similar performance and the main distress was the transverse and longitudinal cracking. This cracking was most likely a result of nonload related causes.

Conclusions

This experimental project was subject to numerous distresses that have been attributed to problems in the pavement's construction. The most significant of these were extensive longitudinal cracking. By 1980, there were also concrete spalls above more than 10 percent of the shoulder tiebars. These problems make it difficult to attribute any of the distresses that were observed during the 1987 field survey to other factors.

Despite the fact that NYDOT wanted to subject the experimental designs on I-88 to a greater volume of traffic than existed on Route 23 at Catskill, the ADT was about the same at both project locations, and the traffic count obtained by Weigh-in-Motion for I-88 was actually lower than that at Catskill, although more trucks were recorded.

Although many of the distresses that occurred were related to materials problems, a direct comparison of performance could be made between the 20-ft (6.1 m) JPCP sections with and without shoulder joint sealant. Survey results indicate better performance in the section with the sealed shoulder joint, which suggests the possible beneficial effects of sealing those joints. This advantage did not extend to the inner lane, where the section without the sealed outer shoulder joint performed better than the section with the sealed joint.

For the given climate and traffic loading, the 1 in (25 mm) epoxy-coated I-beams used for load transfer appeared to have worked well. However, such devices probably would be inappropriate for heavily-trafficked roadways.

Ohio 1

This experimental project, located on U.S. 23 near Chillicothe, Ohio, was constructed in 1973. The roadway is a Principal Arterial, four-lane divided highway. Since being opened to traffic in 1973, the pavement has sustained approximately 3.4 million 18-kip (80 kN) ESAL applications in the outer lane and nearly 0.6 million ESAL applications in the inner lane. These values represent calculations through 1987.

Table 34 provides backcalculated E and k-values and table 35 presents additional deflection results and AASHTO drainage coefficients. The performance indicators for each section are summarized in figure 27.

Observations

The asphalt-treated base course sections performed better than the sections constructed over aggregate base. This is based on the amount of faulting and transverse cracking displayed by each section. Additionally, the sections with asphalt-treated base courses sections provided better support to the slabs, as they showed no voids at the slab corners. The sections with the aggregate base course exhibited voids at an average of 70 percent of the slab corners.

The load transfer efficiencies for all of the sections are good to excellent. It should be noted that while every section has dowel bars, the two sections exhibiting 100 percent load transfer (OH 1-6 and 1-7) are those containing plastic-coated dowel bars.

The sections with the shorter joint spacing (21 ft [6.4 m]) consistently exhibited better performance than the sections with longer joint spacing (40 ft [12.2 m]). The 40-ft (12.2 m) slabs displayed larger faulting and more transverse cracking. Both of these are not unusual as the longer slabs undergo more movement and are susceptible to larger curling stresses. However, it should be pointed out that the PSR and the roughness values show no significant differences for the different spacing. That is, although the longer slabs show more distresses, the larger number of joints on the shorter slabs makes the roughness and PSR approximately the same for each section.

Table 34. Slab modulus and composite k-values for Ohio 1.

Section	E-Value, psi	k-Value, pci
OH 1-1	4,430,000	449
OH 1-3	5,310,000	440
OH 1-4	5,230,000	525
OH 1-6	4,060,000	431
OH 1-7	4,430,000	340
OH 1-9	3,400,000	351
OH 1-10	4,510,000	405

Table 35. Deflection testing results and drainage coefficients for Ohio 1.

Section	Load Transfer Efficiency %	Corner Deflection, mils	% Corners Exhibiting Voids	Drainage Coefficient
OH 1-1	95	13.0	71	0.95
OH 1-3	79	7.8	0	0.90
OH 1-4	70	7.7	0	0.90
OH 1-6	100	20.4	94	0.95
OH 1-7	100	16.4	81	1.07
OH 1-9	86	15.7	100	0.90
OH 1-10	71	9.5	0	1.05

OHIO 1 RTE. 23 CHILLICOTHE

OUTER LANE PERFORMANCE DATA

			BASE TYPE			
			AGGREGATE		ATB	
			OH 1-1	OH 1-9	OH 1-4	AVG.
9 in JRCP 40 ft JT. SPACING	STD. DOWELS	PSR	4.2	4.2	4.1	4.2
		ROUGHNESS, in/mi	109	109	123	114
		FAULTING, in	0.13	0.14	0.07	0.11
		T. CKS./mi	0	106	29	45
		LONG. CKS., ft/mi	0	0	0	0
		% JT. SPALL.	0	0	0	0
			OH 1-7			
	PLASTIC COATED DOWELS	PSR	4.2			4.2
		ROUGHNESS, in/mi	115			115
		FAULTING, in	0.07			0.07
		T. CKS./mi	235			235
		LONG. CKS., ft/mi	0			0
		% JT. SPALL.	0			0
9 in JRCP 21 ft JT. SPACING			OH 1-10		OH 1-3	
	STD. DOWELS	PSR	4.2		4.2	4.2
		ROUGHNESS, in/mi	122		106	114
		FAULTING, in	0.10		0.06	0.08
		T. CKS./mi	0		0	0
		LONG. CKS., ft/mi	0		0	0
		% JT. SPALL.	0		13	7
			OH 1-6			
	PLASTIC COATED DOWELS	PSR	4.2			4.2
		ROUGHNESS, in/mi	104			104
		FAULTING, in	0.03			0.03
		T. CKS./mi	31			31
		LONG. CKS., ft/mi	0			0
		% JT. SPALL.	0			0
AVG.		PSR	4.2		4.2	
		ROUGHNESS, in/mi	112		114	
		FAULTING, in	0.09		0.06	
		T. CKS./mi	74		14	
		LONG. CKS., ft/mi	0		0	
		% JT. SPALL.	0		7	

Figure 27. Outer lane performance data for Ohio 1 (Age = 14 years, ESAL's = 3.4 million).

The effect of the plastic-coated dowel bars is not clear. While it would not be expected that they would reduce faulting, the sections with plastic-coated dowels are exhibiting less faulting than the standard dowel sections. However, it is somewhat surprising that the sections with the plastic-coated dowel bars display far more deteriorated transverse cracking than those with the standard dowels. For JRCP, this distress can often be attributed to the dowel bars freezing up (due to corrosion), and the movements associated with the slab force open any tight transverse cracks. Once the cracks are open, they can deteriorate and break down under traffic much more easily. One possible explanation is that the dowel bars were misaligned at construction, which could also result in the opening of mid-slab cracks.

Conclusions

The most significant conclusion that can be drawn from this experiment is that joint spacing has a major impact on reducing the amount of deteriorated transverse cracks occurring on a pavement. Thus, sections with shorter joint spacings would require less repair of deteriorated cracks than sections with longer joint spacing.

It should be noted that the PSR and roughness information indicates similar performance between the sections with the two different joint spacings. It seems that although the shorter slabs exhibited less individual distress in terms of lower levels of faulting and less transverse cracking, the larger number of joints in the short slab sections increased the roughness and also served to decrease the PSR to a level where the two designs are performing about the same. However, the pavement was only 14 years old at the time of survey and was surveyed before a lot of the transverse cracks on the sections with longer joint spacing had broken down. It is believed that once the transverse cracks on the longer slabs break down, the roughness on those sections will increase.

The asphalt-treated bases exhibited better performance than the aggregate bases. They had less faulting and transverse cracking, and also were less erodible than the aggregate base courses. The aggregate bases typically had slab corners with 70 percent voids, but the asphalt-treated bases displayed no voids.

Finally, while the sections with the plastic-coated dowel bars had less faulting than the ones with the standard dowel bars, there was more slab cracking associated with the plastic coated bars. It is possible that these dowels were misaligned when installed. If this it true, this would serve to open up any cracks occurring in the adjacent panels, much like joint lockup. This, in turn, would lead to the cracks breaking down under additional traffic loading.

Ohio 2

In 1974 an experimental project was constructed on State Route 2 near Vermilion, Ohio. This roadway is a four-lane divided highway, with a functional classification of Rural Minor Arterial. Through 1987, the pavement has sustained 3.3 million 18-kip (80 kN) ESAL applications in the outer lane and nearly 0.6 million ESAL applications in the inner lane.

Deflection testing and coring/boring operations were not performed on these sections due to project constraints. Therefore, no modulus values or deflection information is available. Performance data from the condition survey is summarized in figure 28.

Observations

Section OH 2-33b (AC shoulders) exhibited nearly twice as much joint spalling as section OH 2-33a (tied PCC shoulders). This could be due to the presence of the tied concrete shoulder whose lane-shoulder joint is well sealed. A well-sealed longitudinal lane-shoulder joint will greatly reduce the amount of water that can infiltrate a pavement structure. Also, it is possible that the A-4 and A-6 subgrade beneath OH 2-33b is less permeable and "ponds" or holds water. This would result in a saturated condition which could lead to increased spalling of the joints. A major portion of the spalling can be attributed to "D" cracking. Low-severity "D" cracking was observed in both sections at transverse and longitudinal joints and at some of the transverse and longitudinal cracks.

As observed from figure 28, both sections exhibited the same high amount of average transverse joint faulting. Significant pumping was also observed for both sections, with OH 2-33a exhibiting low-severity pumping and OH 2-33b exhibiting medium-severity pumping. The slab-on-grade design of these sections lends itself easily to pumping, particularly on fine-grained subgrades and in a wet environmental region. The difference in severity levels between the two sections can be attributed to the condition of the lane-shoulder joint sealant. The lane-shoulder joint sealant on section OH 2-33a (tied shoulder) was in excellent condition, whereas the sealant on section OH 2-33b (AC shoulder) was in fair-poor condition.

Very little transverse cracking occurred on the experimental sections. The section with the tied concrete shoulders exhibited no transverse cracking, whereas the section with asphalt shoulders displayed only 11 transverse cracks per mile. The tied concrete shoulders appear to have provided more support to the mainline pavement than the asphalt shoulders, thus reducing the amount of cracking.

Longitudinal cracking was observed in both pavement sections. The cause of the longitudinal cracking is most likely the result of late sawing or insufficient depth of sawing. From plans, the longitudinal joint was required to be sawed to a depth of 3.75 in (95 mm), or 25 percent of the depth. This is not considered adequate for the longitudinal joint and, moreover, it may be possible that thicker slabs require even deeper cuts to ensure the formation of the longitudinal joint. It is also possible that the joint sawing crew was inexperienced or not aware of the sawing requirements and actually sawed less than the depth of 3.75 in (95 mm).

There is no discernible difference between the two sections with respect to roughness or PSR, reflecting the fact that the average faulting for both sections was the same.

OHIO 2 RTE. 2 VERMILLION

		OUTER LANE PERFORMANCE DATA	
		TIED PCC SHOULDER	AC SHOULDER
		OH 2-33a	OH 2-33b
15 in NONDOWELED	PSR	3.4	3.5
JPCP	ROUGHNESS, in/mi	90	85
20 ft	FAULTING, in	0.11	0.11
JT. SPACING	T. CKS./mi	0	11
NO BASE	LONG. CKS., ft/mi	104	148
	% JT. SPALL.	15	23

Figure 28. Outer lane performance data for Ohio 2
(Age = 13 years, ESAL's = 3.3 million).

The tied concrete shoulder (OH 2-33a) is performing much better than the asphalt shoulder. It exhibits no dropoff or separation and is in excellent overall condition. Recall also that there was less cracking on the mainline pavement for the tied concrete shoulder section.

Conclusions

It appears that the tied concrete shoulder provides more support to the mainline pavement than the asphalt shoulder. This resulted in the section with the asphalt shoulders exhibiting more pumping and slightly more transverse cracking. Although the amount of transverse cracking may seem insignificant, recall that the slabs are 15 in (381 mm) thick.

The joint sealant between the lane-shoulder joint was an important factor in the amount of pumping exhibited by each section. The lane-shoulder joint for the section with concrete shoulders was in excellent condition, thus preventing water from entering the pavement system. The lane-shoulder joint sealant was in fair-poor condition for the section with the asphalt shoulders. This allowed for the entry of water into the pavement structure and resulted in increased pumping of the subgrade.

Based on the performance of the two sections in this experiment, thick, nondoweled concrete slabs placed directly on subgrade do not appear to be a favorable alternative to conventional designs which include a base course. Given the climate, traffic loading, and subgrade characteristics of the experiment, it is apparent that the additional concrete thickness is not a substitute for the inclusion of a high-quality base course beneath the slab or the use of mechanical load transfer devices. A high-quality base material will reduce pumping and the use of mechanical load transfer devices will help to limit faulting to acceptable levels.

Ontario 1

This experimental project was constructed on Highway 3N near Ruthven in 1982. This roadway's functional classification is Rural Minor Arterial. It is a two-lane, nondivided highway with unrestricted access. Since being opened to traffic in 1982, the pavement has been subjected to approximately 1 million 18-kip (80 kN) ESAL applications (through 1987).

The study sections in Ontario were not subjected to physical testing due to logistical and budgetary reasons. Therefore, no 1987 core or deflection data is available. However, as part of an ongoing study being conducted by the Ontario Ministry of Transportation and Communications, extensive physical testing has been performed by that agency. Results from this testing are used here for reference purposes only, since testing procedures and methodologies may not have been performed in a manner which allows valid comparisons to the other results.

The deflection data was not used to backcalculate E and k-values, but load transfer efficiencies and void detection results were determined. These are presented in table 36 along with drainage coefficients for the sections. Performance data for each section (by direction) is given in figure 29.

Table 36. Deflection testing results and drainage coefficients for Ontario 1.

Section	Load Transfer Efficiency %	Corner Deflection, mils	% Corners Exhibiting Voids	Drainage Coefficient
ONT 1-1	77	N/A	0	0.95
ONT 1-2	48	N/A	0	1.00
ONT 1-3	48	N/A	0	1.00
ONT 1-4	44	N/A	0	1.00

Note: Testing results were obtained by the Ontario MTC and are provided for informational purposes only. Collection and measurement techniques might not be similar to allow for valid comparisons of deflection data.

ONTARIO 1 HWY. 3N RUTHVEN

PERFORMANCE DATA

		12-13-19-18 ft SKEWED JOINTS								
		AC SHOULDER				PCC SHOULDER				
		8 in JPCP		12 in JPCP		7 in JPCP		8 in JPCP		
		ONT 1-2								AVG.
PATB	DIRECTION	SE	NW							
	PSR	3.8	3.7							3.8
	ROUGHNESS, in/mi	78	70							74
	FAULTING, in	0.05	0.06							0.06
	T. CKS./mi	0	0							0
	LONG. CKS., ft/mi	0	0							0
	% JT. SPALL.	0	1							1
						ONT 1-4		ONT 1-3		
LCB	DIRECTION					SE	NW	SE	NW	
	PSR					3.7	3.8	3.8	3.8	3.8
	ROUGHNESS, in/mi					65	72	57	66	65
	FAULTING, in					0.04	0.07	0.04	0.09	0.06
	T. CKS./mi					45	50	30	25	38
	LONG. CKS., ft/mi					85	260	0	490	209
	% JT. SPALL.					0	0	0	1	0
				ONT 1-1						
NO BASE	DIRECTION			SE	NW					
	PSR			3.8	3.8					3.8
	ROUGHNESS, in/mi			90	62					76
	FAULTING, in			0.05	0.07					0.06
	T. CKS./mi			0	0					0
	LONG. CKS., ft/mi			0	40					20
	% JT. SPALL.			0	0					0
AVG.	PSR			3.8		3.8		3.8		
	ROUGHNESS, in/mi			76		68		62		
	FAULTING, in			0.06		0.06		0.07		
	T. CKS./mi			0		48		28		
	LONG. CKS., ft/mi			20		172		245		
	% JT. SPALL.			0		0		1		

Figure 29. Performance data by direction for Ontario 1 (Age = 5 years, ESAL's = 1.0 million).

Observations

There were insignificant differences in the average center slab deflections for the four sections, although the 12-in (305 mm) slab-on-grade had a much smaller variation in its deflection than did the others. Approximately 95 percent of the project is constructed on fill typical of the silty clay found in the area. It is of low to intermediate plasticity and would have a corresponding AASHTO classification of A-6 to A-7-6.

The load transfer efficiencies were not as good as might be expected for a pavement of this age. The 12-in (305 mm) slab-on-grade had the best load transfer efficiencies. The load transfer efficiencies for the thinner slabs, both with and without concrete shoulders, were similarly poor, ranging from 44 percent on the section with a 7-in (178 mm) slab, to 48 percent on the two sections with 8-in (203 mm) slabs.

None of the sections exhibited particularly good drainability, as shown by table 36. It does appear as if the sections with concrete shoulders exhibited less pumping. All of the sections were surveyed shortly after a rainfall, and it was apparent that excess moisture was not being completely removed from the pavement cross-section for any of the sections. This is especially surprising for ONT 1-2, which has a permeable asphalt treated base yet shows the <u>most</u> pumping.

Since one of the design variables on this project was shoulder type, the condition of the shoulders is of interest. The general design of the shoulder was approximately 2 ft (0.6 m) of paved surface and 8 ft (2.4 m) to 10 ft (3.0 m) of granular surface. The paved surface on ONT 1-1 and 1-2 was asphalt concrete. Both of these shoulders exhibited extensive linear cracking and alligator cracking. The overall condition rating was poor for both of these sections.

The PCC shoulder on 1-3 was in excellent condition, although there was some spalling along the shoulder/gravel edge. The concrete shoulder on 1-4 received a rating of good. It had an average of 35 transverse cracks/mile, most of which extended from transverse cracks in the mainline pavement. There was no lane-shoulder dropoff or separation on these two sections. The aggregate portion of all four sections was weathered and raveled.

The performance of the sections placed on lean concrete bases was the worst of the three base types. This is especially true in terms of transverse cracking and longitudinal cracking. If considerable shrinkage cracking of the base occurs during curing, reflection cracking in the PCC slab can develop. If positive measures are not taken to deter reflective cracking, a potential problem exists for the pavement slab. Another factor to consider is the stiffness of the lean concrete base. This is a fairly stiff material and the combination of curling and warping stresses and load stresses can contribute to cracking in the concrete surface. The other two base types showed no cracking.

The traffic lanes with AC shoulders performed better than the sections with tied PCC shoulders. However, as has been noted elsewhere, those sections with PCC shoulders were also the sections with lean concrete bases. The most notable difference in performance between the sections with the two different shoulder types is in the amount of cracking. Since the occurrence of cracking is related to the presence of the LCB, the true effect of shoulder type can not be determined. It would be interesting to learn what type of curing compound was used on the LCB.

This project included 7-in (178 mm), 8-in (203 mm), and 12-in (305 mm) thick pavement slabs. Unfortunately, no apparent trend related to slab thickness emerges from the survey results. For example, one of the sections that performed the best was the 8-in (203 mm) slab on 4 in (102 mm) of PATB with AC shoulders. The 12-in (305 mm) slab-on-grade with AC shoulders performed as well, however. A direct comparison can be made between the 7-in (178 mm) and the 8-in (203 mm) sections on LCB, as the only design difference is slab thickness. The data shows that the thinner section had more overall transverse cracking, but less longitudinal cracking. The effect of slab thickness will not become known until after more traffic loadings.

While base drainability is not directly a variable in this experimental project, an examination of the effects of drainage characteristics on pavement performance provides interesting results. While none of the sections had good drainability, the section with a permeable layer had arguably the best performance of any of the sections. However, it should be noted that the permeable base was placed directly on the subgrade without the use of a filter layer and that the accompanying edge drains were placed in the dense-graded base course beneath the shoulder, beyond the permeable base material. Thus, water was trapped in the permeable layer and could not be removed from the structure; widespread pumping was the result. The limited results available confirm the positive effects of a permeable base layer on pavement performance, yet also indicate the problems of not placing a filter material beneath the base course and not placing edge drains within the permeable material.

Conclusions

Overall, the section with a permeable, asphalt-treated base performed the best and the sections with concrete shoulders on a lean concrete base performed the worst. The construction of a drainable base layer may have contributed to a slight improvement in the performance of ONT 1-2 over the other sections, although there is no identical design without the permeable layer to which a direct comparison can be made. This section was the only one without both transverse and longitudinal cracking and, along with ONT 1-4, also had the lowest average joint faulting of any of the sections. However, the permeable base section had the highest pumping of all of the sections as a result of water being trapped within the structure and not easily removed.

Ontario 2

Highway 427 is an urban principal arterial highway located in Toronto, Ontario. This 9 in (229 mm) thick section was included in the study because it is one of the oldest, heavily-trafficked JPCP designs with dowel bars. It is estimated that this pavement has sustained roughly 36 million 18-kip (80 kN) ESAL applications in the outer lane.

This highway was one of several on which no physical testing was performed. Therefore, no strength estimates of the various layers are available.

Due to the excessive traffic on this pavement and the unavailability of lane closures, physical measurements were only recorded on the outer lane. These performance results, along with the project design data, are summarized in table 37. The results of the drainability analysis performed on the pavement section yielded an AASHTO drainage coefficient of 0.80.

Observations

For a 16-year-old pavement which has sustained 36 million ESAL applications, this pavement is performing extremely well. Faulting is minimal, virtually no cracking has occurred, and the rideability is reasonably good. The cracking that did occur was only in the 19-ft (5.8 m) slabs.

Conclusions

This pavement is performing very well after 16 years of service and a very high level of heavy traffic. Pavement deterioration is negligible and the only maintenance and rehabilitation required on this section to date has involved the AC shoulders. The low level of faulting on such a heavily trafficked pavement in this cold and wet climate suggests the benefit to be derived from doweling transverse joints. In addition, the edge drains that were added after 11 years of service may also have helped the performance of this section.

Pennsylvania 1

An experimental project was constructed on Route 66 and Route 422 near Kittanning in 1980 to investigate the feasibility of several alternative base designs to Pennsylvania Department of Transportation's existing base design. Both roadways are four-lane highways (two lanes in each direction) and have a functional classification of Urban Minor Arterial.

Estimates of the number of 18-kip (80 kN) ESAL applications sustained by each section (through 1987) are presented in table 38. Table 39 provides backcalculated E and k-values for the sections within the project. Table 40 presents additional results from deflection testing and also drainage coefficients for each section. The performance data for the outer lane of each section is summarized in figure 30.

Observations

The approach joint corner deflections for sections 1-2, 1-3, and 1-5 were almost triple those of the other two sections. However, adjacent load transfer efficiencies (LTE) were the highest for these sections. Of the other two sections, the LTE was lower on the section with the well-graded aggregate base.

Table 37. Design and performance data for the outer lane of Ontario 2 (Age = 16 years, ESAL's = 36 million).

Design Data	Performance Data	
9.0 in JPCP	Joint Spalling, %	0
6.0 in CTB	Joint Faulting, in	0.01
No subbase	Trans. Cracks/mile	5
12-13-19-18 ft joints	Long. Cracks, ft/mile	0
1.00 in dowels	Roughness, in/mile	104
AC shoulders	PSR	3.9
Subgrade not known		

Table 38. Traffic information for Pennsylvania 1.

Highway	Section(s)	Two-Way ADT, 1987	1987 % Trucks	ESAL's, millions Inner	Outer
Rt. 422	PA 1-1	10,300	6	0.08	0.60
Rt. 66	PA 1-2 thru PA 1-5	10,200	4	0.03	0.27

Table 39. Slab modulus and composite k-values for Pennsylvania 1.

Section	E-Value, psi	k-Value, pci
PA 1-1	4,210,000	731
PA 1-2	3,390,000	1040
PA 1-3	3,230,000	538
PA 1-4	4,530,000	747
PA 1-5	3,620,000	540

Table 40. Deflection testing results and drainage coefficients for Pennsylvania 1.

Section	Load Transfer Efficiency %	Corner Deflection, mils	% Corners Exhibiting Voids	Drainage Coefficient
PA 1-1	84	12.9	45	0.80
PA 1-2	100	41.4	100	1.05
PA 1-3	98	40.0	95	1.05
PA 1-4	39	11.1	15	1.00
PA 1-5	98	33.4	95	1.00

PENNSYLVANIA 1 RTE. 422 & RTE. 66 KITTANNING

OUTER LANE PERFORMANCE DATA

			10 in JRCP 46.5 ft JOINTS
BASE TYPE	CTB		PA 1-1
		PSR	4.2
		ROUGHNESS, in/mi	75
		FAULTING, in	0.03
		T. CKS./mi	0
		LONG. CKS., ft/mi	0
		% JT. SPALL.	0
	PATB		PA 1-2
		PSR	3.8
		ROUGHNESS, in/mi	91
		FAULTING, in	0.02
		T. CKS./mi	0
		LONG. CKS., ft/mi	0
		% JT. SPALL.	0
	AGG (UNIFORM-GRADED)		PA 1-3
		PSR	3.7
		ROUGHNESS, in/mi	113
		FAULTING, in	0.03
		T. CKS./mi	0
		LONG. CKS., ft/mi	0
		% JT. SPALL.	0
	AGG (WELL-GRADED)		PA 1-4
		PSR	4.0
		ROUGHNESS, in/mi	109
		FAULTING, in	0.03
		T. CKS./mi	0
		LONG. CKS., ft/mi	0
		% JT. SPALL.	0
	AGG (CONTROL)		PA 1-5
		PSR	4.0
		ROUGHNESS, in/mi	82
		FAULTING, in	0.03
		T. CKS./mi	0
		LONG. CKS., ft/mi	0
		% JT. SPALL.	0
	AVG.	PSR	3.9
		ROUGHNESS, in/mi	94
		FAULTING, in	0.03
		T. CKS./mi	0
		LONG. CKS., ft/mi	0
		% JT. SPALL.	0

Figure 30. Outer lane performance data for Pennsylvania 1.

The sections with the most voids were also the sections with the highest LTE's. They also had the highest approach corner deflections. Permeabilities were not a factor in deflections, load transfer efficiency, or presence of corner voids. The least permeable sections, 1-1 and 1-2, had both low and high percent voids. The mid-slab deflection of 1-2 was the lowest and 1-3 was the highest, but their permeabilities were essentially identical. Well-drained sections had both high LTE's (1-2, 1-3) and low LTE's (1-4). The LTE's of the poor-draining sections were consistently good (84 to 98 percent).

Only one design feature, base type, was varied in this experiment. The primary question of interest is whether or not the five base types had any differential effect on pavement performance. Very little differentiation is noted for the various reported distresses, so the answer can only be that varying the permeability of the base layer has no effect on pavement performance, for the given low level of traffic and slab design. While the base permeabilities varied by several orders of magnitude, all of the sections were constructed with longitudinal edge drains, providing them with identical subdrainage design. Another factor to consider is that all of the sections, with the exception of 1-2, were constructed on a substantial longitudinal grade, varying from 1 percent to over 4 percent. It is believed that these factors combined to provide all of the subdrainage necessary for these sections to remove free moisture.

Conclusions

This project was designed to evaluate the constructability and feasibility of several base designs that were substantially different from Pennsylvania's typical base design at the time. The results certainly support the contention that other base designs can be successfully built, but no trends have begun to develop in the performance of the different designs.

Several factors no doubt contributed to the excellent performance of all of the sections. Construction control on this experimental project was probably very good, as is shown by the care taken in obtaining large volumes of both lab and field results. Each section was constructed with longitudinal subdrains which provided drainage to water entering the pavement system along the lane-shoulder interface, which is considered to be the greatest source of excess moisture in a pavement. The longitudinal grade of this particular portion of Route 66 is quite high. Thus surface runoff is likely to be swift and excess moisture may be removed from the pavement surface rapidly, before it has an opportunity to enter the pavement system. Another good feature of these designs is the use of 1.25-in (32 mm) epoxy-coated dowel bars at the transverse joints. These may also be contributing to the excellent performance of all of these sections.

Finally, it should be pointed out that this experimental project was only 7 years old at the time of its evaluation. While the ADT was fairly high, there is very little heavy truck traffic on this pavement as shown by the accumulative ESAL's. Deterioration that would be accelerated by high volumes of heavy traffic will take longer to develop.

New Jersey 2

Route 130 is a four-lane Principal Arterial highway located in a heavily developed part of central New Jersey, east of Trenton. Since being opened to traffic in 1951, this 10-in (254 mm) JRCP has sustained approximately 35 million 18-kip (80 kN) ESAL applications in the outer lane and nearly 7.2 million ESAL applications in the inner lane. These represent calculations through 1987.

Backcalculated material properties include an elastic modulus of the PCC surface (E) of 6,720,000 psi (46,330 MPa) and a composite k-value on top of the aggregate base of 234 pci (63.5 kPa/mm). The approach corner slab deflection was 7.2 mils. The adjusted load transfer efficiency at the transverse joints was 76 percent. Deflections taken at the slab corners indicated that no voids existed beneath any of the approach or leave corners. The calculated drainage coefficient is 1.05, and the pavement received an overall drainability rating of good.

Observations

Typical of pavements in New Jersey, this pavement incorporated expansion joints at every joint. It is observed from table 41 that the section is performing fairly well. Joint faulting is reasonably low, a small amount of cracking is occurring, and the rideability is fairly high. Joint spalling is becoming more of a problem, but this is not unexpected due to the use of the formed expansion joints.

Conclusions

This section was in remarkably good condition for a pavement that is 36 years old. Distresses were minimal, especially considering the age and traffic loadings of this pavement. Faulting and cracking were lower on the inner lane, although its roughness and PSR were higher than on the outer lane. The need for regular joint resealing is not unexpected, given the design of the expansion joints. At the time of the survey, the average joint opening in the outer lane was 1.27 in (32 mm) for the 78-ft (23.8 m) slabs.

Pavement designs using all expansion joints have historically served New Jersey well and this pavement is no exception. However, one drawback is the high roughness values that are typically associated with the design.

New Jersey 3

This experimental project, located on I-676 near Camden, New Jersey, was constructed in 1979. The roadway's functional classification is Urban Interstate. It is a six-lane divided highway, with three lanes in each direction. Through 1987, the project has sustained an estimated 4.2 million 18-kip (80 kN) ESAL applications in the outer lane, 2.5 million ESAL applications in the middle lane, and nearly 0.6 million ESAL applications in the inner lane.

Table 42 gives backcalculated E and k-values for each section, and table 43 presents additional deflection information as well as drainage coefficients for each section. Figure 31 summarizes the outer lane performance data for the project.

Table 41. Design and performance data for the outer lane of New Jersey 2 (Age = 36 years, ESAL's = 35 million).

Design Data	Performance Data	
10.0 in JRCP	Joint Spalling, %	20
5.0 in AGG base	Joint Faulting, in	0.06
7.0 in AGG subbase	Trans. Cracks/mile	24
78.5 ft joints	Long. Cracks, ft/mile	10
1.25 in dowels	Roughness, in/mile	63
AC shoulders	PSR	3.8
A-4 subgrade		

Table 42. Slab modulus and composite k-values for New Jersey 3.

Section	E-Value, psi	k-Value, pci
NJ 3-1	5,400,000	356
NJ 3-2	5,330,000	210

Table 43. Deflection testing results and drainage coefficients for New Jersey 3.

Section	Load Transfer Efficiency %	Corner Deflection, mils	% Corners Exhibiting Voids	Drainage Coefficient
NJ 3-1	50	9.2	0	1.10
NJ 3-2	57	6.7	0	1.10

NEW JERSEY 3 I-676 CAMDEN

		OUTER LANE PERFORMANCE DATA	
		PERMEABLE AGG BASE	PERMEABLE ATB
		NJ 3-1	NJ 3-2
9 in JRCP 78.5 ft JT. SPACING	PSR	3.6	3.5
	ROUGHNESS, in/mi	134	153
	FAULTING, in	0.05	0.06
	T. CKS./mi	0	0
	LONG. CKS., ft/mi	0	0
	% JT. SPALL.	0	43

Figure 31. Outer lane performance data for New Jersey 3 (Age = 8 years, ESAL's = 4.2 million).

Observations

The center slab deflections were higher for the permeable asphalt-treated base than for the open-graded aggregate base. However, corner deflections were just the reverse; the aggregate base corner deflections were higher than those of the asphalt base.

The load transfer efficiencies were fairly low for doweled joints (50 and 57 percent) indicating that some looseness of the dowels may exist. This may be due, in part, to the use of wide expansion joints in lieu of contraction joints.

No voids, or loss of support, were detected at slab corners. Since the aggregate gradation of both base layers was open-graded and a filter fabric was placed between the base and subbase, these results suggest that no migration or displacement of fines is taking place.

After 8 years of service, the section with permeable aggregate base had no joint spalling. The section with the permeable asphalt-treated base had 43 percent spalled joints (medium- or high-severity). One of the reasons for the difference in joint spalling may be related to the construction of the expansion joints.

Joint faulting in the outer lane was similar for the two sections, 0.05 in (1.3 mm) and 0.06 (1.5 mm). These levels of faulting are not considered very significant for long-jointed, doweled JRCP.

No transverse cracking of any severity occurred on either section. This absence of cracking may be due to the relatively low level of truck traffic over the 8 years of service life of this pavement.

No longitudinal cracking occurred on either section. The plans indicate that these pavements were paved one lane at a time, with keyways built between the lanes. This would eliminate two of the most common causes of longitudinal cracking, insufficient depth of sawcut or late sawing of the longitudinal joint.

There exists a significant roughness problem for both sections. The reason for the high roughness is not readily apparent, since there exists very little surface distress. The high roughness values of both sections are probably not related to the performance of the sections, but may have been built in during construction or may be due to the wide expansion joints utilized in the design. It is not likely that the permeable base type had caused the joint spalling since the concrete aggregate is generally sound.

The only design feature that is evaluated in this project is the type of permeable base. The performance of both of the sections evaluated was very good, with the exception of the greater spalling of transverse joints observed in the section with the asphalt-treated base. In general, both of the permeable base designs have performed similarly well after 8 years and 4.2 million ESAL applications.

Conclusions

Both sections with permeable base courses have performed very well. No significant deterioration has occurred after 8 years and 4.2 million ESAL's, except for spalling of the transverse joints in the permeable asphalt-treated base section. Since this pavement has expansion joints every 78-ft (23.8 m) instead of the more conventional contraction joints, the spalling may be related to the construction methods used with the expansion joints.

The high base permeabilities for both of these sections provide rapid subdrainage to water entering the pavement section. In this manner, excess moisture is removed from the pavement structure and erosion of the base or saturation of the concrete slab is prevented. This ultimately aids in reducing faulting.

5. WET-NONFREEZE ENVIRONMENTAL REGION

Three States in the wet-nonfreeze environmental region contributed projects to the study: California, North Carolina, and Florida. The performance of the projects in these States is discussed in this section.

The projects in this environmental region have a Corps of Engineers Freezing Index of 0, a Thornthwaite Moisture Index ranging from 20 to 40, and annual precipitation ranging from 44 in (1118 mm) to 59 in (1499 mm). The highest average monthly maximum temperature averages about 90 ^{o}F (32 ^{o}C), while the lowest average monthly minimum temperature ranges from 29 ^{o}F (-2 ^{o}C) to 50 ^{o}F (10 ^{o}C).

California 3

In 1975, an experimental project was constructed on U.S. 101 near Geyserville to examine the effects of different shoulder types on the performance of jointed plain concrete pavement (JPCP) performance. The roadway has two lanes in each direction and has a functional classification of Principal Arterial. Through 1987, the pavement sustained approximately 3.6 million 18-kip (80 kN) ESAL applications in the outer lane and nearly 0.7 million ESAL applications in the inner lane.

Table 44 shows backcalculated E and k-values for each section in the project. Table 45 presents additional deflection testing results and gives drainage coefficients for each section. Table 46 provides longitudinal lane-shoulder joint load transfer efficiencies, and figure 32 summarizes performance data for the outer lane of each section.

Observations

As expected, the two sections with tied concrete shoulders showed a higher average load transfer efficiency than the nontied concrete shoulder section. No voids, or loss of support, were detected beneath any of the slab corners tested. The subgrade material for all three sections was well-drained. This helps to provide the sections with average drainability.

Table 44. Slab modulus and composite k-values for California 3.

Section	E-Value, psi	k-Value, pci
CA 3-1	3,530,000	286
CA 3-3	4,170,000	312
CA 3-5	4,380,000	397

Table 45. Deflection testing results and drainage coefficients for California 3.

Section	Load Transfer Efficiency %	Corner Deflection, mils	% Corners Exhibiting Voids	Drainage Coefficient
CA 3-1	79	3.8	0	0.90
CA 3-2	86	3.1	0	1.00
CA 3-5	74	6.0	12	0.90

Table 46. Longitudinal lane-shoulder joint load transfer efficiency for California 3.

Section	Longitudinal Lane-Shoulder Joint Load Transfer Efficiency, %
CA 3-1	70
CA 3-2	85
CA 3-5	55

CALIFORNIA 3 U.S. 101 GEYSERVILLE

OUTER LANE PERFORMANCE DATA

		9 in JPCP 12-13-19-18 ft JOINTS		
		TIED PCC SHOULDER	NON TIED PCC SHOULDER	
JOINTS SEALED		CA 3-1		AVG.
	PSR	3.6		3.6
	ROUGHNESS, in/mi	134		134
	FAULTING, in	0.08		0.08
	T. CKS./mi	27		27
	LONG. CKS., ft/mi	0		0
	% JT. SPALL.	2		2
JOINTS UNSEALED		CA 3-2	CA 3-5	
	PSR	3.9	3.8	3.8
	ROUGHNESS, in/mi	104	123	114
	FAULTING, in	0.11	0.10	0.10
	T. CKS./mi	6	118	62
	LONG. CKS., ft/mi	0	34	17
	% JT. SPALL.	3	6	4
AVG.	PSR	3.8	3.8	
	ROUGHNESS, in/mi	119	123	
	FAULTING, in	0.10	0.10	
	T. CKS./mi	16	118	
	LONG. CKS., ft/mi	0	34	
	% JT. SPALL.	2	6	

Figure 32. Outer lane performance data for California 3 (Age = 12 years, ESAL's = 3.6 million).

The concrete shoulders were in good condition after 12 years of service. The sections with tied concrete shoulders had high longitudinal joint load transfer efficiencies, which may have helped reduce transverse cracking. While the overall condition of the nontied shoulder was excellent, the sealant in the longitudinal lane-shoulder joint was in poor condition. The nontied section did have poor load transfer across that longitudinal lane-shoulder joint and also exhibited high corner deflections.

From the limited data, sealing of the transverse joints appears to reduce both transverse joint faulting and spalling. While the reductions achieved by the section with the sealed joints are not substantial, there does appear to be a correlation. The correlation suggests that sealing joints will reduce the amount of water that can infiltrate the pavement system (thereby reducing pumping, erosion, and subsequent faulting) and will prevent the infiltration of incompressibles (thereby reducing joint spalling).

The amount of transverse cracking was greater in the outer lane than in the inner lane. This is due to the higher truck volumes in the outer lane. The number of spalled joints was also observed to be higher for the outer lanes. The mean PSR is higher for the inner lane than for the outer lane which reflects the quantity and severity of distresses observed in this lane. It should be noted that CA 3-1, which had been flooded and required a substantial amount of bituminous patching, had the highest roughness index in both lanes. It is believed that the large roughness values reflect the roughness of that patched area.

Conclusions

This experiment evaluated the effect of joint sealing and tied concrete shoulders on concrete pavement. It was observed that the section with the sealed joints performed better than the others with respect to transverse joint spalling and joint faulting. However, it should be noted that the reduction was not substantial.

With regards to the tieing of concrete shoulders to the mainline pavement, it was determined that the sections with the tied PCC shoulders performed better than the section with nontied PCC shoulders. The sections with tied concrete shoulders had less transverse cracking and smaller corner deflections than the section with nontied concrete shoulders. The relatively short tiebar spacing 22 in (559 mm) appears to be satisfactory to provide good support to the mainline pavement. Thus, the practice of tieing the concrete shoulder to the mainline pavement is an important aspect of concrete shoulder construction that should not be omitted.

North Carolina 1

This experimental project, located on I-95 near Rocky Mount, was constructed in 1967. The roadway is a four-lane divided highway, with a functional classification of Rural Interstate. Since being opened to traffic in 1967, the pavement has sustained 9.1 million 18-kip (80 kN) ESAL applications in the outer lane and over 1.9 million ESAL applications in the inner lane. These values represent calculations through 1987.

Deflection testing results are presented in table 47 and table 48. Table 48 also includes drainage coefficients for each section. Figure 33 summarizes performance data for the outer lane of each section.

Observations

The load transfer efficiencies were more dependent upon base type than on the presence or absence of dowels (load transfer efficiencies of 78 percent for the doweled sections compared to 63 percent for the nondoweled sections). Voids, or loss of support, were detected beneath the joint corners for several of the pavement sections. Particularly, it appears that significant erosion is occurring beneath the aggregate base course sections.

The drainability for all of the sections was not very good. The aggregate base course had a very low permeability which detracted from the drainability of the sections with that base course. If the subgrade for a section was somewhat well-draining, this served to slightly increase the overall drainability of that section.

The pavement section constructed over the asphalt-treated base displayed superior performance over all of the other base types in every distress category. The other base types performed approximately the same, although the aggregate base course sections may have been in slightly worse condition. The remaining base course sections exhibited their own unique deterioration: the aggregate base course sections had transverse cracking, faulting, and erosion of the base; the soil-cement base course sections had significant longitudinal cracking and faulting; and the cement-treated base course section had longitudinal cracking and faulting.

With the exception of the section constructed over the asphalt-treated base course, the treated bases all had a certain amount of longitudinal cracking. It is believed that the treated bases contribute to the cracking because of their larger coefficient of friction produced at the slab-base interface. The longitudinal joint was sawed to a depth of 2.75 in (70 mm), or 31 percent to 34 percent.

The effect of load transfer devices on the performance of these pavements is unclear. Only two direct comparisons can be made between doweled and nondoweled sections. In one case the doweled section performed better than its nondoweled counterpart (aggregate base course), and in the other case the nondoweled section performed better than the doweled section (soil cement base course). However, it appears likely that erosion is occurring on the aggregate base course sections and subsequently the dowels are effective at reducing faulting. The less erodible soil-cement base course sections are not experiencing any erosion, and therefore uniform support is provided to the slab.

For nondoweled joints, it appears that skewed joints offer some advantages over perpendicular joints, particularly in the level of faulting and measured roughness. This information is shown in figure 33 by comparing NC 1-1 and NC 1-8.

Table 47. Slab modulus and composite k-values for North Carolina 1.

Section	E-Value, psi	k-Value, pci
NC 1-1	4,630,000	538
NC 1-2	5,190,000	347
NC 1-3	3,970,000	494
NC 1-4	4,210,000	570
NC 1-5	5,500,000	628
NC 1-6	5,140,000	672
NC 1-7	5,090,000	128
NC 1-8	4,220,000	513

Table 48. Deflection testing results and drainage coefficients for North Carolina 1.

Section	Load Transfer Efficiency %	Corner Deflection, mils	% Corners Exhibiting Voids	Drainage Coefficient
NC 1-1	50	7.6	0	0.75
NC 1-2	100	5.7	0	0.80
NC 1-3	100	4.0	0	0.80
NC 1-4	59	8.8	15	0.75
NC 1-5	92	4.5	0	0.75
NC 1-6	58	4.6	0	0.90
NC 1-7	77	8.3	0	0.90
NC 1-8	16	11.8	40	0.80

NORTH CAROLINA 1

I-95 ROCKY MOUNT, NC

OUTER LANE PERFORMANCE DATA

			JPCP 30 ft JOINTS			JRCP 60 ft JOINTS	
			SKEWED JOINTS	PERPENDICULAR JOINTS		PERPENDICULAR JOINTS	
			NO LOAD TRANSFER	LOAD TRANSFER	NO LOAD TRANSFER	LOAD TRANSFER	AVG.
8 in PCC	AGG					NC 1-7	
		PSR				3.4	3.4
		ROUGHNESS, in/mi				85	85
		FAULTING, in				0.15	0.15
		T. CKS./mi				0	0
		LONG. CKS., ft/mi				295	295
		% JT. SPALL.				0	0
9 in PCC	AGG		NC 1-1	NC 1-4	NC 1-8		
		PSR	3.4	3.4	3.7		3.5
		ROUGHNESS, in/mi	83	85	95		88
		FAULTING, in	0.12	0.13	0.22		0.16
		T. CKS./mi	5	0	64		23
		LONG. CKS., ft/mi	0	0	0		0
		% JT. SPALL.	3	0	0		1
	SOIL CEM			NC 1-2	NC 1-3		
		PSR		3.5	3.6		3.6
		ROUGHNESS, in/mi		79	77		78
		FAULTING, in		0.16	0.13		0.15
		T. CKS./mi		10	5		8
		LONG. CKS., ft/mi		1451	3068		2260
		% JT. SPALL.		0	0		0
	CTB				NC 1-5		
		PSR			3.2		3.2
		ROUGHNESS, in/mi			105		105
		FAULTING, in			0.16		0.16
		T. CKS./mi			0		0
		LONG. CKS., ft/mi			179		179
		% JT. SPALL.			0		0
	ATB				NC 1-6		
		PSR			3.8		3.8
		ROUGHNESS, in/mi			69		69
		FAULTING, in			0.05		0.05
		T. CKS./mi			0		0
		LONG. CKS., ft/mi			0		0
		% JT. SPALL.			0		0
AVG.		PSR	3.4	3.4	3.6	3.4	
		ROUGHNESS, in/mi	83	82	86	85	
		FAULTING, in	0.12	0.15	0.14	0.15	
		T. CKS./mi	5	0	1	0	
		LONG. CKS., ft/mi	0	725	812	295	
		% JT. SPALL.	3	0	0	0	

Figure 33. Outer lane performance data for North Carolina 1 (Age = 20 years, ESAL's = 9.1 million).

A general comparison of JRCP and JPCP and joint spacing (60-ft [18.2 m] and 30-ft [9.1 m]) can be made, although the two items confound one another and are further confounded by pavement thickness. Nevertheless, the two sections have approximately the same performance in terms of roughness, PSR, and faulting. However, the JRCP section with 60-ft (18.2 m) joint spacing has a moderate amount of longitudinal cracking but no deteriorated transverse cracking. This may be partially attributed to the mild environment, the reduced curling stresses, and the soft aggregate base course.

Conclusions

The performance of the section constructed over asphalt-treated base course was superior to all other designs within this experiment. The other base course sections (soil-cement, CTB, and aggregate) performed approximately the same, although the aggregate base course sections displayed the most distress.

The mild climate is probably the reason for the sections being relatively free of distress. Although some cracking occurred on the soil-cement sections, overall very little cracking and virtually no spalling occurred on the project. The absence of the latter may also be attributed to the mild climate, but in conjunction with good quality aggregates and what appears to have been a regular joint sealing program.

The cause of the longitudinal cracking observed on these sections is probably related to the amount of friction between the base and slab. The soil-cement base courses have a larger friction coefficient than aggregate base courses, but the asphalt- and cement-treated base courses are also thought to have large friction coefficients and they did not display any significant longitudinal cracking. Other possible explanations include late sawing, insufficient depth of sawing, or differential support conditions.

For nondoweled joints, there is evidence to support the advantage of skewed joints over perpendicular joints. In both the outer and inner lane, the nondoweled skewed joints over aggregate base course had less faulting than the nondoweled perpendicular joints over the same base course.

Finally, it appears that for a mild climate, treated base courses can have an effect on reducing faulting. In addition, dowel bars are believed to have a definite effect on reducing faulting when used in conjunction with erodible base courses, such as the aggregate base course used on this project. However, the dowel bars used in this experimental project were only 1 in (25 mm) in diameter, probably of insufficient size to keep faulting to a reasonable level. The use of dowel bars of adequate diameter is essential to achieve the desired levels of load transfer.

North Carolina 2

This single section, located on I-85 near Greensboro, is a doweled, 11-in (279 mm) JPCP with a lean concrete base and tied concrete shoulders. Through 1987, this pavement section has sustained approximately 5.8 million, 1.5 million, and 0.4 million ESAL applications in the outer, center, and inner lanes, respectively.

The modulus of the slab was backcalculated from deflection data to be 5,890,000 psi (40,610 MPa) and the composite k-value on top of the lean concrete base was determined to be 293 pci (80 kPa/mm). There was 100 percent load transfer across the transverse joints and only 5 percent of the slab corners were exhibiting voids. This pavement section had an overall drainability of fair-good, corresponding with an AASHTO drainage coefficient of 0.95.

Observations

Significant amounts of pumping was observed throughout the section. The pumping was occurring at the corners of transverse joints, both in the mainline pavement and in the shoulder. A small amount of transverse cracking was observed, but only in the longer slabs (23 and 25 ft [7.0 and 7.6 m]). Design and performance data for the outer lane of this section is presented in table 49.

Conclusions

The large amount of pumping that is occurring on this pavement section is a cause for concern. The pumping is apparently from moisture which is flowing between the slab and the lean concrete base. It is believed that this distress will soon result in additional pavement deterioration, including faulting and corner breaks. The large diameter (1.38 in [35 mm]) dowels used on this project have thus far kept joint faulting to a very low level.

Florida 2

Located on I-75 near Tampa and built in 1986, this single section is a doweled, 13-in (330 mm) JPCP resting on a sand base course. Through 1987, this pavement has carried approximately 2 million ESAL applications in the outer lane, 0.8 million ESAL applications in the center lane, and 0.2 million ESAL applications in the inner lane.

Backcalculation procedures using deflection data provided a slab modulus value of 5,550,000 psi (38,270 MPa) and a composite k-value of 378 pci (180 kPa/mm). Load transfer efficiency was determined to be only 29 percent with 10 percent of the slab corners exhibiting voids.

Design and performance data for the outer lane of this pavement section is summarized in table 50. As would be expected, this 1-year-old pavement section is in excellent condition. The pavement displays no spalling or cracking and has minimal joint faulting. However, the relatively low PSR may be an indication that some roughness problems were built in to the pavement at construction.

Florida 3

This single section project on I-75 was included in the study in an effort to evaluate the poor performance and rapid failure of this pavement. It had carried 4.1 million 18-kip (80 kN) ESAL applications in the outer lane, 1.7 million ESAL applications in the center lane, and 0.4 million ESAL applications in the inner lane.

Table 49. Design and performance data for the outer lane of North Carolina 2 (Age = 5 years, ESAL's = 5.8 million).

Design Data	Performance Data	
11.0 in JPCP	Joint Spalling, %	0
5.0 in LCB	Joint Faulting, in	0.02
No subbase	Pumping Severity	High
18-25-23-19 ft joints	Transverse Cracks/mile	10
1.38 in dowels	Longitudinal Cracks, ft/mile	0
PCC shoulders	Roughness, in/mile	64
A-4 subgrade	PSR	4.2

Table 50. Design and performance data for the outer lane of Florida 2 (Age = 1 year, ESAL's = 2 million).

Design Data	Performance Data	
13.0 in JPCP	Joint Spalling, %	0
6.0 in sand base	Joint Faulting, in	0.01
No subbase	Transverse Cracks/mile	0
12-18-19-13 ft joints	Longitudinal Cracks, ft/mile	0
1.25 in dowels	Roughness, in/mile	64
PCC shoulder	PSR	3.7
A-3 subgrade		

The backcalculated E of the slab was determined to be 4,160,000 psi (28,680 MPa). The composite dynamic k-value on the top of the lean concrete base was backcalculated to be 529 pci (144 kPa/mm). The load transfer measured on the doweled transverse joints averaged only 19 percent, while the mean slab corner deflection was an extremely high 23.9 mils. Slab corner deflection tests showed that 73 percent of the corners were exhibiting loss of support. From the drainage analysis, the subgrade was determined to be very poorly draining. This information, coupled with the presence of the lean concrete base course, resulted in the assignment of a drainage coefficient of 0.75.

For a pavement section only 5 years old, this section is displaying a large amount of deterioration. The design and performance data for this section is summarized in table 51.

<u>Observations</u>

The major distress that has occurred is deteriorated transverse cracking which has reached an excessive amount (generally 70 transverse cracks/mile is a critical level). These cracks were routed and sealed just before the survey was conducted. Transverse cracking varies across the traffic lanes (as does the accumulated ESAL's). This information is summarized in table 52.

In addition, the joint spacing had an effect on cracking. The slab lengths in the random joint spacing pattern ranged from 16 ft (4.9 m) to 23 ft (7.0 m). This information is tabulated in table 53.

Combined, the results from tables 52 and 53 show the effect of slab length and traffic loading on transverse cracking. Generally speaking, the longer the slabs and the heavier the traffic, the greater the number of cracked slabs. The longer slab lengths of 22 and 23 ft (6.7 and 7.0 m) consistently had larger amounts of transverse cracking, and it should be noted that many of those larger slabs actually had two transverse cracks. The shorter 16- and 17-ft (4.9 and 5.2 m) slabs located on the lighter-trafficked inner lane had no transverse cracks, whereas, for the same lane, the longer 22- and 23-ft (6.7 and 7.0 m) slabs had over 50 percent of the slabs cracked. A general reduction in transverse cracking is observed as lower traffic loadings are experienced.

There also existed a fair amount of longitudinal cracking within the project. A total of 896 ft/mi was recorded for all lanes.

Faulting averaged 0.08 in (2.0 mm) at transverse joints, which is approaching a critical level of 0.13 in (3.3 mm). There existed visible signs of medium-severity pumping in the form of fines on the surface. This is apparently from the lean concrete base, which must be eroding away. The 1-in (25 mm) diameter dowels may be insufficient to handle the heavy traffic loadings that are present on the project. Nearly 3 out of 4 joints had a measured loss of support, which correlates with the presence of pumping, and a low load transfer at the transverse joint certainly contributed to the pumping condition. Also, corner deflection was very high, indicating that extensive pumping and loss of support exists along the outer lane.

Table 51. Design and performance data for the outer lane of Florida 3 (Age = 5 years, ESAL's = 4.1 million).

Design Data	Performance Data	
9.0 in JPCP	Joint Spalling, %	2
6.0 LCB	Joint Faulting, in	0.08
No subbase	Pumping Severity	Medium
16-17-23-22 ft joints	Transverse Cracks/mile	310
1.00 in dowels	Longitudinal Cracks, ft/mile	440
PCC shoulders	Roughness, in/mile	84
A-3 subgrade	PSR	3.2

Table 52. Transverse cracking by lane for Florida 3.

Lane	Cumulative ESAL's, millions	Transverse Cracks/mile
Outer	4.1	310
Center	1.7	250
Inner	0.4	80

Table 53. Effect of slab length on cracking for Florida 3.

	% CRACKED SLABS		
Slab Length, ft	Outer Lane	Center Lane	Inner Lane
16	85	54	0
17	86	71	0
22	100	100	69
23	92	100	50
Cumulative ESAL's	4.1	1.7	0.4

The PCC shoulder was generally in good condition. However, there were some localized areas of joint spalling. Load transfer at the longitudinal lane-shoulder joint was determined to be 44 percent, indicating that the tied PCC shoulder is not lending a substantial amount of support to the mainline pavement in edge and corner loading situations.

Conclusions

This 5-year-old, heavily-trafficked pavement showed extensive transverse cracking and significant pumping and faulting. An examination of the transverse cracking by traffic lane (e.g., loading level) and slab length shows that both factors have contributed to this distress. Additionally, it is possible that the doweled, skewed joint design may have contributed to the cracking by providing additional restraint; with such a design, it can be difficult to properly align the dowel bars.

Based on these results and upon other aspects of the pavement design, the major causes of the transverse cracking is believed to be a combination of thermal curling, shrinkage (frictional restraint), other restraints perhaps induced by the doweled, skewed joints, and traffic loading stresses. There still remains some questions as to why such a large amount of cracking developed, however.

Poor drainability, an erodible lean concrete base, and small diameter dowels have contributed to the erosion of the lean concrete base. This erosion has led to significant faulting and serious loss of support under the outer lane corners, which will eventually lead to corner breaks and additional transverse cracking.

CHAPTER 4 EFFECT OF DESIGN FEATURES ON PAVEMENT PERFORMANCE

1. INTRODUCTION

The experimental projects were originally selected to allow a study of the effect of jointed concrete pavement design features on pavement performance. Of special interest were those factors which could be shown to improve pavement subdrainage and reduce deflection. However, the extensive surveys and data collection allow for an analysis of the performance of the entire concrete pavement system.

Eleven design features are included in this analysis: slab thickness, base type, joint spacing, reinforcement, joint orientation (skewed or perpendicular), transverse joint load transfer, dowel bar coating, longitudinal joint design, joint sealant, shoulders, and subdrainage. The number of sections available for evaluating certain features is limited, so that their direct effect on performance cannot be determined without considerable mechanistic and statistical analyses. Such analyses were performed and are included in volume II.

All of the data presented here, except where noted, is from the outer traffic lane, which represents the heaviest traveled lane.

2. SLAB THICKNESS

Pavement sections having slab thicknesses ranging from 7 to 15 in (178 to 381 mm) were included in the study. Although often confounded with other variables, the following comparisons can be made from the field data.

Minnesota 1 and Minnesota 2

Two experimental sections in Minnesota, Minnesota 1 on I-94 near Rothsay and Minnesota 2 on I-90 near Albert Lea, showed mixed results with respect to slab thickness. For Minnesota 1, the 9-in (229 mm) JRCP nondoweled slabs performed much better than the 8-in (203 mm) JRCP nondoweled slabs in every distress category. However, the doweled 8- and 9-in (203 and 229 mm) JRCP sections did not exhibit a significant difference in performance. It should be noted that the nondoweled pavement sections faulted so severely that they required rehabilitation after 14 years of service. For Minnesota 2, the 8- and 9-in (203 and 229 mm) JPCP slabs displayed nearly identical performance, except that the 9-in (229 mm) slab had more longitudinal cracking. These results are summarized in tables 54 and 55.

Arizona 1

On this experimental project on State Route 360 in Phoenix, 11- and 13-in (279 and 330 mm) nondoweled, concrete sections placed directly on the subgrade are performing better than a nondoweled 9-in (229 mm) concrete pavement on a cement treated base in terms of faulting, longitudinal cracking, and joint spalling. No difference in performance was observed between the 11- and the 13-in (279 and 330 mm) slabs. However, the 9-in (229 mm) pavement was older and had sustained greater traffic loadings. No transverse cracks were observed for any pavement section. These results are summarized in table 56.

Table 54. Outer lane performance data relative to slab thickness for Minnesota 1 (Age = 17 years, ESAL's = 5.5 million).

Slab Thick, in	Dowels	Joint Faulting, in	Transverse Cracks/mi	Longitudinal Cracks, ft/mi	% Joint Spalling	PSR
8	No	0.39*	25	2406	23	3.5
9	No	0.33*	0	968	8	3.7
8	Yes	0.04	32	1464	25	3.4
9	Yes	0.08	47	1744	30	3.3

* Data from 1984 prior to rehabilitation of nondoweled sections.
Longitudinal cracking includes inner and outer lanes.

Table 55. Outer lane performance data relative to slab thickness for Minnesota 2 (Age = 10 years, ESAL's = 2.8 million).

Slab Thick, in	Dowels	Joint Faulting, in	Transverse Cracks/mi	Longitudinal Cracks, ft/mi	% Joint Spalling	PSR
8	Yes	0.06	0	150	9	3.8
9	Yes	0.06	0	746	3	3.9

Longitudinal cracking includes inner and outer lanes.

Table 56. Outer lane performance data relative to slab thickness for Arizona 1.

Slab Thick, in	ESAL's, millions	Age	Joint Faulting, in	Longitudinal Cracks, ft/mi	% Joint Spalling	PSR
9	4.0	15	0.08	233	22	3.4
11	2.8	8	0.03	0	0	3.8
13	3.4	12	0.01	0	0	3.8

Longitudinal cracking includes inner and outer lanes.

California 1

The project on I-5 near Tracy, California included two different slab thicknesses constructed over cement-treated bases. The 8.4-in (213 mm) slabs had significantly more transverse and longitudinal cracking than the 11.4-in (290 mm) slabs. Other distresses were about the same for each section. However, the serviceability was lower for the thicker slab. This information is summarized in table 57.

Ohio 2

On State Route 2, near Vermilion, Ohio, two 15-in (381 mm) JPCP slabs were constructed on grade. They were 13 years old and had carried 3.3 million ESAL's. The slabs showed very little transverse cracking or transverse joint spalling. However, significant longitudinal cracking was observed. Faulting was 0.11 in (2.8 mm) for each of the two sections, which is fairly high. Typically, faulting of 0.13 in (3.3 mm) results in a rough ride for JPCP. Increased slab thickness did not prevent joint faulting for these sections.

Ontario 1

On Highway 3N, near Ruthven, 12-, 8-, and 7-in (305, 203, and 178 mm) nondoweled slabs were built. The 12-in (305 mm) slab on grade displayed no transverse cracking and only a small amount of longitudinal cracking. The 7- and 8-in (178 and 203 mm) slabs placed on lean concrete base courses showed much more longitudinal and transverse cracking. The amounts of spalling, faulting, and PSR were comparable for all of the sections. Table 58 summarizes this information.

North Carolina 2

This 11-in (279 mm) doweled JPCP was built in 1982 on I-85 near Greensboro. While structurally sound, the pavement is displaying an extensive amount of pumping, which is evidently occurring beneath the lean concrete base. The pumping is believed to be caused by the presence of free moisture beneath the lean concrete base; the moisture is trapped there since the pavement cross-section is a bathtub-type design that holds water. The pumping has not resulted in excessive faulting yet because of the large (1.38-in [35 mm] diameter) dowel bars. Thus, while the thick slab appeared to have minimized slab cracking, it could not control the pumping brought about by the poor cross-sectional design of the pavement.

Summary of the Effects of Slab Thickness

Regardless of climatic zone, the thicker slabs reduced the amount of transverse and longitudinal cracking. However, joint spalling, faulting, and pumping did not generally decrease with increased slab thickness. Poor drainage design evidently diminished the effect of the thicker slabs. Thick slabs on grade appeared to perform well in dry climates, although the sections studied were only 8 and 12 years old and had been subjected to only 2.8 and 3.4 million ESAL applications. The thick, slab-on-grade design did not perform as well in wet climates, as the sections exhibited substantial faulting of nondoweled joints after 13 years of service and only 3.3 million ESAL applications.

Table 57. Outer lane performance data relative to slab thickness for California 1 (Age = 16 years, ESAL's = 7.6 million).

Slab Thick, in	Dowels	Joint Faulting, in	Transverse Cracks/mi	Longitudinal Cracks, ft/mi	% Joint Spalling	PSR
8.4	No	0.10	30	940	3	3.0
11.4	No	0.11	0	0	3	2.7

Longitudinal cracking includes inner and outer lanes.

Table 58. Outer lane performance data relative to slab thickness for Ontario 1 (Age = 5 years, ESAL's = 0.84 million).

Slab Thick, in	Dowels	Base Type	Joint Faulting, in	Transverse Cracks/mi	Longitudinal Cracks, ft/mi	PSR
7	No	LCB	0.05	48	345	3.8
8	No	LCB	0.06	27	480	3.8
12	No	None	0.06	0	40	3.8

Longitudinal cracking includes inner and outer lanes.

3. BASE TYPE

The type of base course was found to be a very important factor in the performance of jointed concrete pavements. Distress types that appeared to be affected by the base include joint faulting, transverse cracking, longitudinal cracking, joint spalling from poor drainage and the resultant "D" cracking, and loss of serviceability related to the presence of the above distresses. The base affects the occurrence of these distress types because of the support it provides for the slab, and its large effect on subsurface drainage and erosion potential.

The base types included were dense-graded aggregate base courses (AGG), cement-treated base courses (CTB), asphalt-treated base courses (ATB), lean concrete base courses (LCB), soil-cement base courses (SC), permeable aggregate base courses (PAGG), and permeable asphalt-treated base courses (PATB). In addition, some slabs were constructed directly on the subgrade without a base course.

Detailed design information for the base course was typically not available. It is known that certain factors in the design of the base course influence its performance. This is particularly true for asphalt-treated base courses where factors such as the type and amount of asphalt cement and the type of mix (hot or cold) are expected to play a key role in the performance of the pavement. For cement-treated bases, the cement content is the primary, but certainly not the only, factor of interest. For aggregate base courses, the percent fines and the plasticity index are significant factors. Where possible, the effect of these factors is discussed but, as previously indicated, they were not always available.

One important factor to consider in the analysis of base types is the friction that is produced at the interface between the base and the concrete slab. Stabilized base courses produce a larger amount of friction than aggregate base courses. The larger friction factors often result in slab cracking, because of the restraint to slab movement. This factor, while not directly measured, is discussed where appropriate.

Minnesota 1

This project on I-94 showed a large difference in performance between three different base types: aggregate, asphalt-treated, and cement-treated. The ATB, containing 5 percent of a 120-150 penetration-graded asphalt cement, displayed the most transverse cracking and by far the most longitudinal cracking.

Aggregate bases showed the best performance in terms of low longitudinal cracking and joint spalling. The aggregate base course was a well-drained material which probably enhanced its performance. The aggregate base course and the CTB, containing 5 percent cement, had about the same amount of transverse cracking, although the CTB displayed a significant amount of longitudinal cracking. The cause of the large amount of cracking for the ATB and CTB is probably increased friction produced by those materials at the slab/base interface. Additionally, it is known that the longitudinal joint was formed with a plastic insert which did not completely form the joint; this factor coupled with the large friction factors of the stabilized base courses are believed to have contributed to the longitudinal cracking.

The aggregate base course and the ATB both exhibited about the same amount of faulting. Surprisingly, for both doweled and nondoweled pavements, the CTB exhibited the most faulting. The performance data relative to base type for this project is summarized in table 59.

Minnesota 6

This section on Trunk Highway 15 near Truman is an 8-in (203 mm) JRCP with a PATB. A subbase material with specific gradation requirements was placed beneath the permeable base to serve as a filter material. The pavement section is in excellent condition, displaying no structural deterioration and faulting of only 0.01 in (0.3 mm). However, the section was built in 1983 and has only sustained approximately 0.85 million ESAL applications.

Arizona 1

This project demonstrated that 11- and 13-in (279 and 330 mm) thick slabs placed directly on the subgrade had the same low amount of faulting as 9-in (229 mm) slabs placed on LCB. They also had less faulting than a pavement slab on a CTB base. The reason for the good performance is probably due to the low annual rainfall (8 in [203 mm]), the high load transfer efficiencies (>90 percent), and the relatively mild climate. The performance data for this project is shown in table 60.

California 1

This project on Interstate 5 near Tracy included a direct comparison between a CTB and an LCB with all other factors held constant. The LCB section displayed more than twice as much transverse cracking as the CTB. The increased cracking was due to either reflective cracking from the lean concrete base or increased curling due to a reduced bond between the LCB and the slab. The lean concrete base section had a large amount of base pumping. The CTB section exhibited more longitudinal cracking and more faulting than the lean concrete base. The LCB displayed a lower PSR than the CTB, probably due to the extensive transverse cracking. These results are tabulated in table 61.

California 2

Two experimental sections on Interstate 210 in Los Angeles compared a conventional CTB and a two-layered base system consisting of a dense AC layer over a permeable cement-treated base. Both pavements exhibited a large amount of faulting, averaging 0.11 in (2.8 mm). The dense AC/permeable cement-treated base section was rougher than the standard cement-treated base section. The CTB had a large amount of transverse cracking, but the dense AC/permeable cement-treated base had no cracking. This may be due to the influence of the AC separation layer placed in that section. However, the presence of the AC layer above the permeable base layer probably altered the drainability and erosion characteristics of the permeable base, essentially rendering it ineffective. The performance results for these sections are illustrated in table 62.

Table 59. Outer lane performance data relative to base type for Minnesota 1 (Age = 17 years, ESAL's = 5.5 million).

Base Type	Dowels	Joint Faulting, in	Transverse Cracks/mi	Longitudinal Cracks, ft/mi	% Joint Spalling
AGG	No	0.31*	17	150	6
ATB	No	0.34*	21	2807	20
CTB	No	0.44*	0	2160	20
AGG	Yes	0.06	35	0	15
ATB	Yes	0.03	51	4302	50
CTB	Yes	0.10	44	510	18

* Data from 1984 prior to rehabilitation of nondoweled sections.
Longitudinal cracking includes inner and outer lanes.

Table 60. Outer lane performance data relative to base type for Arizona 1.

Base Type	Slab Thick, in	ESAL's, millions	Age	Joint Faulting, in	Longitudinal Cracks, ft/mi	% Joint Spalling	PSR
CTB	9	4.0	15	0.08	233	22	3.4
NONE	11-13	2.9	9	0.02	0	0	3.7
LCB	9	1.8	5	0.02	0	0	3.6

Longitudinal cracking includes inner and outer lanes.

Table 61. Outer lane performance data relative to base type for California 1 (Age = 16 years, ESAL's = 7.6 million).

Base Type	Dowels	Joint Faulting, in	Transverse Cracks/mi	Longitudinal Cracks, ft/mi	Pumping	PSR
CTB	No	0.10	30	940	None	3.0
LCB	No	0.06	75	230	High	2.7

Longitudinal cracking includes inner and outer lanes.

Table 62. Outer lane performance data relative to base type for California 2 (Age = 7 years, ESAL's = 4.4 million).

Base Type	Dowels	Joint Faulting, in	Transverse Cracks/mi	Longitudinal Cracks, ft/mi	Pumping	PSR
AC/PCTB	No	0.11	0	0	None	3.8
CTB	No	0.11	290	0	None	4.1

Longitudinal cracking includes inner and outer lanes.

California 6

Section CA 6 on Route 14, located in the greater Los Angeles area, was a nondoweled 9-in (229 mm) JPCP with an LCB. Built in 1980, this section has had about the same amount of traffic (4.4 million ESAL applications) as the California 2 sections; however, it displayed a slightly larger amount of faulting (0.15 in [3.8 mm]). The LCB apparently was not effective in reducing faulting. Deflection measurements indicated no voids beneath the slab, so it is believed that pumping was occurring beneath the LCB.

Michigan 1

The sections on U.S. 10 near Clare, Michigan included a PATB and an aggregate base course with far better performance characteristics than pavement sections constructed over a dense-graded ATB. The PATB contained between 2 and 3 percent of an 85-100 penetration-graded asphalt cement and did not utilize a filter layer beneath it as the existing sand subbase closely approximated the gradation of the proposed filter material. This section had very little faulting or joint spalling.

The sections constructed over ATB (containing between 5 and 9 percent of a 250-300 penetration-graded asphalt cement) were the poorest performers, since it was a bathtub design that held water and contributed to a large amount of erosion and spalling from "D" cracking. This section also had a lot of cracking and faulting and a low PSR. The aggregate base course sections had a fair amount of drainability that resulted in far better performance than the sections constructed over the dense-graded ATB. Table 63 includes the performance results for this project. It should be noted that the pavement sections are under relatively light traffic (less than 1 million ESAL applications after 12 years of service).

Michigan 3

This 10-in (254 mm) JRCP located on I-94 has a permeable aggregate base course placed on an existing sand subbase. Built in 1986, this pavement section is performing very well after 1 year of service and 2.8 million ESAL applications. No structural deterioration was observed and faulting was only 0.02 in (0.5 mm).

Michigan 5

Built in 1984, this 10-in (254 mm) JRCP is also located on I-94 and contains a permeable aggregate base course placed on an existing sand subbase. This pavement is beginning to show serious signs of deterioration after only 3 years and 3.1 million ESAL applications. The major problem is the amount of deteriorated transverse cracks that have occurred (144/mile). This is believed to be the result of a combination of locked joints (due to the use of an ineffective bondbreaker on the dowel), offset spacing of the shoulder and pavement joints, and small top size of the coarse aggregate (which could not provide adequate load transfer and would allow the crack to break down rapidly under traffic). In addition, testing performed by MDOT has indicated that the sand subbase has infiltrated into the permeable base course. This will reduce the permeability of the base and not allow for the rapid removal of free moisture, which could contribute to the deterioration of the transverse cracks. Finally, there was some concern that the gradation

Table 63. Outer lane performance data relative to base type for Michigan 1 (Age = 12 years, ESAL's = 0.9 million).

Type	Base Type	Dowels	Joint Faulting, in	Transverse Cracks/mi	Longitudinal Cracks, ft/mi	% Joint Spalling	PSR
JPCP	PATB	No	0.03	0	0	9	3.9
JPCP	ATB	No	0.18	24	0	69	2.9
JPCP	AGG	Yes	0.04	0	0	12	3.6
JRCP	AGG	Yes	0.07	2	0	0	3.4

Longitudinal cracking includes inner and outer lanes.
Dowel bars were epoxy-coated.

and percent crushing specifications for the permeable base may not have been proper and could have resulted in an unstable base course.

New York 1

Experimental sections on Route 23 included asphalt-treated and aggregate base courses. This experiment showed that for JPCP, the sections with ATB performed better than those with aggregate base courses in terms of transverse and longitudinal cracking and joint spalling. However, for JRCP, the ATB sections exhibited more faulting and spalling than the aggregate base courses. The aggregate base course had a plasticity index of 1.7 and contained 7 percent fines. The ATB contained 2.5 to 4.0 percent asphalt cement. The performance data for this section is summarized in table 64.

Ohio 1

This experimental project on U.S. 23 near Chillicothe had asphalt-treated and aggregate base courses. Sections with asphalt-treated bases performed better than the the sections with aggregate bases. The aggregate bases had more faulting and much more cracking, and also displayed many more voids beneath the slab corners. The sections with the asphalt-treated bases did have more spalling on the under side of the joint than did the aggregate base sections, probably due to the lower drainability of the asphalt-treated base. Unfortunately, no information was available concerning the design of either the asphalt-treated or aggregate base courses. Table 65 provides the performance information for this project.

Ontario 1

These nondoweled JPCP sections on Highway 3N near Ruthven compared a PATB, an LCB, and a thick slab placed directly on subgrade. The section with the LCB displayed far more longitudinal cracking and more transverse cracking than the other two base types. This is probably due to either reflective cracking from the base or to the stiffness of the base when the slab is undergoing curling effects.

The section with the PATB had very little distress; however, it did display a lot of pumping. This may be attributed to the permeable base being constructed directly on grade without the use of a filter material and that the longitudinal edge drains intended to drain the permeable base were placed in a dense-graded aggregate material 6 in (152 mm) from the permeable base. Thus, fines could easily infiltrate the permeable base course where excess water could not readily be removed.

The 12-in (305 mm) slab placed directly on grade performed nearly as well as the permeable base section. It did exhibit some longitudinal cracking, but this may be due to the thick slab being sawed only 25 percent of its depth. The performance data for these sections is shown in table 66.

Pennsylvania 1

The pavement sections on Route 66 near Kittanning exhibited no significant differences in performance between five different base sections: CTB, PATB, permeable aggregate base of relatively uniform gradation, a more well-graded, high permeability aggregate base, and a conventional

Table 64. Outer lane performance data relative to base type for New York 1 (Age = 22 years, ESAL's = 2.0 million).

Type	Base Type	Joint Faulting, in	Transverse Cracks/mi	Longitudinal Cracks, ft/mi	% Joint Spalling	PSR
JPCP	ATB	0.02	0	0	6	4.0
JPCP	AGG	0.03	35	264	13	3.9
JRCP	ATB	0.14	0	0	73	3.6
JRCP	AGG	0.09	0	0	9	3.4

Longitudinal cracking includes inner and outer lanes.
Load transfer is provided by ACME devices.

Table 65. Outer lane performance data relative to base type for Ohio 1 (Age = 14 years, ESAL's = 3.4 million).

Joint Spacing, ft	Base Type	Dowels	Joint Faulting, in	Transverse Cracks/mi	Longitudinal Cracks, ft/mi	% Joint Spalling	PSR
21	ATB	Yes	0.06	0	0	13	4.2
21	AGG	Yes	0.10	0	0	0	4.2
40	ATB	Yes	0.07	29	0	0	4.1
40	AGG	Yes	0.14	53	0	0	4.2

Longitudinal cracking includes inner and outer lanes.
Results summarized for standard (painted/greased) dowels only.

Table 66. Average performance data relative to base type for Ontario 1 (Age = 5 years, ESAL's = 0.84 million).

Slab Thick, in	Base Type	Dowels	Joint Faulting, in	Transverse Cracks/mi	Longitudinal Cracks, ft/mi	Pumping	PSR
8	PATB*	No	0.06	0	0	Medium	3.8
8	LCB	No	0.07	28	480	Low	3.8
12	None	No	0.06	0	40	Low	3.8

* PATB placed directly on subgrade; no filter layer used.
Longitudinal cracking includes inner and outer lanes.

dense-graded aggregate base. A conventional dense-graded aggregate subbase was placed beneath each base type.

Essentially no deterioration had taken place and the PSR of all of the sections was very high. This is most likely due to the very low traffic level which, after 7 years of service, was only 0.27 million ESAL's. All five sections have longitudinal underdrains and are located on a significant grade, which combined to provide more than adequate drainage of free water from each of the base layers.

New Jersey 3

This project on Interstate 676 was 8 years old and had carried 4.2 million ESAL's at the time of survey. The project included two permeable bases: aggregate and asphalt-treated. Each of these performed very well and exhibited only minimal faulting. No cracking or pumping occurred. The PATB did exhibit more joint spalling than the permeable aggregate sections, but this is believed to be construction-related since the joints are all expansion joints spaced at 78.5-ft (24 m) intervals. The wide expansion joints may have caused the roughness that resulted in a PSR of 3.5. A filter fabric was placed beneath both the permeable base courses.

North Carolina 1

This project on I-95 had four dense-graded base types: cement-treated, asphalt-treated, soil cement, and aggregate. The sections with the dense-graded asphalt-treated base course displayed superior performance in every category over the sections constructed on the other base courses. The sections constructed on cement-treated, soil cement, and aggregate base courses all had a substantial amount of faulting and lower PSR. The sections constructed on the soil-cement base course displayed a large amount of longitudinal cracking. This information is shown in table 67.

North Carolina 2

This 11-in (279 mm) JPCP on I-85 contained a lean concrete base course. The pavement was in structurally good condition, but there was a substantial amount of pumping observed at the joints in the mainline pavement and in the shoulder. Inspection of the reddish-brown pumped material shows that it is the curing compound placed on the lean concrete base. Therefore, the pumping is believed to be occurring from moisture between the slab and the lean concrete base. The cross-sectional design of the pavement section was a bathtub design that did not allow for the removal of excess moisture.

Florida 3

Lean concrete served as the base course for this 9-in (229 mm) JPCP on I-75 near Tampa. This pavement section has exhibited extensive cracking and pumping. A major portion of the cracking has been attributed to the use of relatively long (22 and 23 ft [6.7 and 7.0 m]) slab lengths with skewed doweled joints. The pumping is believed due to a poor cross-sectional design with no provision for drainage. Water was trapped between the slab and lean concrete base which resulted in the erosion and pumping of the base material. The pumping action has also caused loss of support at 73 percent of the slab corners, resulting in additional slab cracking.

Table 67. Outer lane performance data relative to base type for North Carolina 1 (Age = 20 years, ESAL's = 9.1 million).

Base Type	Dowels	Joint Faulting, in	Transverse Cracks/mi	Longitudinal Cracks, ft/mi	PSR
AGG	Yes	0.13	0	0	3.4
AGG	No	0.17	32	0	3.5
ATB	No	0.05	0	0	3.8
CTB	No	0.16	0	179	3.2
SC	No	0.13	5	3068	3.6
SC	Yes	0.16	10	1451	3.5

Longitudinal cracking includes inner and outer lanes.
Joint spacing = 30 ft (9.1 m), dowel diameter = 1 in (25 mm).

Summary of the Effects of Base Type

The type of base course material usually had a large impact on the distresses observed on the pavement surface, particularly for heavily trafficked pavements. Key variables appear to be base/slab interface friction, stiffness, erodibility, and permeability. These had a large effect on transverse and longitudinal cracking, joint spalling, and faulting. Table 68 provides a subjective ranking of the relative performance of the bases at each project site. The following summarizes the overall results for each base type, starting with the poorest performing base:

VERY POOR: Cement-treated or soil-cement dense-graded bases. Practically every site showed very poor performance in terms of pumping, faulting, and cracking.

POOR: Lean concrete bases. This base was associated with considerable pumping, faulting and transverse and longitudinal cracking. This is particularly true when located in a "bathtub" situation with poor subdrainage.

FAIR-POOR: Asphalt-treated, dense-graded bases. Most sections performed poorly, but there were some that showed good performance. Severe longitudinal cracking and faulting existed at several locations. A bathtub section at Clare, for example, resulted in severe faulting and accelerated "D" cracking at joints.

FAIR: No base. Only three sites existed, one showing good performance, one showing fair performance, and one showing poor performance. However, it should be noted that each of these sections were constructed with thicker slabs (in the range of 11 in [279 mm] to 15 in [381 mm]) and had been subjected to relatively low traffic levels. These two factors skew the performance results of these sections.

GOOD: Aggregate dense-graded bases. Performance ranged from poor to very good, but was generally fair to very good. The more open the gradation, the better the performance.

VERY GOOD: Permeable bases. With one exception (MI 5), the sections with permeable bases displayed virtually no faulting, cracking, or joint spalling. Pumping was observed at one site (ONT 1-2), but this is believed to be caused by the infiltration of the subgrade into the permeable base and the improper location of edge drains to remove the excess water. While the permeable base sections generally performed well, it should be noted that they are not very old and have not been subjected to heavy traffic loadings.

Pavement sections constructed with permeable base layers generally require the use of a filter layer beneath the base. The purpose of such a layer is to arrest the migration of fines into the permeable base course from below. The absence of a filter layer can result in clogging of the permeable base and subsequently pumping and faulting. This phenomenon is believed to have been partially responsible for the pumping observed on the section in Ontario.

Table 68. Overall relative summary of the performance of base types.

PROJECT ID	CTB/SC	ATB	BASE TYPE LCB	NONE	AGG	PERM
MN 1	VP	VP	---	---	G	---
MN 6	---	---	---	---	---	VG
MI 1	---	VP	---	---	VG	VG
MI 3	---	---	---	---	---	VG
MI 5	---	---	---	---	---	VP
NY 1	---	G	---	---	F	---
OH 1	---	G	---	---	F	---
OH 2	---	---	---	P	---	---
ONT 1	---	---	VP	F	---	F
ONT 2	VG	---	---	---	---	---
PA 1	(VG)	---	---	---	(VG)	(VG)
NJ 3	---	---	---	---	---	VG
NC 1	VP	VG	---	---	P	---
NC 2	---	---	P-F	---	---	---
FL 3	---	---	VP	---	---	---
CA 1	VP	---	VP	---	---	---
CA 2	VP	VP	---	---	---	---
CA 6	---	---	VP	---	---	---
AZ 1	P	---	VG	VG	---	---

KEY: CTB: Cement-treated base
ATB: Asphalt-treated base
LCB: Lean concrete base
PERM: Permeable base
SC: Soil-cement
AGG: Dense-graded aggregate base
NONE: No base course, slab on grade

RATINGS:
VG: Very good base performance (no distress)
G: Good base performance
F: Fair base performance
P: Poor base performance
VP: Very poor base performance (extensive distress)

( ): Indication that pavement was subjected to less than one-half million ESAL's in the outer traffic lane.

For the sections constructed with permeable bases, the subbase layer beneath the base was evaluated to determine if it met filter requirements. This evaluation calls for an analysis of the gradation of the permeable base course, the subbase, and the subgrade. The projects incorporating permeable bases and the gradation data necessary for the filter evaluation are shown in table 69. The table is fairly complete with the exception of the Minnesota 6 base course gradation and the Ontario 1-2 subgrade gradation. In addition, this table does not include the New Jersey 3 sections since these both contained a filter fabric that is expected to prevent the infiltration of fines into the permeable base.

The base, subbase, and subgrade gradations were then used to evaluate the necessary filter criteria. The results of this evaluation are presented in table 70. This evaluation consisted of two parts: an evaluation of the base and subbase to determine if the subbase would contaminate the base course, and an evaluation of the subbase and subgrade to determine if the subgrade would contaminate the subbase.

An examination of table 70 shows that, for the base/subbase evaluation, all of the existing subbase materials met the necessary filter requirements, with one exception. The one exception was MI 1-4a, which is a section that is in good condition and is not exhibiting any pumping, but is displaying large corner deflections and substantial loss of support beneath slab corners.

Very few of the projects met the filter criteria for the subbase and subgrade evaluation. This is an indication that fines from the subgrade are likely to migrate into the subbase and contaminate that layer.

4. JOINT SPACING

Existing design procedures provide only very limited guidance on the selection of joint spacing. This is true for both JRCP and JPCP. In theory, longer jointed pavements will experience more transverse cracking than shorter jointed pavements. Furthermore, it is expected that joint deterioration may be greater in long-jointed pavements, as the compressive forces that build up in the slabs from incompressibles increase.

There was wide variation of joint spacings included in the study. Also, joint spacing consisted of both uniform and variable, or random, lengths. A major confounding factor in the analysis of joint spacing was pavement type (JRCP vs. JPCP).

Minnesota 1

The sections on I-94 near Rothsay all were 27-ft (8.2 m) JRCP. However, an adjacent section of 40-ft (12.2 m) JRCP of similar age and traffic (MN 5) was also tested and included in the study. Results in table 71 show that faulting and PSR are about the same while the number of deteriorated transverse cracks and spalled joints is much higher for the section with 40 ft (12.2 m) joint spacing. However, the amount of reinforcement (0.085 percent for MN 1 and 0.04 percent for MN 5) present in both JRCP sections was believed to be inadequate for the large temperature extremes encountered in this climate. The steel was unable to hold the cracks tight, which resulted in their further breakdown and deterioration.

Table 69. Gradation information for permeable bases required for filter criteria evaluation.

	MN 6	MI 1-4a	MI 3	MI 5	ONT 1-2	PA 1-2	PA 1-3	PA 1-4
Base	PATB	PATB	PAGG	PAGG	PATB	PATB	PAGG	PAGG
D_5	N/A	4.3	2.2	0.60	2.0	2.4	5.1	2.3
D_{10}	N/A	7.1	2.5	2.7	4.3	3.7	5.7	5.1
D_{15}	N/A	9.9	3.2	4.4	5.6	4.9	6.2	6.1
D_{50}	N/A	15	8.1	9.8	15	10	12	19
D_{60}	N/A	17	11	11	18	12	15	29
Subbase	AGG	SAND	SAND	SAND	NONE	AGG	AGG	AGG
D_5	0.089	0.096	0.25	0.10		0.095	0.095	0.095
D_{10}	0.15	0.15	0.32	0.16		0.21	0.21	0.21
D_{15}	0.27	0.25	0.37	0.25		0.28	0.28	0.28
D_{50}	0.79	0.46	1.7	0.60		8.5	8.5	8.5
D_{60}	1.3	0.79	2.9	0.84		12	12	12
D_{85}	3.8	6.2	19	4.4		22	22	22
Subgrade	A-2-4	A-2-4	A-2-4	A-2-4	A-7-6	A-4	A-4	A-2-4
D_{15}	0.25	0.21	N/A	N/A	N/A	N/A	N/A	N/A
D_{50}	1.1	0.41	0.082	0.14	N/A	N/A	N/A	0.88
D_{85}	3.6	2.5	1.5	1.9	N/A	0.074	0.23	11

Table 70. Evaluation of filter criteria.

	Base/Subbase Filter Requirements Met					Subbase/Subgrade Filter Requirements Met				
Project	C1	C2	C3	C4	C5	C1	C2	C3	C4	C5
MN 6	N/A	N/A	N/A	N/A	N/A	Y	N	Y	Y	Y
MI 1-4a	Y	Y	N	Y	Y	Y	N	Y	Y	Y
MI 3	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y
MI 5	Y	Y	Y	Y	Y	Y	N	Y	Y	Y
ONT 1-2	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
PA 1-2	Y	Y	Y	Y	Y	Y	N	N	Y	N
PA 1-3	Y	Y	Y	Y	Y	Y	N	N	Y	N
PA 1-4	Y	Y	Y	Y	Y	Y	N	Y	Y	N

* <u>Criteria 1 (C1)</u>: $(D_{15})_{base/subbase} \leq 5 * (D_{85})_{subbase/soil}$

* <u>Criteria 2 (C2)</u>: $(D_{15})_{base/subbase} \geq 5 * (D_{15})_{subbase/soil}$

* <u>Criteria 3 (C3)</u>: $(D_{50})_{base/subbase} \leq 25 * (D_{50})_{subbase/soil}$

* <u>Criteria 4 (C4)</u>: $(D_{5})_{base/subbase} \geq 0.074$ mm

* <u>Criteria 5 (C5)</u>: $(CU)_{base/subbase} = (D_{60})_{base/subbase} / (D_{10})_{base/subbase} \leq 20$

* Criteria from *"Highway Subdrainage Design,"* By L. K. Moulton, FHWA-TS-80-224, August 1980.

Table 71. Outer lane performance data relative to joint spacing for Minnesota 1 (Age = 17 years, ESAL's = 5.5 million).

Joint Spacing, ft	Dowels	Reinf, %	Joint Faulting, in	Transverse Cracks/mi	% Joint Spalling	PSR
27	Yes	0.08	0.10	23	14	3.3
40	Yes	0.04	0.09	53	36	3.3

Sections are 9-in (229 mm) JRCP with aggregate base.

Table 72. Outer lane performance data relative to joint spacing for Minnesota 2 (Age = 10 years, ESAL's = 2.8 million).

Joint Spacing, ft	Dowels	Joint Faulting, in	Transverse Cracks/mi	% Joint Spalling	Pumping	PSR
13-16-14-19	Yes	0.06	0	3	None	3.8
27	Yes	0.05	3	3	None	4.0

Sections are 9-in (229 mm) slabs.

Minnesota 2

These sections on I-90 near Albert Lea provide a comparison between long-jointed JRCP and short-jointed JPCP. Results in table 72 show essentially equal performance. However, the shorter, random joint spacing of 13-16-14-19 ft (4.0-4.9-4.3-5.8 m) resulted in a slightly lower PSR than the longer uniform joint spacing of 27 ft (8.2 m), probably due to increased joint faulting. Minnesota currently uses a joint spacing of 13-16-14-17 ft (4.0-4.9-4.3-5.2 m).

California 1

This project on I-5 allows a comparison of a pavement with an extremely short, random joint spacing of 5-8-11-7 ft (1.5-2.4-3.4-2.1 m) and California's former standard joint spacing of 13-19-18-12 ft (4.0-5.8-5.5-3.7 m). The pavements with the shorter joint spacing had less faulting and less transverse cracking than the pavements with conventional spacing. However, a 1989 reinspection revealed that the section with the extremely short joint spacing is beginning to develop extensive slab cracking.

The 7.75-ft (2.4 m) average section had less joint movement, which helped maintain good aggregate interlock load transfer at the nondoweled joints. Most of the transverse cracks that occurred on the 15-ft (4.6 m) average joint spacing section were in the longer slab segments (i.e., in the 18- and 19-ft [5.5 and 5.8 m] slabs). This comparison is illustrated in table 73. The data shown in this table was measured in the northbound lanes. The southbound lanes recently showed considerably more pumping and cracking, possibly due to a poorer drainage environment. Due to the cracking in the longer slabs, California has modified their joint spacing to 12-15-13-14 ft (3.7-4.6-4.0-4.3 m) in an effort to avoid large differences between maximum and minimum slab lengths.

Michigan 1

The sections on U.S. 10 included long-jointed JRCP and short-jointed JPCP placed on a granular base. Long-jointed pavements, with a joint spacing of 71 ft (21.6 m) performed similarly to the short, random-jointed pavements which had a joint spacing pattern of 13-17-16-12 ft (4.0-5.2-4.9-3.7 m). Faulting was about the same for each section. The long-jointed sections contained preformed joint seals and 0.15 percent reinforcement, a combination that resulted in no spalling and very little deteriorated transverse cracking. The shorter jointed sections had more joint spalling than the longer sections, while the longer jointed sections had a lower PSR. Both sections included epoxy-coated dowel bars which apparently reduced joint lockup and subsequent spalling. The performance information on these sections is summarized in table 74.

New York 1

These sections on Route 23 had 20-ft (6.1 m) JPCP and 60-ft (18.3 m) JRCP. The long-jointed sections did not display any transverse crack deterioration, perhaps because they contained 0.2 percent reinforcing steel. However, the JRCP on ATB did have much more joint spalling and much higher joint faulting than the short-jointed sections. The performance data results for this project are shown in table 75. Load transfer for these sections was provided by ACME devices.

Table 73. Outer lane performance data relative to joint spacing for California 1 (Age = 16 years, ESAL's = 7.6 million).

Joint Spacing, ft	Dowels	Joint Faulting, in	Transverse Cracks/mi	% Joint Spalling	Pumping	PSR
5-8-11-7	No	0.06	5	2	Medium	2.9
13-19-18-12	No	0.10	30	3	None	3.0

Sections consist of 8.4-in (213 mm) PCC slabs over CTB.

Table 74. Outer lane performance data relative to joint spacing for Michigan 1 (Age = 12 years, ESAL's = 0.9 million).

Joint Spacing, ft	Dowels	Joint Faulting, in	Transverse Cracks/mi	% Joint Spalling	Pumping	PSR
13-17-16-12	Yes	0.04	0	11	None	3.6
71	Yes	0.06	2	0	None	3.2

All cracks counted for JPCP, but only medium-high severities for JRCP. Dowel bars are epoxy-coated.

Table 75. Outer lane performance data relative to joint spacing for New York 1 (Age = 22 years, ESAL's = 2.0 million).

Joint Spacing, ft	Base Type	Dowels	Joint Faulting, in	Transverse Cracks/mi	% Joint Spalling	PSR
20	ATB	Yes	0.02	0	6	4.0
60	ATB	Yes	0.14	0	73	3.6
20	AGG	Yes	0.03	35	13	3.9
60	AGG	Yes	0.09	0	9	3.4

Load transfer devices are ACME.

Table 76. Outer lane performance data relative to joint spacing for New York 2 (Age = 12 years, ESAL's = 1.43 million).

Joint Spacing, ft	Dowels	Joint Faulting, in	Transverse Cracks/mi	% Joint Spalling	PSR
20	Yes	0.01	30	0	4.1
27	Yes	0.01	26	0	4.1
63	Yes	0.02	0	0	4.0

Load transfer devices are epoxy-coated I-Beams.

New York 2

Interstate 88 near Otego includes a range of joint spacings from 20-ft (6.1 m) to 63-ft (19.2 m). As illustrated in table 76, the JRCP section with 63-ft (19.2 m) joint spacing exhibited excellent performance. It did not have any deteriorated cracks, although it did have some low severity transverse cracks that were held tight by the large amount of reinforcing steel (0.2 percent). The 20- and 27-ft (6.1 and 8.2 m) JPCP showed some transverse cracking. The faulting was low for all of the sections and the PSR was very high and about the same for all of the sections. Load transfer for these sections is provided by epoxy-coated I-beams.

Ohio 1

U.S. 23 near Chillicothe had JRCP sections with 21-ft (6.4 m) and 40-ft (12.2 m) joint spacing. The short-jointed sections consistently performed better than the long-jointed sections. The 40-ft (12.2 m) slabs displayed higher faulting and much more deteriorated transverse cracking. However, the overall PSR was about the same. The performance data for this project is shown in table 77. The sections presented included 1.25-in (32 mm) noncoated dowel bars and contained the same amount of reinforcement, 0.09 percent. For the section with longer joint spacing, this is a fairly low amount.

New Jersey 2

This 10-in (229 mm) doweled JRCP on Route 130 near Yardville was built in 1951 and is exhibiting only 24 deteriorated transverse cracks per mile after approximately 35 million ESAL applications. This section employs a joint spacing of 78.5-ft (23.9 m) in which every joint is an expansion joint. It also contains stainless steel clad dowel bars and 0.14 percent reinforcement. These factors have apparently combined to provide excellent pavement performance (PSR = 3.8).

North Carolina 1

These sections on I-95 near Rocky Mount had 30-ft (9.1 m) doweled JPCP and 60-ft (18.3 m) JRCP, but the results are confounded by slab thickness. The long-jointed slabs displayed no deteriorated transverse cracking, but a substantial amount of longitudinal cracking. Short jointed sections had no cracking at all. Faulting was observed to be higher for the longer slabs.

Florida 3

This 9-in (229 mm) doweled JPCP on I-75 near Tampa employed a joint spacing of 16-17-23-22 ft (4.9-5.2-7.0-6.7 m) and is exhibiting 310 transverse cracks per mile in the outer lane. It was observed that 100 percent of the 22-ft (6.7 m) slabs and 92 percent of the 23-ft (7.0 m) slabs are cracked, and that the joints exhibited only 19 percent load transfer.

Summary of the Effects of Joint Spacing

Joint spacing was observed to have a large effect on the performance of both JRCP and JPCP for most of the projects. For JPCP, the Tracy sections showed that reducing joint spacing from an average of 15-ft (4.6 m) to an

Table 77. Outer lane performance data relative to joint spacing for Ohio 1 (Age = 14 years, ESAL's = 3.4 million).

Joint Spacing, ft	Base Type	Dowels	Joint Faulting, in	Transverse Cracks/mi	% Joint Spalling	PSR
21	ATB	Yes	0.06	0	13	4.2
40	ATB	Yes	0.07	29	0	4.1
21	AGG	Yes	0.10	0	0	4.2
40	AGG	Yes	0.14	53	0	4.2

Results summarized for standard (painted/greased) dowels only.
All pavements are 9-in (229 mm) JRCP.

average of 7.75 ft (2.4 m) reduced faulting and transverse cracking significantly. This effect was observed on many of the random joint-spaced JPCP sections where the 18- and 19-ft (5.5 and 5.8 m) slabs always exhibited cracking and the 12- and 13-ft (3.7 and 4.0 m) slabs rarely displayed cracking. However, from a project inspection in 1989, the section with extremely short joint spacing has begun to display longitudinal cracking and interior corner breaks. This may be an indication that too short of slab lengths can cause problems as well.

For JRCP, the shorter joint spacings (e.g. 20- to 27-ft [6.1 and 8.2 m]) usually provided better performance in terms of fewer deteriorated transverse cracks, less joint faulting, and a reduced percent of joints spalled. However, several long-jointed reinforced sections in Michigan, New York, and North Carolina displayed surprisingly good performance in terms of minimal faulting, little or no transverse cracking, and little joint deterioration. Some of these sections had high reinforcement content and good joint seals, which probably contributed to their performance.

An analysis was conducted for the JPCP sections to further determine the effect of specific design factors on transverse slab cracking. Table 78 provides a summary of cracking by joint length for the sections with random joint spacing. While other factors such as base type, climate, and traffic levels come into play, the majority of the transverse cracking was observed at joint spacings greater than 18 ft (5.5 m).

In an effort to consider the effect of the foundation stiffness on transverse cracking, the radius of relative stiffness, l, was calculated for each JPCP section. The radius of relative stiffness is defined by:

$$l = [(Eh^3)/(12k(1-u^2))]^{0.25} \quad (1)$$

where:
E = concrete modulus of elasticity, psi
h = slab thickness, in
k = static effective k-value, pci (determined by dividing dynamic k by 2)
u = Poisson's ratio

A ratio of slab length (L) to the radius of relative stiffness (l) was determined for each section and is provided in table 79. An examination of this table indicates that large amounts of transverse cracking occurs when the L/l ratio increases.

The data from table 79 was broken out to consider other factors, most notably base type. Data from the California 1 sections near Tracy is given in table 80 to show cracking for different joint spacings, base types and slab thicknesses. All other factors are constant. A plot of this data for all California 1 sections is shown in figure 34. At high L/l ratios, more transverse cracking is noted.

A similar analysis was conducted for the different base types. Results for cement-treated and lean concrete bases from all sections are shown in figure 35. Even though traffic loadings varied widely, there exists a general relationship between increased L/l and percentage of slabs cracked. L/l ratios of greater than about 5.0 had a rapidly increasing probability of developing transverse cracks.

Table 78. Summary of transverse cracking by slab length for sections with random joint spacing (data for outer lanes only).

Project Section ID	Number Of Slabs	PERCENT SLABS CRACKED BY SLAB LENGTH															TOTAL % SLABS CRACKED
		5 ft	7 ft	8 ft	11 ft	12 ft	13 ft	14 ft	15 ft	16 ft	17 ft	18 ft	19 ft	22 ft	23 ft	25 ft	
AZ 1-1	72						0		0		0						0
AZ 1-2	72						0		0		0						0
AZ 1-4	72						0		0		0						0
AZ 1-5	71						0		0		0						0
AZ 1-6	71						0		0		0						0
AZ 1-7	71						0		0		0						0
AZ 2	70						0		0		0						0
CA 1-1	136	0	0	0	3												1
CA 1-3	68					0	0					6	29				9
CA 1-5	68					0	0					0	0				0
CA 1-7	68					0	12					52	24				22
CA 1-9	68					12	6					71	65				39
CA 2-2	70					0	0					0	0				0
CA 2-3	59					7	53					100	100				66
CA 3-1	60					13	0					13	7				8
CA 3-2	59					0	0					7	0				2
CA 3-5	61					43	40					7	13				27
CA 6	68					0	0	0	12								3
CA 7	69					11	29					47	47				34
CA 8	77					0	0	0	0								0
FL 2	69					0	0					0	0				0
FL 3	54									77	79			92	100		87
MI 1-4a	65					0	0					0	0				0
MI 1-7a	33					0	0			0	0						0
MI 1-7b	34					0	0			0	0						0
MI 1-10a	33					0	0					0	0				0
MI 1-10b	39					0	0					10	10				6
MI 1-25	23					0	0					17	17				8
MN 2-1	68						0	0		0			0				0
MN 2-2	69						0	0		0			0				0
MN 4a	83						0	0		0			0				0
MN 4b	83						0	0		0			0				0
NC 2	50											0	0		8	8	4
ONT 1-1a	69					0	0					0	0				0
ONT 1-1b	69					0	0					0	0				0
ONT 1-2a	68					0	0					0	0				0
ONT 1-2b	68					0	0					0	0				0
ONT 1-3a	68					6	6					18	6				10
ONT 1-3b	68					6	0					18	6				8
ONT 1-4a	68					18	12					18	12				14
ONT 1-4b	68					12	12					18	18				14
ONT 2	68					0	0					0	6				3
AVERAGE		0.0	0.0	0.0	3.0	4.6	4.4	0.0	1.3	11.0	7.9	17.7	12.4	92.0	54.0	8.0	8.7

Table 79. Ratio of slab length to the radius of relative stiffness (L/l).

Section ID	Slab Thick, in	Static k, pci	Slab E, psi	Age, yrs	Cum. ESAL's	Base Type	Slab Length, L ft	Percent Slabs Cracked	Rad. of Rel. Stiff., l in	L/l
AZ 1-1	9.0	273	3140000	15	3968618	CTB	13	0	29.08	5.4
							15	0		6.2
							17	0		7.0
AZ 1-2	13.0	246	3440000	12	3414954	NONE	13	0	40.23	3.9
							15	0		4.5
							17	0		5.1
AZ 1-4	13.0	172	3490000	8	2387085	NONE	13	0	44.15	3.5
							15	0		4.1
							17	0		4.6
AZ 1-5	11.0	219.5	3290000	8	2801246	NONE	13	0	36.11	4.3
							15	0		5.0
							17	0		5.6
AZ 1-6	9.0	310.5	3090000	5	2012173	LCB	13	0	28.04	5.6
							15	0		6.4
							17	0		7.3
AZ 1-7	9.0	292	3690000	5	1522951	LCB	13	0	29.77	5.2
							15	0		6.0
							17	0		6.9
AZ 2	10.0	87	5560000	4	1622817	LCB	13	0	48.31	3.2
							15	0		3.7
							17	0		4.2
CA 1-1	8.4	116	6610000	16	7621830	CTB	5	0	41.19	1.5
							7	0		2.0
							8	0		2.3
							11	3		3.2
CA 1-3	8.4	174.5	5240000	16	7621830	CTB	12	0	35.10	4.1
							13	0		4.4
							18	6		6.2
							19	29		6.5
CA 1-5	11.4	167.5	5280000	16	7621830	CTB	12	0	44.67	3.2
							13	0		3.5
							18	0		4.8
							19	0		5.1
CA 1-7	8.4	216.5	6480000	16	7621830	LCB	12	0	35.07	4.1
							13	12		4.4
							18	52		6.2
							19	24		6.5
CA 1-9	8.4	149	6950000	16	7621830	CTB	12	12	39.18	3.7
							13	6		4.0
							18	71		5.5
							19	65		5.8
CA 2-2	8.4	711.5	7040000	7	4423827	PCTB	12	0	26.59	5.4
							13	0		5.9
							18	0		8.1
							19	0		8.6

Table 79. Ratio of slab length to the radius of relative stiffness (L/l) (cont'd).

Section ID	Slab Thick, in	Static k, pci	Slab E, psi	Age, yrs	Cum. ESAL's	Base Type	Slab Length, L ft	Percent Slabs Cracked	Rad. of Rel. Stiff., l in	L/l
CA 2-3	8.4	286	4980000	7	4423827	CTB	12	7	30.63	4.7
							13	53		5.1
							18	100		7.1
							19	100		7.4
CA 3-1	9.0	143	3530000	12	3641197	CTB	12	13	35.19	4.1
							13	0		4.4
							18	13		6.1
							19	7		6.5
CA 3-2	9.0	156	4170000	12	3641197	CTB	12	0	35.90	4.0
							13	0		4.3
							18	7		6.0
							19	0		6.4
CA 3-5	9.0	198.5	4380000	12	3641197	CTB	12	43	34.22	4.2
							13	40		4.6
							18	7		6.3
							19	13		6.7
CA 6	9.0	147	6710000	7	4434338	PATB	12	0	41.04	3.5
							13	0		3.8
							14	0		4.1
							15	12		4.4
CA 7	10.2	163	6260000	7	10524966	CTB	12	11	43.17	3.3
							13	29		3.6
							18	47		5.0
							19	47		5.3
CA 8	10.2	169.5	6420000	7	1190558	HMAC	12	0	43.02	3.3
							13	0		3.6
							14	0		3.9
							15	0		4.2
FL 2	13.0	189	5550000	1	2005755	SAND	12	0	48.43	3.0
							13	0		3.2
							18	0		4.5
							19	0		4.7
FL 3	9.0	264.5	4160000	5	5967175	LCB	16	77	31.44	6.1
							17	79		6.5
							22	92		8.4
							23	100		8.8
MI 1-4a	9.0	234	5880000	12	885249	PATB	12	0	35.35	4.1
							13	0		4.4
							18	0		6.1
							19	0		6.4
MI 1-7a	9.0	146	6340000	12	885249	AGG	12	0	40.53	3.6
							13	0		3.8
							16	0		4.7
							17	0		5.0

Table 79. Ratio of slab length to the radius of relative stiffness (L/l) (cont'd).

Section ID	Slab Thick, in	Static k, pci	Slab E, psi	Age, yrs	Cum. ESAL's	Base Type	Slab Length, L ft	Percent Slabs Cracked	Rad. of Rel. Stiff., l in	L/l
MI 1-7b	9.0	134.5	6090000	12	885249	AGG	12	0	40.96	3.5
							13	0		3.8
							16	0		4.7
							17	0		5.0
MI 1-10a	9.0	218	6230000	12	885249	ATB	12	0	36.51	3.9
							13	0		4.3
							18	0		5.9
							19	0		6.2
MI 1-10b	9.0	251	5280000	12	885249	ATB	12	0	33.81	4.3
							13	0		4.6
							18	10		6.4
							19	10		6.7
MN 2-1	9.0	64	6780000	10	222420	AGG	13	0	50.65	3.1
							14	0		3.3
							16	0		3.8
							19	0		4.5
MN 2-2	8.0	63.5	8010000	10	222420	AGG	13	0	48.44	3.2
							14	0		3.5
							16	0		4.0
							19	0		4.7
MN 4	7.5	111	6300000	1	218813	AGG	13	0	37.80	4.1
							14	0		4.4
							16	0		5.1
							17	0		5.4
NY 1-1	9.0	274.5	3810000	22	3136345	ATB	20	0	30.48	7.9
NY 1-6	9.0	304.5	4020000	22	3136345	AGG	20	13	30.10	8.0
NY 1-8a	9.0	319	4100000	22	3136345	ATB	20	3	29.90	8.0
NY 1-8b	9.0	274	3880000	22	3136345	ATB	20	0	30.63	7.8
NY 2-3	9.0	170.5	5100000	12	1428074	AGG	20	13	36.92	6.5
NY 2-9	9.0	235.5	5090000	12	1428074	AGG	20	9	34.04	7.0
NY 2-11	9.0	148	6120000	12	1428074	AGG	26.7	13	40.04	8.0
NC 1-1	9.0	269	4630000	20	9137389	AGG	30	3	32.16	11.2
NC 1-2	9.0	173.5	5190000	20	9137389	SC	30	6	36.93	9.7
NC 1-3	9.0	247	3970000	20	9137389	SC	30	3	31.61	11.4
NC 1-4	9.0	285	4210000	20	9137389	AGG	30	0	30.95	11.6
NC 1-5	9.0	314	5500000	20	9137389	CTB	30	0	32.30	11.1
NC 1-6	9.0	336	5140000	20	9137389	ATB	30	0	31.23	11.5
NC 1-8	9.0	256.5	4220000	20	9137389	AGG	30	37	31.80	11.3
NC 2	11.0	146.5	5890000	5	5755336	LCB	18	0	46.22	4.7
							19	0		4.9
							23	8		6.0
							25	8		6.5

Table 80. Effect of joint spacing, slab thickness, and base type on transverse cracking for California 1.

Joint Space, ft	Slab Thick, in	Base Type	Radius of Rel. Stiff, l, in	Percent Slabs Cracked	L/l
5	8.4	CTB	41.1	0	1.5
7	8.4	CTB	41.1	0	2.0
8	8.4	CTB	41.1	0	2.6
11	8.4	CTB	41.1	3	3.2
12	8.4	CTB	35.1	0	4.1
13	8.4	CTB	35.1	0	4.4
18	8.4	CTB	35.1	6	6.2
19	8.4	CTB	35.1	29	6.5
12	8.4	LCB	35.1	0	4.1
13	8.4	LCB	35.1	12	4.4
18	8.4	LCB	35.1	52	6.2
19	8.4	LCB	35.1	24	6.5
12	11.4	CTB	44.7	0	3.2
13	11.4	CTB	44.7	0	3.5
18	11.4	CTB	44.7	0	4.8
19	11.4	CTB	44.7	0	5.1
12	8.4	CTB	39.2	12	3.7
13	8.4	CTB	39.2	6	4.0
18	8.4	CTB	39.2	71	5.5
19	8.4	CTB	39.2	65	5.8

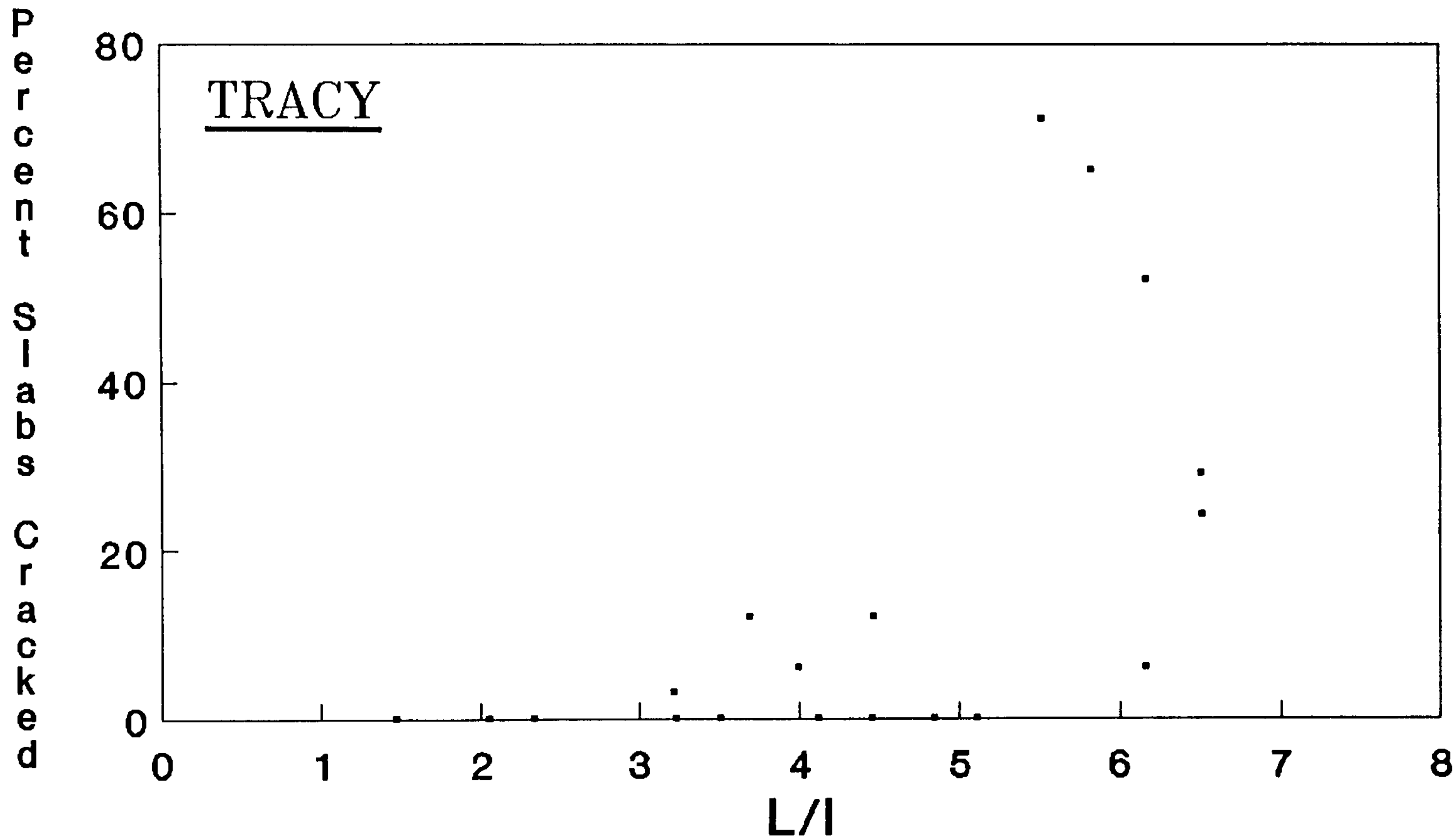

Figure 34. Percent cracked slabs vs. L/l for California 1 (Tracy) sections.

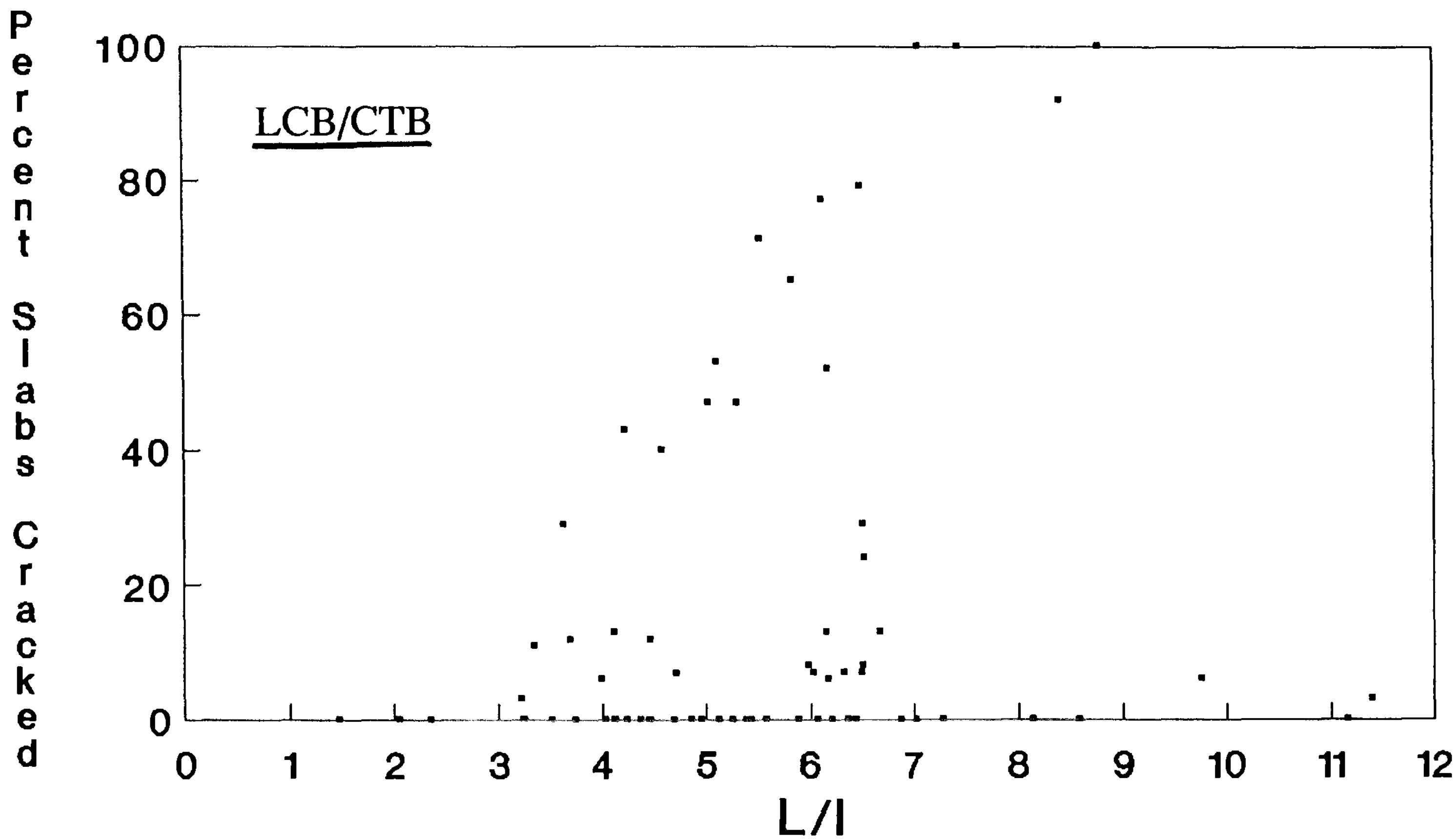

Figure 35. Percent cracked slabs vs. L/l for all sections with cement-treated and lean concrete bases.

As seen in figure 36, transverse cracking occurred for slabs with L/l greater than about 6.5 for sections having either dense-graded aggregate or no base (slab on grade). Asphalt-treated bases showed very little transverse cracking; a similar plot showed that transverse cracking occurred on some sections for L/l ratios of greater than 5.5.

It appears that there exists a general relationship between the L/l ratio and transverse cracking. For each base type there exists a level of L/l that, if exceeded, greatly increases the probability of transverse cracking. The data from this study of over 2500 slabs suggests the following general limits to reduce transverse slab cracking.

Base Type	Maximum L/l for crack control
AGG	6.5
CTB, ATB, LCB	5.0

Using these guidelines, the allowable joint spacing would increase with increased slab thickness, but decrease for increased (stiffer) foundation support conditions. Although little data was available to evaluate permeable bases, it is believed that the L/l value for each type of permeable base construction (stabilized or nonstabilized) would be the same as the L/l values given above for the nonpermeable base designs.

5. REINFORCEMENT DESIGN

Reinforcement is placed in JRCP to keep transverse cracks from opening and deteriorating. It is accepted that these longer slabs will crack from curling and shrinkage stresses; the reinforcement is expected to hold the crack tight to prevent spalling and faulting of the crack. However, there must be an adequate amount of steel present to hold the crack tight; to date, little guidance has been provided on the amount of reinforcement needed to accomplish this.

There are several factors that affect the performance of the JRCP pavements, in addition to the amount of reinforcement used in the slab. The size, coating, spacing and, ultimately, the performance of dowels has an impact in that they determine whether the joints will continue to work to accommodate thermal movements of the slab. If the dowel bars become corroded and lock up, the joint will be rendered ineffective and mid-slab cracks will open up and begin working. The base type also has an effect, as different base materials will have different friction coefficients which again affects slab movement. Finally, the effect of joint spacing should be considered. More movement at joints or working cracks can be expected from longer slabs.

Minnesota 1

The transverse cracking data from this project on I-94 near Rothsay clearly suggests that dowels had locked up due to corrosion. This is because there were more deteriorated transverse cracks in the doweled sections than in the nondoweled sections, which implies that the reinforcement design was unable to accommodate the increased stresses of immobile joints. There existed only 0.08 percent to 0.09 percent reinforcement in these slabs, which is believed to be inadequate for the temperature extremes of the region. In

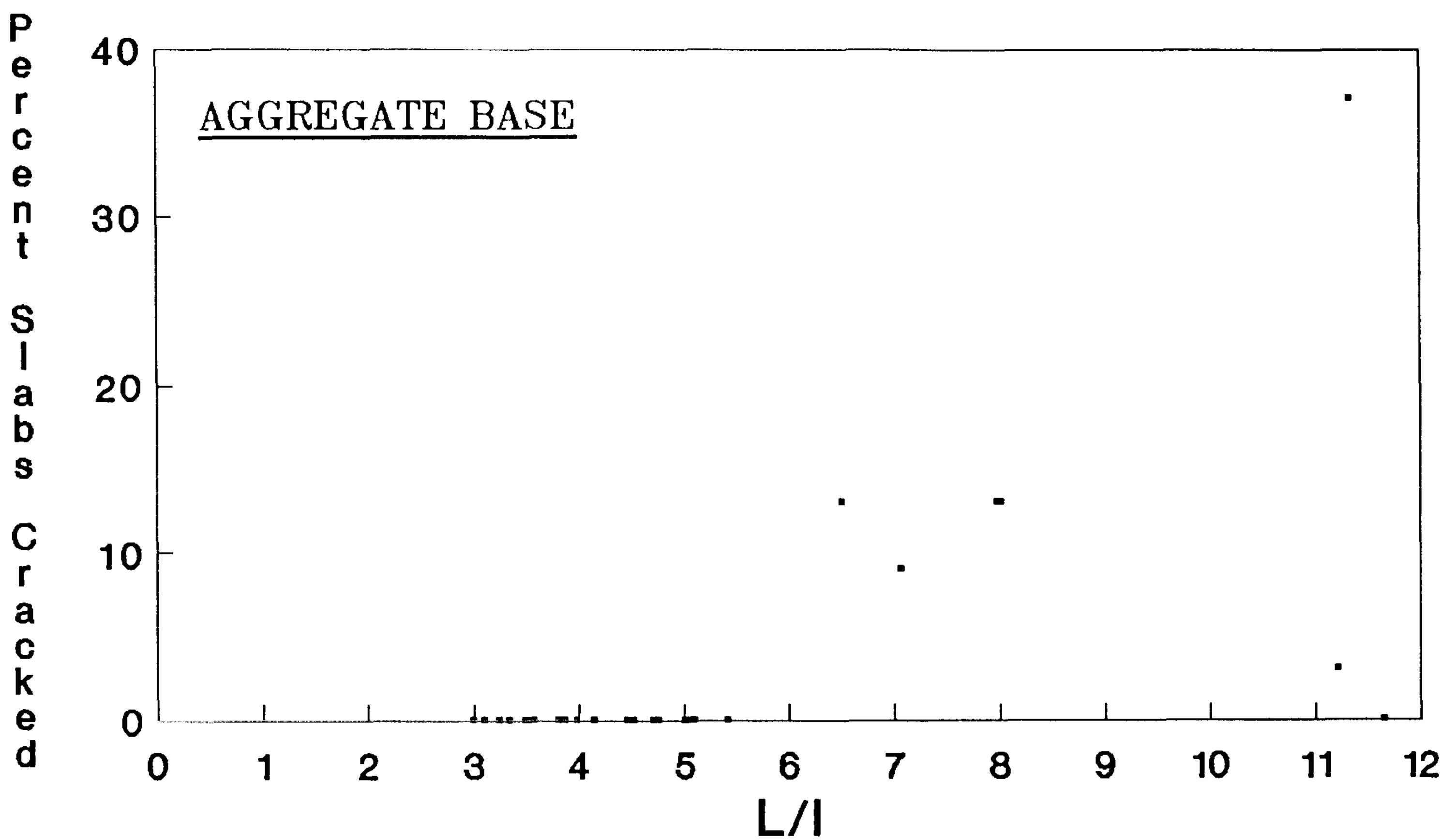

Figure 36. Percent cracked slabs vs. L/l for all sections with aggregate bases.

addition, the aggregate base sections had the least transverse cracking (followed by the CTB and the ATB sections), and longer (40 ft [12.2 m]) JRCP slabs had more transverse cracks than shorter 27 ft (8.2 m).

Ohio 1

The sections with plastic-coated dowels performed much worse in terms of transverse cracking that the sections with noncoated dowels. This is an anomaly that needs to be studied further. Of interest would be the final alignment of the plastic-coated dowels and the current quality and condition of the coating. The results suggest that while the longer jointed slabs do not perform as well, 0.09 percent reinforcement is perhaps adequate to hold cracks tight in the doweled JRCP slabs of this particular climatic region.

New Jersey 2 and New Jersey 3

These heavily trafficked pavements both have a high percentage of reinforcement steel (0.14 percent and 0.16 percent) and 1.25-in (32 mm) diameter, stainless steel clad dowel bars. Since both of these factors are expected to contribute to the reduction of transverse cracking, it is not possible to further differentiate between them. The performance of NJ 2 after 36 years and 35 million ESAL's is particularly noteworthy.

Other Sections

Most other reinforced sections all had fairly high amounts of reinforcement steel, ranging from 0.15 to 0.20 percent. Traffic on these pavements ranges from 0.9 million ESAL's to 9.1 million ESAL's, and joint spacings are all comparatively high, ranging from 60 ft (18.3 m) to 71 ft (21.7 m). These are constructed over a variety of base types. With the exception of MI 4, which had 205 transverse cracks/mile and MI 5, which had 144 transverse cracks/mile, none of these pavements exhibited a significant amount of transverse cracking.

Summary of the Effects of Reinforcement

The isolated effect of reinforcement on transverse crack deterioration and pavement performance cannot be directly determined because there were no direct comparisons available. However, an examination of table 81 indicates that the sections with more than 0.10 percent reinforcement appear to have performed very well regardless of the joint spacing, base type, or dowel coating. At lower levels of reinforcement, it appears that the shorter joint spacings performed better than the longer joint spacings and that bases with higher friction coefficients had more cracking than those with lower friction coefficients. Thus, a <u>minimum</u> of 0.10 percent reinforcement appears to be necessary to control transverse cracking, with higher amounts for extreme climatic conditions or stiff base course materials.

6. JOINT ORIENTATION

Joint orientation refers to the angle that the transverse joint is placed with respect to the centerline of the mainline pavement. Perpendicular joints are constructed at a right angle to the centerline of the mainline pavement. Skewed joints are placed at an angle to the centerline of the mainline pavement, usually offset 2 ft (0.6 m) per lane.

Table 81. Summary of performance data related to reinforcement design.

LOCATION	ESAL's (millions)	BASE TYPE	JOINT SPACING, ft	PERCENT REINF.	DOWEL DIA., in	TRANSVERSE CRACKS/mile
DRY-FREEZE						
MN 1	5.5	AGG	27	0.085	0.00	16
		AGG	27	0.085	1.00 NC	35
		ATB	27	0.085	0.00	21
		ATB	27	0.085	1.00 NC	51
		CTB	27	0.085	0.00	0
		CTB	27	0.085	1.00 NC	44
MN 5	5.5	AGG	40	0.040	1.00 NC	53
MN 2	2.8	AGG	27	0.090	1.00 NC	3
MN 3	1.5	AGG	27	0.050	1.00 EC	0
MN 6	0.9	PATB	27	0.060	1.00 EC	0
WET-FREEZE						
MI 1	0.9	AGG	71	0.150	1.25 EC	3
MI 3	2.8	PAGG	41	0.140	1.25 EC	0
MI 4	4.3	AGG	71	0.150	1.25 NC	205
MI 5	3.1	PAGG	41	0.140	1.25 EC	144
NY 1	2.0	AGG	61	0.200	ACME	0
		ATB	61	0.200	ACME	0
NY 2	1.1	AGG	63	0.200	I-BEAM EC	0
OH 1	3.4	AGG	21	0.090	1.25 NC	0
		AGG	21	0.090	1.25 PC	31
		AGG	40	0.090	1.25 NC	0
		AGG	40	0.090	1.25 PC	235
		ATB	21	0.090	1.25 NC	0
		ATB	40	0.090	1.25 NC	29
NJ 2	35.0	AGG	78	0.140	1.25 SSC	24
NJ 3	4.2	PAGG PATB	78	0.160	1.25 SSC	0
WET-NONFREEZE						
NC 1	9.1	AGG	60	0.170	1.25 NC	0

Key: NC = No Coating (of Dowels)
EC = Epoxy-Coated
PC = Plastic-Coated
SSC = Stainless-Steel Coating
ACME = Proprietary two-part malleable iron load transfer devices

Skewed joints are usually constructed to reduce the number of critical wheel loads occurring at the transverse joint from two to one, and thus should reduce joint deflection. Skewed joints are often placed in a random pattern and are expected to be of most benefit to nondoweled pavements, as it is not believed that they would add any load transfer if adequate diameter, corrosion-resistant dowel bars are present. Similarly, the use of a random joint spacing pattern is not expected to assist in pavement performance for doweled pavements.

The effect of skewed joints on pavement performance is not always clear, as this variable is often confounded by other factors (e.g., dowel bars and pavement type). Few projects made a direct comparison between skewed and perpendicular joints.

New York 1

Route 23 near Catskill included skewed and perpendicular nondoweled joints for JPCP. The section with skewed joints has slightly less transverse cracking and a lower PSR, but slightly more faulting than the section with perpendicular joints. Measured roughness was the same for both sections. This distress information is provided in table 82.

North Carolina 1

Interstate 95 near Rocky Mount included skewed and perpendicular nondoweled joints for JPCP. Skewed joints exhibited about one-half the joint faulting of the perpendicular joints and transverse cracking was much greater for the perpendicular joints. Measured roughness and the panel PSR was less for the skewed joints than for the perpendicular joints. Performance data for these sections is summarized in table 83.

Summary of the Effects of Joint Orientation

The sample size is too small to draw definite conclusions about the effect of skewed joints on pavement performance. In fact, the only two sites with direct comparisons showed different results. The Rocky Mount sections showed that nondoweled skewed joints have much lower faulting than nondoweled perpendicular joints. At the Catskill site, however, the nondoweled perpendicular joints had slightly less faulting than the nondoweled skewed joints, although the faulting for both sections was very small. The skewed joints for both test sites had lower PSR than perpendicular joints.

Skewed joints and random joint spacings are believed to be of most benefit for nondoweled pavements. The performance of pavement slabs with adequate diameter, corrosion-resistant dowel bars is not expected to increase utilizing those features.

7. TRANSVERSE JOINT LOAD TRANSFER

Transverse joint load transfer, the mechanism by which wheel loads are transferred from one side of the joint to the other, is a critical design element for jointed concrete pavements. It is typically achieved by either aggregate interlock, which is the face-to-face interaction of the aggregate particles on either side of the joint, or mechanical load transfer devices, such as dowel bars, which are installed across a joint at the mid-depth of the slab.

Table 82. Outer lane performance data relative to joint orientation for New York 1 (Age = 22 years, ESAL's = 2.0 million).

Joint Orientation	Joint Faulting, in	Transverse Cracks/mi	% Joint Spalling	Roughness, in/mi	PSR
Skewed	0.03	0	0	65	3.8
Perpendicular	0.01	9	0	64	4.1

Sections are 9-in (229 mm) nondoweled JPCP over ATB with 20-ft (6.1 m) joint spacing.

Table 83. Outer lane performance data relative to joint orientation for North Carolina 1 (Age = 20 years, ESAL's = 9.1 million).

Joint Orientation	Joint Faulting, in	Transverse Cracks/mi	% Joint Spalling	Roughness, in/mi	PSR
Skewed	0.12	5	0	83	3.4
Perpendicular	0.22	64	0	95	3.7

Sections are 9-in (229 mm) nondoweled JPCP over an aggregate base with 30-ft (9.1 m) joint spacing.

For cold climates and heavy traffic loadings, aggregate interlock is generally not acceptable for providing adequate load transfer. If aggregate interlock is relied on where it is not suitable, excessive joint faulting will usually occur. However, other factors can also contribute to faulting. These include the diameter and spacing of dowel bars (which determines the dowel/concrete bearing stresses), the base type (stabilized base courses reduce the rate but not the occurrence of faulting), and the drainage conditions (the presence of free water beneath the slab can cause pumping and, ultimately, faulting).

The concrete thermal coefficient of expansion has a major impact on the suitability of aggregate interlock for load transfer. Also, the size and relative quality of the aggregate, as well as the anticipated temperature changes, are very important factors. Existing design methodologies do not currently account for these factors.

Joint load transfer was measured on all sections using a Falling Weight Deflectometer (FWD) during cooler morning hours. Load transfer affects corner and transverse joint deflection and thus is strongly related to pumping, erosion, and faulting. The following discussion addresses the pavement sections included in the study which evaluate the effectiveness of the two primary means of load transfer.

Doweled and Nondoweled Comparisons

<u>Minnesota 1</u>

This project on I-94 clearly demonstrates the effectiveness of dowel bars in reducing faulting in cold climates subject to moderate-heavy traffic levels. The results in table 84 show that doweled joints had greater measured load transfer, lower corner deflections, and a lower percentage of corners with voids (loss of support). As a result, the faulting was reduced by 87 percent on average. In fact, the nondoweled sections required rehabilitation in 1984 due to their severe faulting. However, the 1-in (25 mm) diameter dowel bars may not have been large enough, as they have become loose as indicated by the relatively low load transfer. In addition, doweled sections did display <u>higher</u> levels of joint spalling and <u>more</u> deteriorated transverse cracks due to dowel freeze-up.

<u>North Carolina 1</u>

The results for this section are shown in table 85. Dowel bars 1-in (25 mm) in diameter were fairly effective in reducing faulting on <u>aggregate</u> base courses under heavy traffic as compared to no dowel bars. Doweled joints showed far higher load transfer, lower corner deflections, and one-third fewer corners with loss of support after the very heavy traffic loadings on this project. However, due to the magnitude of faulting for the doweled joints and the loss of load transfer, it appears that, for the given traffic loadings and joint spacing, the 1-in (25 mm) diameter dowels were not of sufficient size to keep joint faulting at an acceptable level.

The 1-in (25 mm) dowels were completely ineffective for the <u>soil cement</u> stabilized bases. This resulted in large faulting and loss of support at the corners. Surprisingly, the nondoweled joints tested had 100 percent load transfer and very low corner deflections. There was no measured loss of support at the corners.

Table 84. Outer lane performance data relative to joint load transfer for Minnesota 1 (Age = 17 years, ESAL's = 5.5 million).

Slab, in	Dowel Dia, in	Joint Faulting, in	Transverse Cracks/mi	% Joint Spalling	L.T. Eff, %	Corner Defl, mils	% Corner Voids
8	0.00	0.39*	25	23	53	12.3	12
8	1.00	0.04	32	25	62	11.1	0
9	0.00	0.33*	0	8	38	13.1	5
9	1.00	0.08	55	30	67	10.5	0

Data is averaged across all base types.
* Data from 1984 prior to rehabilitation of nondoweled sections.

Table 85. Outer lane performance data relative to joint load transfer for North Carolina 1 (Age = 20 years, ESAL's = 9.1 million).

Base Type	Dowel Dia, in	Joint Faulting, in	Transverse Cracks/mi	% Joint Spalling	L.T. Eff, %	Corner Defl, mils	% Corner Voids
AGG	0.00	0.22	64	0	33	9.7	20
AGG	1.00	0.13	0	0	68	8.6	7
SC	0.00	0.13	5	0	100	4.0	0
SC	1.00	0.16	10	0	59	8.8	15

Sections are 9-in (229 mm) JPCP with 30-ft (9.1 m) joint spacing.

There was no joint spalling for both doweled and nondoweled joints. Transverse cracking was much greater for nondoweled joints on an aggregate base course, but about the same for the soil cement base.

Nondoweled Sections

In addition to the above directly comparable results, there existed many sections of JPCP without dowels under medium to heavy traffic that provide informative results. This data is presented in table 86. The following results are suggested from an examination of that table.

- Nondoweled joints, irrespective of other design features or climate, will generally develop significant faulting under heavy traffic loadings (over 1 million ESAL's).

- Although faulting developed in all climates, colder and wetter climates developed more faulting than warmer and dryer climates.

- In general, the higher the joint load transfer, the lower the corner deflection, the fewer the corners with loss of support, and the lower the joint faulting.

- Longer joint spacing, in the range of 18 to 30 ft (5.5 to 9.1 m) has a much greater amount of faulting than joint spacings in the range of 5 to 13 ft (1.5 to 4.0 m).

- There were sections with dense-graded aggregate, ATB or CTB, or lean concrete bases that showed significant faulting. The permeable bases showed low faulting, although they were not very old and had not been subjected to a significant amount of traffic.

Other Doweled Sections

There were also several individual sections of JPCP and JRCP with dowels that generally showed lower faulting. However, dowel bar diameter also appears to be important in controlling faulting. The following results are obtained from an examination of table 87.

- Load transfer is fairly high for doweled joints, with the exception of FL 2 and FL 3.

- The longer jointed pavements tend to have higher faulting. Acceptable faulting for longer jointed pavements is about 0.26 in (6.6 mm) before roughness becomes a problem.

- A 1-in (25 mm) dowel bar diameter may be too small for heavy traffic. For example, the NC 1-4 and 1-7 sections showed considerable faulting after 9 million ESAL's, as did FL 3 after 5.6 million. However, the shorter jointed ONT 2 and MN 1, both with a 1-in (25 mm) diameter dowel, showed very little faulting after 35 million and 5.5 million ESAL's, respectively.

- Several sections with 1.25-in (32 mm) diameter or larger dowel bars showed low faulting after many ESAL loadings.

Table 86. Load transfer performance data for nondoweled pavement sections.

ERES ID	ESAL's (mill)	Base Type	Joint Spacing, ft	L.T. Eff, %	Corner Defl, mils	% Corner Voids	Pumping	Joint Fault, in
WET-NONFREEZE:								
CA 3	3.6	CTB	12-19	80	4.3	4	None	0.10
WET-FREEZE:								
MI 1								
4a	0.9	PATB	12-19	19	21.8	75	None	0.03
10ab	0.9	ATB	12-19	42	17.1	85	Low	0.17
OH 2	3.3	None	20	—	----	—	Medium	0.11
ONT 1								
1	1.0	None	12-19	—	----	—	Low	0.05
2	1.0	PATB	12-19	—	----	—	Medium	0.05
3,4	1.0	LCB	12-19	—	----	—	Low	0.04
NY 1								
8ab	3.1	ATB	20	100	10.9	22	None	0.02
DRY-NONFREEZE:								
CA 1								
1	7.6	CTB	5-11	85	15.9	60	Medium	0.06
3,5,9	7.6	CTB	12-19	87	14.0	55	Low	0.11
7	7.6	LCB	12-19	88	8.2	10	High	0.06
CA 2								
2	4.4	AC/PCTB	12-19	10	14.9	65	None	0.11
3	4.4	CTB	12-19	19	24.4	90	None	0.11
CA 6	4.4	LCB	12-15	79	5.9	0	None	0.15
CA 7	10.5	CTB	12-19	35	9.8	23	None	0.06
CA 8	5.2	HMAC	12-15	92	3.2	0	None	0.04
AZ 1								
1	4.0	CTB	13-17	94	8.6	30	None	0.08
2,4,5	2.9	None	13-17	100	7.8	12	None	0.02
6,7	1.8	LCB	13-17	97	6.0	12	None	0.01
DRY-FREEZE:								
MN 1								
1,3,5, 7,9,11	5.5	AGG ATB CTB	27	46	12.7	8	N-Med	0.36*

* Data from 1984 prior to rehabilitation of nondoweled sections.
Data averaged over sections of similar design.

Table 87. Load transfer performance data for doweled sections.

ERES ID	ESAL's (mill)	Joint Spacing, ft	Dowel Dia, in	L.T. Eff, %	Corner Defl, mils	% Corner Voids	Joint Fault, in
WET-FREEZE:							
OH 1	3.4	40	1.25	88	13.2	63	0.10
MI 3	2.8	41	1.25	88	5.1	0	0.02
MI 4	4.4	71	1.25	59	21.3	58	0.09
MI 5	3.1	41	1.25	94	26.9	95	0.05
NJ 2	34.8	78	1.25	76	7.2	0	0.06
OH 1	3.4	21	1.25	83	12.6	31	0.06
ONT 2	35.6	12-19	1.00	--	----	--	0.01
WET-NONFREEZE:							
NC 1-7	9.1	60	1.00	77	8.3	0	0.15
NC 1-4	9.1	30	1.00	59	8.8	15	0.13
NC 2	5.7	18-25	1.38	100	7.5	5	0.02
FL 2	2.0	12-19	1.25	29	7.6	10	0.08
FL 3	5.6	16-23	1.00	19	23.4	73	0.08
DRY-FREEZE:							
MN 1 2,4,6, 8,10,12	5.5	27	1.00	64	10.8	0	0.06
MN 3	1.5	27	1.00	93	9.6	17	0.01
MN 4	0.2	13-17	1.00	86	7.9	0	0.01
MN 6	0.9	27	1.00	80	7.2	0	0.01
DRY-NONFREEZE:							
AZ 2	1.6	13-17	1.25	72	11.1	31	0.01

Summary Of the Effects Of Load Transfer

Dowel bars were found to substantially reduce the amount of joint faulting when compared directly with nondoweled sections of otherwise similar design. The diameter of the bar was a factor, with 1.25-in (32 mm) diameter bars providing good load transfer and faulting control under heavy traffic, while a 1-in (25 mm) diameter dowel was sometimes ineffective, particularly for heavy traffic loading.

Nondoweled JPCP generally developed significant faulting, regardless of pavement design or climate. Sections with dowels displayed higher levels of joint spalling and more deteriorated transverse cracks. This is believed to be the result of joint lockup caused by either corrosion or dowel misalignment.

Several other observations can be made concerning load transfer. Although significant faulting developed in all climates, colder and wetter climates develop more faulting than warmer and dryer climates. Also, the longer the joint spacing the greater the faulting, both in sections with and without dowel bars. Finally, base type seemed to have an effect on faulting. Nondoweled pavement designs included sections with dense-graded aggregate, ATB or CTB, and lean concrete bases that showed significant faulting. The permeable bases showed lower faulting, although they were not very old and have not been subjected to heavy traffic.

8. DOWEL BAR COATINGS

Nearly all of the doweled pavement sections in the project had conventional dowel coatings of paint and/or grease. However, several sections utilized some sort of corrosion inhibitor, such as stainless steel, plastic, and epoxy. The corrosion inhibitor is expected to prevent the dowel from corroding and subsequently locking up, which can result in joint spalling and the opening of transverse cracks in the slab. The following information is available regarding the performance of these special coatings.

Michigan 1

The project on U.S. 10 near Clare represented one of the earlier uses of epoxy-coated dowel bars. While the short-jointed slabs also performed well, most noteworthy were the 71-ft (21.6 m) JRCP sections which exhibited no deteriorated transverse cracking and no significant joint spalling. The coating is believed to have prevented corrosion and subsequent dowel lockup, which can produce extensive mid-slab cracking. It should be noted that these pavement sections have been subjected to less than 1 million ESAL's.

Michigan 5

This newer JRCP section on I-94 near Paw Paw contained epoxy-coated dowel bars. However, the lubricant placed on the dowel to facilitate movement was ineffective, and this served to increase the restraint in the slab. The restraint was also increased due to the basket tie wires not being severed prior to paving. Subsequently, many of the joints locked up, which served to open adjacent transverse cracks.

Ohio 1

Plastic-coated dowel bars used on U.S. 23 near Chillicothe displayed less faulting than standard-coated (paint/grease) dowel bars. However, it appears that the plastic-coated dowels had an adverse effect on other distresses, such as transverse cracking and joint spalling. The exact cause of the increased deterioration is not known, but a possible explanation could be that the dowel bars were misaligned during construction. Table 88 provides performance data for this project relative to dowel bar coating.

New York 2

This project on I-88 near Otego contained epoxy-coated I-beams. The long-jointed section (63.5-ft [19.3 m] joint spacing) displayed no deteriorated transverse cracks and no transverse joint spalling. The epoxy-coated I-beams and the large amount of reinforcement (0.2 percent) evidently combined to provide excellent performance after 1.4 million ESAL's.

New Jersey 2

The stainless steel dowel coating used in the New Jersey 2 section resulted in excellent performance. Very little deteriorated transverse cracking occurred after 35 million ESAL's and 36 years of service. This performance is typical of other similar pavements in New Jersey.

Summary of the Effects of Dowel Coatings

Good anticorrosion coatings appear to have been very successful in limiting distresses related to the occurrence of dowel lockup. However, insufficient data was available for a direct comparison of coated and noncoated dowels. In addition, an effective dowel lubricant must be placed on the dowel to ensure its movement and thereby prevent joint lockup.

9. LONGITUDINAL JOINT DESIGN

There existed a substantial amount of longitudinal cracking on some sections and none on many others. It is believed that the design and construction of the longitudinal joint and the base/slab friction may greatly influence the occurrence of longitudinal cracking, which can lead to serious pavement deterioration, increased maintenance efforts, and reduced pavement life. The results shown in table 89 were obtained from the field data that has been averaged over sections of similar design.

The results show that base type is a very important factor in the development of longitudinal cracking. This can be related to the friction factor that is an indication of the relative amount of friction that is produced at the slab-base interface; stabilized bases produce higher friction than nonstabilized bases. The average longitudinal cracking for each base type is summarized below:

No Base = 86 ft/mile (sum of inner and outer lanes)
Lean Concrete Base = 226 ft/mile
Aggregate Base = 228 ft/mile
Asphalt-Treated Base = 664 ft/mile
Cement-Treated Base = 729 ft/mile

Table 88. Outer lane performance data relative to dowel coating for Ohio 1 (Age = 14 years, ESAL's = 3.4 million).

Joint Spacing, ft	Dowel Coating	Joint Faulting, in	Transverse Cracks/mi	% Joint Spalling
21	Paint/Grease	0.10	0	0
21	Plastic	0.03	31	0
40	Paint/Grease	0.14	53	0
40	Plastic	0.07	235	0

All sections are 9-in (229 mm) on aggregate base.

Table 89. Summary of longitudinal cracking and related design data.

SECTION	AGE	BASE	LONG. JT. FORMING	LONG. JT. DEPTH, in	DEPTH/ THICK	LONG. CRACKS, ft/mile
OH 2	13	None	Saw	3.75	0.25	258
ONT 1-1	5	None	Saw	3.00	0.25	0
AZ 1-2,3	10	None	Saw	3.25	0.25	0
					NO BASE MEAN = 86	
MN 5	18	AGG	Saw	2.75	0.30	1261
MN 2	10	AGG	Saw	2.75	0.32	224
NY2	12	AGG	Saw	2.25	0.25	194
NY1	22	AGG	Saw	2.00	0.22	132
MI 4	15	AGG	Saw	2.75	0.30	91
MN 1	17	AGG	Insert	2.75	0.32	75
NC 1 1,4,7,8	20	AGG	Saw	2.75	0.30	74
OH 1	14	AGG	Saw	2.25	0.25	0
MI 1	12	AGG	Saw	2.75	0.30	0
					AGG BASE MEAN = 228	
CA 1-5	16	LCB	Insert	2.00	0.24	230
AZ 1-6,7	5	LCB	Saw	2.25	0.25	0
CA 6	7	LCB	Insert	2.00	0.24	0
NC 2	5	LCB	Saw	3.50	0.32	0
FL 3	5	LCB	Saw	2.50	0.28	900
					LCB MEAN = 226	
NC 1-2,3	20	SC	Saw	2.75	0.30	2260
CA 7	7	CTB	Saw	3.00	0.29	2060
MN 1	17	CTB	Insert	2.75	0.32	1320
CA 1-1,3,9	16	CTB	Insert	2.00	0.24	500
AZ 1-1	15	CTB	Insert	2.25	0.25	233
NC 1-5	20	CTB	Saw	2.75	0.30	179
CA 3	12	CTB	Insert	2.00	0.22	11
CA 1-5	16	CTB	Insert	2.00	0.26	0
CA 2-3	7	CTB	Saw	2.00	0.24	0
					CTB MEAN = 729	
MN 1	17	ATB	Insert	2.75	0.32	3550
CA 8	7	AC	Saw	3.00	0.29	1026
NY1	22	ATB	Saw	2.00	0.22	73
OH 1	14	ATB	Saw	2.25	0.25	0
NC 1-6	20	ATB	Saw	2.75	0.30	0
CA 2-2	7	AC/PCTB	Saw	2.00	0.24	0
ONT 1-2	5	PATB	Saw	2.20	0.27	0
					ATB MEAN = 664	

Longitudinal joint depths obtained from plans or State specifications.

There is likely a great difference between slab/base friction for these base types. The slab-on-grade designs are expected to have low friction and in fact show very little longitudinal cracking. The CTB and ATB are known to bond very tightly to the slab and, due to their stiffness, produce a large amount of friction. This is reflected by the large amount of cracking that they display. The LCB and aggregate bases rank between the two extremes.

The longitudinal joint depth ranged from 22 to 34 percent of the slab thickness, and was formed by either sawing or plastic insert. It should be noted that the depths of the longitudinal joints were obtained from plans or State specifications and do not represent actual field-measured depths.

Within each base type, there does not appear to be any relationship between depth/slab thickness ratio and the amount of longitudinal cracking. A trend is noticed, however, concerning the <u>type</u> of joint forming technique used. It was observed that the average longitudinal cracking on all sections with inserts was 658 ft/mi (125 m/km) compared to 364 ft/mi (69 m/km) for the sections with the sawcut longitudinal joint. Thus, it is not believed that the inserts are as effective in creating the longitudinal joint.

Summary of the Effects of Longitudinal Joint Design

Two conclusions can be reached concerning the longitudinal joint design. The first conclusion is that the type of base has a profound effect on the amount of longitudinal cracking, as previously discussed. The second conclusion is that the establishment of the longitudinal joint is critical in order to minimize longitudinal cracking. Every effort must be made to ensure the formation of the longitudinal joint. For inserts, it is critical that the insert is placed to the specified depth. For sawcuts, the important factor is the time of sawing. If the sawcut is not made in sufficient time, a longitudinal crack will not form beneath the cut. When stabilized bases are used, the timing of the longitudinal joint sawing becomes even more critical. If bonding develops or the slab is placed thicker than the plans specify, sawing to the recommended depth of one-third of the slab thickness may not be enough to ensure the formation of the longitudinal joint.

10. TRANSVERSE JOINT SEALANT

Data from the field sections was analyzed to determine the effect of joint sealant on pavement performance. The types of joint sealants included asphalt cement, rubberized asphalt, silicone, and preformed compression seals. In addition, sections were included which contained no joint sealant.

Two projects were studied where joint sealant type was varied and other factors held constant. In addition, several other observations on the effect of joint sealants can be made from other sites. It should be noted that joint sealing maintenance also has an effect on joint spalling as well as the original sealant type, but this information was often not available.

Direct Sealant Comparisons

<u>California 3</u>

Section 3-1 received a rubber asphalt sealant, while CA 3-2 and 3-3 did not have any sealant. Section 3-1 exhibited joint spalling at 2 percent of the joints whereas the nonsealed sections displayed joint spalling at 3 percent and 6 percent of the joints, respectively.

Minnesota 2

These four sections on I-90 were constructed in 1977. Sections 2-1 and 2-2 were sealed with a hot-poured sealant and sections 2-3 and 2-4 were sealed with preformed compression sealants. Spalling was minimal for both sealant types, averaging about 6 percent of all joints for both. However, the preformed sealants were in better condition than the hot poured. Also, the preformed sealant was used on the section with 27-ft (8.2 m) joints, whereas the hot poured was used on the shorter, random joint spacing section. Thus, the preformed seals were able to perform as well as the hot poured while accommodating larger joint movements.

Nonsealed Joints in California

Joint sealant is typically not used in concrete pavement construction in California. Therefore, it is interesting to note that the sections included from California did not display significant spalling, despite the fact that the oldest section was 16 years old. Typical spalling amounts ranged from 0 percent to 10 percent of the joints spalled. Spalling appeared to be slightly less in the inner lane of the projects. However, it should be noted that in the southbound lanes of the I-5 project near Tracy, failure of several of the the 11-ft (3.4 m) slabs was attributed to the absence of longitudinal tiebars between lanes, the absence of joint sealant in the longitudinal and transverse joints, and poor drainage conditions.

Low annual rainfall, moderate temperature extremes, and the absence of deicing materials (salt, sand) applied to the roadway apparently all combine to provide for the good performance of nonsealed joints in California. However, further investigation is required to determine the long-term effects of nonsealed joints on concrete pavement performance.

Preformed Compression Seals

Preformed compression seals were used on several sections. The following summarizes some of the observed results on these sections.

- o Joints that used preformed compression seals included the MI 1 sections. Even though "D" cracking existed, the long-jointed MI 1-1 showed no medium- or high-severity joint spalling after 12 years. Epoxy-coated dowel bars were used at the joints.

- o Sections NY 1-8a and 1-8b were 22 years old, had no load transfer devices (to cause spalling), and had preformed compression sealants. None of the joints exhibited any medium- or high-severity spalling.

- o The OH 1 sections contained preformed compression sealants and displayed virtually no joint spalling. They were 14 years old at the time of survey.

- o The ONT 2 section contained preformed compression seals. After 16 years of service, it did not display any medium- or high-severity joint spalling.

Other Sealant Types

Rubberized asphalt was used as a joint sealant on many of the projects. Its performance ranged from fair to good and generally fell off after about 7 years. However, one section in California had a rubberized asphalt sealant that was performing well after 12 years of service.

A few projects included in the study contained silicone joint sealant. Although not very old (less than 5 years), the sealant was in very good condition.

Summary of the Effects of Joint Sealing

From the limited data, it appears that there may be some benefit gained from sealing joints in harsh climates. Preformed joint sealants were used quite extensively on the projects evaluated and have performed very well. They appear to be able to offer long-term service lives on the order of 10 to 15 years or more. Rubberized asphalt joint sealant typically showed good performance for 5 to 7 years. Silicone sealant was exhibiting good performance through 5 years of service.

California's mild climate, absence of deicing materials, and short joint spacing appears to allow for nonsealed joints to perform relatively well. However, little is known on the effect of nonsealed joints on long-term concrete pavement performance. In one case, nontied longitudinal joints, nonsealed longitudinal and transverse joints, and poor drainage conditions resulted in early pavement failures.

11. TIED PCC SHOULDERS/WIDENED LANES

Several sections included in the research project provided a direct comparison between a tied PCC shoulder and an AC shoulder. Tied PCC shoulders were also included on several other sections and can be included for evaluation.

There were no direct comparisons between sections with widened outside lanes and sections with concrete or asphalt shoulders. In fact, only four sections included in the project had widened outside lanes and these were all relatively new.

Minnesota 2

Four sections in all were constructed, two with tied PCC shoulders and two with AC shoulders. In terms of shoulder condition, PCC shoulders constructed on I-90 near Albert Lea are performing much better than AC shoulders. The PCC shoulders are in excellent condition with minor deterioration, whereas the AC shoulders were in poor condition, due to alligator cracking and lane shoulder dropoff. Some corner breaks and spalling along the longitudinal lane-shoulder joint occurred in the PCC shoulder, but this is attributed to the fact that the paint stripe delineating the transition between the pavement and outside shoulder was located on the shoulder, thus encouraging heavy trucks to ride the paint stripe and cause large edge deflections.

The effect of the concrete shoulders on the performance of the traffic lane was unclear. Very high load transfer across the longitudinal lane-shoulder joint was measured. However, much higher corner deflections were exhibited in the traffic lanes with the PCC shoulders.

The sections with PCC shoulders averaged 448 ft/mi (85 m/km) of longitudinal cracking, while the AC shoulder sections had no longitudinal cracking. The full tied width of a 15-ft (4.6 m) inner traffic lane, a 12 ft (3.7 m) outer traffic lane, and a 10 ft (3.0 m) outer shoulder may have resulted in high tensile stresses. These high tensile stresses are believed to have resulted in longitudinal cracking.

Arizona 1

The tied PCC shoulders on State Route 360 are in better overall condition than the AC shoulders. Corner deflections have been reduced by 25 percent by the PCC shoulders. No apparent influence on the performance of the traffic lane pavement was noted.

Michigan 1

This project on U.S. 10 near Clare included a section with a tied acceleration ramp. This section illustrated that if the shoulders (in this case, the acceleration ramp) are not adequately tied to the mainline pavement, or if poor drainage conditions exists, the shoulders will have no effect on traffic lane performance. Joint faulting on this section was similar to similar sections with AC shoulders.

Michigan 4

Two adjacent sections, one with tied PCC shoulders and the other with AC shoulders, were constructed on I-69 near Charlotte. The section with tied PCC shoulders exhibited higher joint faulting, lower crack faulting, and lower transverse cracking compared to the section with AC shoulders. There was no significant longitudinal cracking for either pavement type. Mainline performance data as influenced by the shoulders is summarized in table 90.

The condition of both shoulders was rated as fair. The tied PCC shoulders had smaller corner deflections (reduced by nearly 60 percent) and far fewer corners with loss of support (22 percent vs. 95 percent). However, for the section with the PCC shoulders, the load transfer efficiency measured across the lane/shoulder joint was only 35 percent. This may be partly due to the small tiebars that were not intended to provide load transfer and to the fact that the face of the pavement was finished very smooth to facilitate differential movement between the slab and shoulder. These factors probably contributed to reduce the beneficial effect of the PCC shoulder edge support.

New York 2

Both PCC and AC shoulders were constructed on I-88 near Otego. However, the pavement designs varied from JPCP to JRCP and thus a direct comparison as to the effect of a tied PCC shoulder is not possible. Very little faulting occurred for any section. However, the PCC shoulders were only 6-in (152 mm) thick which, coupled with the bathtub design, made them

Table 90. Influence of shoulder on performance of mainline pavement for Michigan 4 (Age = 15 years, ESAL's = 4.4 million).

Section	Shoulder Type	L.T. Eff,%	Corner Defl,mils	% Corner Voids	Joint Faulting,in	Crack Faulting,in
MI 4-1	PCC	51	15.6	22	0.12	0.20
MI 4-2	AC	67	27.1	95	0.06	0.31

susceptible to frost heave. The frost heave which occurred was believed responsible for the longitudinal cracking that occurred on each of the concrete shoulder sections.

Ohio 2

Two nondoweled 15-in (381 mm) JPCP sections on State Route 2 near Vermilion compared PCC and AC shoulders. Although the PCC shoulder was in better overall condition than the AC shoulder, it did not reduce transverse joint faulting; both sections exhibited faulting of 0.11 in (2.8 mm).

Ontario 1

PCC shoulders constructed on Highway 3N near Ruthven, Ontario were in much better overall condition than AC shoulders. The AC shoulders were cracked and badly deteriorated. The effect of the shoulders on traffic lane performance is confounded, however, by other pavement design variables. It should be noted that there was an average of 417 ft/mi (79 m/km) of longitudinal cracking on the section with PCC shoulders and only 20 ft/mi (3.8 m/km) on the section with AC shoulders. However, this may be due to the fact that the tied PCC shoulder actually "overhung" the lean concrete base. Thus, half of the shoulder rested on the LCB and the other half rested on a dense-graded aggregate base. The dissimilar support provided by the different materials undoubtedly contributed to the cracking.

California 3

The pavement sections on U.S. 101 near Geyserville had both tied and nontied PCC shoulders. The tied PCC shoulders exhibited higher longitudinal lane-shoulder joint load transfer efficiencies than the nontied pavement sections. The tied PCC shoulders were also shown to reduce corner deflections by 50 percent and appeared to reduce the amount of transverse cracking occurring in the mainline pavement. Joint faulting was about the same for sections with tied and nontied PCC shoulders. Practically no longitudinal cracking was noted on the sections with either tied or nontied shoulders.

Widened Lanes

Four pavement sections, MN 3, MN 4, MN 6, and CA 8, included widened lanes in their design. These sections are all in excellent condition. However, it should be noted that these sections are all less than 4 years old and have sustained less than 1.5 million ESAL applications. Therefore, given the current pavement condition and age and traffic limitations, it is impossible to draw any conclusions regarding the effect of widened lanes on pavement performance.

Summary of the Effects of Tied PCC Shoulders/Widened Lanes

The effect of a tied PCC shoulder on the adjacent traffic lane was mixed. However, it can be said that tied concrete shoulders appear to improve the performance of the adjacent traffic lane pavement, provided that an adequate tie is established and positive subdrainage is provided. A tied PCC shoulder on a dense-graded base can result in a bathtub design and reduce performance.

Corner deflections and the percent of corners with voids were usually less for pavements having tied PCC shoulders. In frost areas, the PCC shoulder should be the same thickness as the traffic lane to avoid differential frost heave and corresponding cracking. In several projects where a direct comparison could be made, there was a larger amount of longitudinal cracking on the sections with tied PCC shoulders than on the section with AC shoulders.

The overall shoulder condition exhibited by the concrete shoulders was consistently better than that of the AC shoulders. The AC shoulders exhibited extensive amounts of alligator cracking, other surface deterioration, and severe lane-shoulder dropoff. There was practically no patching observed on the PCC shoulders.

Due to the young age of the sections, no conclusions could be reached regarding the effect of widened lanes on concrete pavement performance.

12. SUBDRAINAGE

Very few projects directly compared drained and nondrained pavement sections. Those that did are included for discussion. The subdrainage provided to a pavement was accomplished either by the installation of longitudinal edge drains or by the inclusion of a drainable base layer and edge drains. Many of the projects that did not directly include subdrainage as a variable did include sections with quite different drainage coefficients (C_d). The difference in drainage coefficients was largely affected by the construction of different base types. These projects can also be compared.

Michigan 1

This experimental project allows a determination of the effect of drained sections and a section constructed with a permeable asphalt-treated base. Drained sections displayed 21 percent less faulting on average, reduced the number of slab corners exhibiting loss of support, and, for dense-graded asphalt-treated base courses, reduced the amount of spalling from 61 percent to 41 percent, due to better drainage and less "D" cracking.

The performance of the pavement section constructed with a permeable asphalt-treated base material was far superior to that of sections with dense-graded asphalt-treated base course in terms of faulting and joint spalling. The dense-graded aggregate section with dowels performed similarly to the permeable nondoweled asphalt base in terms of faulting and spalling. Most notably, faulting in the PATB section was held to 0.03 in (0.8 mm) without the use of dowel bars versus 0.20 in (5.1 mm) faulting for dense asphalt base. The composite k-value for the permeable asphalt-treated base was 468 pci (127 kPa/mm), about the same as for the dense-graded asphalt-treated base. Performance data for this project is summarized in table 91.

The high percentage of corners indicating a loss of support for the JPCP with either a granular base and doweled joints (95 percent) or a permeable asphalt base without dowels (75 percent), and high corner deflections, causes concern for the long-term performance of the sections under heavy traffic. Traffic for these sections has been relatively light, only 0.9 million ESAL's over a 12-year period.

Table 91. Performance data relative to drainage for Michigan 1 (Age = 12 years, ESAL's = 0.9 million).

Pavement Type	Base Type	Drained	Joint Faulting, in	% Joint Spalling	% Corner w/ Voids	Corner Defl, mils
JPCP	ATB	Yes	0.14	41	70	13.7
JPCP	ATB	No	0.19	62	100	20.4
JPCP	PATB	Yes	0.03	9	75	21.8
JRCP	AGG	Yes	0.05	0	0	6.7
JRCP	AGG	No	0.08	0	15	10.3
JPCP	AGG	No	0.04	12	100	31.8
JPCP	AGG	Yes	0.04	11	94	26.3

Positive drainage was provided by French drains installed in 1981, except for PATB section which contained edge drains.

Michigan 5

This newer single section on I-94 was built in 1984 with a permeable aggregate base course. After three years of service, the section is in poor condition, exhibiting extensive cracking. This is believed to be due, in part, to the fact that specifications allowed the permeable base to contain a low percentage of crushed material and an improper gradation. This resulted in poor support conditions which contributed to the poor performance of the section.

Arizona 1

On State Route 360 in Phoenix, no difference in performance was observed between sections with and without edge drains. However, the sections are only 5 years old and are placed in a mild, dry climate where edge drains may not be appropriate.

California 2

Section CA 2, on I-210 near Los Angeles showed that the construction of a strong permeable cement-treated base course can be accomplished (k = 1400 pci [380 kPa/mm]). However, the pavement consisted of a dense graded AC layer directly beneath the PCC slab and on top of the permeable CTB. This layer has apparently eroded or rutted at the joint which has contributed to the amount of faulting (0.11 in [2.8 mm]) observed on this pavement after only 4.4 million ESAL's.

Pennsylvania 1

On Route 66 near Kittanning, it appears that any positive means of removing water, either in the form of edge drains or a permeable base layer, is enough to provide good performance. However, the sections are only 7 years old and under relatively light traffic. The composite k-value for the permeable asphalt-treated base was 1000 pci (271 kPa/mm), much higher than that of either the cement-treated or aggregate bases.

New Jersey 3

Permeable base courses constructed on I-676 near Camden demonstrated good performance under heavy traffic (4.4 million ESAL's). No significant distress was observed. Composite k-values of 350 pci (95 kPa/mm) for the permeable aggregate base course and 210 pci (54 kPa/mm) for the permeable asphalt-treated base course were obtained.

Summary of the Effects of Subdrainage

Positive subdrainage, in the form of either edge drains or permeable base layers with edge drains, generally showed a marked effect in reducing faulting and other distresses, most notably "D" cracking. Several sections having a permeable base are performing very well. Most of these are under lighter traffic, however. The structural support provided by a permeable base is excellent at the slab center. However, the corner deflections are quite high and there is indication of loss of support at the corners also. Thus, it may be necessary to dowel these pavements when they are to be loaded with medium to heavy traffic.

Placement of the permeable drainage layer directly beneath the slab is critical, as shown by the results of CA 2. No intermediate layer of AC should be placed between the drainage layer and the slab, or the expected benefits of the permeable base may not be realized.

As previously discussed, for sections with a permeable base course, the importance of a properly-designed filter material beneath the base cannot be overemphasized. The absence of such a layer or an improperly-designed layer will allow the infiltration of fines into the permeable base course, resulting in the decreased permeability of that layer. In addition, maintenance of the outlets for the underdrains and the proper location of the underdrains (within the permeable base layer) are critical to prevent the ponding of water in the permeable bases. Finally, as illustrated by MI 5, the gradation of a permeable base course is critical to its performance. It must consist of a stable gradation to ensure that long-term performance is achieved.

Another way of looking at the performance data is to organize the projects with different base types by AASHTO drainage coefficient, C_d. The drainage coefficient is an overall measure of the drainability of the pavement and takes into account more than just the permeability of the base layer. Volume V discusses the determination of the drainage coefficients. Calculation of the drainage coefficient by a rational method allows these values to be used for comparative purposes between sections. In table 92, the projects with different base types are organized by drainage coefficient. Several key distresses are averaged for each base type, and the drainage coefficients for each section are reported for that type. For the pavements which have sustained enough traffic loads for the trends to become clear, it can generally be stated that as the drainage coefficient improves or increases, the load-related distresses decrease. This appears to hold true for joint faulting and transverse cracks with very few exceptions.

The different designs whose distresses are averaged in table 92 represent a wide range of design variables, none of which was the overall drainability of the section. The effects of these variables are obscured, however, by averaging the performance results by base type only. The trend that emerges suggests that the overall drainage of the section, almost regardless of the other variables, has a large effect on the performance of the section.

Table 92. Summary of selected performance data related to C_d.

LOCATION	ESAL's (millions)	BASE TYPE	JOINT FAULTING, in	% JOINT SPALLING	TRANSVERSE CRACKS/MI	AVERAGE C_d
DRY-FREEZE						
MN 1	5.5	CTB	0.27	19.0	22	0.80
		ATB	0.19	34.8	36	0.85
		AGG	0.19	10.3	28	1.05
MN 2	2.8	AGG	0.06	5.8	1	0.84
MN 3	1.5	AGG	0.02	0.0	0	0.80
WET-FREEZE						
MI 1	0.9	ATB	0.18	59.7	16	0.80
		AGG	0.05	5.8	1	0.90
		PATB	0.03	9.0	0	1.10
MI 3	2.8	PAGG	0.02	0.0	0	1.13
MI 4	4.3	AGG	0.09	3.0	205	0.73
MI 5	3.1	PAGG	0.05	0.0	144	1.05
NJ 2	35.0	AGG	0.06	20.0	24	1.05
NJ 3	4.2	PAGG	0.05	0.0	0	1.10
		PATB	0.06	43.0	0	1.10
NY 1	2.0	AGG	0.06	11.0	18	0.78
		ATB	0.05	19.8	2	0.93
NY 2	1.4	AGG	0.02	0.0	43	0.88
OH 1	3.4	ATB	0.07	6.5	74	0.90
		AGG	0.09	0.0	15	0.96
OH 2	3.3	NONE	0.11	19.0	5	0.92
ONT 1	1.0	NONE	0.05	0.0	0	0.95
		PATB	0.05	0.0	0	1.00
		LCB	0.04	0.0	38	1.00
ONT 2	36.0	CTB	0.01	0.0	5	0.80
PA 1	0.3	CTB	0.03	0.0	0	0.90
		AGG	0.03	0.0	0	1.02
		PATB	0.02	0.0	0	1.05
WET-NONFREEZE						
NC 1	9.1	CTB	0.16	0.0	0	0.75
		AGG	0.16	0.0	17	0.80
		SC	0.15	0.0	8	0.89
		ATB	0.05	0.0	0	0.90
NC 2	5.8	LCB	0.02	0.0	10	0.95
FL 2	2.0	AGG	0.01	0.0	0	0.80
FL 3	4.1	LCB	0.08	2.0	310	0.75

CHAPTER 5 SUMMARY AND CONCLUSIONS

Performance results obtained from a number of experimental projects constructed over the past 20 years were studied to assess the effect of a large number of pavement variables on pavement performance. Ninety-five pavement sections located in 4 major climatic regions were thoroughly evaluated. This analysis is not complete, as it will be extended to include statistical and mechanistic analyses of the field data. However, the interim results which are presented in this study provide some revealing insights into pavement performance.

1. SUMMARY OF THE EFFECT OF DESIGN FEATURES

Specific findings on the effect of several design features have been summarized in the text. Based upon these findings, some general conclusions can be made relative to the design and performance of jointed plain and reinforced concrete pavements.

The effect of <u>slab thickness</u> on pavement performance was unambiguous. It was found that increasing slab thickness helped to reduce transverse and longitudinal slab cracking in all cases, in every climatic zone. This effect was much more noticeable when slab thicknesses were increased an inch on thinner pavements, say from 8 in to 9 in (203 mm to 229 mm), then when they were increased several inches, from 9 to 11 in (229 to 279 mm) or 11 to 13 in (279 to 330 mm). It was not possible to directly compare the performance of the thinner slabs and the thicker slabs, as the thick slabs were all constructed directly on the subgrade and the thinner slabs were all constructed on base courses.

Increasing the thickness of the slab did not appear to reduce joint spalling or joint faulting. Thick slabs on grade, especially in wet climates and exposed to heavy traffic, faulted as much as thinner pavements constructed on a base course.

The <u>base type</u> seemed to have a very large effect on the performance of jointed concrete pavements. The aspects of base type which affected pavement performance included: base/slab interface friction; base stiffness; base erodibility; and base permeability or drainability. The major performance indicators which were affected by variations in base type were transverse and longitudinal cracking, joint spalling, and faulting.

The worst-performing base type was the cement-treated or soil-cement bases. These tended to exhibit excessive pumping, faulting, and cracking. This is most likely a result of the use of an impermeable layer that traps moisture and yet can break down and contribute to the movement of fines beneath the slab.

The use of lean concrete bases generally produced poor concrete pavement performance. Large curling and warping stresses have been associated with slabs constructed over lean concrete bases. These stresses result in considerable transverse and longitudinal cracking of the slab. These bases can also contribute to a bathtub section in which moisture is trapped within the pavement cross section, particularly where there is poor subdrainage.

Dense-graded, asphalt-treated base courses ranged in performance from very poor to good. The fact that these type of bases were often constructed as a bathtub-type design contributed to their poor performance. This improper design often resulted in severe cracking, faulting, and pumping. However, a few sections displayed good performance, perhaps due to an adequate cross section. More investigation is required before any recommendations or guidelines concerning the use of asphalt-treated base courses are proposed.

The construction of thicker slabs directly on the subgrade (no base) resulted in a pavement that performed marginally. These pavement were especially susceptible to faulting, even under low traffic levels. However, the performance of these pavements is not directly comparable to other pavement sections, as the slabs were thicker (11 in [279 mm] to 15 in [381 mm]) than conventional pavements.

Pavements constructed over aggregate bases had varied performance, but were generally in the fair to very good category. In general, the more open the gradation of the aggregate in the base, the better the pavement performed. An advantage of the aggregate bases is that they contribute the least to the high curling and warping stresses in the slab. Also, even aggregate bases that are not open-graded tend to perform fairly well because they are more permeable and have a lower friction factor than stabilized base courses.

The best bases in terms of pavement performance were those designed to be permeable. Typical base courses have permeabilities ranging from 0 to less than 1 ft/day (0.3 m/day); good permeable bases have permeabilities upwards of 1000 ft/day (305 m/day). The highly permeable bases typically performed very well, although it should be noted that the sections are generally not very old and have not been subjected to a significant amount of traffic. Specific areas of concern, however, were the high corner deflections and the low load transfer exhibited by the permeable base courses. Since these could impact their long-term performance, the use of dowel bars may be required for these types of bases.

The analysis of the effect of permeable base types is confounded by the presence of other design variables, but the limited data available suggests that the use of a permeable base layer may provide an effective means of improving pavement performance. An unexpected benefit of the use of permeable bases was the reduction in "D" cracking on pavements susceptible to that distress.

A carefully-designed filter material is recommended beneath the permeable base course. Such a filter layer will prevent the intrusion of fines into the permeable base. If the permeable base becomes contaminated with fines, then it will no longer remove water as quickly from the pavement structure. Regular maintenance of the drainage outlets is necessary to allow for the water to be removed from the pavement structure. Finally, the edge drains placed to remove the water from the permeable base layer must be located within the permeable base so that water can reach the drain and be removed.

Several experimental projects evaluated the effects on performance of different **slab lengths**. The range of slab lengths for JPCP pavements was from an average of 7.75 ft (2.4 m) to 30 ft (9.1 m). For JPCP pavements, reducing the slab length reduces both the magnitude of joint faulting and the amount of transverse cracking. Shorter slabs may also reduce joint spalling, although there was insufficient data to support this conclusion. On sections with random joint spacing, it was found that if the longer slabs were greater than 18 ft (5.5 m) long, they experienced more transverse cracking than the shorter slabs.

Jointed reinforced concrete pavements (JRCP) included in this study ranged from 21 ft (6.4 m) to 78 ft (23.8 m) long slabs. Generally, the shorter joint spacings performed better, as measured by deteriorated transverse cracks, joint faulting, and joint spalling. However, several long-jointed JRCP slabs performed quite well. In particular, the long-jointed pavements in New Jersey which were constructed with expansion joints displayed excellent performance. Factors to consider for enhanced JRCP performance are the amount of reinforcement, the dowel coating, the base type, the durability and top size of the aggregate, and the coefficient of expansion of the concrete.

An examination of the stiffness of the foundation was made through the use of the radius of relative stiffness. Generally speaking, when the ratio of L/l was greater than 5, transverse cracking occurred within the slab. This factor was further examined for different base types, which showed that stiffer base courses required shorter joint spacing to reduce or eliminate transverse cracking.

The amount of **reinforcement** appeared to have an effect in controlling the amount of deteriorated transverse cracking. Although often confounded by the presence of corrosion-resistant dowel bars, pavement sections that contained more than 0.1 percent reinforcing steel exhibited less deteriorated transverse cracking; sections with less than that amount often displayed a significant amount of transverse cracking, particularly in cold climates. A minimum of 0.1 percent reinforcing steel is therefore recommended, with larger amounts required for harsher climates and longer slabs. Corrosion-resistant dowel bars are also recommended. The combination of corrosion-resistant dowel bars, an adequate amount of reinforcement, and preformed compression seals appears to enhance the performance of jointed reinforced concrete pavements.

The issue of **joint orientation** has been of interest in pavement design for many years. Conventional wisdom has it that skewed joints reduce the number of wheel loads being applied to the pavement from two to one and thus may reduce load-associated distresses. While the results from the limited sample size in this study are ambiguous, all of the nondoweled sections with skewed joints had a lower PSR than similar designs with perpendicular joints. The available data provides no definitive conclusions on the effectiveness of skewing transverse joints for nondoweled slabs. Skewed joints, and random joint spacing for that matter, are not believed to provide any benefit to doweled slabs.

Many sections in the study allowed for the direct comparison of the two methods of **transverse joint load transfer**. Dowel bars were found to substantially reduce the amount of joint faulting when compared directly with nondoweled sections of otherwise similar design. The impact of load transfer devices becomes more significant at higher traffic levels.

The diameter of the dowel bar had an effect on performance, as larger diameter bars provided better load transfer and control of faulting under heavy traffic than did smaller dowels. It appeared that a minimum 1.25-in (32 mm) diameter dowel bar was necessary to provide good performance.

Nondoweled JPCP slabs generally developed significant faulting, regardless of pavement design or climate. This effect was somewhat mitigated by the use of permeable bases. However, the sections in this group had a much lower number of accumulated ESAL's, so no conclusions can be drawn yet.

Some sections with dowels displayed higher levels of joint spalling and more deteriorated cracks than sections of similar design without load transfer devices. It is strongly suspected that the load transfer devices in these sections have corroded and locked up, or were misaligned at the time of placement. This has resulted in the development of working cracks instead of working joints. However, the use of dowel coatings that inhibit corrosion and the proper placement of the load transfer devices will certainly contribute to improved performance.

Two projects in New York used load transfer devices other than dowels. On these particular projects, these nonconventional devices performed fairly well, but the pavements have not been subjected to a large number of accumulated ESAL's. For instance, on other high-volume roadways, it is known that the ACME devices were unable to provide adequate load transfer and failed. At this time, circular, corrosion-resistant dowel bars are believed to provide the best means of transverse joint load transfer.

Dowel bar coatings are applied to dowel bars to provide resistance to corrosion brought about by moisture and deicing chemicals. While most of the sections in this study did not contain corrosion-resistant dowel bars, those that did generally exhibited enhanced performance. Very little deteriorated transverse cracking was identified on these sections, which would seem to indicate that the transverse joint was still functioning properly. In fact, one section in New Jersey with stainless-steel clad dowel bars was performing satisfactorily after 36 years of service.

The **longitudinal joint design** was found to be a critical design element. Both inadequate forming techniques or insufficient longitudinal joint depths can contribute to the development of longitudinal cracking. There is evidence to support the advantage of sawing techniques over the use of inserts. However, no clear trend was identified pertaining to the minimum depth needed to ensure the formation of the longitudinal joint. The presence of stabilized base courses also contributed to the development of longitudinal cracking, and their use makes the timing of sawing operations even more critical, as the joint must be established before a strong bond develops. The depth of the longitudinal joint is generally recommended to be one-third of the actual (not plan) slab thickness, and may have to be deeper when stabilized bases are included in the design.

Joint sealing appeared to have a beneficial effect on the performance of concrete pavements. This was particularly true in harsh climates with excessive amounts of moisture. Preformed compression sealants were shown to perform for 15 or more years on heavily-trafficked pavements. Except where a "D" cracking problem existed, the pavement sections containing the preformed sealants generally exhibited little joint spalling and were in good overall condition.

Rubberized asphalt joint sealants showed good performance for 5 to 7 years, although falling off after that time. Silicone joint sealant was included on a few projects and was performing well for up to 5 years of service.

It is a widespread belief that the construction of tied shoulders on jointed concrete pavements serves to reduce edge stresses and edge and corner deflections by providing more lateral support to the mainline pavement. This is thought to improve pavement performance and increase pavement life. Surprisingly, this study showed that while portland cement concrete (PCC) shoulders performed better than asphalt concrete shoulders, many of the tied shoulder designs were deficient and actually contributed to poorer performance in the mainline pavement. Several factors are critical if the benefits of tied concrete shoulders are to be realized. The spacing of the tiebars must not be too great. A spacing of 40-in (1016 mm) on NY 2 was found to be too great for the shoulder to provide support to the mainline pavement. Tiebars must be strategically located near slab corners to provide support. Also, in areas susceptible to frost heave action, it is recommended that the shoulder have the same thickness as the slab to avoid the effects of differential frost heave.

The use of tied shoulders on concrete pavements also allows for the establishment of a sealable joint reservoir between the mainline pavement and the shoulder. This joint is the major source of moisture in a pavement and sealing of this joint should help to reduce the occurrence and deterioration of moisture-related distresses.

Tied PCC shoulders did not appear to have any effect on the faulting of the transverse joints in the mainline pavement. In some cases, tied PCC shoulders were constructed over a stabilized dense-graded base course in a bathtub design that resulted in the poor performance of the mainline pavement.

Widened traffic lanes are expected to enhance concrete pavement performance by reducing edge and corner loadings. By allowing for a more interior-loading condition, edge and corner stresses are reduced. However, the sections that incorporated widened lanes in their design were relatively new and had been subjected to low traffic loadings; hence, the effect of widened lanes on concrete pavement performance could not be evaluated. Nevertheless, it is believed that the use of widened lanes should be considered in the design of a concrete pavement.

One area of pavement design for which little guidance exists is subdrainage. The provision of positive subdrainage, either in the form of longitudinal edge drains or the combination of a drainage layer and edge drains, generally resulted in a reduction in faulting and spalling related to "D" cracking.

This study also evaluated subdrainage indirectly, by the rational calculation of a coefficient of drainage (C_d) for every pavement section. This coefficient not only takes into account the permeability of the layer directly beneath the slab, it also considers the permeability of the other layers, the presence of edge drains, the transverse and longitudinal slopes of the pavement surface, the drainage distance, and the climate.

The results showed that, with very few exceptions, as the drainage characteristics of the pavement sections improved, the load-related distresses decreased. This was especially true for faulting and transverse cracking. The effect of improved drainage characteristics on other performance indicators will be examined in greater depth. However, it appears that overall pavement performance can be improved by the construction of a base layer with a high drainage coefficient. Two major components of such a base are an open-graded material with a high permeability and the restriction of the percent fines. A filter layer must be provided below the permeable base and regular maintenance of the outlets must be performed.

The trends that are reported above result from preliminary analyses of the data collected. More thorough mechanistic and statistical analyses are required to remove the effects of confounding factors and also allow comparison of projects with different traffic levels located in different areas. The goal is to develop specific conclusions regarding the effect of the variables studied in this project on pavement performance. Performance models will be developed using the significant effects and significant interactions between these effects to aid in the determination of performance curves, estimates of remaining life, and life-cycle cost estimates. Volume II documents the results of this investigation.

2. FUTURE RESEARCH

A great deal of information, knowledge, and insight on the design and performance of jointed concrete pavements emerged from this study. However, as is often the case, there were many areas where the results were either inconclusive or could not adequately be assessed due to the limited number of sections (and associated design features) included in the study. The following list has been prepared to identify those areas of concrete pavement design and performance requiring further or additional study:

- Ongoing evaluations of permeable base courses (both stabilized and nonstabilized) to determine their long-term performance capabilities and the incorporation of their design considerations into current design procedures.

- Detailed drainage studies in each climatic region to quantify the life extension gained by providing positive subdrainage and to determine the appropriateness of the range of C_d values.

- A more thorough evaluation of slab/base frictional characteristics and their effect on slab cracking.

- An analysis of the effectiveness of skewing joints.

- A long-term evaluation of the effectiveness of corrosion-resistant dowel bars.

- Performance evaluations regarding the effect of widened lanes on long-term concrete pavement performance.

- A thorough evaluation of the longitudinal joint design and forming procedures, including the depth of joint, type of base, timing of sawing, and the effect of large temperature changes during curing.

- Improved guidance on the selection of joint spacing for JRCP.

- More complete analysis of the effectiveness of different types of joint sealant materials.

- Collection and examination of mid-slab edge deflections.

- Further evaluation of the accuracy of weigh-in-motion (WIM) studies.

- An evaluation of the effect of the concrete coefficient of expansion, the maximum aggregate size, and the aggregate quality (durability) on concrete pavement performance.

To address some of these areas, the current project database could be extended by including additional sections from less well-represented regions, particularly with specific design variables. This would allow for an evaluation of many of the above items by providing a more balanced representation of design features.

3. SUGGESTIONS FOR FUTURE DATA COLLECTION AND TESTING

Every effort was made to make the data collection and testing procedures for this project as complete as possible. However, as is often the case, several uncollected data items and unperformed tests were later identified which would have contributed to the performance evaluation of the projects. These items include:

- Actual field measurements of the transverse and longitudinal joint depths.

- Mid-slab edge deflections with the FWD.

- Measurements of the volume of the cores before and after subjecting them to heating and cooling to determine the thermal coefficient of expansion.

- Distinction between deformed and smooth welded wire reinforcement.

- Site-specific temperature data for the months of the concrete paving and curing.

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