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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), deflections, 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 detailed summary reports documenting the design, construction, and performance of the 95 concrete pavement sections included in the study. Also presented are summary tables which compile the design and performance data from every section into a convenient tabular format.

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

FHWA No.	Vol. No.	Short Title
FHWA-RD-89-136	I	Evaluation of Concrete Pavement Performance and Design Features
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-89-140	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

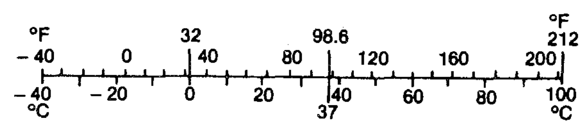
Symbol	When You Know	Multiply By	To Find	Symbol
	LENGTH			
in	inches	25.4	millimetres	mm
ft	feet	0.305	metres	m
yd	yards	0.914	metres	m
mi	miles	1.61	kilometres	km
	AREA			
in^2	square inches	645.2	millimetres squared	mm^2
ft^2	square feet	0.093	metres squared	m^2
yd^2	square yards	0.836	metres squared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

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

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

APPROXIMATE CONVERSIONS FROM SI UNITS

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



* SI is the symbol for the International System of Measurement

(Revised April 1989)

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CHAPTER 1 INTERSTATE 94 — ROTHSAY, MINNESOTA

1. INTRODUCTION

This experimental project, located on I-94 near Rothsay, Minnesota, was constructed in 1970. The experiment was conducted by the Minnesota Department of Transportation (Mn/DOT) to evaluate the effects of base type, slab thickness, and load transfer on concrete pavement performance. Design variables include aggregate, asphalt-treated, and cement-treated base courses; 8- and 9-in (203 and 229 mm) pavement slabs; and doweled and nondoweled joints. The cement-treated base course was stabilized with 5 percent portland cement; the asphalt-treated base consisted of 5 percent AC-1, 120/150 pen. All sections were reinforced, had 27-ft (8.2 m) skewed joints, and were constructed on a clay loam/clay subgrade (AASHTO Classification A-4 and A-6).

Twelve sections in the westbound direction, designated as MN 1-1 through MN 1-12, were surveyed and tested in 1987. In addition, MN 5, a "control" section (9-in [229 mm] slab over granular base, doweled joints, and 40-ft [12.2 m] joint spacing at right angles), was included for comparison with the 27-ft (8.2 m) joint spacing of the experimental sections. It is located on I-94 near the experimental sections and shares the same traffic levels. A complete listing of the original design and construction information is provided in the summary tables in chapter 32.

2. CLIMATE

The pavement sections are located in western Minnesota in the dry-freeze environmental zone. The project site has a Thornthwaite Moisture Index of 0, a Corps of Engineers Freezing Index of 2188, and receives 23 in (584 mm) of rainfall annually. The highest average monthly maximum temperature is 84 ^{o}F (29 ^{o}C) while the lowest average monthly minimum temperature is -3 ^{o}F (-19 ^{o}C).

Climatological data compiled at Rothsay during July and August 1970 (the period that the project was paved) indicated that the maximum daily temperatures ranged from 47 ^{o}F to 97 ^{o}F (8 ^{o}C to 36 ^{o}C), and the minimum daily temperatures ranged from 30 ^{o}F to 72 ^{o}F (-1 ^{o}C to 22 ^{o}C).

3. TRAFFIC

The roadway is a four-lane, divided highway (two lanes in each direction), and has a functional classification of Rural Interstate. Since being opened to traffic in 1970, the pavement has 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. These figures represent estimates through 1987.

The two-way Average Daily Traffic (ADT) in 1986 was approximately 5000 vehicles per day, including 21 percent heavy trucks. Weigh-in-motion data collected at the project site in 1987 provided a traffic count of 5100 vehicles per day, including 30 percent heavy trucks.

4. MAINTENANCE AND REHABILITATION

Rehabilitation was performed on the project in 1984. Due to severe faulting of the nondoweled pavement sections, diamond grinding was performed on those sections. A tied concrete edge beam, 2.5-ft (0.61 m) wide, 6-in (152 mm) thick and containing three 1-in (25 mm) diameter dowel bars, was added to those nondoweled pavement sections which were constructed over either granular or cement-treated base courses. Partial-depth concrete repairs were placed at both transverse and longitudinal cracks and some joints, to repair localized spalling distress. Finally, all random cracks and transverse joints were sealed with a hot-poured rubberized asphalt sealant.

5. PHYSICAL TESTING RESULTS

Cores from slab centers and from typical transverse joints were retrieved from the pavement sections in 1987. The center cores were tested in split tensile to provide an indication of concrete strength. The joint cores were visually inspected for signs of deterioration beneath the joint or for any signs of material durability distress (microcracking in the coarse aggregate).

The split tensile strength values were used to obtain an estimate of the modulus of rupture. A mean modulus of rupture of 688 psi (4.7 MPa) was calculated for all sections.

The joint cores taken from the sections indicated that a small amount of deterioration was present directly beneath the joint, particularly for the nondoweled pavement sections. The joint cores also displayed microcracking in the coarse aggregate ("D" cracking) for almost every section.

Deflection testing was performed on the sections in 1987 for purposes of layer moduli characterization, determination of load transfer efficiencies, and void detection. The elastic modulus of the concrete (E), the composite k-value, and the load transfer values are summarized for each section in the summary tables in chapter 32.

The elastic modulus values of the slab (obtained from FWD testing) averaged 7,690,000 psi (53,020 MPa) for the experimental sections and 7,560,000 psi (52,130 MPa) for the control section (MN 5). The composite dynamic k-values (on top of the base) varied somewhat for the different base types. The aggregate base was 200 pci (54 kPa/mm), the asphalt-treated base was 296 pci (80 kPa/mm), and the cement-treated base was 265 pci (72 kPa/mm).

There was no subbase constructed beneath any of the bases. As previously mentioned, the subgrade is an AASHTO Classification A-4 to A-6 material. The k-value for MN 5 (aggregate base) was calculated to be 156 pci (42.3 kPa/mm).

As would be expected, the average load transfer efficiencies were higher for the doweled pavement sections (64 percent) than for the nondoweled pavement sections (46 percent). Load transfer for MN 5 (doweled joints) was 62 percent. However, for doweled joints, load transfer efficiencies of 62 to 64 percent are not considered high. It is likely that the 1-in (25 mm) dowels are of insufficient size to contribute significantly to load transfer.

In addition, voids under the slab corners were identified using deflection-based void detection procedures. The pavement sections exhibiting voids are shown in table 1. Voids were not detected under any corners of the doweled pavement sections.

Table 1. Corner voids on MN 1 sections at Rothsay.

Section	Base Type	Dowels	% Corners Exhibiting Voids
MN 1-3	Aggregate	No	7
MN 1-9	CTB	No	14
MN 1-11	CTB	No	29

6. DRAINABILITY OF PAVEMENT SECTIONS

A drainage analysis was performed on each section to evaluate the section's ability to remove water from the pavement structure. It was found that the subgrade was surprisingly well-draining and that the sections are in an area that has the potential for the pavement to be saturated only about 10 percent of the time. Table 2 summarizes the findings of the drainage analysis.

Table 2. Drainage summary for MN 1 and MN 5 sections.

Section	Base Permeability, ft/hr	AASHTO Drainage Coefficient	Overall Drainability
MN 1-1	0.62	1.05	Fair-Good
MN 1-2	0.17	1.05	Fair-Good
MN 1-3	0.72	1.05	Fair-Good
MN 1-4	0.78	1.05	Fair-Good
MN 1-5	0.00	0.85	Fair
MN 1-6	0.00	0.85	Fair
MN 1-7	0.00	0.85	Fair
MN 1-8	0.00	0.85	Fair
MN 1-9	0.00	0.80	Fair
MN 1-10	0.00	0.80	Fair
MN 1-11	0.00	0.80	Fair
MN 1-12	0.00	0.80	Fair
MN 5	0.78	0.95	Fair

The aggregate base course sections were somewhat drainable and thus contributed to the drainability of those sections. Other factors contributing to the drainability of the sections are the well-drained subgrade, the small amount of time during the year that the pavement has the potential to be saturated, and the fact that the aggregate base beneath the shoulder was "daylighted" to the ditchline. The control section, MN 5, had slightly less drainability than its aggregate base course counterparts because of the presence of a dense-graded subbase located beneath the base course.

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.				WITHOUT MN 5	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 1. Outer lane performance data for Minnesota 1.

MINNESOTA 1 AND MINNESOTA 5
I-94 ROTHSAY

INNER 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.
		PSR	3.3	3.6	3.4	3.5	3.4
		ROUGHNESS, in/mi	108	100	124	86	104
		FAULTING , in	0.06	0.02	0.03	0.03	0.04
		T. CKS./MI.	0	47	0	0	12
		LONG. CKS., ft/mi	0	0	0	0	0
		% JT. SPALL.	24	8	8	4	15
	ATB		MN 1-5	MN 1-6	MN 1-7	MN 1-8	
		PSR	3.4	3.3	3.5	3.5	3.4
		ROUGHNESS, in/mi	98	92	95	80	91
		FAULTING , in	0.07	0.03	0.04	0.04	0.05
		T. CKS./MI.	16	0	0	55	18
		LONG. CKS., ft/mi	880	1299	1190	2644	1503
		% JT. SPALL.	16	23	15	81	34
	CTB		MN 1-11	MN 1-12	MN 1-9	MN 1-10	
		PSR	3.2	3.4	3.4	3.4	3.3
		ROUGHNESS, in/mi	50	104	118	88	90
		FAULTING , in	0.04	0.02	0.03	0.03	0.03
		T. CKS./MI.	0	40	0	35	19
		LONG. CKS., ft/mi	2874	842	0	0	929
		% JT. SPALL.	74	50	15	18	39
40 ft JT. SPACING	AGG. BASE					MN 5	
		PSR				3.7	3.7
		ROUGHNESS, in/mi				94	94
		FAULTING , in				0.04	0.04
		T. CKS./MI.				33	33
		LONG. CKS., ft/mi				0	0
		% JT. SPALL.				23	23

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
PSR	3.3	3.4	3.4	3.5	3.5
ROUGHNESS, in/mi	85	99	112	85	87
FAULTING , in	0.06	0.02	0.03	0.03	0.04
T. CKS./MI.	5	29	0	30	31
LONG. CKS., ft/mi	1251	714	397	881	661
% JT. SPALL.	38	27	13	34	32

ALL DATA FROM 1987 CONDITION SURVEY.

Figure 2. Inner lane performance data for Minnesota 1.

7. DETERIORATION OF PAVEMENT SECTIONS

The results of the distress survey conducted on each pavement section are given in the summary tables. The primary performance indicators are tabulated in figure 1 (outer lane) and figure 2 (inner lane). The relative performance of each pavement section with respect to the primary performance indicators are discussed below for the outer lane only.

Joint Spalling

The amount of medium- and high-severity joint spalling was larger for the asphalt-treated base course sections than for the aggregate or cement-treated base course sections. It was also larger for doweled sections than for the nondoweled sections. In addition, the 8-in (203 mm) sections had a much higher incidence of spalling than the 9-in (229 mm) sections. These observations are illustrated in table 3.

Table 3. Transverse joint spalling, by design variable.

Distress	AGG	ATB	CTB	Dowels	No Dowels	8-in Slab	9-in Slab
Joint Spalling, %	10	35	19	27	15	24	19

Finally, there was more joint spalling on the control section (40-ft [12.2 m] joint spacing) than on the 27-ft (8.2 m) section with the same design features (9-in [229 mm] slab, aggregate base course, dowel bars).

Best Performance

The pavement sections displaying the best performance, in terms of joint spalling, had the following characteristics:

- 9-in (229 mm) slab on any base course without load transfer devices.

Worst Performance

The worst performance in terms of joint spalling was exhibited by the pavement sections with the following design features:

- Any 8- or 9-in (203 or 229 mm) slab on asphalt-treated base course with load transfer devices.

Joint Faulting

Joint faulting was larger for the cement-treated base course sections than for either the aggregate or asphalt-treated base course sections. In addition, the amount of faulting for the nondoweled sections was much larger than for the doweled sections. Also, the 8-in (203 mm) sections exhibited slightly larger faulting than the 9-in (229 mm) sections. These results are tabulated in table 4. Finally, the faulting of the 40-ft (12.2 m) control section was greater than that of the comparable 27-ft (8.2 m) sections.

Table 4. Transverse joint faulting, by design variable.

Distress	AGG	ATB	CTB	Dowels	No Dowels	8-in Slab	9-in Slab
Joint Faulting, in	0.19	0.19	0.27	0.06	0.36	0.21	0.20

It should be noted that the faulting values for the outer lane of the 12 experimental sections represents those collected by the Minnesota Department of Transportation prior to the diamond grinding and rehabilitation performed in 1984. The faulting values shown for the inner lane are from 1987.

Best Performance

Pavement sections with the best performance, in terms of joint faulting, had the following design features:

- o Any 8- or 9-in (203 or 229 mm) slab on either aggregate or asphalt-treated base course with load transfer devices.

Worst Performance

The pavement sections displaying the worst performance, in terms of joint faulting, had the following characteristics:

- o Any 8- or 9-in (203 or 229 mm) slab on cement-treated base course without load transfer devices.

Transverse Cracking

Transverse cracking is measured by the number of deteriorated (medium- and high-severity) 12-ft (3.7 m) cracks per mile. It was observed to be much greater for the asphalt-treated base course sections than for either the aggregate or cement-treated base course sections. It also was observed to be greater for the sections with dowel bars than for those without dowel bars. The amount of deteriorated transverse cracks was about the same for the 8- and 9-in (203 and 229 mm) sections. Table 5 summarizes these results.

Table 5. Transverse cracking, by design variable.

Distress	AGG	ATB	CTB	Dowels	No Dowels	8-in Slab	9-in Slab
Transverse Cracks/mile	26	36	21	43	12	27	28

It was also observed that there were more transverse cracks on the 40-ft (12.2 m) control section than on the 27-ft (8.2 m) section of the same design. The longer jointed section had 0.04 percent reinforcement mesh, as compared to the 0.08 to 0.09 percent found in the shorter slabs. This low level of reinforcement, combined with the small diameter, corrodible dowels in the doweled sections, may help to explain the higher level of cracking in these sections.

Best Performance

In terms of deteriorated transverse cracks, the best performance was exhibited by pavement sections with the following characteristics:

- Any 9-in (229 mm) slab without load transfer devices.

Worst Performance

The worst performance, in terms of deteriorated transverse cracks, was displayed by pavement sections with the following design features:

- 9-in (229 mm) slab over asphalt-treated base course with load transfer devices.

Longitudinal Cracking

When evaluating longitudinal cracking, the total amount occurring in both traffic lanes was added together for each pavement section. This provides a clearer indication of the effect of the different design features on longitudinal cracking, as table 6 shows.

Table 6. Longitudinal cracking, by design variable.

Base Type	8-in Nondoweled	8-in Doweled	9-in Nondoweled	9-in Doweled
AGG	301	0	0	0
ATB	2656	3372	2958	5233
CTB	4260	1018	0	0

The amount of longitudinal cracking, measured in feet per mile, was much larger for the asphalt-treated base course sections than for either the aggregate or cement-treated base course sections. The aggregate base course sections had almost no longitudinal cracking while the cement-treated base course sections had about half that of the asphalt-treated base course sections. The amount of longitudinal cracking was approximately the same for the sections with and without dowel bars. The incidence of longitudinal cracking was higher for the 8-in (203 mm) sections than for the 9-in (229 mm) sections. Finally, there was far more longitudinal cracking on the 40-ft (12.2 m) section than on the 27-ft (8.2 m) section.

The longitudinal joint for all sections was formed using a plastic insert placed to a depth of 2.75 in (70 mm). This 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.

Best Performance

The pavement sections with the best performance, in terms of longitudinal cracking, had the following characteristics:

- Any 8- or 9-in (203 or 229 mm) slab on aggregate base course.

- Any 9-in (229 mm) slab on cement-treated base course.

<u>Worst Performance</u>

The worst performance, in terms of longitudinal cracking, was recorded by pavement sections with the following design features:

- Any 8- or 9-in (203 or 229 mm) slab on asphalt-treated base course.

- Any 8-in (203 mm) slab on cement-treated base course.

Present Serviceability Rating (PSR) and Roughness

Since 1984 (pre-grinding) PSR and roughness values were not available, the rideability of the diamond ground sections (all nondoweled sections) could not be evaluated with regard to the other sections. However, for those sections not diamond ground (including the 40-ft (12.2 m) section), there is no discernible difference between the PSR or roughness values.

Other Distress Types

In addition to the distress types previously discussed, there was low severity "D" cracking 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 within most of the sections, and medium severity pumping was reported for one asphalt-treated base course section.

8. EFFECT OF DESIGN FEATURES ON PAVEMENT PERFORMANCE

Each of the various design features had an influence on the performance of the pavement. The relative effect of these design features is summarized below.

Base Type

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

Joint Load Transfer

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 devices were too small in diameter. Also, the sections without load transfer devices actually had less joint spalling and less deteriorated transverse cracking than the doweled sections.

Slab Thickness

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 in the smaller amount of cracking (both transverse and longitudinal) as compared to the 8-in (203 mm) sections.

Joint Spacing

A direct comparison of joint spacing can be made when including the 40-ft (12.2 m) control section. 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, considering the amount of cracking (both transverse and longitudinal), spalling, and faulting.

9. COMPARISON OF OUTER LANE AND INNER LANE PERFORMANCE

Although being subjected to far fewer traffic loadings than the outer lane, the inner lane sections still can provide some insight into the performance of the design sections. In fact, many of the performance trends observed in the outer lane hold true for the inner lane. For instance, the asphalt-treated base course sections demonstrated the worst overall deterioration; the doweled sections performed better than the nondoweled sections; the 9-in (229 mm) pavement sections exhibited superior performance over the 8-in (203 mm) sections; and 27-ft (8.2 m) sections outperformed the 40-ft (12.2 m) sections in every distress category. An overall comparison of the two traffic lanes by base type is provided in table 7.

Table 7. Comparison of performance by lane.

AGGREGATE BASE COURSE

Variable	Inner Lane	Outer Lane
ESAL's (millions)	0.5	5.5
Joint Spalling, %	15	10
Joint Faulting, in	0.04 (1987)	0.19 (1984)
Trans. Cracks/mile	12	26
Long. Cracks, ft/mile	0	75

ASPHALT-TREATED BASE COURSE

Variable	Inner Lane	Outer Lane
ESAL's (millions)	0.5	5.5
Joint Spalling, %	34	35
Joint Faulting, in	0.05 (1987)	0.19 (1984)
Trans. Cracks/mile	18	36
Long. Cracks, ft/mile	1503	2051

Table 7. Comparison of performance by lane. (Cont'd)

CEMENT-TREATED BASE COURSE

Variable	Inner Lane	Outer Lane
ESAL's (millions)	0.5	5.5
Joint Spalling, %	39	19
Joint Faulting, in	0.03 (1987)	0.27 (1984)
Trans. Cracks/mile	19	44
Long. Cracks, ft/mile	929	391

Overall, the two traffic lanes exhibited similar performance trends in the relative amounts of the various performance indicators. Slightly larger amounts of joint spalling occurred in the inner lane than the outer lane, indicating the insensitivity of spalling to traffic. Also, the faulting for the inner lane is far less than the outer lane. Longitudinal cracking is about the same for the two lanes, while there are more deteriorated transverse cracks in the outer lane than the inner lane. It is believed for reinforced pavements that once a crack break downs in the outer lane, it propagates to the inner lane as the reinforcement is ruptured across the lane.

10. SUMMARY AND CONCLUSIONS

The most obvious 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 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.(1) The performance of the aggregate base course sections may have been enhanced by the better overall drainage characteristics of those sections.

There was an extensive amount of longitudinal cracking observed within the pavement sections. The cause of the longitudinal cracking 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 centerline joint was formed with a plastic insert, which 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 Minnesota Department of Transportation 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 techniques are needed for pavement slabs constructed over treated base courses.

It is also known that the asphalt-treated base courses were not "whitewashed," resulting in large AC temperatures at laydown. 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 nighttime every other joint), and the temperature at the time of paving. Temperatures at the time of paving may have ranged as high as the 80's; temperatures as low as 30 ^{o}F (-1 ^{o}C) were also encountered during the paving period. This temperature differential may have contributed to the large amount of cracking which occurred on the project.

The study also indicates the benefit of doweling 27-ft (8.2 m) transverse joints in this 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 the 1-in (25 mm) diameter dowels are not sufficient to provide reliable load transfer efficiencies.

The nondoweled pavement sections had less spalling and transverse cracking than the doweled pavement sections. Since the dowel bars utilized on this project had no treatment to inhibit corrosion, this is probably a sign that the dowel bars have corroded and "locked-up," resulting in spalling at the joint above the dowel bar and also serving to open any transverse cracks in the adjacent panel. The use of coated dowel bars is recommended to inhibit corrosion of the devices.

Another conclusion that can be drawn from the study 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. However, the performance of these reinforced sections was perhaps adversely influenced by the small amount of reinforcing steel in the pavement (less than 0.10 percent). This small amount of steel is probably insufficient for the temperature extremes experienced by a pavement in this climate.

11. ADDITIONAL READING

Krauthammer, T., and H. Khanlarzadeh, "Numerical Assessment of Pavement Test Sections," Transportation Research Record 1117, Transportation Research Board, 1987.

Raisanen, D. L., and D. D. Anderson, Concrete Pavements on Treated Bases," Minnesota Department of Transportation, Investigation No. 193, Final Report, 1972.

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CHAPTER 2 INTERSTATE 90 — ALBERT LEA, MINNESOTA

1. INTRODUCTION

In 1977, the Minnesota Department of Transportation constructed four experimental concrete pavement sections on I-90 between Albert Lea and Fairmont, Minnesota. The experiment was conducted to evaluate the effects of widened lanes, tied concrete shoulders, and slab thickness on pavement performance. Design variables include plain and reinforced slabs, skewed random (13-16-14-19-ft [4.0-4.1-4.3-5.8 m]) and skewed uniform (27-ft [8.2 m]) joint spacings, tied PCC shoulders and AC shoulders, and 8- and 9-in (203 and 229 mm) thick pavement slabs.

This project was the subject of a field monitoring program. The test sections were instrumented to measure load-induced strains and deflections. In addition, pavement temperature and slab curl were monitored.(2) These items were measured at the outer and inner lane slab edges and at the outer and inner lane transverse joints.

Four sections, designated as MN 2-1 through MN 2-4, were surveyed and tested in 1987. Two sections, MN 2-1 and MN 2-2, were constructed with tied concrete shoulders. All sections contained dowel bars, except for the inner lanes of MN 2-1 and 2-2. The pavement sections had 15-ft (4.6 m) inner lanes, aggregate base courses, and an AASHTO Classification A-6 subgrade in common. Complete project information is provided in the summary tables presented in chapter 32.

2. CLIMATE

The pavement sections are located in the south-central part of Minnesota. The project site has a Thornthwaite Moisture Index of 16, a Corps of Engineers Freezing Index of 1688, and receives an average of 30 in (762 mm) of rainfall annually. The highest average monthly maximum temperature is 85 ^{o}F (29 ^{o}C) and the lowest average monthly minimum temperature is 6 ^{o}F (-14 ^{o}C).

3. TRAFFIC

Interstate 90 is a four-lane divided highway (two lanes in each direction). The roadway's functional classification is Rural Interstate. From 1977 through 1987, the pavement has sustained approximately 2.8 million 18-kip (80 kN) Equivalent Single-Axle Load (ESAL) applications in the outer lane and over 0.2 million 18-kip (80 kN) ESAL's in the inner lane.

The two-way Average Daily Traffic (ADT) in 1986 was approximately 3900 vehicles per day, including 20 percent heavy trucks. Weigh-in-motion data collected at the project site in 1987 provided a traffic count of 3950 vehicles per day, including 29 percent heavy trucks.

4. MAINTENANCE AND REHABILITATION

According to records provided by the Minnesota Department of Transportation (Mn/DOT), transverse joint resealing with a hot-poured rubberized asphalt sealant was performed on sections 2-1 and 2-2. No other maintenance or rehabilitation activity has been performed on the sections.

5. PHYSICAL TESTING RESULTS

Cores from slab centers and from typical transverse joints were retrieved from the pavement sections in 1987. The center cores were tested in split tensile to provide an indication of concrete strength. The joint cores were visually inspected for signs of deterioration beneath the joint or for any signs of material durability distress.

The split tensile strength values were used to obtain an estimate of the modulus of rupture. A mean modulus of rupture of 717 psi (4.9 MPa) was calculated for all sections. The joint cores taken from the sections indicated no deterioration beneath the joint, nor any signs of microcracking in the coarse aggregate.

Deflection testing was performed on the sections in 1987 for purposes of layer moduli characterization, determination of load transfer efficiencies, and void detection. The elastic modulus of the concrete (E), the composite k-value, and the load transfer efficiency values are summarized for each section in the tables of chapter 32.

The slab elastic modulus (E) values averaged 7,200,000 psi (46,640 MPa) for all sections. The composite k-value (on top of the base) averaged 149 pci (40.4 kPa/mm), but varied by section. This is shown in table 8.

Table 8. Composite k-values.

Section	Base Thickness, in	k-value, pci
MN 2-1	5	128
MN 2-2	6	127
MN 2-3	5	162
MN 2-4	6	178

The base course and subgrade type for all four sections were aggregate and silty clay (AASHTO A-6), respectively. Center slab deflections were observed to be highest for section MN 2-2 (whose slab thickness was 8 in [203 mm]), and lowest for MN 2-4 (thickest pavement system of the group: 9-in [229 mm] slab and 6-in [152 mm] base course).

The load transfer efficiencies and percent corners exhibiting voids are reported in table 9.

Table 9. Load transfer efficiency and corners with voids.

Section	Load Transfer Efficiency, %	% Corners Exhibiting Voids
MN 2-1	79	7
MN 2-2	86	7
MN 2-3	99	0
MN 2-4	86	0

It is observed that load transfer efficiency was good to excellent for all four sections. The magnitude of the corner deflections was largest for MN 2-1 and MN 2-2, which contained the random joint spacing and tied concrete shoulders.

Deflection-based void detection procedures were used to identify voids under the slab corners. Only a small number of slab corners are exhibiting voids.

As previously stated, MN 2-1 and MN 2-2 both had tied and keyed concrete shoulders. The shoulders were tied with 0.62-in (116 mm) diameter, 30-in (762 mm) tiebars, spaced at 30-in (762 mm) intervals.

6. DRAINABILITY OF PAVEMENT SECTIONS

A drainage analysis was performed on each section to evaluate the section's ability to remove water from the pavement structure. The subgrade soils were found to be well-drained for MN 2-3 and MN 2-4, but were poorly-drained for MN 2-1 and MN 2-2. The pavement sections are located in an area that has the potential for the pavement to be saturated approximately 19 percent of the time. Table 10 summarizes the findings of the drainage analysis.

Table 10. Drainage summary for MN 2 sections.

Section	Base Permeability, ft/hr	AASHTO Drainage Coefficient	Overall Drainability
MN 2-1	0.93	0.85	Poor-Fair
MN 2-2	0.08	0.75	Very Poor
MN 2-3	0.06	0.80	Very Poor-Poor
MN 2-4	0.41	0.95	Fair

While the aggregate base course was somewhat drainable for MN 2-1, the poor drainability of the subgrade detracted from the overall drainability of that section. The effect of the poor-draining subgrade on MN 2-2 was compounded by a very low base permeability, thus giving it the worst rating of the group. The drainability of MN 2-3 and MN 2-4 was assisted by the well-drained subgrade. It should be noted that the aggregate base beneath the shoulder of every section was "daylighted" to the ditchline.

7. DETERIORATION OF PAVEMENT SECTIONS

The results of the distress survey conducted on each pavement section are given in the summary tables. The primary performance indicators are tabulated in figure 3 for both the outer and inner lane. The relative performance of each pavement section with respect to the primary performance indicators are discussed below for the outer lane only.

Joint Spalling

A small amount of medium- and high-severity joint spalling was observed on the pavement sections, as illustrated in table 11.

Table 11. Transverse joint spalling on MN 2 sections.

Section	Joint Spacing, ft	Joint Spalling, %
MN 2-1	13-16-14-19	3
MN 2-2	13-16-14-19	9
MN 2-3	27	3
MN 2-4	27	8

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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

INNER 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		72			82	77
JT. SPACING	FAULTING, in		0.03			0.03	0.03
DOWELED	T. CKS./mi		0			0	0
JOINTS	LONG. CKS., ft/mi		0			0	0
AGGREGATE BASE	% JT. SPALL.		0			1	1
				MN 2-3	MN 2-4		
JRCP	PSR			4.0	4.0		4.0
27 ft	ROUGHNESS, in/mi			62	74		68
JT. SPACING	FAULTING, in			0.03	0.02		0.02
DOWELED	T. CKS./mi			0	0		0
JOINTS	LONG. CKS., ft/mi			0	0		0
AGGREGATE BASE	% JT. SPALL.			3	5		4

Figure 3. Outer and inner lane performance data for Minnesota 2.

However, the amount of joint spalling that did occur is of very little consequence. Sections 2-1 and 2-2 had hot-poured rubberized asphalt joint sealant and sections 2-3 and 2-4 had preformed joint seals. Overall, the hot-poured joint sealant was rated as fair-good, as it was displaying some distress. The preformed joint seals were in good condition.

Joint Faulting

The amount of joint faulting occurring on every pavement section was very small. In fact, the average joint faulting for all sections was only 0.06 in (2 mm). Therefore, in terms of faulting, all sections are performing about the same. It should be noted that all of the outer lane joints within the project contain 1-in (25.4 mm) diameter dowel bars.

Transverse Cracking

For JRCP, transverse cracking is measured by the number of deteriorated (medium- and high-severity) 12-ft (3.7 m) cracks per mile, whereas for JPCP, all 12-ft (3.7 m) cracks are counted. The only section displaying any transverse cracking was the 9-in (229 mm) JRCP with 27-ft (8.2 m) skewed joints and 6-in (152 mm) aggregate base course (MN 2-4). However, the amount of transverse cracking occurring on this section was so small (5 cracks/mile) that it is considered insignificant.

Longitudinal Cracking

Including both outer and inner lanes, two sections exhibited longitudinal cracking. This is illustrated in table 12.

Table 12. Longitudinal cracking at MN 2, by design variable.

Section	Slab Thickness	Shoulder Type	LONGITUDINAL CRACKING, ft/mi Outer Lane	Inner Lane
MN 2-1	9	PCC	0	746
MN 2-2	8	PCC	150	0
MN 2-3	9	AC	0	0
MN 2-4	9	AC	0	0

In each case where longitudinal cracking was observed, it occurred over a localized stretch of several consecutive slabs. It is not believed that the cracking was due to the friction of the base course or to the construction of the longitudinal joint. The longitudinal joints were sawed to a depth of 2.75 in (70 mm). Rather, it is believed that the cause of the longitudinal cracking was the presence of both the tied shoulders and widened lanes on sections MN 2-1 and MN 2-2.

For the longitudinal cracking in the inner lanes, it is believed that the widened lane would produce larger curling stresses along the length of the slab which could result in extensive cracking of the slab. For MN 2-1, the cracking occurred at two locations (over 8 and 3 consecutive slabs), while for MN 2-2 it occurred over 2 adjacent slabs. In each case, at least one short slab (either 13 or 14 ft [4.0 or 4.3 m] long) was exhibiting cracking. It is thought that the cracking probably initiated in the shorter slabs of the random joint spacing and propagated to the adjacent slabs. The best way to reduce the potential for this longitudinal cracking would be to decrease the slab width slightly (say from 15 ft [4.6 m] to 13.5 ft [4.1 m]) when using joint spacing as short as was used here.

The tied PCC shoulders may have contributed to the cracking in the outer lanes by actually restraining the slab as curling stresses act on the slab. This restraint could result in the longitudinal cracking exhibited in the outer lane. However, note that the amount of cracking occurring in the outer lane is much less than that occurring in the inner lane.

Present Serviceability Rating (PSR) and Roughness

The PSR and roughness values for the four pavement sections were all very close to one another and indicated a "good" rideability. The PSR's all hovered around the 3.9 mark, while the roughness values ranged from 72 to 96 in/mile (1137 to 1515 mm/km). However, the pavement sections were all relatively smooth, as evidenced by the small amount of faulting occurring on the sections.

Overall Shoulder Condition

Since one of the design variables in the experiment was shoulder type, the condition of the shoulders should be discussed. Sections MN 2-1 and 2-2 had tied PCC shoulders, whereas MN 2-3 and 2-4 had AC shoulders. The relative shoulder performance indicators are summarized in table 13 below.

Table 13. Shoulder performance at MN 2 sections.

Section	Shoulder Type	Lane-Shoulder Drop-off, in	Lane-Shoulder Separation, in	Overall Rating
MN 2-1	PCC	0.00	0.00	Excellent
MN 2-2	PCC	0.00	0.00	Excellent
MN 2-3	AC	0.20	0.11	Good
MN 2-4	AC	0.59	0.49	Poor

The concrete shoulders were in excellent overall condition, whereas the asphalt shoulders were displaying some distress. The asphalt shoulders, particularly on MN 2-4, frequently displayed long segments of medium- and high-severity alligator cracking along the length of each section. The lane-shoulder joint sealant was in excellent condition for the concrete shoulders and in poor condition for the asphalt shoulders.

8. EFFECT OF DESIGN FEATURES ON PAVEMENT PERFORMANCE

Four different pavement design features were evaluated within this project. The effect that each of the various design features had on the performance of the pavement is not always readily apparent, particularly since the experiment was not set up as a full factorial design to isolate the different design variables. However, some general comments can still be made on the influence of the various design features on pavement performance.

Pavement Type

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 occurring were insignificant. While both sections exhibited about the same amount of joint faulting (recall that they were both doweled), the JPCP sections did display a substantial amount of longitudinal cracking. However, 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.

Joint Spacing

As reported above, the JRCP sections with 27-ft (8.2 m) joint spacing performed slightly better than the JPCP sections with 13-16-14-19-ft (4.0-4.9-4.3-5.8 m) joint spacing. A small amount of transverse cracking and joint spalling occurred on the 27-ft (8.2 m) sections, but not enough to significantly detract from their performance. This is to be expected with longer slab lengths, as more movement must be accommodated. The longitudinal cracking occurring on the sections with short random joint spacing was enough to detract from their performance. It is not believed that the longitudinal cracking is load related, but is instead associated with the presence of the outside tied PCC shoulder and inside widened lane.

Slab Thickness

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.

Shoulder Type

The intent in constructing a tied PCC shoulder is to provide support to the outer portion of the slab, thus reducing edge and corner deflections. The presence of a tied PCC shoulder also provides a very tight joint between the shoulder and the outer lane which can easily be maintained and sealed. Studies have indicated that 60-80 percent of the water that gets into a pavement structure infiltrates through the longitudinal lane-shoulder joint. Thus, if a tight, easily maintained joint is established at the lane-shoulder joint, the amount of moisture that infiltrates at that location will be reduced.

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 deflections were 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 outer shoulder had at one time been painted on the outer shoulder for all of the sections. This would result 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 rated as "excellent." 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).

Widened Inside Lanes

It is not possible to evaluate the effect of the widened inside lane on pavement performance since this design feature was included on every pavement section. The concept behind the use of the widened lanes is similar to that employed by the tied PCC shoulders—to reduce edge deflections. Widened lanes accomplish this by creating a more interior loading situation where larger deflections or stresses are not critical. In this project, a 15-ft (4.6 m) lane was used with the paint stripe placed at 12 ft (3.7 m), thus providing a 12-ft (3.7 m) wide traffic lane and a 3-ft (0.9 m) wide inner shoulder. However, the sections with the widened inside lanes and tied PCC shoulder were the only ones that exhibited longitudinal cracking. The reasons for this have been discussed previously.

9. COMPARISON OF OUTER LANE AND INNER LANE PERFORMANCE

An overall comparison of the two traffic lanes is provided in table 14. The two traffic lanes exhibited similar performance trends in the relative amounts of the various performance indicators. Longitudinal cracking occurred only on the random 13-16-14-19 ft (4.0-4.9-4.3-5.8 m) sections with the tied PCC shoulders. As would be expected, the faulting for the inner lane is less than that of the outer lane, although it is very small for the project as a whole. No transverse cracking occurred in the inner lane, and only a small amount occurred in the outer lane. Joint spalling was slightly less for the inner lane than for the outer lane. It should be recalled that on MN 2-1 and 2-2 only the outer lanes were doweled.

10. SUMMARY AND CONCLUSIONS

One conclusion drawn from this experimental project is the benefits gained from using short-jointed slabs, whether plain or reinforced. Both JRCP and JPCP sections, using 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.

Table 14. Comparison of performance by lane at MN 2.

13-16-14-19 FT JOINT SPACING, Dowels only in outer lane

Variable	MN 2-1 Inner Lane No Dowels	MN 2-1 Outer Lane Dowels	MN 2-2 Inner Lane No Dowels	MN 2-2 Outer Lane Dowels
ESAL's (millions)	0.2	2.8	0.2	2.8
Joint Spalling, %	1	3	0	9
Joint Faulting, in	0.03	0.06	0.03	0.06
Trans. Cracks/mi	0	0	0	0
Long. Cracks, ft/mi	746	0	0	150
PSR	3.8	3.8	3.9	3.9
Roughness, in/mi	82	72	72	82

27-FT JOINT SPACING, Dowels

Variable	MN 2-3 Inner Lane	MN 2-3 Outer Lane	MN 2-4 Inner Lane	MN 2-4 Outer Lane
ESAL's (millions)	0.2	2.8	0.2	2.8
Joint Spalling, %	3	3	5	8
Joint Faulting, in	0.03	0.05	0.02	0.06
Trans. Cracks/mi	0	0	0	5
Long. Cracks, ft/mi	0	0	0	0
PSR	4.0	4.0	4.0	4.0
Roughness, in/mi	62	76	74	96

The 9-in (229 mm) pavement section performed better than the 8-in (203 mm) pavement section of the same design. This is based primarily on the thinner section containing some longitudinal cracking. These sections have been subjected to a relatively low level of traffic (2.8 million ESAL's over 10 years), which may be a contributory factor to their overall good performance.

The presence of the widened lanes and tied PCC shoulders appears to be the reason for the longitudinal cracking which occurred on the project. While widened lanes do reduce edge stresses and deflections, it is clear that they cannot be widened to the point where longitudinal cracking will occur. Based on the information on this project, a 13-ft (4 m) widened lane may be the maximum width allowable for this short random joint spacing. The possibility of a combined effect of late joint sawing and widened lanes should also be considered. If the longitudinal joint sawing was not performed in a timely fashion, the effect of the widened lanes on the occurrence of longitudinal cracking may become critical.

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

It is possible that the drainability of the sections with PCC shoulders (MN 2-1 and MN 2-2) reduced or eliminated the potential benefits of the tied shoulders. Both of these sections were constructed over poorly-draining subgrade, which may have detracted from the performance of the tied concrete shoulders. The overall drainability of these sections ranged from very poor to fair. 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.

The large corner deflections on the sections with tied concrete shoulders is even more perplexing in light of a 1978 report.(2) That report, which analyzed the pavement sections discussed here and several other projects in Minnesota, concluded, among other things, that tied PCC shoulders reduce edge strains by 28 percent, that lane widening was as effective as concrete shoulders in reducing strains and deflections, and that the presence of tied PCC shoulders may allow for a thinner mainline pavement thickness to be constructed.(2) Since the load transfer across the longitudinal joint was excellent, it is possible that the higher deflections may be due to the improper placement of the tiebars at the corners.

11. ADDITIONAL READING

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CHAPTER 3 INTERSTATE 90 — AUSTIN, MINNESOTA

1. INTRODUCTION

This pavement section is located on I-90 in Austin, Minnesota, in the southeastern part of the State. The section is a four-lane divided highway with a functional classification of Urban Interstate. It was constructed and opened to traffic in 1984 and is designated as MN 3.

2. DESIGN

The pavement is a 9-in (229 mm) JRCP with 27-ft (8.2 m) skewed joints. A 4-in (102 mm) aggregate base rests beneath the slab on a 10-in (254 mm) aggregate subbase. The subgrade for the project is an AASHTO A-4 material.

Load transfer is accomplished through the use of 1-in (25 mm) diameter epoxy-coated dowel bars. The joints are sealed with a preformed compression seal. The slabs are reinforced with approximately 0.05 percent steel.

This pavement section is constructed with widened lanes. The outer lane is 14 ft (4.3 m) wide and the inner lane is 13 ft (4.0 m) wide; however, both lanes are striped such that they are 12 ft (3.6 m) wide. The shoulders outside of the widened lanes consist of 3 in (76 mm) of asphalt concrete over 10 in (254 mm) of aggregate base. No subsurface drainage exists on this pavement.

3. CLIMATE

This section is located in the dry-freeze environmental zone. The Corps of Engineers Freezing Index for this area is 1250, and the Thornthwaite Moisture Index is 21. This area averages 30 in (762 mm) of rainfall annually.

4. TRAFFIC

The pavement carried a two-way ADT of 10,600 vehicles in 1987, including 15 percent heavy trucks. Weigh-in-motion collected on the project site in 1987 indicated a two-way traffic count of 6400 vehicles per day, including 25 percent trucks. Since opening to traffic in 1984, this section has sustained approximately 1.5 million 18-kip (80 kN) ESAL applications in the outer lane and nearly 0.25 million 18-kip (80 kN) ESAL applications in the inner lane.

5. DRAINABILITY AND OTHER PHYSICAL TESTING RESULTS

The subgrade material was well-drained. However, since the base and subbase layers of this pavement were dense-graded and had poor to marginal permeability, this reduced the overall drainability of the section. This section was assigned an AASHTO Drainage Coefficient of 0.80, corresponding to an overall drainability rating of very poor to fair.

Testing performed on the pavement showed that the modulus of elasticity of the slab was 8,810,000 psi (60,740 MPa) and the modulus of subgrade reaction on top of the base was 256 pci (70 kPa/mm). Corner deflections averaged 9.6 mils and the adjusted load transfer efficiency of the transverse joints was 93 percent. Voids were detected under 17 percent of the corners.

6. MAINTENANCE AND REHABILITATION

According to records provided by the Minnesota Department of Transportation, no maintenance or rehabilitation has been performed on this section to date.

7. PAVEMENT PERFORMANCE

The pavement section is in excellent condition. The average PSR was 3.8 in the outer lane and 4.0 in the inner lane. The Mays Meter Roughness Index was 44 and 51 in/mi for the outer and inner lane, respectively. Transverse joint faulting was extremely low for the project, averaging 0.02 in (0.5 mm) for the outer lane and 0.01 in (0.3 mm) for the inner lane. There was no pumping, joint spalling, transverse cracking, or longitudinal cracking observed in the section.

8. CONCLUSIONS

Although this pavement is only 3 years old, it has experienced 1.5 million ESAL applications in the outer lane. However, even under this relatively heavy traffic loading, the pavement is performing very well, as no major distress was observed. With the low levels of faulting, the rideability of the pavement is very good.

CHAPTER 4 TRUNK HIGHWAY 15 — NEW ULM, MINNESOTA

1. INTRODUCTION

Located in south central Minnesota on Trunk Highway 15 near New Ulm, this pavement section is a two-lane highway. It was constructed and opened to traffic in 1986 and is designated as MN 4.

2. DESIGN

The pavement is a 7.5-in (191 mm) JPCP with 13-16-14-17 ft (4.0-4.9-4.3-5.2 m) skewed joints. The slab rests on a 5-in (127 mm) aggregate base course. The subgrade for the project is an AASHTO A-2-6 material.

Load transfer is accomplished through the use of 1-in (25 mm) diameter epoxy-coated dowel bars. The joints are sealed with a hot-poured sealant material. Longitudinal edge drains, 4 in (102 mm) in diameter, run continuously along the project, with outlets at approximately 500-ft (152 m) intervals.

Widened lanes were included in the design of this pavement section. Each lane is actually 13.5 ft (4.1 m) wide, but the paint stripe is placed such that the travelling lanes are 12 ft (3.7 m) wide, with a 1.5-ft (0.46 m) shoulder. The shoulders adjacent to the slab consist of 3 in (76 mm) of asphalt concrete over 9.5 in (241 mm) of aggregate base.

3. CLIMATE

This section is located in the dry-freeze environmental zone. The Corps of Engineers Freezing Index for this area is 1800, and the Thornthwaite Moisture Index is 0. It averages 28 in (711 mm) of rainfall annually.

4. TRAFFIC

The pavement carried a two-way ADT of 2,900 vehicles in 1987, including 13.5 percent heavy trucks. Weigh-in-motion data collected on the project site in 1987 indicated a two-way traffic count of 4058 vehicles per day, including 15.4 percent trucks. Since opening to traffic in 1986, this section has sustained approximately 0.22 million 18-kip (80 kN) ESAL applications in each direction.

5. DRAINABILITY AND OTHER PHYSICAL TESTING RESULTS

The subgrade material for this section was poorly drained. The permeability of the base course was such that it had a marginal drainability. The presence of the edge drains elevate the overall drainability of the section to fair, corresponding to an AASHTO Drainage Coefficient of 0.90.

Testing performed on the pavement showed that the modulus of elasticity of the slab was 6,300,000 psi (43,440 MPa) and the modulus of subgrade reaction on top of the base was 222 pci (61 kPa/mm). Corner deflections averaged 7.9 mils and the adjusted load transfer efficiency of the transverse joints was 86 percent. Voids were not detected under any slab corners.

6. MAINTENANCE AND REHABILITATION

According to records provided by the Minnesota Department of Transportation, no maintenance or rehabilitation has been performed on this section to date.

7. PAVEMENT PERFORMANCE

The pavements in both directions of this section were in excellent condition. No difference in performance was observed between the two directions. The average PSR (both directions) was 4.7, the Mays Meter Roughness averaged 40 in/mi, and transverse joint faulting averaged only 0.01 in (0.3 mm). There was no pumping, joint spalling, transverse cracking, or longitudinal cracking observed within the pavement section.

8. CONCLUSIONS

As would be expected for a new pavement, this section is in excellent condition. No distresses of any kind were observed. After 1 year of service, the pavement has sustained only 0.22 million 18-kip (80 kN) load applications.

CHAPTER 5 TRUNK HIGHWAY 15 — TRUMAN, MINNESOTA

1. INTRODUCTION

Located in south central Minnesota on Trunk Highway 15 near Truman, this pavement section is a two-lane highway. It was constructed and opened to traffic in 1983 and is designated as MN 6.

2. DESIGN

The pavement is an 8-in (203 mm) JRCP with 27-ft (8.2 m) skewed joints. The slab contains approximately 0.06 percent reinforcement. A 4-in (102 mm) permeable asphalt-treated base course and a 4-in (102 mm) dense-graded aggregate subbase course lie directly beneath the slab. The permeable asphalt-treated base contained between 1.5 and 3.0 percent of a 60/70 penetration-graded asphalt cement. The aggregate subbase was required to meet specific gradation for a filter layer. The subgrade for the project is an AASHTO A-2-4 material.

Load transfer is provided by 1-in (25 mm) diameter epoxy-coated dowel bars. The joints are sealed with a preformed joint sealant. Longitudinal edge drains (4 in [102 mm] in diameter) run continuously along the project, with outlets at approximately 500-ft (152 m) intervals.

Widened lanes were included in the design of this pavement section. Each lane is actually 13.5 ft (4.1 m) wide, but the paint stripe is placed such that the travelling lanes are 12 ft (3.7 m) wide, with a 1.5-ft (0.46 m) shoulder. The shoulders adjacent to the slab consist of 4 in (102 mm) of asphalt concrete over 12 in (305 mm) of aggregate base.

3. CLIMATE

This section is located in the dry-freeze environmental zone. The Corps of Engineers Freezing Index for this area is 1800, and the Thornthwaite Moisture Index is 20. It averages 30 in (762 mm) of rainfall annually.

4. TRAFFIC

The 1987 ADT for the section was 3900 vehicles per day, including 17 percent heavy trucks. Since opening to traffic in 1983, this section has sustained approximately 0.85 million 18-kip (80 kN) ESAL applications in each direction.

5. DRAINABILITY AND OTHER PHYSICAL TESTING RESULTS

The subgrade material for this section was poorly drained. The base course was constructed to be very permeable, and the subbase was rated as marginal in terms of drainability. The fact that the base was permeable and that edge drains were present on the project results in an overall drainability rating of good. This corresponds to an AASHTO Drainage Coefficient of 1.05.

Testing performed on the pavement showed that the modulus of elasticity of the slab was 6,570,000 psi (45,300 MPa) and the modulus of subgrade reaction on top of the base was 199 pci (54 kPa/mm). Corner deflections averaged 7.2 mils and the adjusted load transfer efficiency of the transverse joints was 80 percent. Voids were not detected under any slab corners.

6. MAINTENANCE AND REHABILITATION

According to records provided by the Minnesota Department of Transportation, no maintenance or rehabilitation has been performed on this section to date.

7. PAVEMENT PERFORMANCE

This pavement section was performing very well. Both directions were exhibiting similar performance. The average PSR (both directions) was 4.4, the Mays Meter Roughness averaged 51 in/mi, and transverse joint faulting averaged only 0.01 (0.3 mm). There was no pumping, joint spalling, transverse cracking, or longitudinal cracking observed within the pavement section.

8. CONCLUSIONS

This 4-year-old pavement is exhibiting outstanding performance after 0.85 million ESAL applications. No major distresses were observed.

CHAPTER 6 STATE ROUTE 360 (SUPERSTITION FREEWAY) — PHOENIX, ARIZONA

1. INTRODUCTION

Experimental pavement sections were built on the Superstition Freeway in Phoenix, Arizona, over a number of years from the mid-1970's through the early 1980's. These sections differed from Arizona's traditional design in that they introduced a number of experimental features, including different base types and pavement thicknesses. One section was constructed with edge drains and five of the sections incorporated the now-standard tied PCC shoulders. Design variables evaluated in this project include lean concrete base courses and slabs built on grade; 9-, 11-, and 13-in (229, 279, and 330 mm) pavement slabs; and edge drains and no edge drains. All sections were JPCP, had random joint spacing of 13-15-17-15 ft (4.0-4.6-5.2-4.6 m), no dowels, and had skewed joints. The subgrade for all sections was an A-6 silty clay soil. There are three lanes in each direction of traffic. The standard design at the time these experimental sections first started being built was 9-in (229 mm) of JPCP constructed on 6-in (152 mm) of cement-treated base and a 4-in (102 mm) aggregate subbase. This design is represented by AZ 1-1.

In total, six sections, designated as AZ 1-1, AZ 1-2, and AZ 1-4 through AZ 1-7, were surveyed and tested in 1987. Other experimental designs on the Superstition Freeway which were <u>not</u> included in this project include an experimental prestressed section and a retrofitted CRCP outer lane and shoulder. A complete listing of the original design and construction information for each section, as well as other pertinent project information, is provided in chapter 32.

2. CLIMATE

The pavement sections are located in the south-central portion of Arizona, in the dry nonfreeze environmental zone. The project site has a Thornthwaite Moisture Index of -47, a Corps of Engineers Freezing Index of 0, and receives an average of 8 in (203 mm) of rainfall annually. The highest average monthly maximum temperature is 105 $^{\circ}$F (41 $^{\circ}$C) and the lowest average monthly minimum temperature is 36 $^{\circ}$F (2 $^{\circ}$C).

3. TRAFFIC

This roadway's functional classification is Urban Principal Arterial. As previously indicated, the sections included in this project were constructed over a period of 10 years, from 1972 to 1981. All were constructed originally as four-lane divided highway (two lanes in each direction), but a third lane was retrofitted to several of the sections.

The traffic for each section varies considerably over the length of the project. This is due to the numerous interchanges occurring on the roadway and is also a function of the proximity of the section to central Phoenix. The sections all carry about 3 percent heavy trucks.

The traffic on each section as well as the number of 18-kip (80 kN) Equivalent Single-Axle Load (ESAL) applications sustained by each section since being opened to traffic (through 1987) are shown in table 15.

Table 15. Traffic summary of AZ 1 sections.

Section	Year Built	2-Way ADT, 1987	Lanes Each Direction	Year Third Lane Added	ESAL's, millions Outer Lane	Lane 2
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

4. MAINTENANCE AND REHABILITATION

The information provided by Arizona DOT shows that several maintenance activities have taken place on AZ 1-1 and 1-2. These include transverse joint resealing, crack sealing, lane-shoulder joint resealing, and partial depth spall repair. No major maintenance or rehabilitation has been performed on the four other sections.

5. PHYSICAL TESTING RESULTS

Cores from slab centers were retrieved from the pavement sections in 1987. The center cores were tested in split tensile to provide an indication of the concrete strength. A joint core from AZ 1-2 was also retrieved at that time. It was visually inspected for signs of deterioration beneath the joint or for any signs of material durability distress.

There were no visible signs of deterioration at the bottom of the joint which might indicate a concrete durability problem. There was no sign of microcracking observed in the core's aggregate.

The split tensile strength values were used to obtain an estimate of the modulus of rupture. These values are reported in chapter 32. A mean modulus of rupture of 753 psi (5.2 MPa) was obtained, which indicates that sound concrete was present.

Deflection testing was performed on the sections in 1987 for purposes of layer moduli characterization, determination of load transfer efficiencies, and void detection. The elastic modulus of the concrete (E), the composite k-value (on top of the base), and the load transfer values are summarized for each section in the summary tables of chapter 32.

The slab E values averaged 3,360,000 psi (23,170 MPa) for all of the sections. The composite k-values for the sections are given in table 16.

Table 16. Summary of k-value by base type.

Base Type	k-value, pci
None (slab on grade)	425
Cement-Treated	546
Lean Concrete	603

The average center slab deflections for the different base course sections varied considerably. The lowest deflections were found on the sections with the lean concrete base. The next lowest deflections were found on the thick slab sections constructed directly on the subgrade. Among the thick slab sections, the center slab deflections of the 13-in (330 mm) slabs were much lower than those of the 11-in (279 mm) slab. The section with the highest deflections was the one with the cement-treated base.

The transverse joint load transfer efficiencies were excellent for all of the sections. The sections constructed on grade had the highest values, followed by the sections on LCB and then the section on CTB. However, these only varied from 94 to 100 percent, which suggests that there was no significant difference among the various sections in terms of load transfer efficiency. The load transfer efficiency across the tied PCC shoulders ranged from an average of 79 percent for the thick slabs without a base, to 97 percent for the 9-in (229 mm) slabs on LCB.

Using deflection-based void detection procedures, voids were detected under 30 percent of the joint corners in the section with a CTB, under 12 percent of the joint corners on the sections with no base, and under 12 percent of the joint corners in the sections with a LCB. The thicker slabs on grade appear to have inhibited the development of voids as well as the sections on LCB, and both were much better than the section on CTB.

6. DRAINABILITY OF PAVEMENT SECTIONS

A drainage analysis was performed on each section. This was performed to evaluate the section's ability to remove water from the pavement structure. It was determined that the subgrade soil was somewhat well-drained and that with the low annual rainfall there exists little potential for the pavement to be saturated. Table 17 summarizes the results of the drainage analysis.

Table 17. Drainage summary for AZ 1 sections.

Section	Base Permeability, ft/hr	AASHTO Drainage Coefficient	Overall Drainability
AZ 1-1	0.0	1.00	Fair
AZ 1-2	- -	1.10	Fair-Good
AZ 1-4	- -	1.10	Fair-Good
AZ 1-5	- -	1.10	Fair-Good
AZ 1-6	0.0	1.10	Fair-Good
AZ 1-7	0.0	1.15	Fair-Good

Although the base courses were impermeable, the drainage provided by the subgrade in combination with the fact that the pavements will not normally be subjected to saturated conditions provides the sections with good overall drainability.

7. DETERIORATION OF PAVEMENT SECTIONS

The results of the extensive distress survey conducted on each pavement section are given in chapter 32. In order to better present the results, as well as to demonstrate the limited extent of the factorial design of the experiment, the primary distress results are tabulated in figure 4 (outer lane) and figure 5 (middle lane). The relative performance of each pavement section with respect to the primary distress types is discussed below for

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 4. Outer lane performance data for Arizona 1.

ARIZONA 1 RTE. 360 PHOENIX

INNER LANE PERFORMANCE DATA

			13-15-17-15 ft SKEWED JOINTS: NO EDGE DRAINS, AC SHOULDER	13-15-17-15 ft SKEWED JOINTS: NO EDGE DRAINS, PCC SHOULDER	13-15-17-15 ft SKEWED JOINTS: EDGE DRAINS, AC SHOULDER	13-15-17-15 ft SKEWED JOINTS: EDGE DRAINS, PCC SHOULDER	AVG.
9 in JPCP	CTB		AZ 1-1				
		PSR	3.4				3.4
		ROUGHNESS, in/mi	102				102
		FAULTING, in	N/A				N/A
		T. CKS./mi	0				0
		LONG. CKS., ft/mi	84				84
		% JT. SPALL.	26				26
	LCB			AZ 1-6		AZ 1-7	
		PSR		3.8		3.8	3.8
		ROUGHNESS, in/mi		83		74	79
		FAULTING, in		N/A		N/A	N/A
		T. CKS./mi		0		0	0
		LONG. CKS., ft/mi		0		0	0
		% JT. SPALL.		6		0	3
11 in JPCP	NO BASE			AZ 1-5			
		PSR		3.8			3.8
		ROUGHNESS, in/mi		83			83
		FAULTING, in		N/A			N/A
		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 / 3.4			3.6
		ROUGHNESS, in/mi		76 / 100			88
		FAULTING, in		N/A / N/A			N/A
		T. CKS./mi		0 / 0			0
		LONG. CKS., ft/mi		0 / 0			0
		% JT. SPALL.		8 / 0			4
AVG.		PSR	3.4	3.7		3.8	
		ROUGHNESS, in/mi	102	85		74	
		FAULTING, in	N/A	N/A		N/A	
		T. CKS./mi	0	0		0	
		LONG. CKS., ft/mi	84	0		0	
		% JT. SPALL.	26	4		0	

Figure 5. Inner lane performance data for Arizona 1.

the outer lane only. It can not be overemphasized that for this set of sections the validity of any comparisons is highly suspect. Not only were the sections built and exposed to traffic at different times, they are also located between different interchanges. They therefore share neither an accumulated ESAL or an ADT. In other projects with an incomplete factorial design, comparisons still have some validity because all of the sections share the same age and approximately the same daily traffic (and accumulated ESAL's). When these factors are not held constant, the evaluation of the effects of the design variables on performance becomes tenuous, at best.

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.

Joint Spalling

Substantial amounts of medium- and high-severity joint spalling occurred only on the section constructed on CTB (22 percent of the joints). This is the oldest section (15 years) and has received the most traffic. The inner lane also had about the same amount of spalling, so that it appears not to be related to traffic. None of the other sections exhibited any significant spalling. It is known that all of the joints were sealed sometime after construction. However, at the time of survey, incompressibles were present in the joints.

Joint Faulting

The average transverse joint faulting was much higher for the section on CTB. The range of joint faulting measured on the other five sections is from 0.01 in (0.3 mm) to 0.02 in (0.5 mm). The section with 0.02-in (0.5 mm) faulting was the one with edge drains. However, the magnitude of the measured faulting and its uniformity provides no basis from which any conclusions can be drawn.

Transverse Cracking

Transverse cracking was not observed on any of the sections on Superstition Freeway. This is also the case when the second lane is considered.

Longitudinal Cracking

Longitudinal cracking was observed on only one section, AZ 1-1. This section exhibited 149 ft of longitudinal cracking per mile (28.2 m/km) in the outer lane and 84 ft of longitudinal cracking per mile (15.9 m/km) in the second (middle) lane. The transverse and longitudinal joints on this section only were formed with inserts, which frequently broke during paving. No other sections displayed any longitudinal cracking. It is suspected that the cause of the longitudinal cracking is insufficient depth of the longitudinal lane-lane joint; all joints were constructed to a depth of 25 percent of the slab thickness.

Present Serviceability Rating (PSR) and Roughness

There was very little difference between the PSR and roughness values on the different sections. The best performance was measured on one of the 13-in (330 mm) slab-on-grade sections, and the section with the worst performance in terms of PSR and roughness was the 9-in (229 mm) slab on CTB. The effect of the other variables, shoulder type and drainage, can not be separated.

8. EFFECT OF DESIGN FEATURES ON PAVEMENT PERFORMANCE

The various design features evaluated in this group of pavement sections included shoulder type, base type, slab thickness, and presence of subdrainage. Because of the confounding effect of different ages, accumulated traffic, and heavy traffic loadings, and the absence of a filled experimental design matrix, analysis of the the effect of the various design features on the performance of the different sections is not straightforward. Some comments about the relative effect of these design features are summarized below.

Base Type

There were no sections built in which the slab thickness, drainage characteristics, and shoulder type were held constant while the base type was varied. 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. This section has an AC shoulder, while the others have a tied PCC shoulder. This may have some effect on the performance of that section.

Load Transfer Devices

The use of load transfer devices was not a variable in this project and thus no comparison can be made between sections with and without load transfer devices. The fact does stand out, however, that in pavements ranging in age from 5 to 15 years and subject to a medium level of traffic loadings, load transfer efficiency was uniformly excellent, varying from 94 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

Slab thickness varied from 9 in (229 mm) to 13 in (330 mm); however, 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. However, there were no corner voids under the 13-in (330 mm) section and there were voids under 36 percent of the 11-in (279 mm) sections.

Shoulder Type

Two shoulder types were included in this project, AC shoulders and tied PCC shoulders. 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.

Subdrainage

One of the pavement sections, AZ 1-7, was constructed with edge drains. It is a 9-in (229 mm) slab on 4-in (102 mm) of LCB and its performance can be readily compared to AZ 1-6, as they share the same cross-section and were both built in 1982. There was no cracking or spalling observed on either section and faulting is insignificant on both. Both the PSR and roughness were better for the section with edge drains, although not by a significant amount.

9. COMPARISON OF OUTER LANE AND MIDDLE LANE PERFORMANCE

The Superstition Freeway is a four- and six-lane divided highway. Due to the heavy traffic volumes, limited performance data was collected from the inner two lanes. In particular, the nature of the traffic control and the volume of the traffic precluded the collection of faulting data on these lanes. However, very little distress was observed in the middle lane and even less in the innermost lane. Thus it is difficult to say if any of the performance trends observed in the outer lane hold true for the middle lane. Average distress data is summarized in table 18.

Table 18. Comparison of performance by lane at AZ 1.

Performance Indicator	Middle Lane	Outer Lane
PSR	3.67	3.65
Roughness, in/mile	86	92
Trans. Cracks/mile	0	0
Long. Cracks, ft/mile	25	14
Pumping	None	None
Joint Spalling, %	5	4
Joint Faulting, in	N/A	0.02

10. SUMMARY AND 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 is AZ 1-1, with a 9-in (229 mm) slab on a CTB. This section has relatively significant faulting and joint spalling. The joint spalling may be caused by infiltration of incompressibles into the joints. The other sections, which vary considerably in design, 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 to date 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 pavement performance in this project, although these sections have not carried sufficient traffic, perhaps, for the differential effect of the variables to be clear.

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 longitudinal lane-lane joint (only 25 percent of the slab thickness) and the use of inserts 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.

11. ADDITIONAL READING

Delton, J. P., "Non-Conventional vs. Conventional Concrete Pavements in Arizona," 2nd International Conference on Concrete Pavement Design, April 14-16, 1981, Purdue University.

Mueller, P., and L. Scofield, "An Expanded Evaluation of Arizona's Ten Mile Concrete Test Roadway," A paper prepared for the 4th International Conference on Concrete Pavement Design and Rehabilitation, Purdue University, April 18-20, 1989.

CHAPTER 7 INTERSTATE 10 — PHOENIX, ARIZONA

1. INTRODUCTION

This pavement is a six-lane Urban Interstate highway located on I-10 just outside of Phoenix. This section was constructed and opened to traffic in 1983. It is designated as AZ 2.

2. DESIGN

This section was constructed as a 10-in (254 mm) JPCP. The transverse joints are skewed and spaced at 13-15-17-15 ft (4.0-4.6-5.2-4.6 m) intervals. The transverse joints contain 1.25-in (32 mm) diameter epoxy-coated dowels bars and are sealed with a hot-poured polymerized asphalt sealant that has a shape factor of 0.67.

The base course for the section consists of 5 in (127 mm) of lean concrete. The subgrade is an AASHTO Classification A-6 material. The section has tied concrete shoulders that are 10 in (254 mm) thick and rest on the 5-in (127 mm) lean concrete base. No subdrainage was provided on the section.

3. CLIMATE

This section is located in a dry-nonfreeze climatic zone. The area has a Thornthwaite Moisture Index of -49, a Freezing Index of 0, and receives an average annual precipitation of 7 in (178 mm). The highest average monthly maximum temperature is 105 ^{o}F (41 ^{o}C) while the lowest average monthly minimum temperature is 39 ^{o}F (4 ^{o}C).

4. TRAFFIC

In 1987, this pavement section had a two-way ADT of approximately 50,000 vehicles, including 9 percent trucks. The pavement section was 4 years old at the time of survey and had sustained approximately 1.6 million 18-kip (80 kN) ESAL's in the outer lane, 0.8 million ESAL's in the center lane, and 0.2 million ESAL's in the inner lane.

5. DRAINABILITY AND OTHER PHYSICAL TESTING RESULTS

The pavement consists of an impermeable lean concrete base and contains no positive subsurface drainage. However, the subgrade is somewhat well-draining, which should assist in the removal of excess moisture. Additionally, in a given year the pavement section is not exposed to saturated conditions very often. Thus, an AASHTO drainage coefficient of 1.05 is assigned to this section, which corresponds to an overall drainability of fair-good.

Deflection testing, coring, and material sampling were performed on the pavement section. From deflection testing, the backcalculated E of the slab was determined to be 5,560,000 psi (38,340 MPa). The modulus of rupture estimated from the split tensile testing of cores was 725 psi (5.0 MPa). The composite dynamic k-value on the top of the lean concrete base was 174 pci (47 kPa/mm), which appears low.

The average transverse joint load transfer for the section was 72 percent. Recall that these joints were doweled. The mean slab corner deflection (under a 9-kip [40 kN] load) was measured to be 11.1 mils. Slab corner deflection tests showed that 31 percent of the corners were exhibiting voids.

6. MAINTENANCE AND REHABILITATION

According to records provided by the Arizona Department of Transportation, this project has not received any maintenance or rehabilitation.

7. PAVEMENT PERFORMANCE

This pavement section is in excellent condition after 4 years of service. Joint faulting is very small and there are no other distresses present within the section. The relevant performance information is summarized in table 19.

Table 19. Comparison of performance by lane at AZ 2.

Indicator	Inner Lane	Center Lane	Outer Lane
ESAL's, millions	0.2	0.8	1.6
Joint Spalling, %	N/A	0	0
Joint Faulting, in	N/A	N/A	0.01
Pumping	N/A	None	None
Transverse Cracks/mile	0	0	0
Longitudinal Cracks, ft/mile	0	0	0
Roughness, in/mile	N/A	N/A	71
PSR	N/A	N/A	3.6

Several of the distress items could not be collected in the inner two lanes due to high traffic volumes.

The tied concrete shoulder was in excellent condition. The lane-shoulder joint was well-sealed and had a load transfer efficiency of 100 percent. The shoulders were tied with 0.50-in (13 mm) diameter, 24-in (610 mm) long tiebars, spaced at 30-in (762 mm) intervals.

8. CONCLUSIONS

This 4-year-old pavement section is performing very well. It has sustained heavy traffic levels and does not exhibit any distress. The only item that may be of concern is the relatively low serviceability value (3.6). However, this is believed to be the result of construction problems. For instance, it is known that the contractors had little experience with the installation of dowel bars and that subsequently there were problems in achieving adequate consolidation of the concrete in the area of the dowel bars. Furthermore, the Arizona DOT's construction specifications at the time allowed for the contractor to pave 48 ft (14.6 m) wide. This created problems in keeping an adequate supply of concrete in front of the paver, resulting in small surface irregularities. The pavement was later ground to meet roughness specifications, but the overall rideability was still adversely affected by these construction-related incidents.

The extent of the loss of support at the corners is a cause for concern in the future. Given the corner deflection (11 mils), it is likely that erosion is occurring beneath the slab, which may adversely affect the long-term performance of this section.

CHAPTER 8 INTERSTATE 5 — TRACY, CALIFORNIA

1. INTRODUCTION

Interstate 5 near Tracy, California, was the site of an experimental project constructed in 1971. The experiment was conducted to evaluate the performance of jointed plain concrete pavements (JPCP) placed on different base types and constructed with different joint spacing, concrete strength, and slab thickness. Experimental design variables that can be evaluated to some extent include different base types (cement treated aggregate, lean concrete), joint spacing (5-8-11-7 ft [1.5-2.4-3.4-2.1 m] and 12-13-19-18 ft [3.7-4.0-5.8-5.5 m]), normal and high strength concrete, and slab thickness (8.4 in [213 mm] and 11.4 in [290 mm]).

The setup of the experiment included a complete replicate of each section in the northbound and southbound traffic lanes. Only the northbound sections were included in this study. A recent report by Caltrans has indicated a distinct difference in the performance of some sections in the southbound lanes versus the replicate sections in the northbound lanes.(3) The reasons for this difference are that the southern end of the project has much poorer drainage, and has exhibited much more pumping and slab cracking. This has greatly affected several of the sections in that area, including the short jointed section and the high strength concrete section.

Five sections in the northbound lanes, designated as CA 1-1 through CA 1-9, were surveyed and tested in 1987. One section, CA 1-3 was essentially a control section to represent California's existing design of 8.4-in (213 mm) JPCP constructed on a cement-treated aggregate base, with skewed joints at 12-13-19-18 ft (3.7-4.0-5.8-5.5 m) intervals (current California design uses 12-13-15-14 ft [3.7-4.0-4.6-4.3 m] joint spacing). All of the sections incluced 24 in (610 mm) of aggregate subbase. The subgrade was an A-1-a soil. No sealant was placed in the transverse joints and a plastic insert was used to form the longitudinal joint. A complete listing of the original design and construction information for each section is provided in chapter 32.

2. CLIMATE

The pavement sections are located in the central valley of California, south of Sacramento, in a dry-nonfreeze environmental zone. The project site has a Thornthwaite Moisture Index of -42, a Corps of Engineers Freezing Index of 0, and receives 9.7 in (246 mm) of precipitation annually. The highest average monthly maximum temperature is 94 ^{o}F (34 ^{o}C) and the lowest average monthly minimum temperature is 37 ^{o}F (3 ^{o}C).

3. TRAFFIC

The roadway is a four-lane divided highway, with a functional classification of Interstate Rural. Since being opened to traffic in 1971, the pavement has sustained approximately 7.6 million 18-kip (80 kN) Equivalent Single-Axle Load (ESAL) applications in the outer lane lane and over 1.1 million ESAL applications in the inner lane. These figures represent calculations through 1987. The two-way Average Daily Traffic (ADT) in 1987 was approximately 12,000 vehicles per day, including 19 percent heavy trucks.

4. MAINTENANCE AND REHABILITATION

According to records provided by the California Department of Transportation, some repairs were placed in the southbound lanes (not the direction included in the study) in 1987. Also, in 1988, 40 percent of the 11-ft (3.4 m) slabs in the sections with joint spacing of 5-8-11-7-ft (1.5-2.4-3.4-2.1 m) had interior corner breaks and received full-depth repair. However, this work was done after the condition survey was conducted.

5. PHYSICAL TESTING RESULTS

Cores from slab centers and from typical transverse joints were retrieved from the pavement sections in 1987. The center cores were tested in split tensile to provide an indication of the concrete strength. The split tensile strength values were used to obtain an estimate of the modulus of rupture as provided in summary tables of chapter 31. A mean modulus of rupture of 807 psi (5.6 MPa) was obtained from three cores taken from the sections with "normal" strength concrete, while a modulus of rupture of 802 psi (5.5 MPa) was obtained from the one core from the section of "high" strength concrete. This appears to be an anomaly as the original construction data showed that at 7 and 28 days the high strength section did have a higher strength concrete. However, slab modulus values backcalculated using deflection data from each of the different sections indicated that the high strength section had a larger E-value as illustrated in table 20.

Table 20. Backcalculated E-value by concrete type.

Section	Concrete Strength	E-value, psi
CA 1-1, 1-3, 1-5, 1-7	Normal	5,903,000
CA 1-9	High	6,950,000

It should be noted that a normal concrete mix design was used for the high strength concrete section, with the change to a higher cement content (a 7.5-sack mix instead of a 5.5-sack mix). The high strength section did not exhibit a higher amount of shrinkage cracking immediately after construction.

The joint cores were visually inspected for signs of deterioration beneath the joint or for any signs of material durability distress (microcracking in the aggregate). The joint cores taken from the pavement sections indicated that no concrete deterioration was present directly beneath the joint. There did exist, however, a considerable amount of microcracking in the aggregate for every joint core. This may be the beginning of a reactive aggregate problem, as there were signs on the slab surface of fine cracks similar to map cracking.

Deflection testing was performed on the sections in 1987 for purposes of layer moduli characterization, determination of load transfer efficiencies, and void detection. The elastic modulus of the concrete (E), the composite k-value (on top of the base), and the load transfer values are summarized for each section in chapter 32.

The composite k-values for the sections varied by base type as shown in table 21.

Table 21. Composite k-value by base type.

Base Type	k-value, pci
Cement-treated	303
Lean concrete	433

The lean concrete base showed a greater slab support than the cement-treated aggregate base. There was a 24-in (610 mm) aggregate subbase beneath each base type and, as noted earlier, the subgrade was classified primarily as an A-1-a soil from boring tests.

The load transfer efficiencies for all sections ranged from 85 to 89 percent, which is very high for nondoweled joints. The stiff, stabilized bases may be aiding in the development of joint load transfer.

Voids were identified beneath slab corners using deflection-based void detection procedures. These results are shown in table 22. All sections

Table 22. Corners with voids and pumping severity by base type.

Base Type	% Corners Exhibiting Voids	Pumping Severity
Cement-Treated	56	Low-Medium
Lean Concrete	15	High

showed some signs of pumping, from blowholes to extensive fines on the shoulder. There is a significant difference between the cement-treated base and lean concrete base in terms of the development of loss of support. 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.

6. DRAINABILITY OF PAVEMENT SECTIONS

A drainage analysis was performed on each section to evaluate the section's ability to remove water from the pavement structure. It was determined that the subgrade for the section is a well-drained material. Since every section has a base that is considered impermeable, this factor assists tremendously in the overall drainability of each section. In addition, the aggregate subbase located beneath the base also influences the drainability of each section. The results of the drainage analysis are shown in table 23.

Table 23. Drainage summary for CA 1 sections.

Section	Subbase Permeability, ft/hr	AASHTO Drainage Coefficient	Overall Drainability
CA 1-1	26.34	1.10	Fair-Good
CA 1-3	0.75	1.00	Fair-Good
CA 1-5	2.32	1.10	Fair-Good
CA 1-7	1.65	1.15	Fair-Good
CA 1-9	2.72	1.10	Fair-Good

Reasons for the good drainage of the sections include the drainability of the subbase, the drainability of the subgrade, and the fact that the pavement is not likely to be saturated in a typical year. The annual average rainfall is a low 9.7 in (246 mm) per year. However, it should be noted that the cross-section of the pavement sections is not a crown; rather, both traffic lanes in each direction are sloped toward the outer shoulder. This results in all water being directed toward either the longitudinal lane-lane joint or the longitudinal lane-shoulder joint.

7. DETERIORATION OF PAVEMENT SECTIONS

In order to better present the results of the extensive distress survey conducted on each pavement section, as well as to provide the factorial design of the experiment, the primary performance indicators are tabulated in figure 6 (outer lane) and figure 7 (inner lane). The relative performance of each pavement section with respect to the primary performance indicators are discussed below for the outer lane only.

Joint Spalling

There was very little joint spalling on any of the sections. The percent of joints spalled ranged from 1 to 9. The section with 9 percent spalling contained the high strength concrete. It is possible that the microcracking observed in the large aggregate of the joint cores may lead to increased spalling in the future.

Joint Faulting

Joint faulting varied greatly between test sections. This is shown in table 24. The cement-treated base sections with 12- to 19-ft (3.7 to 5.8 m)

Table 24. Transverse joint faulting.

Section	Base Type	Joint Spacing, ft	Slab Thickness, in	Subbase Permeability, ft/hr	Joint Faulting, in
CA 1-1	CTB	5-11	8.4	26.34	0.06
CA 1-3	CTB	12-19	8.4	0.75	0.10
CA 1-5	CTB	12-19	11.4	2.32	0.11
CA 1-7	LCB	12-19	8.4	1.65	0.06
CA 1-9	CTB	12-19	8.4	2.72	0.13

joint spacing had the highest faulting. A maximum value of about 0.13 in (3 mm) average faulting for JPCP is critical to good rideability of the pavement. Slab thickness did not appear to reduce joint faulting at all. The shorter joint spacing section had 50 percent less joint faulting, as did the lean concrete base section. Thus, either shortening the joint spacing or providing a less erodible base course seems to have a significant effect on reducing joint faulting.

While none of the sections contained any positive subdrainage, there was an aggregate subbase exhibited fairly good permeability, as shown in table 24. The good drainability of this subbase layer may have contributed to reducing joint faulting. In addition, the subgrade was classified as an A-1-a soil, which should provide significant bottom drainage.

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 6. Outer lane performance data for California 1.

CALIFORNIA 1 I-5 TRACY

INNER 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	3.8	4.0	3.3	3.7
		ROUGHNESS, in/mi	88	86	103	92
		FAULTING, in	N/A	N/A	N/A	N/A
		T.CKS./mi	0	0	0	0
		LONG. CKS., ft/mi	0	440	0	147
		% JT. SPALL.	1	0	0	0
				CA 1-7		
	LCB	PSR		3.4		3.4
		ROUGHNESS, in/mi		115		115
		FAULTING, in		N/A		N/A
		T.CKS./mi		0		0
		LONG. CKS., ft/mi		0		0
		% JT. SPALL.		3		3
				CA 1-9		
HIGH STRENGTH PCC	CTB	PSR		3.2		3.2
		ROUGHNESS, in/mi		120		120
		FAULTING, in		N/A		N/A
		T.CKS./mi		0		0
		LONG. CKS., ft/mi		70		70
		% JT. SPALL.		1		1
AVG.		PSR	3.8	3.5	3.3	
		ROUGHNESS, in/mi	88	107	103	
		FAULTING, in	N/A	N/A	N/A	
		T.CKS./mi	0	0	0	
		LONG. CKS., ft/mi	0	170	0	
		% JT. SPALL.	1	1	0	

Figure 7. Inner lane performance data for California 1.

Best Performance

Pavement sections with the best performance, in terms of joint faulting, had the following design features:

- o JPCP slab on lean concrete base.
- o JPCP slab with extra short joint spacing (5 to 11 ft [1.5 to 3.4 m]).

Worst Performance

The pavement section displaying the worst performance, in terms of joint faulting, had the following characteristics:

- o JPCP slab on cement-treated aggregate base course without load transfer devices.

Transverse Cracking

Transverse cracking, measured by counting all severity levels of 12-ft (3.7 m) cracks per mile, occurred on some of the experimental sections. A summary of results are given in table 25.

Table 25. Transverse cracking on CA 1 sections.

Section	Slab Thickness, in	Joint Spacing, ft	Concrete Strength	Base Type	Transverse Cracks/mile
CA 1-1	8.4	5-11	Normal	CTB	5
CA 1-3	8.4	12-19	Normal	CTB	30
CA 1-5	11.4	12-19	Normal	CTB	0
CA 1-7	8.4	12-19	Normal	LCB	75
CA 1-9	8.4	12-19	High	CTB	190

The section with "high" strength concrete had considerably more cracking than the other sections. This might be due to higher shrinkage of the concrete slab at the time of construction, although none was noted by Caltrans. The sections on the lean concrete base also had some significant cracking, which may be due to stiffer support provided by the lean concrete base. This stiffer support induces higher curling stresses in the slab, that result in transverse cracking. Most of the cracked slabs were those of 18- to 19-ft (5.8 to 5.5 m) joint spacing, whereas the 12- and 13-ft (3.7 and 4.0 m) slabs rarely cracked transversely. The best sections were the thicker slabs and the shorter joint spacing.

Best Performance

In terms of transverse cracks, the best performance was exhibited by pavement sections with the following characteristics:

- o 11.4-in (290 mm) JPCP slab on cement-treated base course.
- o 8.4-in (213 mm) JPCP slab on cement-treated base with short joint spacing (5-11 ft [1.5-3.4 m]).

Worst Performance

The worst performance, in terms of deteriorated transverse cracks, was displayed by pavement sections with the following design features:

- 8.4-in (213 mm) JPCP slab containing "high" strength concrete.
- 8.4-in (213 mm) JPCP slab having a lean concrete base.

Longitudinal Cracking

Longitudinal cracking occurred to a significant degree on three sections. The amount of longitudinal cracking occurring in both lanes by section is shown in table 26.

Table 26. Longitudinal cracking on CA 1 sections.

Section	Slab Thickness, in	Joint Spacing, ft	Concrete Strength	Base Type	Longitudinal Cracking, ft/mi
CA 1-1	8.4	5-11	Normal	CTB	0
CA 1-3	8.4	12-19	Normal	CTB	940
CA 1-5	11.4	12-19	Normal	CTB	0
CA 1-7	8.4	12-19	Normal	LCB	230
CA 1-9	8.4	12-19	High	CTB	519

For the most part, the longitudinal cracking occurred near the centerline, indicating that inadequate joint forming procedures are the likely cause of the majority of the cracking. It is known that the longitudinal joints were formed with plastic inserts placed to a depth of 2 in (51 mm) for the 8.4-in (213 mm) slabs (24 percent of the slab thickness), and to a depth of 3 in (76 mm) for the 11.4-in (290 mm) slabs (26 percent of the slab thickness).

Present Serviceability Rating (PSR) and Roughness

Roughness was measured on each pavement section using a Mays Ridemeter. In addition, Present Serviceability Ratings were performed in conjunction with the roughness measurements. This information is presented in table 27.

Table 27. Roughness and present serviceability on CA 1 sections.

Section	Base Type	Joint Spacing, ft	Slab Thickness, in	Roughness, in/mi	PSR
CA 1-1	CTB	5-11	8.4	102	2.9
CA 1-3	CTB	12-19	8.4	94	3.0
CA 1-5	CTB	12-19	11.4	122	2.7
CA 1-7	LCB	12-19	8.4	102	2.7
CA 1-9	CTB	12-19	8.4	116	2.5

The smoothest riding (and highest panel PSR) section after 16 years is the JPCP having a cement-treated base with 12 to 19 ft (3.7 to 5.8 m) joint spacing. This is hard to explain, because this section had a considerable amount of faulting and some transverse cracking. The next two smoothest sections were those with either the lean concrete base or the short joint spacing, which also had the least joint faulting. The roughest section had the high strength concrete with the cement treated base and long joint spacing. This section had the highest faulting and transverse cracking. However, it should be pointed out that the PSR values are all approaching the critical lower limits for high-volume, Interstate highways, indicating that none of the sections are performing particularly well.

8. EFFECT OF DESIGN FEATURES ON PAVEMENT PERFORMANCE

Several of the design features had an influence on the performance of the pavement sections. The general effects of these design features are summarized in the following sections.

Base Type

The two different base types produced mixed results for different distress types. Holding everything else constant, the average performance data for the sections having 8.4-in (213 mm) slabs, 12-19 ft (3.7-5.8 m) joint spacing, and normal strength concrete are shown in table 28.

Table 28. Performance summarized by base type.

Base Type	Faulting, in	Transverse Cracks/mi	Longitudinal Cracks, ft/mi	Pumping	Roughness, in/mi	PSR
CTB	0.10	30	500	None	94	3.0
LCB	0.06	75	230	High	102	2.7

These results indicate that the cement-treated base section has performed better in all cases except faulting and longitudinal cracking. In terms of longitudinal cracking, it is suspected to be a construction-related problem and not due to the base type. With respect to faulting, overall roughness and serviceability rating shows the cement-treated base section to have performed better.

Joint Spacing

A direct comparison of joint spacing can be made between the 5 to 11 ft (1.5 to 3.4 m) JPCP and the 12 to 19 ft (3.7 to 5.8 m) JPCP with cement-treated bases and normal strength concrete, as shown in table 29.

Table 29. Performance summarized by joint spacing.

Joint Spacing, ft	Joint Faulting, in	Transverse Cracks/mi	Joint Spalling, %	Roughness in/mile	PSR
5-11	0.06	5	2	102	2.9
12-19	0.10	30	3	94	3.0

Faulting is less for the shorter joint spacing section. Measured joint opening after construction for the shorter jointed pavements was about one-half that of the normal joint spacing sections, which would help maintain good load transfer and thus result in lower faulting. However, recall that the 11-ft (3.4 m) slabs on the shorter-jointed section required rehabilitation in the form of full-depth repairs because of interior corner breaks. The longer-joint spaced section has greater transverse cracking, probably due to increased thermal curling of the longer slabs.

High Strength Concrete

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. Since the backcalculated E is based on much more data from the sections, it is more reliable than a single core test.

The 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.

Slab Thickness

One section had a slab thickness of 11.4 in (290 mm) as compared to 8.4 in (213 mm) for the rest. A direct comparison can be made for both sections on cement-treated bases, 12- to 19-ft (3.7 to 5.8 m) joint spacing and normal strength concrete. Table 30 makes this comparison.

Table 30. Performance summarized by slab thickness.

Slab Thickness, in	Joint Faulting, in	Transverse Cracks/mi	Longitudinal Cracks, ft/mi	Roughness, in/mile	PSR
8.4	0.10	30	500	94	3.0
11.4	0.11	0	0	122	2.7

The thicker slab had far less slab cracking, but increased roughness and a lower PSR. The reason for increased roughness is not clear, as after construction it was not rougher than the other sections. With increased traffic loadings, the level of cracking present in the thinner section will most likely accentuate the difference in performance between these sections.

9. COMPARISON OF OUTER AND INNER LANE PERFORMANCE

Although being subjected to far fewer traffic loadings than the outer lane, the inner lane sections provide an interesting comparison. Table 31 summarizes the average performance for all of the sections.

Table 31. Comparison of performance by lane at CA 1.

Variable	Inner Lane	Outer Lane
ESAL's (millions)	1.1	7.6
Joint Spalling, %	1	4
Trans. Cracks/mile	0	60
Long. Cracks, ft/mile	102	205
Roughness, in/mi	102	111
PSR	3.5	2.8

Transverse cracking is much greater in the outer lane than the inner lane, probably due to the greater truck loadings. Longitudinal cracking is also higher in the outer lane, but it is believed that the cracking is due to insufficient depth of the longitudinal joint. The roughness is greater and the PSR much lower in the outer traffic lane also. Thus, the increased traffic in the outer lane is taking its toll on overall performance.

10. SUMMARY AND CONCLUSIONS

The most significant conclusions that can be drawn from the experiment are related to the individual design parameters, rather than to any given pavement section.

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. Neither of these pavement designs has performed particularly well, especially in consideration of the 24-in (610 mm) granular subbase and the A-1-a subgrade soil.

The thicker slab reduced slab transverse cracking greatly. As traffic increases in the future, this performance difference should 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 similar. Most of the cracking occurred in the 18- and 19-ft (5.5 and 5.8 m) slabs. Therefore, 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 formed 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.

11. ADDITIONAL READING

Spellman, D. L., J. H. Woodstrom, B. F. Neal, and P. E. Mason, "Recent Experimental PCC Pavements in California," California Division of Highways, Interim Report, June 1973.

Neal, B. F., and J. H. Woodstrom, "Performance of PCC Pavements in California," Report No. FHWA-CA-TL-78-06, California Department of Transportation, February 1978.

Neal, B. F., "Evaluation of Design Changes and Experimental PCC Construction Features," Final Report No. FHWA/CA/TL-85/07, California Department of Transportation, July 1987.

CHAPTER 9 INTERSTATE 210 — LOS ANGELES, CALIFORNIA

1. INTRODUCTION

This experimental project, located on I-210 near Los Angeles, California, was constructed in 1980. The experiment was conducted to evaluate the effect of drainable base layers in conjunction with edge drains on jointed plain concrete pavements (JPCP). The experimental project included two replicates of the drainage section and one control section.

The drainage sections (designated CA 2-1 and CA 2-2) consisted of an 8.4-in (213 mm) portland cement concrete (PCC) surface layer constructed over a 2.4-in (61 mm) asphalt concrete separation layer, a 5.4-in (137 mm) permeable cement treated base (PCTB) course layer, and a 3-in (76 mm) aggregate subbase. These sections were constructed with lateral edge drains. The control section (CA 2-3) consisted of an 8.4-in (213 mm) PCC surface layer constructed over a 5.4-in (137 mm) nonpermeable cement-treated base (CTB) course layer and a 6-in (152 mm) aggregate subbase layer.

Since CA 2-1 and CA 2-2 are replicate sections, only CA 2-2 was surveyed along with the control section (CA 2-3). In addition to the 8.4-in (213 mm) slab, each section had in common a randomized, skewed joint pattern of 12-13-19-18 ft (3.7-4.0-5.8-5.5 m) and were constructed over an AASHTO classification A-4 subgrade material. A complete listing of the original design and construction information for each section, as well as other pertinent project information, is provided in chapter 32.

2. CLIMATE

The pavement sections are located in southern California in the dry-nonfreeze environmental zone. The project site has a Thornthwaite Moisture Index of -29, a Corps of Engineers Freezing Index of 0, and receives an average of 15.6 in (396 mm) of precipitation annually. The highest average monthly maximum temperature is 89 °F (32 °C) and the lowest average monthly minimum temperature is 41 °F (5 °C).

3. TRAFFIC

The roadway is an eight-lane divided highway (four lanes in each direction), and has a functional classification of Urban Interstate. Since being opened to traffic in 1980, the pavement has sustained approximately 4.4 million 18-kip (80 kN) Equivalent Single-Axle Load (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. These values represent calculations through 1987. The two-way Average Daily Traffic (ADT) in 1985 was approximately 42,000 vehicles per day, including 9 percent heavy trucks.

4. MAINTENANCE AND REHABILITATION

According to records provided by the California Department of Transportation, no rehabilitation or maintenance has been performed on the sections to date.

5. PHYSICAL TESTING RESULTS

Cores from slab centers were retrieved from the pavement sections in 1987. The center cores were tested in split tensile to provide an indication of the concrete strength. The split tensile strength values were used to obtain an estimate of the modulus of rupture as provided in the summary tables of chapter 32. A mean modulus of rupture of 687 psi (4.7 MPa) was obtained, which is indicative of sound concrete (at the slab center). Since the project is relatively new, no joint cores were taken from this experimental project.

Deflection testing was performed on the sections in 1987 for purposes of layer moduli characterization, determination of load transfer efficiencies, and void detection. The elastic modulus of the concrete (E), the composite k-value, and the load transfer values are summarized for each section in chapter 32.

The slab E values averaged 6,010,000 psi (41,440 MPa) for both sections, which is indicative of strong concrete. The composite k-values (on top of the base) for the sections are shown in table 32.

Table 32. Summary of composite k-values of CA 2.

Section	Base Type	k-value, pci
CA 2-2	AC/PCTB	1423
CA 2-3	CTB	572

Both sections display a high k-value, although the support provided by the AC/PCTB is much larger. Recall that the subgrade was classified as an A-4 soil from boring tests, which is a fairly stiff soil.

The load transfer efficiencies and percent corners with voids, or loss of support, are shown in Table 33.

Table 33. Load transfer efficiencies and corners with voids at CA 2.

Section	Base Type	Edge Drains	Load Transfer Efficiency, %	% Corners Exhibiting Voids
CA 2-2	AC/PCTB	Yes	10	65
CA 2-3	CTB	No	19	90

The load transfer efficiencies are extremely low. It is clear that aggregate interlock is not providing adequate load transfer.

Both sections exhibit a substantial amount of loss of support/erosion beneath the slab corners. Although there is loss of support within the permeable base sections, the permeable base/edge drain combination appears to reduce the amount of loss of support. The voids were identified using deflection-based void detection procedures.

6. DRAINABILITY OF PAVEMENT SECTIONS

A drainability analysis was performed on the project to determine the overall drainability of each section. The subgrade for both sections was 50 percent well-drained; the other 50 percent of the soils have been altered or modified. The borrow material, however, is also well-drained. The sections are in a location where the pavement would be expected to be saturated only 11 percent of the time. The results of the drainage analysis are tabulated below:

Table 34. Drainage summary for CA 2 sections.

Section	Aggregate Subbase Permeability, ft/hr	AASHTO Drainage Coefficient	Overall Drainability
CA 2-2	232.00	1.10	Fair-Good
CA 2-3	2.50	0.90	Fair-Good

No information is available pertaining to the permeability of the permeable CTB. The conventional CTB was assumed to be impermeable. Both sections contained well-draining subbases (beneath the cement-treated base) and a well-drained subgrade; however, CA 2-2 contained edge drains. The combination of longitudinal edge drains and the permeable CTB on CA 2-2 resulted in a better overall drainability for that section. The longitudinal edge drains present on CA 2-2 were 8-in (203 mm) perforated metal pipe, connecting to 8-in (203 mm) cross-drains at approximately 400-ft (122 m) intervals. These cross-drains discharge into drop inlets and storm drains.

7. DETERIORATION OF PAVEMENT SECTIONS

The relative performance of each pavement section with respect to the primary performance indicators are discussed in the following sections. This information is tabulated in figure 8.

Joint Spalling

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.

Joint Faulting

Both sections exhibited a large amount of faulting, as indicated in table 35. The large amount of faulting is not surprising, after examination

Table 35. Transverse joint faulting at CA 2.

Section	Average Faulting, in	Highest Faulting, in	Lowest Faulting, in
CA 2-2	0.11	0.27	0.01
CA 2-3	0.11	0.18	0.00

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.

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

		MIDDLE 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	N/A	N/A
	ROUGHNESS, in/mi	N/A	N/A
	FAULTING, in	N/A	N/A
	T. CKS. /mi	0	162
	LONG. CKS., ft/mi	0	0
	% JT. SPALL.	0	0

Figure 8. Outer and middle lane performance data for California 2.

Transverse Cracking

No transverse cracking occurred on the permeable base/edge drain section (CA 2-2). The nonpermeable control section, however, experienced transverse mid-panel cracking in nearly every long slab. The cracks were all of low or medium severity.

This difference in cracking may be partially attributed to the use of the 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 likely 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.

Longitudinal Cracking

No longitudinal cracking was observed on either of the pavement sections.

Present Serviceability Rating (PSR) and Roughness

The rideability of the pavement sections was rated by the survey crew and also was measured by a Mays Ridemeter. The results of these measurements are reported in table 36.

Table 36. Roughness and present serviceability at CA 2.

Section	Base Type	Roughness, in/mi	PSR
CA 2-2	AC/PCTB	107	3.8
CA 2-3	CTB	98	4.1

The smoothest riding (and highest panel PSR) section after 7 years is the nonpermeable base section. However, the values are relatively close to one another, indicating that both sections have about the same rideability. Since the sections had the same levels of faulting and CA 2-3 had more transverse cracking, the expectation is that CA 2-3 would have a lower PSR (and higher roughness readings). The actual results may be attributed to the larger magnitude of faulting occurring in the permeable base section. Recall, in section CA 2-2, that the largest faulting value was greater than 0.25 in (6 mm).

Other Pavement Distress

Low-severity reactive aggregate distress was observed in both sections. No other distresses were observed on this experimental project.

8. EFFECT OF DESIGN FEATURES ON PAVEMENT PERFORMANCE

This experiment evaluated the effect of two different base types on pavement performance. As the two sections were constructed adjacent to one another and were subjected to the same traffic loading, the effect of base type is isolated and can be evaluated.

Upon examination of the performance data, it is evident that the section with a permeable CTB and edge drains (CA 2-2) performed better than the section constructed over conventional CTB (CA 2-3). It would be convenient to state that this is due to the inclusion of drainage into the design of CA 2-2, but there is one confounding factor: the presence of the AC separation layer. This confounds the experiment entirely, as its presence alters the drainage characteristics of the permeable CTB, and may also affect some of its other properties. Therefore, it can be stated that CA 2-2 performed better than CA 2-3, but the reason cannot be clearly attributed to the presence of drainage, although it may have been a contributing factor.

9. COMPARISON OF OUTER AND MIDDLE LANE PERFORMANCE

Large traffic volumes on the four-lane roadway prevented the measurement of faulting in the three inner lanes. For the same reason, PSR and roughness information was not collected in these lanes. Nevertheless, other distresses occurring in the second lane were noted. This information is presented in table 37, which summarizes the outer lane and the second lane distresses for both sections.

Table 37. Comparison of performance by lane at CA 2.

CA 2-2

Variable	Second Lane	Outer Lane
ESAL's (millions)	2.1	4.4
Joint Spalling, %	1	0
Joint Faulting, in	N/A	0.11
Trans. Cracks/mile	0	0
Long. Cracks, ft/mile	0	0
Roughness, in/mi	N/A	107
Pumping	None	None
PSR	N/A	3.8

CA 2-3

Variable	Second Lane	Outer Lane
ESAL's (millions)	2.1	4.4
Joint Spalling, %	2	4
Joint Faulting, in	N/A	0.11
Trans. Cracks/mile	28	50
Long. Cracks, ft/mile	0	0
Roughness, in/mi	N/A	98
Pumping	None	None
PSR	N/A	4.1

In terms of spalling, there was no discernible difference between the two lanes. It is interesting to note that no transverse cracking occurred on the PCTB section in either lane, but a significant amount occurred on the CTB section. Furthermore, all transverse cracking occurred on the conventional CTB section (CA 2-3).

10. SUMMARY AND CONCLUSIONS

Due to the limited nature of the study, it is difficult to draw any strong conclusions. However, enough results are available to make several observations.

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. The AC separation layer appears to have assisted in reducing transverse cracking, however.

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 within the section was in the 18- and 19-ft (5.5 and 5.8 m) slabs.

11. ADDITIONAL READING

Neal, B. F., "Evaluation of Design Changes and Experimental PCC Construction Features," Report No. FHWA/CA/TL-85/07, California Department of Transportation, July 1987.

Wells, G. K., "Evaluation of Edge Drain Performance," Report No. FHWA/CA/TL-85/15, California Department of Transportation, November 1985.

Neal, B. F., and J. H. Woodstrom, "Performance of PCC Pavements in California," Report No. FHWA/CA/TL-78/06, California Department of Transportation, February 1978.

CHAPTER 10 ROUTE 14 — SOLEMINT, CALIFORNIA

1. INTRODUCTION

This pavement section is located in the greater Los Angeles area. It is designated as CA 6 and was constructed and opened to traffic in 1980. The roadway is a six-lane divided highway with an Urban Freeway functional classification.

2. DESIGN

The pavement was constructed as a 9-in (229 mm) JPCP. It has a random, skewed joint spacing of 12-13-15-14 ft (3.7-4.0-4.6-4.3 m). The transverse joints were not sealed and did not contain load transfer devices.

The plans called for a 5-in (127 mm) permeable asphalt-treated base, but on-site coring and boring in the survey area revealed that a 5-in (127) lean concrete base actually existed. A 2-in (51 mm) asphalt concrete subbase course was located beneath the lean concrete. The subgrade for the project was an AASHTO Classification A-2-4 silty sand material. Asphalt shoulders, 3.6 in (91 mm) thick and 8 ft (2.4 m) wide, run continuously along the project.

Subsurface drainage is provided by 3-in (76 mm) diameter longitudinal edge drains that run continuously along the project. Outlets are spaced at approximately 150-ft (46 m) intervals.

3. CLIMATE

This section is located in a dry-nonfreeze climatic zone. The area has a Thornthwaite Moisture Index of -43, a Freezing Index of 0, and receives an average annual precipitation of 7 in (178 mm). The highest average monthly maximum temperature is 98 °F (37 °C) while the lowest average monthly minimum temperature is 32 °F (0 °C).

4. TRAFFIC

In 1987, this section had a two-way ADT of 46,000 vehicles, including 9 percent trucks. At the time of survey, the pavement was 7 years old and had carried 4.4 million 18-kip (80 kN) ESAL's in the outer lane, 2.2 million ESAL's in the center lane, and 0.5 million ESAL's in the inner lane.

5. DRAINABILITY AND OTHER PHYSICAL TESTING RESULTS

The lean concrete base is impermeable, which hinders the overall drainability of the pavement section. However, the longitudinal edge drains, the well-draining subgrade, and the small amount of annual rainfall will all contribute positively to the drainability of the section. The pavement section was assigned an AASHTO drainage coefficient of 1.00, indicating an overall drainability of fair to good.

The slab modulus value backcalculated from deflection testing was determined to be 6,710,000 psi (46,260 MPa). The modulus of rupture estimated from slab cores was 791 psi (5.4 MPa). The composite dynamic k-value on the top of the lean concrete base was 294 pci (80 kPa/mm).

Deflection testing also provided information on load transfer and loss of support. The average transverse joint load transfer was determined to be 79 percent. The mean slab corner deflection (under a 9-kip [40 kN] load) was measured to be 5.9 mils. Slab corner deflection tests showed that none of the corners were exhibiting loss of support.

6. MAINTENANCE AND REHABILITATION

According to records provided by the California Department of Transportation, no maintenance or rehabilitation has been performed on this project.

7. PAVEMENT PERFORMANCE

After 7 years of service, this pavement section is beginning to exhibit significant roughness problems. The roughness in the outer lane, as measured by the Mays Meter, is 158 in/mi. The serviceability of the pavement was rated at 3.4, which is fairly low for such a new pavement. The roughness can be attributed to the substantial amount of faulting that is occurring (0.15 in [3.8 mm]) in the outer lane. A small amount of joint spalling (3 percent) and transverse cracking (10 cracks/mi) did occur in the outer lane. Small, closely-spaced cracks were identified throughout the section, which is believed to be reactive aggregate distress. No longitudinal cracking or pumping was observed within the section. Performance data for the section is summarized below:

Table 38. Comparison of performance by lane at CA 6.

Indicator	Inner Lane	Center Lane	Outer Lane
ESAL's, millions	0.5	2.2	4.4
Joint Spalling, %	N/A	13	3
Joint Faulting, in	N/A	N/A	0.15
Pumping	N/A	None	None
Transverse Cracks/mile	0	0	10
Longitudinal Cracks, ft/mile	0	0	0
Roughness, in/mile	N/A	N/A	158
PSR	N/A	N/A	3.4

Certain distress information could not be collected for the two inner lanes due to high traffic volumes.

8. CONCLUSIONS

While structurally sound, this pavement section has faulted significantly. The excessive faulting has resulted in poor rideability, as indicated by the high roughness index and low PSR. The cause of the large joint faulting is not clear, since pumping was not observed and the deflection measurements indicate full support. It is possible that erosion may be occurring beneath the lean concrete base.

The reactive aggregate distress that was identified has not yet caused extensive damage to the pavement. However, it is expected that this distress will ultimately result in a substantial amount of joint spalling.

CHAPTER 11 INTERSTATE 5 — SACRAMENTO, CALIFORNIA

1. INTRODUCTION

One experimental section was evaluated on I-5 about 15 miles (24 km) south of Sacramento, California. It is designated as CA 7 in this study. This section of I-5 is a four-lane divided highway with a functional classification of Rural Interstate. It was constructed and opened to traffic in 1979.

2. DESIGN

This jointed plain concrete pavement (JPCP) was constructed 10.2 in (259 mm) thick over a 5.4-in (137 mm) cement-treated base (CTB). The fine-grained clay subgrade (AASHTO classification A-7) present at the project site was treated with lime to a depth of 5.4 in (137 mm) to act as a working platform during construction and also to provide additional structural support to the pavement system.

The transverse joints were skewed with a random spacing of 12-13-19-18 ft (3.7-4.0-5.8-5.5 m). The joints were not sealed at construction and no load transfer devices were placed at the transverse joints. The outer shoulder was constructed of 4 in (102 mm) of asphalt concrete placed over 12 in (305 mm) of asphalt-treated permeable material. The lane-shoulder joint was sealed at construction.

The section was constructed with longitudinal drains which were placed within the permeable asphalt material at a depth of 15.6 in (396 mm). 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.

Additional project information is contained in the summary tables in chapter 32.

3. CLIMATE

This section of I-5 is located in the dry-nonfreeze environmental region. This area has a Corps of Engineers Freezing Index of 0 and a Thornthwaite Moisture Index of -31. The average annual precipitation for the site is 17 in (432 mm). The highest average monthly maximum temperature is 93 ^{o}F (34 ^{o}C) and the lowest average monthly minimum temperature is 38 ^{o}F (3 ^{o}C).

4. TRAFFIC

Traffic records provided by the California Department of Transportation indicate that, in 1985, the Average Daily Traffic (ADT) for the project was 20,000 vehicles. This includes 22 percent heavy trucks. Since its opening to traffic in 1979, the pavement has sustained approximately 10.5 million 18-kip (80 kN) Equivalent Single-Axle Loads (ESAL) applications in the outer lane and nearly 2.4 million ESAL applications in the inner lane. These estimates are based on calculations through 1987.

5. DRAINABILITY AND OTHER PHYSICAL TESTING RESULTS

The strength properties of the surface and the support layers were backcalculated using the center slab deflections as measured by the Falling Weight Deflectometer (FWD). 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 calculated as 326 pci (88.5 kPa/mm).

A loss of support analysis was performed using the procedures outlined in NCHRP 1-21. Twenty-three percent of the slab corners exhibited voids or loss of support beneath the corners.

The load transfer efficiencies were calculated using the FWD deflections. The average load transfer efficiency for the transverse joints was 35 percent. This is a typical value for joints relying strictly on aggregate interlock for load transfer.

Core samples were retrieved from the stabilized pavement layers. The center cores were tested in split tensile to provide an indication of the concrete strength. The split tensile strength values were used to obtain an estimate of the modulus of rupture. A mean modulus of rupture of 552 psi (3.8 MPa) was obtained, which is indicative of sound concrete (at the slab center). No joint cores were retrieved from this section.

A drainage analysis was performed on the section to determine the pavement's ability to remove excess moisture from the pavement structure. The subgrade was determined to be somewhat poorly-drained and the presence of the lime-treated material decreased the drainability of the section. However, the edge drains assist in draining away excess moisture, thus providing for an overall drainability rating of "fair." An AASHTO drainage coefficient of 0.85 was assigned to the section.

6. MAINTENANCE AND REHABILITATION

In 1986 the longitudinal and transverse cracks and randomly selected joints were sealed with a hot-pour rubberized asphalt sealant. The cracks were not routed before application of the sealant. No other significant rehabilitation or maintenance has been performed on this section to date.

7. PAVEMENT PERFORMANCE

The performance of both the inner and outer traffic lanes is summarized in table 39.

Table 39. Summary of performance by lane at CA 7.

Variable	Inner Lane	Outer Lane
ESAL's (millions)	2.4	10.5
Joint Spalling, %	0	0
Joint Faulting, in	N/A	0.06
Trans. Cracks/mile	74	119
Long. Cracks, ft/mile	1426	598
Roughness, in/mi	N/A	87
PSR	3.9	3.8

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 at the time of survey for either lane. The average transverse joint faulting for the outer lane of the project was 0.06 in (1.5 mm). Joint faulting measurements were not collected in the inner lane due to high traffic volumes.

8. CONCLUSIONS

The large amounts of longitudinal and transverse cracking, which both the inner and outer lane exhibited, may be attributed to a construction problem such as late sawing of the transverse and longitudinal joints or improper depth of saw cut of the joints. This is evident in that all of the cracking occurred before placement of a construction joint, which was located in the middle of the section.

The construction problems aside, the section is performing quite well in terms of spalling and roughness/serviceability. Given the high traffic volumes, it is possible that faulting could reach significant levels before the pavement reaches its design life. The low load transfer efficiency for this section and its relatively low number of years in service are not encouraging indicators for future good performance.

CHAPTER 12 U.S. 101 — THOUSAND OAKS, CALIFORNIA

1. INTRODUCTION

This section of U.S. 101, designated as CA 8, is an urban principal arterial highway located near Thousand Oaks, California in the greater Los Angeles area. It consists of four lanes in each direction. A condition survey was conducted on the outermost lane, which serves as an exit lane. This project was constructed and opened to traffic in 1983.

2. DESIGN

The design of the pavement cross-section is 10.2 in (259 mm) of JPCP on 5.4 in (137 mm) of asphalt-treated base. A 9-in (229 mm) aggregate subbase is located beneath the base course. The subgrade is an AASHTO A-7 sandy clay material.

Transverse joints are skewed, with a random joint spacing of 12-13-15-14 ft (3.7-4.0-4.6-4.3 m). The joints are not sealed and do not contain load transfer devices. Longitudinal edge drains, (2-in [51 mm] in diameter) with lateral outlets spaced at approximately 300-ft (91 m) intervals, were included in the original construction.

This pavement section was constructed with a widened (15 ft [4.6 m]) outside lane, which is actually an exit lane. The outer shoulders are asphalt concrete, 4.2 in (107 mm) thick and 7.5 ft (2.3 m) wide.

3. CLIMATE

This section is located in the dry-nonfreeze environmental zone. It has a Corps of Engineers Freezing Index of 0 and a Thornthwaite Moisture Index of -25. The average annual precipitation is 15 in (381 mm). The highest average monthly maximum temperature is 76 ^{o}F (24 ^{o}C) and the lowest average monthly minimum temperature is 43 ^{o}C (6 ^{o}C). As the section is not far from the coast, it appears that coastal environmental effects greatly influence the local climate.

4. TRAFFIC

The 1987 two-way ADT for this section was 135,000 vehicles, including 7 percent commercial vehicles. It is estimated that this pavement has sustained approximately 1.2 million 18-kip (80 kN) Equivalent Single-Axle Load (ESAL) applications in the outer lane, 4.1 million ESAL's in the center two lanes, and 0.6 million ESAL's in the inner lane.

5. DRAINABILITY AND OTHER PHYSICAL TESTING RESULTS

This pavement section was constructed with a relatively impermeable asphalt-treated base course over a poorly-draining aggregate subbase. The presence of the longitudinal edge drains coupled with a subgrade that was somewhat well-draining improved the overall drainability of the section. It was rated as fair, with an AASHTO drainage coefficient of 0.95.

The elastic modulus of the slab was backcalculated from deflection testing to be 6,420,000 psi (44,260 MPa). The modulus of rupture was estimated to be 727 psi (5.0 MPa) from split tensile testing of cores. The dynamic k-value on top of the asphalt-treated base was determined to be 339 pci (92 kPa).

Load transfer across the transverse joints was measured to be 92 percent. Loaded corner deflections averaged only 3.2 mils. Using deflection-based void detection procedures, no voids were identified beneath the slab corners.

6. MAINTENANCE AND REHABILITATION

From records provided by the California Department of Transportation, no maintenance or rehabilitation has been performed on the pavement section.

7. PAVEMENT PERFORMANCE

Due to the large traffic volumes on this pavement, distress items were not collected on the innermost lanes; limited data was collected for lane 2. The results of the distress survey are summarized in table 40.

Table 40. Comparison of performance by lane at CA 8.

Indicator	Outer Lane	Lane 2
ESAL's (millions)	1.2	4.1
Joint Spalling, %	0	0
Joint Faulting, in	0.04	N/A
Pumping	None	None
Trans. Cracks/mi	0	0
Long. Cracks, ft/mi	0	0
Roughness, in/mi	92	N/A
PSR	3.8	N/A

The pavement section is performing fairly well after 4 years of service. No cracking was present and faulting is fairly low. However, there was a small amount of joint spalling observed along the longitudinal joint between the outermost lanes.

8. CONCLUSIONS

Overall, this pavement is in good condition. No major distresses were observed and it has maintained an acceptable level of rideability. The favorable condition of the pavement could be due to the widened traffic lane which reduces the amount of critical edge and corner loading situations.

The high load transfer, low corner deflections, and absence of voids are indicative of adequate support. However, a small amount of faulting is being exhibited, which may indicate that erosion has begun to occur.

CHAPTER 13 U.S. 10 — CLARE, MICHIGAN

1. INTRODUCTION

This experimental project is located on U.S. 10 near Clare, Michigan. Constructed in 1975, this experiment was conducted primarily to evaluate the performance of jointed plain concrete pavements (JPCP) placed on different base types, including untreated aggregate, permeable asphalt-treated aggregate, and a dense-graded asphalt-treated aggregate base. Conventional jointed reinforced concrete pavement (JRCP) sections were also included. The experimental design variables evaluated in this project include different base types (dense asphalt-treated, permeable asphalt-treated, aggregate), JPCP vs. JRCP, and doweled and nondoweled joints. One section that had a concrete on-ramp adjacent to the outer lane was also included. Some experimental sections were located in areas of poorer drainage or where the water table approached within 5 feet (1.5 m) of the subbase. In 1981, these areas had vertical French drains added in the subbase, directly beneath the lane-shoulder interface. The inclusion of these drains effectively introduced subdrainage as a design variable. All of the sections were constructed with full-depth AC shoulders.

The setup of the experiment included 3 replicates of each of 4 different designs, with drained and nondrained sections located in each design group, for a total of 24 possible sections. With the addition of the section with a tied acceleration ramp, there were actually 25 candidate sections for this study. Eight of these sections, designated as MI 1-1a through MI 1-25, were surveyed and tested in 1987. Two sections, MI 1-1a and 1-1b, were included as control sections. They represented Michigan DOT's traditional design of 9-in (229 mm) JRCP constructed on a 4-in (102 mm) aggregate base, with perpendicular joints at 71.2-ft (21.7 m) intervals. Michigan 1-1a was constructed with longitudinal subdrainage. Common to the experimental designs was JPCP, 9-in (229 mm) slabs, and skewed transverse joints. The transverse joint spacing pattern was 13-19-18-12 ft (4.0-5.8-5.5-3.7 m), except for the two JPCP sections with dowels (1-7a and 1-7b), which had 13-17-16-12 ft (4.0-5.2-4.9-3.7 m) joint spacing. A complete listing of the original design and construction information for each section, as well as other pertinent project information, is provided in the summary tables of chapter 32.

2. CLIMATE

The pavement sections are located in the central part of Michigan in a wet-freeze environmental zone. The project site has a Thornthwaite Moisture Index of 29, a Corps of Engineers Freezing Index of 875, and receives an average of 32 in (813 mm) of precipitation annually. The highest average monthly maximum temperature is 82 ^{o}F (27 ^{o}C) and the lowest average monthly minimum temperature is 11 ^{o}F (-11 ^{o}C).

3. TRAFFIC

U.S. 10 is a four-lane divided highway (two lanes in each direction). The roadway's functional classification is Rural Principal Arterial. From its opening to traffic in 1975 through 1987, the pavement has experienced an estimated 0.89 million cumulative 18-kip (80 kN) Equivalent Single-Axle Loads (ESAL) applications in the outer lane and 0.08 million ESAL's in the inner lane.

The two-way Average Daily Traffic (ADT) in 1987 was approximately 5100 vehicles, including 8 percent heavy trucks. Weigh-in-motion data collected at the project site in 1987 provided a traffic count of 5900 vehicles per day, including 14 percent heavy trucks.

4. MAINTENANCE AND REHABILITATION

In 1981, vertical French drains were installed as a rehabilitation measure through the shoulders of the pavement sections with the dense-graded asphalt-treated base courses (MI 1-10a and MI 1-10b). These sections had shown moisture-related distresses from early on in their life, and by 1980 it had become clear that an effort needed to be made to remove excess moisture from the pavement. Additionally, some of the sections have received longitudinal joint sealing.

5. PHYSICAL TESTING RESULTS

Cores from slab centers and from typical transverse joints were retrieved from the pavement sections in 1987. The center cores were tested in split tensile to provide an indication of the concrete strength. The split tensile strength values were used to obtain an estimate of the modulus of rupture. The mean modulus of rupture was 748 psi (5.2 MPa), indicative of sound concrete.

The joint cores were visually inspected for signs of deterioration beneath the joint or for any signs of material durability distress such as microcracking in the aggregate. The joint cores taken from the pavement sections indicated that a significant amount of concrete deterioration was present directly beneath the joint, particularly for the sections with a dense asphalt-treated base course. These joint cores also displayed some cracking in the coarse aggregate, typical of the initial stages of "D" cracking.

Deflection testing with a Falling Weight Deflectometer (FWD) was performed on the sections in 1987 for purposes of layer moduli characterization, determination of load transfer efficiencies, and void detection. The elastic modulus of the concrete (E), the composite k-value (on top of the base), and the load transfer efficiency values are summarized for each section in chapter 32.

The slab E-values averaged 5,865,000 psi (40,440 MPa) for all of the sections, which is indicative of strong concrete. The composite k-values for the sections varied as shown in table 41.

Table 41. Summary of composite k-value by base type.

Base Type	k-value, pci
Aggregate (AGG)	303
Asphalt-treated dense graded (ATB)	469
Asphalt-treated permeable (PATB)	468

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, which is very similar to the subbase material.

The two types of asphalt-treated bases showed greater slab support than the aggregate base. Surprisingly, the permeable asphalt-treated base showed the same support as the dense-graded, asphalt-treated base. The PATB was constructed with 2 to 3 percent 85-100 pen AC, 2 to 6 percent fly ash, and a uniform-graded aggregate. The dense-graded layer was constructed with 6 to 8 percent of 250-300 pen AC. The center slab deflections were highest for the aggregate base course sections, and much lower for the asphalt-treated dense and permeable bases. The trend shown in the center slab deflections mirrors the k-value results.

The average load transfer efficiencies were much higher for the doweled sections (92 percent) than for the nondoweled pavement sections (34 percent). These results correspond with the transverse joint faulting measured on nondoweled sections, where erosion occurred. Complete field survey results are presented in the next section.

Voids were identified beneath slab corners using deflection-based void detection procedures. These results are shown in table 42.

Table 42. Corners with voids at MI 1.

Section	Base Type	Dowels	Edge Drains	% Corners Exhibiting Voids
MI 1-1a	AGG	Yes	Yes	0
MI 1-1b	AGG	Yes	No	15
MI 1-4a	PATB	No	Yes	85
MI 1-7a	AGG	Yes	Yes	85
MI 1-7b	AGG	Yes	No	100
MI 1-10a	ATB	No	Yes	70
MI 1-10b	ATB	No	No	100

All of the sections, except the long-jointed JRCP, showed voids at most corners. This was also the case in the permeable base section. The edge drains appeared to reduce the amount of loss of support somewhat. A comparison of the percent voids at corners in sections with similar base types with and without edge drains shows 0 vs. 15, 85 vs. 100, and 70 vs. 100 percent corners with voids.

6. DRAINABILITY OF PAVEMENT SECTIONS

A drainage analysis was performed on each section to evaluate the section's ability to remove water from the pavement structure. Among the factors considered in the determination of a pavement drainage coefficient (Cd) for each section were: environment, layer drainage, time of saturation, longitudinal and transverse slopes, and material characteristics. The resulting drainage coefficients are presented in table 43.

Table 43. Drainage summary for MI 1 sections.

Section	Base Type	Edge Drains	Base Permeability, ft/hr	Drainage Coefficient	Overall Drainability
MI 1-1a	AGG	Yes	0.04	1.00	Fair
MI 1-1b	AGG	None	0.01	0.90	Poor-Fair
MI 1-4a	PATB	Yes	N/A	1.10	Fair-Good
MI 1-7a	AGG	Yes	1.22	0.90	Poor-Fair
MI 1-7b	AGG	None	0.02	0.80	Poor-Fair
MI 1-10a	ATB	Yes	0.00	0.85	Poor-Fair
MI 1-10b	ATB	None	0.00	0.85	Poor-Fair
MI 1-25	ATB	None	0.00	0.70	Very Poor

It should be noted that no filter layer was used between the PATB and the sand subbase because the gradation of the proposed filter material was very similar to that of the sand subbase.

7. DETERIORATION OF PAVEMENT SECTIONS

In order to better present the results of the distress survey conducted on each pavement section, the primary performance indicators are tabulated in figure 9 (outer lane) and figure 10 (inner lane). The relative performance of each pavement section with respect to the primary performance indicators are discussed below for the outer lane only.

Joint Spalling

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.

The 71.2-ft (21.7 m) JRCP showed no medium- and high-severity joint spalling. However, the JPCP joint spalling varies greatly with base type. The dense-graded asphalt-treated bases had much greater transverse joint spalling than either the aggregate or permeable base, which had about the same amount of joint spalling. Table 44 summarizes this information.

Table 44. Transverse joint spalling on MI 1 sections.

Section	Base Type	Joint Spacing, ft	Joint Spalling, %
MI 1-1a	AGG	71.2	0
MI 1-1b	AGG	71.2	0
MI 1-4a	PATB	12-19	9
MI 1-7a	AGG	12-17	12
MI 1-7b	AGG	12-17	11
MI 1-10a	ATB	12-19	41
MI 1-10b	ATB	12-19	63
MI 1-25	ATB	12-19	75

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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 9. Outer lane performance data for Michigan 1.

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INNER 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		3.8		4.0	3.9
		ROUGHNESS, in/mi		50		44	47
		FAULTING, in		0.02		0.03	0.03
		T. CKS./mi		0		0	0
		LONG. CKS., ft/mi		0		0	0
		% JT. SPALL.		0		0	0
9 in JPCP 13-19-18-12 ft JT. SPACING	PERM ATB		MI 1-4a				
		PSR	4.0				4.0
		ROUGHNESS, in/mi	41				41
		FAULTING, in	0.00				0.00
		T. CKS./mi	0				0
		LONG. CKS., ft/mi	0				0
		% JT. SPALL.	2				2
	ATB		MI 1-10a		MI 1-10b / MI 1-25		
		PSR	3.0		3.6 / 3.4		3.3
		ROUGHNESS, in/mi	78		80 / 103		87
		FAULTING, in	0.03		0.03 / 0.01		0.02
		T. CKS./mi	0		0 / 0		0
		LONG. CKS., ft/mi	0		0 / 0		0
		% JT. SPALL.	32		43 / 25		33
9 in JRCP 71 ft JT. SPACING	AGG			MI 1-1a		MI 1-1b	
		PSR		3.9		3.2	3.6
		ROUGHNESS, in/mi		76		74	75
		FAULTING, in		0.04		0.04	0.04
		T. CKS./mi		0		10	5
		LONG. CKS., ft/mi		0		0	0
		% JT. SPALL.		0		0	0
AVG.		PSR	3.5	3.8	3.5	3.6	
		ROUGHNESS, in/mi	60	63	92	59	
		FAULTING, in	0.02	0.03	0.02	0.04	
		T. CKS./mi	0	4	0	5	
		LONG. CKS., ft/mi	0	0	0	0	
		% JT. SPALL.	17	0	34	0	

Figure 10. Inner lane performance data for Michigan 1.

Those sections with dense-graded, asphalt-treated bases result in bathtub-type designs, from which water does not readily drain. This results in continual saturation of the PCC slab when excess moisture is available which, in turn, causes acceleration of the deterioration of both the transverse and longitudinal joints. The aggregate bases had greater permeability and thus drained free water more rapidly. The permeable asphalt section was constructed to have much higher permeability, over 1000 ft/day (305 m/day), than either of the other two base designs. This design can be considered free-draining, essentially reducing the time the concrete slab is saturated to zero.

Best Performance

The pavement sections displaying the best performance, in terms of joint spalling, had the following characteristics:

- 9-in (229 mm) JRCP or JPCP slab on either aggregate or asphalt-treated permeable base course.

Worst Performance

The worst performance, in terms of joint spalling, was demonstrated by the pavement sections with the following design features:

- 9-in (229 mm) JPCP slabs on dense-graded, asphalt-treated base course.

Joint Faulting

Joint faulting varied greatly between test sections, as shown in table 45. The permeable base JPCP section with no dowels had practically no

Table 45. Transverse joint faulting on MI 1 sections.

Section	Base Type	Joint Spacing, ft	Dowels	Joint Faulting, in
MI 1-1a	AGG	71.2	Yes	0.05
MI 1-1b	AGG	71.2	Yes	0.08
MI 1-4a	PATB	12-19	No	0.03
MI 1-7a	AGG	12-17	Yes	0.04
MI 1-7b	AGG	12-17	Yes	0.04
MI 1-10a	ATB	12-19	No	0.14
MI 1-10b	ATB	12-19	No	0.19
MI 1-25	ATB	12-19	No	0.20

faulting. The JPCP and JRCP with 1.25-in (32 mm) dowel bars and an aggregate base also had very little faulting. The dense-graded asphalt base JPCP sections showed a large amount of faulting and are candidates for rehabilitation due to excessive roughness from faulting. While it might not be expected that a concrete slab would fault on a stabilized base, evidently the dense asphalt-treated base held free moisture that caused freeze-thaw damage to the joint faces and erosion of the bottom of the slab.

Some of the sections contained the added subdrainage. Comparisons of faulting for these sections is shown in table 46. The short-jointed sections

Table 46. Transverse joint faulting summarized by subdrainage.

Section	Sub-Drains	Base Type	Joint Spacing, ft	Dowels	Joint Faulting, in
MI 1-1a	Yes	AGG	71.2	Yes	0.05
MI 1-1b	No	AGG	71.2	Yes	0.08
MI 1-7a	Yes	AGG	12-17	Yes	0.04
MI 1-7b	No	AGG	12-17	Yes	0.04
MI 1-10a	Yes	ATB	12-19	No	0.14
MI 1-10b	No	ATB	12-19	No	0.19
MI 1-25	No	ATB	12-19	No	0.20

on aggregate bases showed no difference in faulting with or without edge drains. However, among the long-jointed sections and the sections on dense-graded asphalt bases, the sections having no subdrainage had about 33 percent more faulting than the sections with drains added.

Best Performance

Pavement sections with the best performance, in terms of joint faulting, had the following design features:

- 9-in (229 mm) JPCP slab on asphalt-treated permeable base course without load transfer devices.
- 9-in (229 mm) JPCP slab on aggregate base course with load transfer devices.
- 9-in (229 mm) JRCP slab on aggregate base course with load transfer devices and subdrainage.

Worst Performance

The pavement section displaying the worst performance, in terms of joint faulting, had the following characteristics:

- 9-in (229 mm) slab on dense-graded, asphalt-treated base course without load transfer devices.

Transverse Cracking

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. Generally, joint lockup, when present, results in opening up of transverse cracks in JRCP. The opened transverse cracks, under heavy traffic, then further deteriorate and eventually cause a rupture of the reinforcement. This has not occurred on these sections.

The JPCP sections with either permeable base or aggregate base sections had no transverse cracking. The three JPCP sections with asphalt-treated bases had 0, 18 and 29 cracks/mile. These can be compared to a critical level of about 70 cracks/mile, at which time rehabilitation is needed. The absence of cracking in most sections is probably due to the relatively low level of truck traffic over the 12 years of service life.

Best Performance

In terms of deteriorated transverse cracks, the best performance was exhibited by pavement sections with the following characteristics:

- 9-in (229 mm) JRCP slab on aggregate base course.
- 9-in (229 mm) JPCP slab on aggregate or permeable base course.

Worst Performance

The worst performance, in terms of deteriorated transverse cracks, was displayed by pavement sections with the following design features:

- 9-in (229 mm) JPCP slab over dense-graded, asphalt-treated base course.

Longitudinal Cracking

No longitudinal cracking occurred on any of the pavement sections. Plans show that the longitudinal joint was sawcut to a depth of 2.75 in (70 mm), or 31 percent of the slab thickness.

Present Serviceability Rating (PSR) and Roughness

The rideability of the sections varied considerably, as illustrated in table 47. The smoothest riding (and highest panel PSR) section after 12 years is the JPCP with a permeable base. This section had the least faulting and joint spalling, and no cracking. The roughest sections were those with the dense asphalt bases. These sections had the most faulting, cracking, and joint spalling.

Table 47. Roughness and present serviceability at MI 1.

Section	Base Type	Joint Spacing, ft	Dowels	Roughness, in/mi	PSR
MI 1-1a	AGG	71.2	Yes	96	3.6
MI 1-1b	AGG	71.2	Yes	91	3.3
MI 1-4a	PATB	12-19	No	47	3.9
MI 1-7a	AGG	12-17	Yes	58	3.6
MI 1-7b	AGG	12-17	Yes	37	3.7
MI 1-10a	ATB	12-19	No	98	2.9
MI 1-10b	ATB	12-19	No	104	2.8
MI 1-25	ATB	12-19	No	159	2.9

8. EFFECT OF DESIGN FEATURES ON PAVEMENT PERFORMANCE

Several of the design features had a significant influence on the performance of the pavement sections. The relative effect of these design features is summarized below.

Base Type

The dense-graded, asphalt-treated base course sections were the poorest performers of all base types. This conclusion 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. These aggregate base sections all contained dowels, however, which have a major effect on reducing faulting.

Joint Load Transfer

Sections with dowels had much higher measured deflection load transfer. However, no direct comparison can be made between identical sections with and without dowels. The only comparisons that can be made evaluate faulting in terms of load transfer and drainage. These results are shown in table 48.

Table 48. Faulting as a function of load transfer and drainage.

Section	Joint Spacing, ft	Base Type	Dowels	Faulting, in	Drainage Coefficient
MI 1-4a	12-19	PATB	No	0.03	1.10
MI 1-7a	12-17	AGG	Yes	0.04	0.90
MI 1-7b	12-17	AGG	Yes	0.04	0.85
MI 1-10a	12-19	ATB	No	0.14	0.80
MI 1-10b	12-19	ATB	No	0.19	0.80
MI 1-25	12-19	ATB	No	0.20	0.70

Faulting is clearly affected by the drainability of the base course and by the presence of dowels. 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 much less permeable aggregate base, however, did prevent faulting.

Subdrainage

The replicate sections with and without added subdrainage showed the significant benefit of the inclusion of those drains. The use of drains resulted in approximately 33 percent less faulting compared to the identical sections without edge drains. The sections with drains also showed about 15 percent fewer corners with voids, or loss of support. The effect of subdrainage on any other distresses is not apparent, except that for the dense asphalt-treated bases, there was less joint spalling with drains (47 percent of the joints) than without drains (65 to 87 percent).

Joint Spacing

A direct comparison of joint spacing can be made between the 71.2-ft (21.7 m) JRCP and the 12 to 17 ft (3.7 to 5.2 m) JPCP. These sections all contain dowel bars and were constructed over aggregate base courses. The comparison is shown in table 49.

Table 49. Comparison of performance by transverse joint spacing.

Section	Joint Spacing, ft	Joint Faulting, in	Transverse Cracks/mi	Joint Spalling, %	Roughness, in/mile	PSR
MI 1-1a	71.2	0.05	0	0	96	3.6
MI 1-1b	71.2	0.08	5	0	91	3.3
MI 1-7a	12-17	0.04	0	12	58	3.6
MI 1-7b	12-17	0.04	0	11	37	3.7

The overall performance of these sections is roughly comparable, the only significant difference being 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 above-average drainage characteristics of these sections may have mitigated some of the distresses often associated with long-jointed pavements.

Concrete Acceleration Ramp

One short section included a tied concrete acceleration ramp adjacent to the outer traffic lane. This section also contained the dense-graded, asphalt-treated base course, no subdrainage, and no dowel bars. There was no joint sealant between the lane and acceleration ramp, thus allowing moisture to freely enter the pavement structure. Also, the tiebars across the lane and ramp failed shortly after construction and this ramp was no longer tied. The performance of this section was as poor as the two other sections with asphalt shoulders and asphalt bases; the tied acceleration ramp had no effect on performance.

9. COMPARISON OF OUTER AND INNER LANE PERFORMANCE

Although being subjected to far fewer traffic loadings than the outer lane, the performance of the inner lane sections provide an interesting comparison. Table 50 summarizes performance for three different section designs.

Faulting in the inner lane is far less than in the outer lane for the dense, asphalt-treated section, but only slightly less for the aggregate base sections. The effect of heavy truck loads on erosion of the underside of the slab is apparent. More joint spalling occurred in the outer lane than in the inner lane for the JPCP sections, indicating that truck traffic can break down concrete that has been weakened from microcracks from freeze-thaw damage. However, the main cause of joint spalling is not related to truck loadings, but to the disintegration of the concrete near the joints. Roughness and PSR were more serious in the outer lane than the inner lane, probably due to increased faulting and joint spalling.

Table 50. Comparison of performance by lane at MI 1.

JRCP WITH AGGREGATE BASE

Variable	Inner Lane	Outer Lane
ESAL's (millions)	0.08	0.90
Joint Spalling, %	0	0
Joint Faulting, in	0.04	0.07
Trans. Cracks/mile	5	3
Long. Cracks, ft/mile	0	0
Roughness, in/mi	75	94
PSR	3.6	3.4

JPCP WITH AGGREGATE BASE

Variable	Inner Lane	Outer Lane
ESAL's (millions)	0.08	0.90
Joint Spalling, %	0	12
Joint Faulting, in	0.03	0.04
Trans. Cracks/mile	0	0
Long. Cracks, ft/mile	0	0
Roughness, in/mi	47	48
PSR	3.9	3.7

JPCP WITH DENSE ASPHALT-TREATED BASE

Variable	Inner Lane	Outer Lane
ESAL's (millions)	0.08	0.90
Joint Spalling, %	33	60
Joint Faulting, in	0.02	0.18
Trans. Cracks/mile	0	16
Long. Cracks, ft/mile	0	0
Roughness, in/mi	87	120
PSR	3.3	2.9

For the sections constructed with the permeable asphalt-treated base material, there was no discernible difference in performance between the two traffic lanes.

10. SUMMARY AND 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 to 19 ft [3.7 to 5.8 m]) and no dowels. The design of this section provided rapid subdrainage to 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. However, the presence of high deflections and voids at the slab corners are cause for concern about the satisfactory long-term performance of this design.

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 a serious erosion problem, 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 slab transverse cracking.

The aggregate base sections with dowels performed very well; much better than the asphalt base pavements. The granular base of these sections provided some vertical drainage. 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.

Vertical French drains placed along some of the sections reduced joint faulting by about 33 percent. Some evidence is 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 attributable to the good performance of the preformed compression seals that apparently kept out incompressibles, and the granular base that provided some subdrainage so that significant "D" cracking has not developed as rapidly.

The drainage coefficients calculated for these sections demonstrates a trend that is followed in the performance of the sections. That is, 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. Their excellent performance, in both the long-jointed and short-jointed sections, are a testimony to the advantage of applying a rust inhibitor to such devices.

11. ADDITIONAL READING

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CHAPTER 14 INTERSTATE 94 — MARSHALL, MICHIGAN

1. INTRODUCTION

This section of I-94 is a four-lane divided highway, with two lanes in each direction, located in south central Michigan between Kalamazoo and Battle Creek. It is classified as Rural Interstate. It was constructed (using recycled aggregate from the previous pavement) and opened to traffic in 1986. Its project designation is MI 3.

2. DESIGN

The pavement slab is JRCP, 10 in (254 mm) thick, with 0.14 percent steel reinforcement. The transverse joints are perpendicular, with 41-ft (12.5 m) joint spacing. The base layer is constructed with an open-graded aggregate and is 4 in (102 mm) thick. There is also a 3-in (76 mm) granular subbase layer. The subgrade has been identified as an AASHTO A-2-4.

The outer and inner shoulders are JRCP, paved 9 and 5 ft (2.7 and 1.5 m) wide, respectively, and have 41-ft (12.5 m) joint spacing. The outer shoulder tapers from a thickness of 10 in (254 mm) at the pavement's outer edge to a thickness of 6 in (152 mm) at the shoulder's outer edge.

Load transfer at the transverse joints is provided by 1.25-in (32 mm) diameter epoxy-coated dowels, placed at 12-in (305 mm) centers. The transverse joints were sealed with a preformed compression sealant. Subsurface drainage is effected by a combination of the permeable base design and 6-in (152 mm) diameter longitudinal edge drains.

3. CLIMATE

This project is located in the wet-freeze environmental zone. It has a Thornthwaite Moisture Index of 27 and a Corps of Engineers Freezing Index of 563. The average annual precipitation is 34 in (864 mm).

4. TRAFFIC

The original design ADT on this pavement was 15,700 vehicles, increasing to 24,000 vehicles after 20 years. The volume of heavy truck traffic was estimated to be 25 percent. Data provided by the Michigan DOT showed a 1986 2-way ADT of 31,300 vehicles, with 22 percent trucks. The estimated cumulative 18-kip (80 kN) Equivalent Single-Axle Load (ESAL) applications through 1987 is 2.8 million for the outer lane and 0.83 million for the inner lane.

5. DRAINABILITY AND OTHER PHYSICAL TESTING RESULTS

As noted above, the base layer was designed to be permeable. The field investigation showed no signs of poor drainage and the overall drainage coefficient (C_d) assigned to this section was 1.10. This section received an overall drainability rating of good.

Field testing of the base and subbase provided an elastic modulus of the slab of 4,380,000 psi (30,200 MPa) and an effective k-value (on top of the base) of 186 pci (50 kPa/mm). Corner deflections averaged 5.1 mils. Load transfer efficiency at the transverse joints was 88 percent and no voids were detected under the corners. Load transfer from the mainline pavement to the tied concrete shoulders was 90 percent.

6. MAINTENANCE AND REHABILITATION

This is a new pavement and no maintenance or rehabilitation has been performed to date.

7. PAVEMENT PERFORMANCE

The outer lane of this section was in excellent condition. It received an average PSR of 4.8, and a Mays Roughness of 37 in/mi. The average transverse joint faulting was 0.02 in (0.5 mm). No deteriorated transverse or longitudinal cracking was observed and there were no spalled joints.

The inner lane was also in excellent condition. Its average PSR was 4.8, and the Mays Roughness was 40 in/mi. The average transverse joint faulting was 0.01 in (0.3 mm). No other distresses were noted in this lane.

8. CONCLUSIONS

This new pavement section is in excellent condition. There are no problems associated with its performance.

CHAPTER 15 INTERSTATE 69 — CHARLOTTE, MICHIGAN

1. INTRODUCTION

In 1970, the Michigan Department of Transportation began construction of an experimental project on I-69, between Charlotte and Olivet, Michigan. The purpose of the study was to evaluate the effect of several different shoulder designs on the reduction of distresses commonly found with the DOT's typical shoulder design. The project included the construction of a tied, jointed plain concrete (JPC) shoulder, two experimental asphalt concrete (AC) shoulders, as well as Michigan's traditional shoulder design at the time. These were all built alongside a mainline pavement of jointed reinforced concrete pavement (JRCP).

The typical Interstate Class AA shoulder design in use at the time consisted of an AC aggregate mix (170 lb/sq yd [92 kg/sq m], 9-ft [2.7 m] wide), on approximately 7.5-in (190 mm) of granular material. One of the experimental AC shoulders (Type A) was constructed as a seal coat placed on an asphalt-stabilized soil aggregate. The other experimental AC shoulder design (Type B) consisted of a 1.5-in (38 mm) AC aggregate wearing course placed on an asphalt-stabilized soil aggregate. Both the concrete and asphalt shoulders were designed as 9-in (229 mm) of JPCP at the lane edge, tapering to 6-in (152 mm) at the shoulder's outer edge. Contraction joints in the concrete shoulder were constructed at 17.75-ft (5.4 m) intervals, and only matched those of the mainline pavement at every fourth joint. The shoulder was tied to the mainline pavement with 0.56-in (14 mm) diameter hook bolts placed at 40-in (1016 mm) intervals. Corrugated rumble strips were shaped into the plastic concrete at every fourth shoulder slab.

Three replicates of each of the four different designs were constructed for a length of approximately 0.5 mile (0.8 km) each along the outer edge of this Interstate pavement. Of the 12 available sections, this study was most interested in evaluating the benefits of tied concrete shoulders as compared to AC shoulders. Since one of the purposes of Michigan's project was "to evaluate the cost and performance of the experimental concrete and bituminous shoulders" it was anticipated that good cost information would also be available to assist in a cost/benefit analysis based on the known costs and the observed performance.(4)

This experimental project isolates shoulder type as a variable. The design of the mainline pavement does not change over the length of the project, so the analysis of this variable is not confounded by other factors. For this project, only the Type B shoulder design section (MI 4-2) and the concrete shoulder section (MI 4-1) were surveyed and tested in 1987. Common to both designs was the mainline JRCP, 9-in (229 mm) slabs with 71.2-ft (21.7 m) joint spacing, doweled joints, a 4-in (102 mm) select granular base material (which extends 3-ft [0.9 m] under the shoulder), and a subbase of 10-in (254 mm) of granular material. Expansion joints were constructed in the mainline pavement at every fifth joint. The contraction joints were sealed at construction with preformed neoprene compression seals. A more complete listing of the original design and construction information, along with other project information, is provided in the summary tables of chapter 32.

2. CLIMATE

The pavement sections are located in the south-central part of Michigan, in a wet-freeze environmental zone. The project site has a Thornthwaite Moisture Index of 22, a Corps of Engineers Freezing Index of 563, and receives an average of 32 in (813 mm) of precipitation annually. The highest average monthly maximum temperature is 84 ^{o}F (29 ^{o}C) and the lowest average monthly minimum temperature is 14 ^{o}F (-10 ^{o}C).

3. TRAFFIC

The roadway is a four-lane divided highway with a functional classification of Rural Interstate. Since opening to traffic in 1972, the pavement has sustained approximately 4.4 million 18-kip (80 kN) Equivalent Single-Axle Load (ESAL) applications in the outer lane and roughly 0.75 million ESAL applications in the inner lane. These estimates represent calculations through 1987.

The two-way Average Daily Traffic (ADT) in 1986 was approximately 13700 vehicles, including 11 percent heavy trucks. Weigh-in-Motion data collected at the project site in 1987 provided a traffic count of 9120 vehicles per day, including 31 percent heavy trucks.

4. MAINTENANCE AND REHABILITATION

According to records provided by the Michigan DOT, no significant rehabilitation has been performed on the surveyed sections. Maintenance patching, in the form of placement of a bituminous patching material at some of the severely deteriorated transverse cracks, has been performed on the sections over the years. The survey suggests that no effort has been made to keep the transverse or lane-shoulder joints sealed.

5. PHYSICAL TESTING RESULTS

Center slab cores and cores from typical transverse joints were retrieved from the pavement sections in 1987. The center cores were tested in split tensile to provide an indication of the concrete strength. The split tensile strength values were used to obtain an estimate of the modulus of rupture, as provided in chapter 32. A mean modulus of rupture of 676 psi (4.7 MPa) was obtained from the center slab cores.

The joint cores were visually inspected for any signs of deterioration beneath the joint and for any signs of material durability distress, such as microcracking in the aggregate. No deterioration was noted on the joint cores taken from MI 4-1. The joint cores removed from MI 4-2 indicated low-severity deterioration taking place beneath the joint. Low-severity microcracking was also present.

Deflection testing was performed on the sections in 1987 for the purpose of layer moduli characterization, determination of load transfer efficiencies, and void detection. The elastic modulus of the concrete (E), the composite k-value, and the load transfer efficiency values are summarized for each section in chapter 32.

The slab E-values averaged 4,680,000 psi (32,270 MPa) for the two sections. The composite k-values (on top of the base) for the sections were 283 pci (76.8 kPa/mm) for the section with the tied concrete shoulder, and 189 pci (51.3 kPa/mm) for the section with AC shoulders. The same aggregate base/subbase design is used throughout this project

From boring tests, the subgrade for both sections was classified as an AASHTO A-4, which is a fairly stiff soil. The center slab deflections were higher for the section with the AC shoulder than for the section with the tied PCC shoulder. This result is consistent with the variation noted in the above discussion of the compossite k-values.

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 for the section with PCC shoulders. It averaged 35 percent, which is very low. This indicates that the tied shoulders are providing very little structural support to the edge of the mainline pavement.

Voids were identified beneath slab corners using deflection-based void detection procedures. These results are shown in table 51.

Table 51. Corner void detection on MI 4 sections.

Section	Shoulder Type	Dowels	Edge Drains	% Corners Exhibiting Voids
MI 4-1	PCC	Yes	None	22
MI 4-2	AC	Yes	None	95

In addition to the clear difference in percent voids (or loss of support) shown here, an examination of the raw data is informative. On MI 4-1, of those joints exhibiting voids, none of the voids occurred under <u>both</u> the approach and the leave slab; 75 percent of the voids detected were located under the leave slab. However, on MI 4-2 voids were detected under both the approach and the leave corner of every joint but one.

6. DRAINABILITY OF PAVEMENT SECTIONS

A drainage analysis was performed on each section to evaluate the section's ability to remove water from the pavement structure. Layer permeabilities were calculated for both the base and subbase layers in each section and, combined with other inputs from the field survey, the overall drainability characteristics of these two sections were identified. The results available suggest that the section with AC shoulders had better base and subbase permeabilities. The two sections' drainage characteristics are presented in table 52.

Table 52. Drainage summary for MI 4 sections.

Section	Permeability, ft/hr Base	Permeability, ft/hr Subbase	Drainage Coefficient	Overall Drainability
MI 4-1	0.01	0.35	0.75	V.Poor-Fair
MI 4-2	0.06	0.85	0.70	V.Poor-Fair

7. DETERIORATION OF PAVEMENT SECTIONS

The primary performance indicators are tabulated in figure 11 for both the outer lane and inner lane. The relative performance of the pavement sections with respect to the primary performance indicators are discussed below for the outer lane only. The performance of the inner lane will be briefly discussed in section 9.

Joint Spalling

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. For all practical purposes, there was no difference in performance between the two sections in terms of joint spalling. The transverse joint sealant on this project was a hot-poured rubberized product in the shoulders and neoprene in the mainline pavement. The pavement's sealant was in "fair" condition on MI 4-1 and "good to fair" condition on MI 4-2.

Joint Faulting

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. It has been postulated that tied concrete shoulders would provide additional support to a mainline pavement.(2,5,6,7) Concrete shoulders also provide for a more easily maintained and tighter joint between the lane and shoulder, which should reduce the amount of moisture able to infiltrate into the base. These are factors which are thought to limit faulting.

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 is too long to provide effective load transfer. 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.

Transverse Cracking

JRCP pavements are expected to develop transverse shrinkage cracks between transverse joints; the reinforcement mesh is designed to hold the cracks tightly together. These two sections are no exception and exhibited extensive transverse cracking. Because a certain amount of shrinkage cracking is expected, low-severity transverse cracks are not considered a major distress in JRCP pavements. In the section with tied PCC shoulders there were an average of 124 low-severity transverse cracks/mile. The AC shoulder section exhibited 173 low-severity transverse cracks/mile.

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

		INNER LANE PERFORMANCE DATA	
		PCC SHOULDER	AC SHOULDER
		MI 4-1	MI 4-2
9 in JRCP 71.2 ft JT. SPACING	PSR	2.6	2.6
	ROUGHNESS, in/mi	114	103
	FAULTING, in	0.05	0.06
	T. CKS./mi	69	54
	LONG. CKS., ft/mi	0	143
	% JT. SPALL.	0	0

Figure 11. Outer and inner lane performance data for Michigan 4.

Medium and high severity transverse cracks are those which show signs of deterioration. These 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. However, both of these values 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." This is not an uncommon occurrence on long-jointed pavements, especially where there are many working transverse cracks. The steel mesh reinforcement holding the slab together is much more likely to yield before the corroded 1.25-in (32 mm) diameter dowels slide. The steel mesh then corrodes and fatigues, and ultimately fails. The transverse cracks then effectively behave as contraction joints and allow the necessary space for the contraction and expansion of the long slabs.

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. "D" cracking is an aggregate durability distress in which freeze-thaw cycling exacerbates microcracking and subsequent deterioration of susceptible aggregates. An excess of moisture is required for this deterioration to take place, and it usually begins at the slab/base interface and works upward to the surface. The fact that this phenomenon is observed only at the cracks confirms that the transverse joints are tight and are not a source of excess moisture. The measured transverse crack widths averaged 0.45-in (11 mm) for the PCC shoulder section and 0.52-in (13 mm) for the AC shoulder section.

Transverse Crack Faulting

Since there was extensive deteriorated transverse cracking, and few deteriorated transverse joints or transverse joint faulting, it is interesting to consider the amount of transverse <u>crack</u> 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 cumulative faulting was almost double in the section with PCC shoulders.

Longitudinal Cracking

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.

Present Serviceability Rating and Roughness

The Present Serviceability Rating (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 (2084 mm/km) for the PCC shoulder section and 112 in/mi (1768 mm/km) 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.

Shoulder Condition

The preceding deterioration indicators are related to the performance of the mainline pavement. Since the experimental project was designed to evaluate the performance of different shoulder designs, it is worthwhile to examine the shoulder condition of the two different sections.

The section with tied PCC shoulders averaged 5 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. 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."

The concrete shoulder was not paved integrally with the mainline pavement. In order to mitigate the potential detrimental effects of differential movement, the slab edges on the pavement were given a smoother finish than were later jobs. This would help to explain the low lane-shoulder load transfer efficiency of 35 percent.

MI 4-2, with AC shoulders, had extensive linear cracking and alligator cracking of the shoulder surface. There was an average drop-off of 0.9 in (23 mm) from the pavement to the shoulder and an average lane-shoulder separation of 0.3 in (8 mm). There was no sealant in the lane-shoulder joint, earning it a "poor" rating. The overall rating given to the outer shoulder in this section was "poor."

8. EFFECT OF DESIGN FEATURES ON PAVEMENT PERFORMANCE

There was only one design feature, 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 shoulder type is evaluated in terms of its effect on pavement performance, an examination of figure 11 shows that the section with the AC shoulder performed better than the section with the tied PCC shoulder. While PSR, roughness, joint spalling, and longitudinal cracking are similar for both sections, transverse cracking and faulting of both joints and cracks is much worse for the section with tied shoulders.

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.

Two factors may 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. As is shown in section 6, layer permeabilities were better in the AC shoulder section, although their overall drainabilities were the same. This difference in permeabilities may have had an affect on the performance of the two sections. Another factor that may account for these results is the fact that the spacing of the shoulder ties of the PCC shoulder section was too great. Thus, the expected benefits of the tied shoulders were not realized.

9. COMPARISON OF OUTER AND INNER LANE PERFORMANCE

The inner lane has been subjected to far fewer traffic loadings than the outer lane, but its performance still provides some interesting comparisons. Recall that the difference between cumulative ESAL's is 4.4 million to 0.75 million The distresses were summarized in figure 11. It can be seen that there is no difference in the PSR and very little difference in the roughness for the two inner lanes, and that these are both better than their respective outer lanes. There was no joint deterioration noted on the inner lane sections, which is similar to the result found in the outer lanes. The faulting on the inner lane of the section with the PCC shoulder was more than half that of the outer lane of the same section, while the joint faulting on the section with AC shoulders was the same for both lanes and approximately the same as the faulting on the inner lane of the PCC shoulder section. The number of medium- and high-severity transverse cracks in the inner lanes was 30 percent of the transverse cracking in the outer lanes for both sections. Longitudinal cracking is the only distress that is greater in the inner lanes than in the outer lanes. The inner lane of the section with AC shoulders has 143 ft/mi of longitudinal cracking; there is none in the inner lane of the other section.

These results confirm that faulting and deteriorated transverse cracking are load-related distresses and that the traffic volumes have had no effect on the amount of joint spalling. The effect of traffic on longitudinal cracking is ambiguous, but this distress seems not to be related to loading. It may be a result of a localized differential settlement. It is not believed that longitudinal cracking is a result of the friction produced by the base, as aggregate base courses typically have low coefficients of friction. It is possible that the depth of cut on the longitudinal joint was insufficient or that this joint was sawed late. As-built plans show that it was cut to a depth of 2.75 in (70 mm), or 31 percent of the slab thickness. While this approximates the commonly recommended value of 1/3 the slab thickness, deeper cuts may be required to ensure formation of the longitudinal joint. The inner lane shoulders are AC, paved 3-ft (0.9 m) wide on both sections.

10. SUMMARY AND CONCLUSIONS

The major purpose of this experimental project was to evaluate the performance of tied PCC shoulders against that of AC shoulders. Examination of the primary performance indicators suggests that the AC shoulders improve pavement performance, as there are fewer transverse cracks and less faulting on this section. However, this is not the conclusion that should be drawn.

A study of the permeabilities of the base and subbase courses provided an explanation for several of the anomalies observed on this project. While these two sections are approximately 2000 ft (610 m) apart and share all design characteristics with the exception of the shoulders, the properties of the base and subbase materials are very different. The base permeability for the AC shoulder section is much better than the base permeability for the section with PCC shoulders. This trend is continued with the subbase permeabilities, where again the subbase on the section with the AC shoulder has a greater permeability than the section with the PCC shoulder.

How does the difference in permeabilities affect the pavement performance? If the drainage characteristics of the section with the concrete shoulders are not as good, this section is not removing free moisture from the pavement system as rapidly as the other. The presence of excess moisture would account for the greater faulting of this section. It has already been suggested that the joints in both sections are "frozen" and that the transverse cracks are working. The large number of faulted transverse cracks and the fact that the deteriorated cracks show greater faulting than the transverse joints on that section are also explained by the presence of excess moisture.

There are also problems that may be associated with the performance of the tie bars connecting the shoulders and the mainline pavement in the section with PCC shoulders. Consider the greater number of deteriorated transverse cracks on the section with concrete shoulders. A possible explanation of this is that the tie bars used to keep the shoulder from separating from the mainline pavement are corroded and "frozen." Combined with the different joint spacing of the shoulder and mainline pavement, the differential movements which occur between these two sections of concrete could cause transverse cracks to occur where the pavements are tied together. Assuming that the PCC shoulder was placed level with the pavement edge, the heave that was measured on the shoulder implies the presence of a lifting force being applied to the shoulder, most likely as a result of freeze-thaw action from trapped moisture.

The tie bars were also spaced too far apart and the slab edge was too smooth to enable the tied PCC shoulder to provide adequate support to the mainline pavement. There was poor load transfer across this joint and the result was that the PCC shoulders did not perform as expected. They were, however, in better overall condition than the AC shoulders.

The conclusion suggested by these results then, is not that AC shoulders have a better 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 tie bars must be carefully designed to ensure that the desired benefits are achieved. When paved integrally with the mainline pavement, tied PCC shoulders will also provide support from aggregate interlock.

Also, overall pavement drainability has an enormous effect on pavement performance that can overcome other significant design features. In effect, this experiment contained an extra design variable, drainability of the pavement layers. If this variable were including in the experimental design factorial, the results would then show that the difference in permeabilities was the controlling design feature.

11. ADDITIONAL READING

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CHAPTER 16 INTERSTATE 94 — PAW PAW, MICHIGAN

1. INTRODUCTION

This section of I-94 is a four-lane, divided, rural Interstate. It is located in southwestern Michigan, west of Kalamazoo. The roadway was constructed and opened to traffic in 1984. It is designated as MI 5.

2. DESIGN

This pavement is JRCP, with 41-ft (12.5 m) joint spacing and perpendicular transverse joints. The slab is 10-in (254 mm) thick and has 0.14 percent reinforcing steel. The concrete was produced using recycled aggregate with a top size of 1 in (25 mm). Transverse joint load transfer is provided by 1.25-in (32 mm) diameter, epoxy-coated dowel bars. The transverse joints are sealed with a preformed compression sealant.

The base is a 4-in (102 mm) open-graded (permeable) aggregate layer. It rests on a sand layer 21 in (533 mm) thick. No filter layer was placed between the permeable layer and the sand layer. The subgrade is an AASHTO A-2-4. In addition to the permeable base layer, 6-in (152 mm) diameter longitudinal edge drains run continuously along the project.

The inner and outer shoulders are JPCP, 5 and 10 ft (1.5 and 3.0 m) wide respectively. The joint spacing for the concrete shoulders averaged 13.6 ft (4.1 m), so that only every third joint matched a joint in the mainline pavement.

3. CLIMATE

This section is located in the wet-freeze environmental zone. The Corps of Engineer Freezing Index for this area is 563, and the Thornthwaite Moisture Index is 40. It receives an average of 38 in (965 mm) of rainfall annually.

4. TRAFFIC

The design 2-way ADT for this section of highway was 18,400 vehicles, with 22 percent trucks. Data provided by the Michigan DOT showed a 1986 2-way ADT of 19,300, with 20 percent truck traffic. Weigh-in-Motion data was also collected at this location in 1987. This data showed a 1-way ADT of 21,900, with 28.5 percent trucks. The estimated cumulative 18-kip (80 kN) Equivalent Single-Axle Load (ESAL) applications through 1987 was 3.1 million ESAL's in the outer lane and 0.75 million in the inner lane.

5. DRAINABILITY AND OTHER PHYSICAL TESTING RESULTS

The base layer of this pavement was designed to be very permeable. It had a calculated permeability of 1.85 ft/hr (0.56 m/hr). The permeability of the subbase was 0.85 ft/hr (0.26 m/hr). Overall, this section was assigned an AASHTO Drainage Coefficient (C_d) of 1.05 and received an overall drainability rating in the range from good to excellent.

Testing performed on the pavement showed that the modulus of elasticity of the slab was 4,490,000 psi (30,960 MPa) and the effective k-value (on top of the base) was 233 pci (63 kPa/mm). Corner deflections averaged 26.9 mils and the adjusted load transfer efficiency of the transverse joints was 94 percent. Loss of support was detected under 95 percent of the corners. Load transfer from the mainline pavement to the concrete shoulder was 48 percent.

6. MAINTENANCE AND REHABILITATION

The field surveys showed that some partial-depth repairs were placed along the longitudinal joint. It is not known why these repairs were necessary. No other maintenance or rehabilitation has been performed on this section, according to information provided by the Michigan DOT.

7. PAVEMENT PERFORMANCE

The average PSR in the outer lane was 4.2. The Mays Meter Roughness Index was 36 in/mi. The average transverse faulting measured on this section was 0.05 in (1.3 mm). There were an average of 144 deteriorated transverse cracks/mi and another 158 low severity transverse cracks/mi. No longitudinal cracking was noted. Joint spalling or pumping was not observed.

The PSR in the inner lane was 4.4, with a Roughness Index of 50 in/mi. The transverse joint faulting was 0.03 in (0.8 mm) There were 5 deteriorated transverse cracks/mi, but 263 low severity transverse cracks/mi. Neither longitudinal cracking nor joint spalling was observed in this lane.

The majority of the transverse cracks occurred at locations where the transverse joints in the shoulder and mainline pavement did not match. Recall that the joint spacing was much shorter for the shoulder and that only one in three joints matched. The offset joints in the shoulder induced the formation of transverse cracks in the mainline pavement, which rapidly deteriorated under traffic.

The transverse cracks typically follow a straight line across the pavement and through the depth of the slab. With the top size of the aggregate only 1 in (25 mm), very little aggregate interlock exists so that the cracks rapidly deteriorated.

8. CONCLUSIONS

This pavement is performing poorly, especially for its young age and low traffic. There are many corner voids, excessive transverse cracking, and some repairs already in place on this 3-year old pavement. It is believed that the major cause for the poor performance of this pavement section is the offset joints in the shoulder forcing a crack in the mainline pavement and the small top size of the aggregate, which reduces the amount of aggregate interlock at opened transverse cracks. Furthermore, limited pavement removal performed by the Michigan DOT indicated that the sand layer had infiltrated into the permeable base. Thus, the favorable drainage characteristics of the base would be diminished.

9. ADDITIONAL READING

McCarthy, G. J. and W. J. MacCreery, "Michigan Department of Transportation Recycles Concrete Freeways," Proceedings, Third International Conference on Concrete Pavement Design and Rehabilitation, Purdue University, pp. 643-647.

CHAPTER 17 ROUTE 23 — CATSKILL, NEW YORK

1. INTRODUCTION

This project was constructed to evaluate the effects of base and subbase type, slab length, slab thickness, reinforcement, and mechanical load transfer devices on pavement performance. Design variables included aggregate, asphalt-treated, and soil-cement base courses; joints with and without load-transfer devices; and skewed and perpendicular joints. Construction was begun on the experimental project on Route 23, between Catskill and Cairo, New York, in 1965, and the road was opened to traffic in 1968. Over time, material testing showed that there were not sufficient differences between the two subbase materials, there was not sufficient strength gain with the soil-cement base material, and there was little thickness difference between the 8- and 9-in (203 and 229 mm) slabs to continue to use these as experimental design variables.

The original set up of the experiment included 4 replicates of each of 8 different designs (except for two designs which had only 3 replicates), for a total of 30 possible sections. Elimination of slab thickness and subbase material as variables simply increased the number of replicates of the existing designs.

Six sections, designated as NY 1-1 through NY 1-8b, were surveyed and tested in 1987. The average section length was 599 ft (183 m). No control sections, in the strict sense of the term, were included. Two sections with 60.8-ft (18.5 m) joint spacing and reinforced slabs, NY 1-3 and NY 1-4, representing New York's typical design, were included. The remaining sections had 20-ft (6.1 m) joint spacing. Other variables include base type (either asphalt-treated or aggregate) and transverse joint load transfer. The asphalt-treated base consisted of 2.5 to 4.0 percent AC. It should be noted that the sections that were constructed with load-transfer devices used a two-part malleable steel device (ACME) that enjoyed some popularity in New York through the 1960's. Since then, the performance of this device was found to be unsatisfactory and it has not been used by the New York State Department of Transportation since the early 1970's.

A complete listing of the original design and construction information for each section, as well as other pertinent project information, is provided in the summary tables of chapter 32.

This project does not utilize a full factorial experimental design in which each design variable is isolated so that its effect can be studied. Therefore, drawing conclusions about the comparative performance of various design variables is not entirely valid.

2. CLIMATE

This experimental section of Route 23 is located in the east central part of New York, in the wet-freeze environmental zone. The project site has a Thornthwaite Moisture Index of 54, a Corps of Engineers Freezing Index of 500, and receives an average of 39 in (991 mm) of rainfall annually. The highest average monthly maximum temperature is 80 $^{\circ}$F (27 $^{\circ}$C) and the lowest average monthly minimum temperature is 10 $^{\circ}$F (-12 $^{\circ}$C).

3. TRAFFIC

The roadway is a four-lane divided highway, with a functional classification of Rural Collector. Since opening to traffic in 1968, the pavement has sustained 3.1 million cumulative 18-kip (80 kN) Equivalent Single-Axle Load (ESAL) applications in the outer lane (through 1987). The inner lane has carried nearly 0.4 million cumulative ESAL's.

The two-way Average Daily Traffic (ADT) in 1986 was approximately 7250 vehicles, including 10 percent heavy trucks. Weigh-in-motion data collected at the project site in 1987 provided a traffic count of 10,100 vehicles per day, including 10 percent heavy trucks.

4. MAINTENANCE AND REHABILITATION

According to records provided by New York State DOT, no major maintenance or rehabilitation has been performed on these sections. The only maintenance work performed to date has been the placement of surface treatments on the asphalt concrete shoulders in 1970, 1975, 1982-83, and 1985.

5. PHYSICAL TESTING RESULTS

Cores from slab centers and from typical transverse joints were retrieved from the pavement sections in 1987. The center cores were tested in split tensile to provide an indication of the concrete strength. The joint cores were visually inspected for signs of deterioration beneath the joint or for any signs of material durability distress, such as microcracking in the aggregate.

There were no visible signs of deterioration at the bottom of the joint which might indicate a concrete durability problem. Some microcracking was observed in the core's aggregate.

The split tensile strength values were used to obtain an estimate of the modulus of rupture. These values are reported in chapter 32. A mean modulus of rupture of 786 psi (5.4 MPa) was obtained.

Deflection testing was performed on the sections in 1987 for purposes of layer moduli characterization, determination of load transfer efficiencies, and void detection. The elastic modulus of the concrete (E), the composite k-value (on top of the base), and the load transfer values are summarized for each section in chapter 32.

The slab E-values averaged 4,080,000 psi (28,130 MPa) for all of the sections. The support k-values for the sections are given in table 53.

Table 53. Composite k-value by base type.

Base Type	k-value, pci
Asphalt-Treated	560
Aggregate	577

There were insignificant differences in the average center slab deflections for the different base course sections. The subgrade soil AASHTO classification was generally an A-2-4 material, which is fairly stiff soil.

Table 54 provides the load transfer efficiencies for each section and the voids, or loss of support, detected for each section.

Table 54. Load transfer efficiencies and corners with voids at NY 1.

Section	Base Type	Load Transfer Devices	Joint Load Transfer, %	% Corners Exhibiting Voids
NY 1-1	ATB	Yes	100	55
NY 1-3	ATB	Yes	47	20
NY 1-4	AGG	Yes	94	25
NY 1-6	AGG	Yes	100	30
NY 1-8a	ATB	No	100	20
NY 1-8b	ATB	No	100	25

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, it is observed from the table that voids were detected in all of the sections.

6. DRAINABILITY OF PAVEMENT SECTIONS

A drainage analysis was performed on each section to evaluate the section's ability to remove water from the pavement structure. Layer permeabilities were calculated for each draining layer below the slab and, with other inputs from the field survey, the overall drainability characteristics of the section were identified. Results from this evaluation are summarized in table 55.

Table 55. Drainage summary for NY 1 sections.

Section	Base Permeability, ft/hr	AASHTO Drainage Coefficient	Overall Drainability
NY 1-1	0.00	0.90	Fair
NY 1-3	0.00	0.90	Fair
NY 1-4	0.11	0.80	V.Poor-Poor
NY 1-6	N/A	0.75	V.Poor-Poor
NY 1-8a	0.00	1.00	Fair-Good
NY 1-8b	0.00	1.05	Fair-Good

In general, the overall drainage characteristics of the sections with impermeable bases were better than those of the sections with aggregate base and subbase layers. These granular materials were fairly impervious to moisture, as is shown in their low drainage coefficient and overall drainability rating.

NEW YORK 1 RTE. 23 CATSKILL

OUTER LANE PERFORMANCE DATA

Pavement	Base	Measure	SKEWED JOINTS: NO LOAD TRANSFER	SKEWED JOINTS: LOAD TRANSFER	PERPENDICULAR JOINTS: NO LOAD TRANSFER	PERPENDICULAR JOINTS: 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.	Measure	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 12. Outer lane performance data for New York 1.

NEW YORK 1 RTE. 23 CATSKILL

INNER LANE PERFORMANCE DATA

			SKEWED JOINTS		PERPENDICULAR JOINTS		
			NO LOAD TRANSFER	LOAD TRANSFER	NO LOAD TRANSFER	LOAD TRANSFER	
9 in JPCP 20 ft JT. SPACING	AGG BASE					NY 1-6	AVG.
		PSR				3.6	3.6
		ROUGHNESS, in/mi				79	79
		FAULTING, in				0.01	0.01
		T. CKS./mi				9	9
		LONG. CKS., ft/mi				264	264
		% JT. SPALL.				10	10
	ATB		NY 1-8b		NY 1-8a	NY 1-1	
		PSR	4.1		4.0	4.0	4.0
		ROUGHNESS, in/mi	59		64	57	60
		FAULTING, in	0.01		0.01	0.01	0.01
		T. CKS./mi	0		9	0	3
		LONG. CKS., ft/mi	0		290	0	97
		% JT. SPALL.	0		0	3	1
9 in JRCP 60.8 ft JT. SPACING	AGG BASE					NY 1-4	
		PSR				4.0	4.0
		ROUGHNESS, in/mi				66	66
		FAULTING, in				0.02	0.02
		T. CKS./mi				9	9
		LONG. CKS., ft/mi				0	0
		% JT. SPALL.				9	9
	ATB					NY 1-3	
		PSR				4.0	4.0
		ROUGHNESS, in/mi				76	76
		FAULTING, in				0.03	0.03
		T. CKS./mi				0	0
		LONG. CKS., ft/mi				0	0
		% JT. SPALL.				73	73

LOAD TRANSFER DEVICES ARE ACME DEVICES.

		SKEWED NO LOAD TRANSFER	SKEWED LOAD TRANSFER	PERPENDICULAR NO LOAD TRANSFER	20 ft JOINTS	60.8 ft JOINTS
AVG.	PSR	4.1		4.0	3.8	4.0
	ROUGHNESS, in/mi	59		64	68	70
	FAULTING, in	0.01		0.01	0.01	0.03
	T. CKS./mi	0		9	5	5
	LONG. CKS., ft/mi	0		290	132	0
	% JT. SPALL.	0		0	7	41

Figure 13. Inner lane performance data for New York 1.

7. DETERIORATION OF PAVEMENT SECTIONS

In order to better present the results of the extensive distress survey conducted on each pavement section, as well as to demonstrate the limited extent of the factorial design of the experiment, the primary distress results are tabulated in figure 12 (outer lane) and figure 13 (inner lane). The relative performance of each pavement section with respect to the primary distress types are discussed below for the outer lane only.

Joint Spalling

Medium and high severity joint spalling are much higher for the sections with load-transfer devices than for the sections without them. This is the overriding factor affecting joint spalling; there is no clear trend which suggests an effect of base types. The data shows that the long-jointed sections had a higher percentage of spalled joints (9 to 73 percent) than the shorter slabs. The one short-jointed section with skewed joints and no load transfer exhibited no joint spalling, nor did the short-jointed section with perpendicular joints and no load transfer.

Best Performance

The pavement section displaying the best performance, in terms of joint spalling, had the following characteristics:

- o The sections with 20-ft (6.1 m) joint spacing on an asphalt-treated base, without load-transfer devices.

Worst Performance

The worst performance, in terms of joint spalling, was demonstrated by the pavement sections with the following design features:

- o The section with 60.8-ft (18.5 m) joint spacing, on an asphalt-treated base, with load-transfer devices. This section had spall repairs on 73 percent of the transverse joints.

Joint Faulting

Generally speaking, joint faulting for the project was low. The average transverse joint faulting was much higher for the long-jointed sections than for the short-jointed sections. There is no significant difference in the faulting observed on the two different base types. The faulting was higher on the sections with the load-transfer devices, which is somewhat surprising. However, if the long-jointed sections are not considered, then there is no real difference between the faulting on the sections with or without the devices. The effect of skewed joints could not be isolated from other factors in order to assess its effect on faulting.

Best Performance

Pavement sections with the best performance, in terms of joint faulting, had the following design features:

- Any of the short-jointed sections on either aggregate or asphalt-treated base course, both with and without load transfer devices.

Worst Performance

The pavement sections displaying the worst performance, in terms of joint faulting, had the following characteristics:

- Either of the long-jointed sections.

Transverse Cracking

Transverse cracking, measured by the number of deteriorated 12-ft (3.7 m) cracks per mile (low-severity transverse cracks on JRCP are not included) was not observed on the sections with 60.8-ft (18.5 m) joint spacing. Among the sections with 20-ft (6.1 m) joint spacing and an ATB, only the section with perpendicular joints and no load transfer exhibited any transverse cracking. The section with load-transfer devices and an AGG base had more transverse cracking than either of the other sections built on an ATB base. With the exception of the short-jointed doweled section on an aggregate base, all of the sections had transverse cracks which radiated from old core holes in the center of slabs. These were not counted in the distress summaries.

Best Performance

In terms of deteriorated transverse cracks, the best performance was exhibited by pavement sections with the following characteristics:

- Either of the sections with 60.8-ft (18.5 m) joint spacing. These both had load transfer devices, and there was no difference in performance between the two base types. Two of the short-jointed sections on ATB also had no deteriorated transverse cracks, although the effect of the other variables is confounded.

Worst Performance

The worst performance, in terms of deteriorated transverse cracks, was displayed by pavement sections with the following design features:

- The short-jointed sections with perpendicular joints. There is not enough data to draw a conclusion about the effect of the other variables.

Longitudinal Cracking

Longitudinal cracking was not observed in the outer lane of any of the six sections evaluated. The depth of sawcut of the longitudinal joint was 2 in (51 mm).

Present Serviceability Rating (PSR) and Roughness

The Present Serviceability Rating (PSR) and roughness measurements for each section are summarized in table 56.

Table 56. Roughness and present serviceability on NY 1 sections.

Section	Base Type	Joint Spacing, ft	Roughness, in/mi	PSR
NY 1-1	ATB	20	56	4.0
NY 1-3	ATB	60.8	79	3.6
NY 1-4	AGG	60.8	90	3.4
NY 1-6	AGG	20	82	3.9
NY 1-8a	ATB	20	64	4.1
NY 1-8b	ATB	20	65	3.8

The measured roughness was generally low and the PSR high for these sections. The exceptions are the two sections constructed with 60.8-ft (18.5 m) joint spacing. These sections exhibited the lowest PSR and the highest roughness of the group. The best performance was measured on the short-jointed sections with an ATB. The sections with load-transfer devices had lower PSR's and higher roughness values, consistent with the increased amount of other distresses noted on these sections.

8. EFFECT OF DESIGN FEATURES ON PAVEMENT PERFORMANCE

The effect of the various design features on the performance of the different sections is not always readily apparent. As was mentioned earlier, the experiment was not set up as a full factorial design, so that some of the variables are not isolated. Some comments about the relative effect of these design features are summarized below.

Base Type

The asphalt-treated base course sections performed better than the aggregate base types on the short-jointed sections. This is primarily based on the amount of cracking (both transverse and longitudinal) and joint spalling present in the aggregate sections. On the long-jointed sections, the aggregate base had less faulting and spalling than the ATB section, but had a higher Roughness Index and a lower PSR.

Load Transfer Devices

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 were the cause of many joint-related performance problems and were not effective in reducing faulting.

Joint Spacing

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. These sections also had higher faulting and joint spalling than the other sections.

Joint Orientation

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.

9. COMPARISON OF OUTER LANE AND INNER LANE PERFORMANCE

The inner lanes have been subjected to many fewer traffic loadings than the outer lanes. However, the performance of the inner lane sections can still provide some insight into the performance of the design features. In fact, many of the performance trends observed in the outer lane hold true for the inner lane. For instance, the asphalt-treated base course sections generally showed better performance than the aggregate base sections. Also, the short-jointed sections performed better than the long-jointed sections, although the long-jointed sections performed fairly well. Finally, the sections with load-transfer devices performed much worse than the section that did not have any load-transfer device. This particular type of load-transfer device is no longer used.

An overall comparison of the two traffic lanes is provided in table 57. The amount of joint spalling which occurred in the inner lane is about

Table 57. Comparison of performance by lane at NY 1.

Variable	Inner Lane	Outer Lane
ESAL's (millions)	0.4	3.1
Joint Spalling, %	16	17
Joint Faulting, in	0.02	0.05
Trans. Cracks/mile	5	7
Long. Cracks, ft/mile	92	0
Roughness, in/mi	67	73
PSR	4.0	3.8

the same as that found in the outer lane. This suggests a relative insensitivity of spalling to traffic. The faulting for the inner lane is about half that of the outer lane, which is consistent with the load-associated causes of faulting. The number of deteriorated transverse cracks was higher in the outer lane than in the inner lane, indicating some effect of the higher truck traffic levels. As would be expected, the rideability and roughness are slightly better for the inner lane than for the outer lane.

Longitudinal cracking was observed only in the inner lane. This distress is most likely caused by construction problems, such as insufficient depth of the longitudinal lane-lane joint. Recall that the longitudinal lane-lane joint was sawed to a depth of 2 in (51 mm), or about 22 percent of the slab thickness.

10. SUMMARY AND CONCLUSIONS

Analysis of the sections in this project is somewhat difficult due to the limited design of the experiment and other confounding variables. However, several observations can be made.

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 (two-part malleable iron devices) were not effective in reducing faulting.

11. ADDITIONAL READING

Bryden, J. E. and R. G. Phillips, "The Catskill-Cairo Experimental Rigid Pavement: A Five-Year Progress Report," Report NYSDOT-ERD-73-PR-17, November 1973.

Vyce, J. M. and R. G. Phillips, "The Catskill-Cairo Experimental Rigid Pavement: A Ten-Year Progress Report," Report FHWA/NY/RR-81/91, July 1981.

Vyce, J. M., "A Summary of Experimental Concrete Pavements in New York," Report FHWA/NY/RR-88/141, Final Report, June 1988.

CHAPTER 18 INTERSTATE 88 — OTEGO, NEW YORK

1. INTRODUCTION

This project was designed by the New York State Department of Transportation (NYSDOT) as part of an ongoing evaluation of concrete pavement performance in New York. An earlier experimental project had been constructed on Rte. 23, between Catskill and Cairo, to evaluate the effects of base and subbase type, slab length, slab thickness, reinforcement, and mechanical load transfer devices on pavement performance. While the DOT had found the results from this project encouraging, there was some concern that this rural collector highway was not carrying enough heavy truck loads (AADT of 5000, with 8 percent trucks) to evaluate the performance of these pavement designs under Interstate conditions. Thus in 1975, construction of this project on I-88 from Otego to Unadilla was initiated and completed.

The design variables on I-88 include joint spacing, shoulder type, and pavement type. Jointed plain concrete pavement (JPCP) slabs were constructed with three different joint spacings, 20 ft (6.1 m), 23.3 ft (7.1 m), and 26.7 ft (8.1 m), and concrete shoulders. A control section was built which consisted of 63.5-ft (19.3 m) JRCP slabs and an asphalt concrete shoulder. Other variables which New York included in this experimental project were a 7-in (178 mm) thick JPCP local road, sections with each of the different joint spacings constructed without a longitudinal joint, sawing and sealing of the lane shoulder joint on one project and hand-forming and leaving unsealed the joint on the other, and construction of rumble strips on the concrete shoulder at 60-ft (18.3 m) intervals and at 100-ft (30.5 m) intervals.

All told, 15 different designs, designated NY 2-1 through 2-15, were constructed in this project, covering 3 different construction contracts. The construction was done over more than one season, as the total length of the three projects was in excess of 15 miles (24 km). Four sections, NY 2-3, 2-9, 2-11, and 2-15, were selected for inclusion in this study and surveyed and tested in 1987. NY 2-15, consisting of New York's traditional long-jointed reinforced pavement design at the time, was the control section. A complete listing of the original design and construction information for each section, as well as other pertinent project information, is provided in the summary tables of chapter 32. While this is not a full factorial experimental design, it is intended that the performance of the designs with the different joint spacings can be compared to the conventional design.

Common to all of the designs evaluated are 9-in (229 mm) thick pavement slabs and an aggregate base. In the construction process, additional material was added to the subgrade to reach grade, resulting in the creation of a subbase for some of the sections. Two short slab sections with 20-ft (6.1 m) joint spacing are included, one with and one without a sealed lane-shoulder joint. One section with 26.7-ft (8.1 m) joint spacing and an unsealed lane-shoulder joint is included. Inner shoulder type is the same as the outer shoulder for all sections. The load transfer devices used on all of the sections are 1-in (25 mm), epoxy-coated I-beams. These were the standard in New York at the time of construction, having replaced the two-part malleable load transfer devices used earlier at Catskill.

2. CLIMATE

This group of sections on I-88 is located near the Pennsylvania border, in the eastern third of New York. This is part of the wet-freeze environmental zone. The project site has a Thornthwaite Moisture Index of 53, a Corps of Engineers Freezing Index of 500, and receives an average of 41 in (1041 mm) of rainfall annually. For the project site, the highest average maximum monthly temperature is 83 ^{o}F (28 ^{o}C) and the lowest average monthly minimum temperature is 13 ^{o}F (-11 ^{o}C).

3. TRAFFIC

This roadway's functional classification is Rural Interstate. It is a four-lane divided highway with two through lanes in each direction. The three projects include a number of interchanges, which means that the through traffic is not the same over all of the sections. However, the difference in traffic levels is probably not that significant. Also, while the sections were not all opened at the same time, the differences in traffic levels 12 years later are not significant.

Since all of the sections have been open to traffic in 1975, the outer lanes have sustained 1.4 million 18-kip (80 kN) Equivalent Single-Axle Load (ESAL) applications (through 1987). The inner lanes have been subjected to approximately 0.14 million ESAL's.

The projected two-way Average Daily Traffic (ADT) in 1987 was approximately 8500 vehicles, including 10 percent heavy trucks. The Weigh-in-Motion data collected at the project site in 1987 provided a traffic count of 9447 vehicles per day, including 18 percent heavy trucks.

4. MAINTENANCE AND REHABILITATION

According to records provided by the NYSDOT, the entire project was the subject of a joint seal replacement contract in 1985. A silicone sealant was used. Transverse and longitudinal cracking which occurred early in the life of the pavement was sealed on an ongoing basis. Extensive spalling occurred along the tied concrete shoulders where the ties entered either the shoulder or the traffic lane. These spalls were repaired with several cementitious and bituminous patching materials as they occurred.

Differential settlement on the AC shoulder along the control section caused a step-down between the traffic lane and the shoulder. It was repaired by the application of a wedge of bituminous material on the shoulder along the pavement edge. No other maintenance and rehabilitation has been performed to date.

5. PHYSICAL TESTING RESULTS

Cores from slab centers and from typical transverse joints were retrieved from the pavement sections in 1987. The center cores were tested in split tensile to provide an indication of the concrete strength. The joint cores were visually inspected for signs of deterioration beneath the joint or for any signs of material durability distress (microcracking in the aggregate).

There were no visible signs of deterioration at the bottom of the joint cores. Low-severity microcracking was observed in the aggregate from the cores on sections 2-11 and 2-15.

The split tensile strength values were used to obtain an estimate of the modulus of rupture. These values are reported in chapter 32. A mean modulus of rupture of 794 psi (5.5 MPa) was obtained from the testing performed on these sections.

Deflection testing was performed on the sections in 1987 for purposes of layer moduli characterization, determination of load transfer efficiencies, and void detection. The elastic modulus of the concrete (E), the composite k-value (on top of the base), and the load transfer efficiency values are summarized for each section in chapter 32.

The slab E-values averaged 5,460,000 psi (37,650 MPa) for all of the sections. An aggregate base course was used for all of the sections, but the composite k-values for the sections varied as shown in table 58.

Table 58. Composite k-values and average mid-slab deflections at NY 2.

Section	k-value, pci	Mid-Slab Deflection, mils
NY 2-3	341	3.3
NY 2-9	471	2.7
NY 2-11	296	3.2
NY 2-15	273	3.5

The aggregate material came from different locations and had different gradations. The average center slab deflections varied from 2.7 to 3.5 mils, as shown above, and follow the same pattern as the k-values. The subgrade soil throughout the project was generally classified as an AASHTO A-1-a material. This is a stiff subgrade and may explain the high k-values that were obtained.

The load transfer efficiencies were "good" to "excellent" for all of the sections. Using deflection-based void detection procedures, voids were detected under the joint corners of three of the sections. The percent of slab corners with voids in a section ranged from highs of 90 percent in NY 2-3 and 50 percent in NY 2-9, to 5 percent in 2-11 and none in 2-15. These results are summarized in table 59.

Table 59. Load transfer efficiencies and corners with voids at NY 2.

Section	Joint Load Transfer, %	% Corners Exhibiting Voids
NY 2-3	100	90
NY 2-9	97	50
NY 2-11	81	5
NY 2-15	99	0

6. DRAINABILITY OF PAVEMENT SECTIONS

A drainage analysis was performed on each section to estimate the section's ability to remove water from the pavement structure. Layer permeabilities were calculated for each draining layer below the slab and, with other inputs from the field survey, the overall drainability characteristics of the section were identified. The results from this evaluation are summarized in table 60.

Table 60. Drainage summary for NY 2 sections.

Section	Base Permeability, ft/hr	AASHTO Drainage Coefficient	Overall Drainability
NY 2-3	N/A	1.00	Poor-Good
NY 2-9	0.25	0.85	V.Poor-Fair
NY 2-11	N/A	0.80	V.Poor-Fair
NY 2-15	0.17	0.85	V.Poor-Fair

Insufficient base material was recovered from sections 2-3 and 2-11 to compute a base permeability. Since the specifications for this layer were identical, the drainabilities obtained from 2-9 and 2-15 were used to provide a guideline.

7. DETERIORATION OF PAVEMENT SECTIONS

The primary distress results are also shown in figure 14 (for the outer lane) and figure 15 (for the inner lane). These tables show that with the exception of the two different 20-ft (6.1 m) sections, no direct comparisons can be made between the designs. The relative performance of each pavement section with respect to the primary distress types are discussed below for the outer lane.

Joint Spalling

No joint spalling was recorded for any of these sections which, as noted earlier, all have epoxy-coated I-beams across the transverse joints. The transverse joints on all of the sections were resealed in 1985 with a silicone sealant and the sealant is in excellent condition throughout the project.

Joint Faulting

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.

Transverse Cracking

Transverse cracking distress is recorded as the number of deteriorated 12-ft (3.7 m) cracks per mile (low-severity transverse cracks on JRCP are <u>not</u> included). The three JPCP sections with tied PCC shoulders showed roughly the same amount of transverse cracking; none was recorded on the JRCP section

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 14. Outer lane performance data for New York 2.

NEW YORK 2 I-88 OTEGO

INNER LANE PERFORMANCE DATA

			PCC SHOULDER	AC SHOULDER
9 in JPCP	20 ft JT. SPACING SEALED LANE-SHLDR. JOINT		NY 2-3	
		PSR	4.0	
		ROUGHNESS, in/mi	78	
		FAULTING, in	0.01	
		T. CKS./mi	35	
		LONG. CKS., ft/mi	0	
		% JT. SPALL.	0	
	20 ft JT. SPACING UNSEAL. LANE-SHLDR. JOINT		NY 2-9	
		PSR	4.1	
		ROUGHNESS, in/mi	62	
		FAULTING, in	0.01	
		T. CKS./mi	10	
		LONG. CKS., ft/mi	0	
		% JT. SPALL.	0	
	26.6 ft JT. SPACING		NY 2-11	
		PSR	4.0	
		ROUGHNESS, in/mi	58	
		FAULTING, in	0.01	
		T. CKS./mi	13	
		LONG. CKS., ft/mi	0	
		% JT. SPALL.	0	
9 in JRCP	63.5 ft JT. SPACING			NY 2-15
		PSR		4.2
		ROUGHNESS, in/mi		57
		FAULTING, in		0.00
		T. CKS./mi		0
		LONG. CKS., ft/mi		161
		% JT. SPALL.		6

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

		PCC SHOULDER	AC SHOULDER
AVG.	PSR	4.0	4.2
	ROUGHNESS, in/mi	66	57
	FAULTING, in	0.01	0.00
	T. CKS./mi	19	0
	LONG. CKS., ft/mi	0	161
	% JT. SPALL.	0	6

Figure 15. Inner lane performance data for New York 2.

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, as reported in New York's 5-year Performance Summary.(8) 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. Table 61 summarizes the amount of transverse cracking in the outer lane and in the inner lane to show the effect of traffic.

Table 61. Transverse cracking summarized by lane and transverse joint spacing.

Section	Joint Spacing, ft	Transverse Cracks/mile Outer Lane	Inner Lane
NY 2-3	20	35	35
NY 2-9	20	25	10
NY 2-11	26.7	26	13
NY 2-15	63.5	0	0

Longitudinal Cracking

Longitudinal cracking, measured in linear feet per mile, was also 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 (13.9 m/km) of longitudinal cracking. Longitudinal cracking is summarized in table 62.

Table 62. Longitudinal cracking summarized by lane and transverse joint spacing.

Section	Joint Spacing, ft	Longitudinal Cracks, ft/mi Outer Lane	Inner Lane
NY 2-3	20	0	0
NY 2-9	20	573	0
NY 2-11	26.7	0	0
NY 2-15	63.5	73	161

Longitudinal cracking, like transverse cracking, was a distress which occurred prematurely in the life of this project. While the quantities of longitudinal cracking reported by NY DOT are not directly comparable to those from the field surveys because of different measuring techniques, the relative amounts of cracking among the projects found by the DOT's survey is very similar to that found during the 1987 field surveys. 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.

Present Serviceability Rating (PSR) and Roughness

Each pavement section was evaluated in terms of rideability (PSR) and roughness (using a Mays Ridemeter). These results are shown in table 63.

Table 63. Roughness and present serviceability at NY 2.

Section	Joint Spacing, ft	Roughness, in/mi	PSR
NY 2-3	20	61	4.2
NY 2-9	20	66	4.0
NY 2-11	26.7	76	4.1
NY 2-15	63.5	63	4.0

The measured roughness was universally low and the PSR quite high for these 12-year old sections. 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.

Concrete Shoulder Distress Summary

The concrete shoulders also experienced a significant amount of distress in the form of extensive longitudinal cracking and some transverse cracking. The cracking for the three sections with concrete shoulders is summarized in table 64. It is believed that these cracks occurred shortly after construction.

Table 64. Summary of slab cracking on sections with concrete shoulders.

Section	Transverse Cracks/mile	Longitudinal Cracks, ft/mile
NY 2-3	30	299
NY 2-9	0	1420
NY 2-11	0	396

8. EFFECT OF DESIGN FEATURES ON PAVEMENT PERFORMANCE

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 compared 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, as seen from table 65. 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 not a result of load-related causes.

Table 65. Performance comparison of sections with 20-ft transverse joint spacing.

Section	Joint Spacing, ft	Lane-Shoulder Joint Sealed	Joint Faulting, in	Transverse Cracks/mi	Longitudinal Cracks, ft/mi
NY 2-3	20	Yes	0.01	35	0
NY 2-9	20	No	0.02	25	573

The effects of joint spacing and shoulder type are confounded by pavement type.

9. COMPARISON OF OUTER LANE AND INNER LANE PERFORMANCE

The inner lanes have been subjected to fewer traffic loadings than the outer lanes. However, the performance of the inner lane sections can provide some insight into the performance of the design features, especially those whose deterioration is load-related. An overall comparison of the two traffic lanes is provided in table 66. Performance indicators are separated for the JPCP sections and the JRCP sections.

Table 66. Comparison of performance by lane and pavement type at NY 2.

	JPCP		JRCP	
Variable	Inner Lane	Outer Lane	Inner Lane	Outer Lane
ESAL's (millions)	0.14	1.4	0.14	1.4
Joint Spalling, %	0	0	6	0
Joint Faulting, in	0.01	0.01	0.00	0.02
Trans. Cracks/mi	19	29	0	0
Long. Cracks, ft/mi	20	181	161	73
Roughness, in/mi	68	66	57	63
PSR	4.1	4.0	4.2	4.0

In general, the PSR and roughness are slightly better in the inner lane than in the outer lane. The average faulting is also lower, although the faulting was very low on both sections. The trend for transverse and longitudinal cracking is ambiguous, perhaps because as they occurred on this project these distresses are most likely not related to traffic loading.

10. SUMMARY AND CONCLUSIONS

This experimental project exhibits 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 tie bars.

These problems make it difficult to attribute any of the distresses that were observed during the 1987 field survey to other factors. This abundance of distress also means that the selection of the sections might have had an inadvertent undesired effect on pavement performance. Every effort was made in section selection to identify a length of pavement on a flat tangent. This was not always possible, and the distress density described by Vyce was much different for left curves, right curves, and tangents.

Despite the fact that NYSDOT wanted to subject the experimental designs on I-88 to a greater volume of traffic than existed on Rte. 23 at Catskill, the AADT 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.

Other pertinent conclusions from the Vyce report include: the observation that there did not appear to be any difference in performance between the 20-ft (6.1 m), 23.3-ft (7.1 m) and 26.7-ft (8.1 m) slabs; recommendations on changes in the tied shoulder design to improve its performance; the attribution of most of the transverse and longitudinal cracking to subgrade discontinuities and moisture problems.

Further observations from a 1987 update on the status of New York's experimental projects include the fact that increased cracking had occurred in the longer JPCP slabs, suggesting that 20-ft (6.1 m) joint spacing was a practical limit for JPCP construction, and the observation that the PCC shoulders have required less maintenance than the AC shoulders and that the distresses can be avoided by sawing and sealing the lane-shoulder joint and constructing the shoulder to the same thickness as the mainline pavement.(9)

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, it is believed that such devices would be inappropriate on heavily-trafficked roadways. Round dowels provide greater load transfer than the I-beam cross-section and can be placed using automatic dowel inserters. It is believed that round, coated dowels of sufficient diameter are more suitable devices to provide good load transfer.

CHAPTER 19 U.S. 23 — CHILLICOTHE, OHIO

1. INTRODUCTION

This experimental project, located on U.S. 23 near Chillicothe, Ohio, was constructed in 1973. The experiment was conducted to evaluate several design parameters which affect the behavior of rigid pavements, with the objective being to develop an improved contraction joint for PCC pavements. The principle design variables include joint spacing, base type, and type of dowel bar coating. The dowel bars were either plastic-coated or painted and greased (the Ohio Department of Transportation's "standard" coating).

Twelve experimental sections were constructed in the southbound roadway in 1973. The sections varied in length from 40 ft (12.2 m) to 400 ft (121.9 m), and incorporated different design features. The short length of these sections is a limitation in obtaining a representative sample of performance data.

Of the 12 sections available, 7 sections, designated as OH 1-1 through OH 1-10, were selected for inclusion in this study. These sections were surveyed and tested in 1987. All of the sections are 9-in (229 mm) JRCP with 0.09 percent reinforcing steel, contain 1.25-in (32 mm) diameter dowel bars (although the coating varies), and are constructed on an AASHTO Classification A-2-4 subgrade. It should be noted that the subgrade in this instance is actually about 20 to 35 ft (6.1 to 10.7 m) of granular fill material; therefore, the actual in-situ subgrade soil does not directly impact the performance of these pavement sections. A complete listing of design variables for the sections is provided in the summary tables of chapter 32.

2. CLIMATE

The pavement sections are located in south-central Ohio, in a wet-freeze environmental zone. The project site has a Thornthwaite Moisture Index of 33, a Corps of Engineers Freezing Index of 25, and receives an average of 39 in (991 mm) of precipitation annually. The highest average monthly maximum temperature for the project site is 84 ^{o}F (29 ^{o}C) while the lowest average monthly minimum temperature is 19 ^{o}F (-7 ^{o}C).

3. TRAFFIC

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) Equivalent Single-Axle Load (ESAL) applications in the outer lane and nearly 0.6 million ESAL applications in the inner lane. These values represent calculations through 1987.

The two-way Average Daily Traffic (ADT) in 1986 was approximately 12,320 vehicles, including 12 percent heavy trucks. Weigh-in-motion data collected at the project site in 1987 provided a traffic count of 14,650 vehicles per day, including 10 percent heavy trucks.

4. MAINTENANCE AND REHABILITATION

According to records provided by the Ohio Department of Transportation, no major rehabilitation has been performed on the sections to date.

5. PHYSICAL TESTING RESULTS

Cores from slab centers and from typical transverse joints were retrieved from the pavement sections in 1987. The center cores were tested in split tensile to provide an indication of the concrete strength. The split tensile strength values were used to obtain an estimate of the modulus of rupture, as provided in chapter 32. A mean modulus of rupture of 783 psi (5.4 MPa) was determined for all of the sections.

The joint cores were visually inspected for signs of deterioration beneath the joint and for any signs of material durability distress (microcracking in the aggregate). Examination of the joint cores taken from the pavement sections indicated that only one section exhibited deterioration beneath the joint and microcracking in the aggregate. This was OH 1-1, which had 40-ft (12.2 m) joint spacing and was constructed on an aggregate base course. The joint cores from the remaining sections had no such indications of deterioration or microcracking.

Deflection testing was performed on the sections in 1987 for purposes of layer moduli characterization, determination of load transfer efficiencies, and void detection. The elastic modulus of the concrete (E), the composite k-value, and the load transfer efficiency values are summarized for each section in chapter 32 of this volume.

The slab E values averaged 4,481,000 psi (30,900 MPa) for all of the sections. The composite k-value (on top of the base) varied by base type, as shown in table 67.

Table 67. Composite k-values at OH 1.

Base Type	k-value, pci
Aggregate	395
Asphalt-Treated (ATB)	482

The asphalt-treated bases typically displayed greater slab support than the aggregate base. The center slab deflections were highest for the aggregate base course sections (averaging 3.4 mils) and much lower for the asphalt treated base course sections (averaging 2.6 mils). Recall that the sections were all constructed on 20 to 35 ft (6.1 to 10.7 m) of granular fill material (AASHTO Classification A-2-4).

The transverse joint load transfer efficiencies and the percent of slab corners exhibiting voids (loss of support) are reported in table 68.

Table 68. Load transfer efficiencies and corners with voids at OH 1.

Section	Base Type	Joint Load Transfer, %	% Corners Exhibiting Voids
OH 1-1	Aggregate	95	71
OH 1-3	ATB	79	0
OH 1-4	ATB	70	0
OH 1-6	Aggregate	100	94
OH 1-7	Aggregate	100	85
OH 1-9	Aggregate	86	95
OH 1-10	Aggregate	71	0

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 voids (loss of support) for the sections were determined using deflection-based void detection procedures. Generally speaking, the aggregate base course sections are displaying an extensive amount of loss of support; the asphalt-treated base course sections did not display any voids. However, one aggregate base course section with 21-ft (6.4 m) joint spacing (OH 1-10) is also not exhibiting any voids.

6. DRAINABILITY OF PAVEMENT SECTIONS

A drainage analysis was performed on each section to evaluate the section's ability to remove water from the pavement structure. The fill was determined to be a well-drained material. The sections have the potential to be saturated 19 percent of the time. The results of the drainability analysis are reported in table 69.

Table 69. Drainage summary for OH 1 sections.

Section	Base Permeability, ft/hr	AASHTO Drainage Coefficient	Overall Drainability
OH 1-1	0.27	0.95	Fair-Good
OH 1-3	0.00	0.90	Fair
OH 1-4	0.00	0.90	Fair
OH 1-6	0.06	0.85	V. Poor-Poor
OH 1-7	7.25	1.07	Good
OH 1-9	0.16	0.90	Poor-Fair
OH 1-10	0.58	1.05	Fair-Good

The ATB was assumed to be impermeable, but was assisted by the well-drained subgrade. This resulted in those sections with ATB (OH 1-3 and OH 1-4) being rated as fair. The aggregate base course varied in permeability for the remaining sections. Those sections with a good aggregate base permeability were rated from fair-good in terms of overall drainability (e.g., OH 1-1, OH 1-7, OH 1-10); those sections with poor aggregate base permeability were rated poor-fair in terms of overall drainability (OH 1-6 and OH 1-9).

7. DETERIORATION OF PAVEMENT SECTIONS

The primary performance indicators are tabulated in figure 16 (outer lane) and figure 17 (inner lane). The relative performance of each pavement section with respect to the primary performance indicators are discussed below for the outer lane only.

Joint Spalling

Only one section exhibited medium- and high-severity joint spalling. This was OH 1-3, which was constructed on an ATB, had 21-ft (6.4 m) joint spacing, and the standard dowel coating. It is somewhat surprising that a section with short joint spacing would be the only one displaying joint spalling. However, only 13 percent of the joints in this section exhibited joint spalling after 14 years of service.

OHIO 1 RTE. 23 CHILLICOTHE

OUTER LANE PERFORMANCE DATA

			BASE TYPE			
			AGGREGATE		ATB	
9 in JRCP 40 ft JT. SPACING	STD. DOWELS		OH 1-1	OH 1-9	OH 1-4	AVG.
		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
	PLASTIC COATED DOWELS		OH 1-7			
		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	STD. DOWELS		OH 1-10		OH 1-3	
		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
	PLASTIC COATED DOWELS		OH 1-6			
		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 16. Outer lane performance data for Ohio 1.

OHIO 1 RTE. 23 CHILLICOTHE

INNER LANE PERFORMANCE DATA

			BASE TYPE			
			AGGREGATE		ATB	
9 in JRCP 40 ft JT. SPACING	STD. DOWELS		OH 1-1	OH 1-9	OH 1-4	AVG.
		PSR	4.2	4.4	4.2	4.3
		ROUGHNESS, in/mi	113	108	115	112
		FAULTING, in	0.06	0.07	0.04	0.06
		T. CKS./mi	22	40	29	30
		LONG. CKS., ft/mi	0	0	0	0
		% JT. SPALL.	0	0	0	0
	PLASTIC COATED DOWELS		OH 1-7			
		PSR	4.4			4.4
		ROUGHNESS, in/mi	105			105
		FAULTING, in	0.06			0.06
		T. CKS./mi	73			73
		LONG. CKS., ft/mi	0			0
		% JT. SPALL.	0			0
9 in JRCP 21 ft JT. SPACING	STD. DOWELS		OH 1-10		OH 1-3	
		PSR	4.2		4.2	4.2
		ROUGHNESS, in/mi	108		113	111
		FAULTING, in	0.07		0.03	0.05
		T. CKS./mi	0		0	0
		LONG. CKS., ft/mi	0		0	0
		% JT. SPALL.	0		0	0
	PLASTIC COATED DOWELS		OH 1-6			
		PSR	4.4			4.4
		ROUGHNESS, in/mi	88			88
		FAULTING, in	0.04			0.04
		T. CKS./mi	0			0
		LONG. CKS., ft/mi	0			0
		% JT. SPALL.	0			0
AVG.		PSR	4.3		4.2	
		ROUGHNESS, in/mi	104		114	
		FAULTING, in	0.06		0.04	
		T. CKS./mi	27		14	
		LONG. CKS., ft/mi	0		0	
		% JT. SPALL.	0		0	

Figure 17. Inner lane performance data for Ohio 1.

Although not exhibited by the joint cores retrieved on this project, reports indicate that a substantial amount of spalling was occurring on the underside of the joints, especially for the sections with the asphalt-treated base course.(10,11,12) It is possible that those sections constructed with asphalt-treated bases resulted in bathtub type designs, so called because water cannot drain easily from this cross-section. This results in the saturation of the PCC slab that could accelerate the deterioration of the transverse joints. However, the good drainability of the subgrade would help somewhat to mitigate that situation.

Joint Faulting

Joint faulting for the sections varied considerably. The average joint faulting for the sections is summarized in table 70.

Table 70. Transverse joint faulting at OH 1.

Section	Base Type	Joint Spacing, ft	Dowel Coating	Average Faulting, in
OH 1-3	ATB	21	Standard	0.06
OH 1-4	ATB	40	Standard	0.07
OH 1-6	Aggregate	21	Plastic	0.03
OH 1-7	Aggregate	40	Plastic	0.07
OH 1-10	Aggregate	21	Standard	0.10
OH 1-1/1-9	Aggregate	40	Standard	0.14

Several observations are apparent upon an examination of the faulting data. First, the plastic-coated dowel sections average a smaller amount of faulting than the standard dowel sections. Also, the asphalt-treated base course sections displayed less faulting than the aggregate base sections. Finally, for the aggregate base sections, the shorter section with 21-ft (6.4 m) joint spacing clearly has less faulting than the longer, 40-ft (12.2 m) slab sections.

Best Performance

In terms of joint faulting, the best performance was exhibited by pavement sections with the following characteristics:

- o 21-ft (6.4 m) joint spacing, aggregate base course, and plastic-coated dowels.

Worst Performance

The worst performance, in terms of joint faulting, was displayed by pavement sections with the following design features:

- o 40-ft (12.2 m) joint spacing, aggregate base course, and standard dowels.

Transverse Cracking

Deteriorated transverse cracking was most prevalent on the 40-ft (12.2 m) slabs. Three of the four sections with 40-ft (12.2 m) slabs displayed a significant amount of deteriorated transverse cracking (29, 106, and 235 deteriorated cracks per mile, compared to a critical level of about 70 per mile where rehabilitation is needed). Only one short-jointed section exhibited transverse cracking, and it was one that contained plastic-coated dowel bars. Interestingly, both sections with plastic-coated dowel bars exhibited more transverse cracking than the sections with standard dowels. Although it is possible that the plastic-coated dowels have "locked-up" due to corrosion, it may be more likely that the dowels were misaligned when installed. This would provide the same result as dowel lockup: opening up the transverse cracks which results in further crack breakdown and deterioration. On the section with plastic-coated dowels which exhibited 235 cracks per mile, the faulting of those cracks was 0.05 in (1.3 mm), approximately the same as the joint faulting for that same section (0.07 in [1.8 mm]). Finally, the pavement sections with asphalt-treated base courses performed better than the sections with aggregate base courses.

Best Performance

In terms of deteriorated transverse cracks, the best performance was exhibited by pavement sections with the following characteristics:

- 21-ft (6.4 m) joint spacing, aggregate base course, and standard dowels.

Worst Performance

The worst performance, in terms of deteriorated transverse cracks, was displayed by pavement sections with the following design features:

- 40-ft (12.2 m) joint spacing, aggregate base course, and plastic-coated dowels.

Longitudinal Cracking

No longitudinal cracking was observed on any of the pavement sections. The longitudinal joint was sawed to a depth of 2.25 in (57 mm).

Present Serviceability Rating (PSR) and Roughness

The rideability was uniform for all of the pavement sections. The Mays Roughness Index and the PSR are presented in table 71.

Table 71. Roughness and present serviceability at OH 1.

Joint Spacing, ft	Base Type	Roughness, in/mile	PSR
40	Aggregate	111	4.2
21	Aggregate	113	4.2
40	ATB	123	4.1
21	ATB	106	4.2

The PSR values are all very similar. No significant difference in terms of rideability or roughness was observed between the different joint spacings. This would seem to indicate that the sections with shorter joint spacings, although displaying much less joint faulting, are actually contributing to increased roughness, since there are more joints over a given length of section. While the joints on the 40-ft (12.2 m) sections may be rougher, they are only encountered half as often as the 21-ft (6.4 m) joints.

8. EFFECT OF DESIGN FEATURES ON PAVEMENT PERFORMANCE

This experimental project provided a factorial design that allows for the effects of some of the different design features to be evaluated. The relative effect of these design features are summarized below.

Base Type

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 ATB 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 under an average of 70 percent of the slab corners.

Joint Spacing

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.

Dowel Coating

The pavement sections all utilized 1.25-in (32 mm) diameter dowels, but with two different coatings. The standard dowel was simply painted and coated with grease and had no corrosion inhibitor. The other dowels were coated with plastic to reduce or inhibit corrosion. A proprietary coating manufactured by 3M was used on dowels in other test sections not included in this study.

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-coating 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. It is believed that the plastic-coated dowels may have been misaligned when installed, which could also result in the opening of mid-slab cracks.

9. COMPARISON OF OUTER AND INNER LANE PERFORMANCE

As part of the condition survey conducted on the pavement sections, the distresses in the inner lanes were also recorded. Although being subjected to far fewer traffic loadings than the outer lane, the inner lane sections provide an interesting comparison. Table 72 summarizes inner and outer lane performance by joint spacing:

Table 72. Comparison of performance by lane and base type of OH 1.

40-FT JOINT SPACING

	AGGREGATE BASE		ASPHALT-TREATED BASE	
Variable	Inner Lane	Outer Lane	Inner Lane	Outer Lane
ESAL's (millions)	0.6	3.4	0.6	3.4
Joint Spalling, %	0	0	0	0
Joint Faulting, in	0.06	0.11	0.04	0.07
Trans. Cracks/mile	45	113	29	29
Long. Cracks, ft/mi	0	0	0	0
Roughness, in/mi	109	111	115	123
PSR	4.3	4.2	4.2	4.1

21-FT JOINT SPACING

	AGGREGATE BASE		ASPHALT-TREATED BASE	
Variable	Inner Lane	Outer Lane	Inner Lane	Outer Lane
ESAL's (millions)	0.6	3.4	0.6	3.4
Joint Spalling, %	0	0	0	13
Joint Faulting, in	0.05	0.06	0.03	0.06
Trans. Cracks/mile	0	15	0	0
Long. Cracks, ft/mi	0	0	0	0
Roughness, in/mi	98	113	113	106
PSR	4.3	4.2	4.2	4.2

Faulting for the inner lane is less than that of the outer lane, although there is not a significant difference. There is less transverse cracking in the inner lane, with the most cracking occurring on the sections with the longer joint spacing. Joint spalling was practically nonexistent on either lane. There was no discernible difference in the roughness measurements and PSR readings obtained for each pavement section.

10. SUMMARY AND CONCLUSIONS

The most significant conclusion that can be drawn from this experiment is that joint spacing has a significant influence 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 bears repeating 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 (smaller levels of faulting, 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 were performing about the same at the time of survey. However, the pavement was only 14 years old at the time of survey and was surveyed before a lot of the transverse cracks had broken down on the sections with longer joint spacing. It is believed that once the transverse cracks break down on the longer slabs, 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 were typically displaying 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.

It should be noted that all of the sections within this project were extremely short in length. Therefore, if there are any localized problems such as subgrade failure or drainage problems, it would serve to detract too much from its overall performance. Similarly, if a section performs well over only a couple of hundred feet, this may not be representative of its true performance.

Conclusions drawn in various reports on the experimental project include:(10,11,12,13)

- The asphalt-treated base courses delay cracking on 40-ft (12.2 m) joints.
- The 21-ft (6.4 m) slabs are effective in reducing transverse cracking.
- There was no benefit gained from the coating of dowel bars.
- The asphalt-treated base course offers less resistance to horizontal movement than aggregate base so that stresses are reduced and there is less transverse cracking.
- The asphalt-treated base course reduces joint deflection under loading.
- There was no significant difference in the movement of a joint with standard dowels and coated dowels.

A follow-up research project has been approved that will study all of the test sections at this location.

11. ADDITIONAL READING

Minkarah, I., and A. Bodocsi, "Final Evaluation of the Field Performance of Ross 23 Experimental Concrete Pavement," State Job No. 14451(0), University of Cincinnati Research Proposal, June 1988.

CHAPTER 20 STATE ROUTE 2 — VERMILION, OHIO

1. INTRODUCTION

In 1974, an experimental project was constructed on State Route 2 near Vermilion, Ohio. The purpose of the experiment was to monitor the development of "D" cracking in concrete pavements in an effort to determine what factors influence the development of this distress. To accomplish this, pavement sections were constructed with various combinations of eleven different typical cross-sections, 10 different material variables, and eleven miscellaneous variables.

Two pavement sections from this project were included for study. These two sections both were thick slabs (15 in [381 mm]) placed directly on the subgrade. One section (designated OH 2-33a) was constructed with tied concrete shoulders, while the other section (designated OH 2-33b) was constructed with asphalt shoulders. The tied shoulders were 10 in (254 mm) thick at the mainline pavement edge, tapering to 6 in (152 mm) at the outer shoulder edge. The asphalt shoulder was 3 in (76 mm) thick.

Both sections had skewed, 20-ft (6.1 m) nondoweled joints that were sealed with a hot-poured rubberized asphalt joint sealant. No subdrainage was provided for either pavement section. The subgrade for OH 2-33a was determined to be an AASHTO Classification A-4 material, while the subgrade for OH 2-33b included both A-4 and A-6 materials. A complete listing of the original design and construction information for each section is provided in the summary tables of chapter 32.

2. CLIMATE

The pavement sections are located in extreme northern Ohio along Lake Erie, in the wet-freeze environmental zone. The project site has a Thornthwaite Moisture Index of 19, a Corps of Engineers Freezing Index of 380, and receives an average of 33.8 in (859 mm) of precipitation annually. The highest average monthly maximum temperature is 84 ^{o}F (29 ^{o}C) and the lowest average monthly minimum temperature is 19 ^{o}F (-7 ^{o}C).

3. TRAFFIC

The roadway is a four-lane divided highway, with a functional classification of Rural Minor Arterial. Since being opened to traffic in 1974, the pavement has sustained 3.3 million 18-kip (80 kN) Equivalent Single-Axle Load (ESAL) applications in the outer lane and nearly 0.6 million ESAL applications in the inner lane. These values represent calculations through 1987. The two-way Average Daily Traffic (ADT) in 1984 was approximately 12,700 vehicles, including 16 percent heavy trucks.

4. MAINTENANCE AND REHABILITATION

According to records provided by the Ohio Department of Transportation, no major rehabilitation or maintenance has been performed to date on the surveyed sections.

5. PHYSICAL TESTING RESULTS

Deflection testing and coring/boring operations were not performed on the sections due to financial constraints. Therefore, the data and conclusions presented here are based only on the condition survey conducted on each pavement section.

6. DRAINABILITY OF PAVEMENT SECTIONS

A drainage analysis was performed on both sections to evaluate the section's ability to remove water from the pavement structure. The sections are located in an area that has the potential for the pavement to be saturated approximately 14 percent of the time. Recall that the sections were constructed directly on subgrade and thus had no base course. Considering that the subgrade for OH 2-33a was somewhat poorly-drained and that the subgrade for OH 2-33b was somewhat poorly-drained to moderately-drained, the overall drainability for OH 2-33b is slightly better. This is illustrated in table 73.

Table 73. Drainage summary for OH 2 sections.

Section	AASHTO Drainage Coefficient	Overall Drainability
OH 2-33a	0.90	Poor
OH 2-33b	0.95	Poor

7. DETERIORATION OF PAVEMENT SECTIONS

The primary performance indicators obtained from the extensive condition survey performed on the sections are presented in figure 18. These primary performance indicators are discussed below for the outer lane only for each pavement section.

Joint Spalling

The percentage of transverse joints that exhibited medium- and high-severity joint spalling are shown in table 74.

Table 74. Transverse joint spalling at OH 2.

Section	Shoulder Type	Sealant Condition	Joint Spalling, %
OH 2-33a	Tied PCC	Fair	15
OH 2-33b	AC	Fair	23

Section OH 2-33b (AC shoulders) exhibited nearly twice as much joint spalling as 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. It appears that most of the spalling can be attributed to "D" cracking.

OHIO 2 RTE. 2 VERMILLION

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

		INNER LANE PERFORMANCE DATA	
		TIED PCC SHOULDER	AC SHOULDER
		OH 2-33a	OH 2-33b
15 in NONDOWELED JPCP 20 ft JT. SPACING NO BASE	PSR	3.3	3.5
	ROUGHNESS, in.mi	86	75
	FAULTING, in	0.04	0.03
	T. CKS./mi	0	0
	LONG. CKS., ft/mi	264	126
	% JT. SPALL.	3	11

Figure 18. Outer and inner lane performance data for Ohio 2.

As previously mentioned, the transverse joints in both sections were sealed with a hot-poured, rubberized asphalt sealant. At the time of the survey, the sealant was in fair condition. This would appear to be very good performance for a 13-year old product.

Joint Faulting

Both sections exhibited a large amount of faulting, although the average transverse joint faulting for both sections was exactly the same. The faulting information by section is presented in table 75.

Table 75. Transverse joint performance at OH 2.

Section	Average Faulting, in	Maximum Faulting, in	Pumping Severity
OH 2-33a	0.11	0.26	Low
OH 2-33b	0.11	0.24	Medium

Since the joints do not contain dowels and significant pumping was observed, the large amount of faulting present is not unexpected. However, this is a large amount of faulting for pavements of this age and traffic loading. There does not appear to be a difference in performance of the two sections with respect to faulting.

Transverse Cracking

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

Longitudinal cracking was observed in both pavement sections. Section OH 2-33a exhibited 104 linear feet of longitudinal cracking per mile (20 m/km) and OH 2-33b exhibited 148 linear feet of longitudinal cracking per mile (28 m/km). The cause of the longitudinal cracking is most likely the result of late sawing or insufficient depth of sawing. The plans show that the longitudinal joint was to be sawed to a depth of 3.75 in (95 mm), or 25 percent of the slab thickness. 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).

Present Serviceability Rating (PSR) and Roughness

Roughness was measured on the pavement sections using a Mays Ridemeter. The roughness of the pavement sections, along with the Present Serviceability Rating (PSR) for the sections, is given in table 76.

Table 76. Roughness and present serviceability on OH 2 sections.

Section	Shoulder Type	Roughness, in/mi	PSR
OH 2-33a	Tied PCC	90	3.4
OH 2-33b	AC	85	3.5

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

Other Pavement Distress

Several other pavement distresses were observed in the condition surveys. These distresses are noted below.

"D" Cracking

Low-severity "D" cracking was observed at several transverse joints throughout both sections. It was also observed at the transverse and longitudinal joint intersections, and at other locations along the longitudinal joint. In addition, low-severity "D" cracking was beginning to develop at some of the transverse and longitudinal cracks.

Pumping

As previously noted, OH 2-33a exhibited low-severity pumping while OH 2-33b exhibited medium-severity pumping. The slab-on-grade design of these sections easily lends itself 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.

The lane-shoulder joint sealant on OH 2-33b had deteriorated to the point where water easily infiltrated the pavement system, thus allowing fines beneath the slab to pump upward through the joints under traffic loading. The reason that the lane-shoulder joint sealant on OH 2-33a was in such excellent condition can be attributed to the fact that the joint between a concrete shoulder and concrete mainline pavement is tied and generally tight; thus less movement occurs at this joint and it is therefore much easier to seal and maintain. Large differential movements are known to occur between an asphalt shoulder and a concrete mainline pavement, thereby making the task of maintaining a good seal on that lane-shoulder joint very difficult.

Longitudinal Joint Spalling

Both sections exhibited a substantial amount of longitudinal joint spalling, as summarized in table 77.

Table 77. Summary of longitudinal joint spalling at OH 2.

Section	Shoulder Type	Low-Severity Spalling, ft/mi	Medium-Severity Spalling, ft/mi
OH 2-33a	Tied PCC	120	184
OH 2-33b	AC	63	95

The longitudinal joint spalling exhibited by these sections is due to "D" cracking, which is occurring at the intersections of the longitudinal and transverse joints, and also at other locations along the longitudinal joint.

8. EFFECT OF DESIGN FEATURES ON PAVEMENT PERFORMANCE

Only one design feature, shoulder type, can be evaluated when comparing the performance of these two sections. Corner deflections and load transfer across the longitudinal lane-shoulder joint are not available, but it is still possible to compare the overall performance of each shoulder type.

The information collected for the performance of the outer shoulder is shown in table 78.

Table 78. Outer shoulder performance at OH 2.

Section	Longitudinal Joint Sealant Condition	Mean Shoulder Drop-off, in	Mean Shoulder Separation, in	Shoulder Condition
OH 2-33a	Excellent	0.00	0.00	Excellent
OH 2-33b	Fair-Poor	0.34	0.15	Fair

The tied concrete shoulder (OH 2-33a) is performing much better than the asphalt shoulder. It exhibits no drop-off 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.

The concrete shoulders are tied to the mainline pavement with 0.63-in (16 mm) diameter tiebars. The tiebars are 30-in (762 mm) long and spaced at 30-in (762 mm) intervals.

9. COMPARISON OF OUTER AND INNER LANE PERFORMANCE

As part of the condition survey, distress data was also recorded for the inner traffic lane. Although being subjected to far fewer traffic loadings than the outer lane, the inner lane sections provide an interesting comparison. Table 79 summarizes the inner and outer lane performance for both sections.

As expected, the amount of joint spalling and faulting is far less in the inner lanes of both sections. There is no discernible difference between the roughness index and PSR of the inner and outer traffic lanes. Section OH 2-33b exhibits no longitudinal cracking in the inner lane, whereas OH 2-33a exhibits more longitudinal cracking in the inner lane than the outer lane. The cause of the longitudinal cracking is most likely the insufficient depth of the sawcut used to establish the longitudinal joint.

Table 79. Comparison of performance by lane at OH 2.

OH 2-33a

Variable	Inner Lane	Outer Lane
ESAL's (millions)	0.6	3.3
Joint Spalling,%	3	15
Joint Faulting, in	0.04	0.11
Trans. Cracks/mile	0	0
Long. Cracks, ft/mile	264	104
Roughness, in/mi	86	90
Pumping	None	Low
PSR	3.3	3.4

OH 2-33b

Variable	Inner Lane	Outer Lane
ESAL's (millions)	0.6	3.3
Joint Spalling, %	11	23
Joint Faulting, in	0.03	0.11
Trans. Cracks/mile	0	11
Long. Cracks, ft/mile	0	148
Roughness, in/mi	75	85
Pumping	None	Med
PSR	3.5	3.5

10. SUMMARY AND CONCLUSIONS

It is difficult to substantiate any conclusions drawn from this experimental project since information from physical testing (core information and deflection data) was not available. This information would provide invaluable information in this evaluation. For example, void detection analysis would provide information regarding the loss of support beneath the slabs. Also, core information would help provide information regarding subgrade classification and the joint construction could be determined (particularly the longitudinal joint). However, in the absence of this information, some conclusions can still be stated.

It appears that the tied concrete shoulder provided 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 reduce the faulting to acceptable levels.

Major conclusions in a report on this experimental project include the following: (14)

- o Reduction in maximum particle size has been observed to reduce the severity and extent of "D" cracking. However, where the maximum aggregate size was reduced to prevent "D" cracking, a greater incidence of transverse cracking, with its attendant faulting, was found to occur.
- o Degree of saturation measurements do not appear to show any particular relationship with the development of "D" cracking.
- o Typical sections, joint sealant type, vapor barriers, and type of subbase material appear to have little or no bearing on the development of "D" cracking.

11. ADDITIONAL READING

Majidzadeh K. and R. Elmitiny, "Long Term Observations of Performance of Experimental Pavements in Ohio," Final Report, Report No. FHWA/OH-81/009, Ohio Department of Transportation, July 1982.

CHAPTER 21 HIGHWAY 3N — RUTHVEN, ONTARIO

1. INTRODUCTION

This experimental project, constructed by Ontario's Ministry of Transportation and Communications (MTC), was designed to evaluate the effects of base type, slab thickness, shoulder type, and two surface textures on pavement performance. The purpose in building and maintaining this project was to test the improved performance claims for pavements with tied shoulders and nonerodible bases and to evaluate their ability to replace the conventional flexible pavements that were being built in Ontario at the time. Design variables included: slab-on-grade, an open-graded drainage layer (asphalt-treated permeable base), and lean concrete base courses; 12-in (305 mm), 8-in (203 mm), and 7-in (178 mm) thick concrete slabs; 3-in (76 mm) asphalt concrete (AC) shoulders and tied concrete shoulders; astroturf and burlap mat texturing. The asphalt-treated base included 2 percent AC. It was placed directly on the subgrade, without the use of a filter layer. This project was completed and opened to traffic in November 1982.

None of the designs were replicated in the traditional sense employed at other experimental projects. However, three of the four sections were constructed over three miles (4.8 km) long, with only the 8-in (203 mm) JPCP on 5-in (127 mm) LCB with tied concrete shoulders being relatively short in length (0.5 mi [0.8 km]).

Common to all of the designs is a random, skewed joint spacing of 12-13-19-18 ft (3.7-4.0-5.8-5.5 m). No dowels were used at the transverse joints. Subdrainage along the entire length of the project was provided by a 4-in (102 mm) diameter perforated plastic pipe, wrapped with a geotextile. It was installed 12 in (305 mm) outside of each slab edge and 2 in (51 mm) into the subgrade. Transverse drainage outlets were located at an average of 300-ft (91.4 m) intervals.

No control section as such was constructed for this project. This project was intended to compare these designs to older concrete pavements that were performing quite well under heavy traffic, but were more expensive to build. The traditional MTC design was 9-in (229 mm) slabs, on 5-in (127 mm) of CTB or LCB, with skewed, doweled joints at 12-13-19-18 ft (3.7-4.0-5.8-5.5 m) intervals. A section representing this design was included in this study as ONT 2.

A complete listing of the original design and construction information for all of the sections, as well as other pertinent project information, is provided in the summary tables of chapter 32. As with other experimental projects included in this study, this project does not include a full factorial design. The intent is more to study the performance of a pavement design as a whole, rather than to evaluate the performance of specific design variables.

2. CLIMATE

This experimental section of Highway 3N is located in the southernmost tip of Ontario, just south of Detroit/Windsor, in the wet-freeze environmental zone. The project site has a Thornthwaite Moisture Index of 22, a Corps of Engineers Freezing Index of 1000, and receives an average of

32 in (813 mm) of rainfall annually. The highest average monthly maximum temperature is 80 °F (27 °C) and the lowest average monthly minimum temperature is 19 °F (-7 °C).

3. TRAFFIC

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) Equivalent Single-Axle Load (ESAL) applications (through 1987).

The two-way Average Daily Traffic (ADT) in 1987 was approximately 5400 vehicles. The most recent truck percentage (1984) showed 13 percent heavy trucks. These traffic loadings are relatively low. Data presented by Kazmierowski and Wrong show an AADT in 1988 of 9450, with 10 percent trucks. (15)

4. MAINTENANCE AND REHABILITATION

Soon after initial construction, both transverse and longitudinal cracking occurred at several locations. These cracks were routed and sealed in 1984. No other maintenance work has been performed since then, according to records provided by the MTC.

5. PHYSICAL TESTING RESULTS

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. As part of the ongoing study being conducted by the Ontario MTC, extensive physical testing has been performed by that agency. Results from this testing are included here for reference purposes only, because testing procedures and methodologies may not have been performed in a manner which allows comparison to the other results.

Cores taken from both the concrete pavement and the lean concrete base were tested for compressive strength. These were not separated by test section, so the results do not identify variations in pavement strength among the sections. The compressive strength reported can be used to give an estimate of the modulus of rupture. For the concrete slabs, a mean modulus of rupture of 523 psi (3.6 MPa) was computed from the 28-day compressive strength. The estimated mean modulus of rupture of the lean concrete base from 28-day compressive strengths was 342 psi (2.3 MPa).

Deflection testing was performed several years after initial construction of the sections (in 1985). The deflection equipment used was of the same general configuration as that used in this study and the testing pattern was also similar. The data was used to determine load transfer efficiencies and corner void detection, but was not used for layer moduli characterization because of insufficiencies in the reported data. The load transfer values and void detection results are summarized in chapter 32.

There were minor differences in the average center slab deflections for all but the section on the open-graded drainage layer, which had average deflections over 60 percent higher than the other three sections. However, the absolute values of all of the deflections were very low.

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. As shown in table 80, 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. Using deflection-based void detection procedures, no voids were detected in any of the sections.

Table 80. Load transfer efficiency and corners with voids at ONT 1.

Section	Base Type	Shoulder Type	Slab Thickness, in	Joint Load Transfer, %	% Corners Exhibiting Voids
ONT 1-1	None	AC	12	77	0
ONT 1-2	PATB	AC	8	48	0
ONT 1-3	LCB	PCC	8	48	0
ONT 1-4	LCB	PCC	7	44	0

Selection of the aggregate used in the concrete paving was done in consideration of long-term performance and the desire to avoid materials with "D" cracking potential. Aggregate durability distress was not observed during the field survey.

6. DRAINABILITY OF PAVEMENT SECTIONS

Part of the field survey consisted of a visual drainage analysis, performed on each section. The purpose of the drainage analysis is to evaluate the section's ability to remove water from the pavement structure. Layer permeabilities were not calculated for the base and subbase layers, as there was no physical sample retrieval done. However, using the results from the visual survey and information provided on the layer material properties, some interesting insights into the drainage characteristics of these sections can be inferred. The results are summarized in table 81.

Table 81. Summary of drainage characteristics at ONT 1.

	PROJECT SECTION			
Component	ONT 1-1	ONT 1-2	ONT 1-3	ONT 1-4
Tied Concrete Shoulder	No	No	Yes	Yes
Blowholes in AC Shoulder	None	None	N/A	N/A
Cattails in Ditch	Yes	Yes	Yes	Yes
Pumping	Low	Med	Low	None
Standing Water in Ditch	None	Yes	Yes	Yes
Clogged Outlet Drains	Yes	Yes	Yes	Yes
Base Type	None	PATB	LCB	LCB
Base Permeability, ft/hr	N/A	14.6	0	0
Drainage Coefficient	0.95	1.00	1.00	1.00
Overall Drainability	V.Poor-Poor	Fair-Good	Fair	Fair

None of the sections exhibited good drainability, as shown by the above table. 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 most pumping. This base was placed directly on the subgrade without benefit of a filter layer. A possibility to consider is that the permeable layer has become clogged with fines and is no longer free draining.

7. DETERIORATION OF PAVEMENT SECTIONS

Highway 3N is a two-lane highway so there is only an outer lane in each direction. This project was completely surveyed in both directions. In order to present the results as completely as possible, the primary distress results are tabulated in figure 19 for each lane within a design as if it were a separate section. The effect is as if each design has a replicate. The relative performance of each pavement section with respect to the primary distress types is discussed below.

Joint Spalling

The percent of spalled transverse joints of medium to high severity was almost nonexistent, ranging from 0 to 1 percent. This data does not support the advantage of any one design over the others in terms of reducing joint spalling, as all of the designs appeared to perform well. The transverse joint sealant was a hot-poured rubberized asphalt. This sealant was in "excellent" condition on the sections with AC shoulders and "good" condition on the sections with PCC shoulders. One observation that can be made is that what little spalling there was occurred only in the westbound direction.

Joint Faulting

In general, the average joint faulting on all of these sections was low, a not unexpected result given the low level of traffic. The range of faulting was from 0.04 in (1.0 mm) to 0.09 in (2.3 mm) for the eight sections considered (a value of 0.13 in [3.3 mm] is considered to cause sufficient roughness to require rehabilitation). Overall, the lowest faulting was found on the sections with the permeable asphalt base and the 7-in (178 mm) slab on LCB. A directional effect is also present with the faulting, as the joint faulting is higher in the westbound direction than it is in the eastbound section for the same design. This is shown in table 82.

Table 82. Summary of transverse joint faulting by direction of traffic.

Section	Slab Thickness, in	Base Type	Direction	Joint Faulting, in
ONT 1-1	12	None	EB	0.05
			WB	0.07
ONT 1-2	8	PATB	EB	0.05
			WB	0.06
ONT 1-3	8	LCB	EB	0.04
			WB	0.09
ONT 1-4	7	LCB	EB	0.04
			WB	0.07

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 19. Performance data by direction for Ontario 1.

Transverse Cracking

Transverse cracking was not observed on either of the sections with AC shoulders. These are also the sections which were not built on LCB, which may be a confounding factor. Of the two sections with tied PCC shoulders on LCB, the section with a 7-in (178 mm) slab had slightly more transverse cracking than the section with an 8-in (203 mm) slab.

Longitudinal Cracking

Longitudinal cracking, measured in linear feet per mile, was observed on three of the four sections evaluated. The one design that did not have longitudinal cracking was the section with the permeable ATB and AC shoulders. The other section with AC shoulders and a 12-in (305 mm) slab-on-grade had the next lowest amount of longitudinal cracking. The two sections with tied concrete shoulders and LCB had the most longitudinal cracking.

The 12-in (305 m) slab-on-grade had transverse and longitudinal joints cut to a depth of 3 in (76 mm); the remaining sections had transverse and longitudinal joints sawed to a depth of 2.2 in (56 mm), with the exception of the two sections with tied concrete shoulders and LCB. These sections were paved one lane at a time and hence required no longitudinal lane-lane joint sawing; however, they did require sawing of the joint between the lane and shoulder (to a depth of 2.2 in [56 mm]).

Present Serviceability Rating (PSR) and Roughness

The measured roughness was generally low and the variation in the PSR between sections was minor, as observed in table 83.

Table 83. Roughness and present serviceability at ONT 1.

Section	Slab Thickness, in	Base Type	Direction	Roughness, in/mi	PSR
ONT 1-1	12	None	EB	90	3.8
			WB	62	3.8
ONT 1-2	8	PATB	EB	78	3.8
			WB	70	3.7
ONT 1-3	8	LCB	EB	57	3.8
			WB	66	3.8
ONT 1-4	7	LCB	EB	65	3.7
			WB	72	3.8

In the field survey, the sections with the different pre-tining surface finishing methods were <u>not</u> identified. This may have contributed to some of the variation in roughness.

Shoulder Condition

Since one of the design variables on this project was shoulder type, the condition of the shoulders is of interest. Shoulder performance information is summarized in the table 84. 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. They were also weathered and ravelled, with the AC shoulders on ONT 1-2 being in worse condition than those on 1-1. The overall condition rating was "poor" for both of these sections.

Table 84. Shoulder performance at ONT 1.

Section	Shoulder Type	Lane-Shoulder Drop-off, in	Lane-Shoulder Separation, in	Overall Rating
ONT 1-1	AC	0.58	0.07	Poor
ONT 1-2	AC	0.47	0.00	Poor
ONT 1-3	PCC	0.00	0.00	Exc
ONT 1-4	PCC	0.00	0.00	Good

The PCC shoulder on ONT 1-3 was in excellent condition, although there was some spalling along the shoulder/gravel edge. The concrete shoulder on ONT 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 drop-off or separation on these two sections. The aggregate portion of all four sections was weathered and ravelled.

Other Distresses

On ONT 1-2 and ONT 1-3 a longitudinal patched area was observed which continued for several slabs. These are a result of problems which occurred at the time of construction and are not related to the performance of the sections.

8. EFFECT OF DESIGN FEATURES ON PAVEMENT PERFORMANCE

The independent effect of the various design features on the performance of the different sections cannot be directly determined. While the differences among the performance of each of the four designs as a whole can be discussed, this experimental study was not set up as a full factorial design to isolate the different variables.

Base Type

The performance of the sections 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 of 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 performance of the 8-in (203 mm) slab on the permeable asphalt treated base was slightly better than that of the 12-in (305 mm) slab-on-grade. However, the construction of the permeable base without a filter layer to prevent the intrusion of fines could effectively render that design ineffective.

Shoulder Type

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.

Tied PCC shoulders are thought to decrease mainline slab deflections due to the increased edge support that they provide. An examination of the average approach slab corner deflections shows that the sections with tied shoulders had deflections 1/2 that of the thin section with AC shoulders, although they were twice the deflection of the 12-in (305 mm) slab-on-grade section. Higher slab deflections cause increased fatigue of concrete slabs and should hasten pavement deterioration. Thus the sections with the concrete shoulders should, in theory, contribute to longer pavement life. The AC shoulders themselves showed extensive deterioration, while the PCC shoulders had no deterioration.

Slab Thickness

This project included 7-in (178 mm), 8-in (203 mm), and 12-in (305 mm) thick pavement slabs. Unfortunately, no readily 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 more traffic has been carried by these sections.

Base Drainability

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 the best performance of any of the sections. Given the widespread occurrence of clogged drainage outlets and the amount of pumping present, water may be trapped in the permeable layer as a result of clogging of the voids. Additional information is needed as to the performance of the edge drains along the entire project and the condition of the drainage layer on ONT 1-2.

The limited results available confirm the positive effects of a permeable base layer on pavement performance, but also the danger in not maintaining the drainage outlets, leaving out the filter layer under the permeable base, and placing the underdrain 6 in (152 mm) beyond the permeable base in the dense-graded aggregate shoulder material.

9. COMPARISON OF LANE PERFORMANCE BY DIRECTION

It has been suggested in section 7 that the distresses are higher in the westbound lanes than in the eastbound lanes. It would be interesting to obtain further traffic information to see if heavy truck volume is actually higher in this direction, or if heavier loads are traveling in this direction. It may be relevant that these lanes were not paved integrally; the concrete paving operation for the sections with tied concrete shoulders consisted of paving the westbound lane and shoulder in one pass and then turning around and paving the eastbound lane. The lanes were tied together by epoxy-coated tiebars, 30 in (762 mm) long, placed at 24-in (610 mm) intervals.

An overall comparison of the eastbound and westbound traffic lanes is provided in table 85.

Table 85. Comparison of performance by lane at ONT 1.

Variable	Eastbound Lane	Westbound Lane
ESAL's (millions)	1.0	1.0
Joint Spalling, %	0	1
Joint Faulting, in	0.045	0.073
Trans. Cracks/mile	18.8	28.8
Long. Cracks, ft/mile	21.3	197.5
Roughness, in/mile	68	72
PSR	3.8	3.8

10. SUMMARY AND 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. This result is similar to that found in 1985 and the inclusion of a free draining layer in future concrete pavement construction is one of these key recommendations made at that time. Other recommendations from those familiar with the project since its construction include the suggestion that all joints be sawed as soon as the concrete reaches a nonplastic state and that the longitudinal joint (lane-shoulder) be sawcut to a depth 1/3 the thickness of the slab. Recall that the depth of sawcut for the longitudinal and transverse cuts was 2.2 in (56 mm) for the 7-in (178 mm) and 8-in (203 mm) sections, and 3 in (76 mm) for the 12-in (305 mm) section.

It was further recommended that concrete shoulders be constructed integral with the pavement. While the results observed here do not support the theoretical benefits from the addition of concrete shoulders, it is very likely that other factors contributed to the poor performance of these sections. These factors include late and shallow sawing of joints, which may have caused random transverse and longitudinal cracking, and the failure to arrest reflective cracking of the lean concrete bases. Further physical testing would be desirable to evaluate the depth of the sawcut and materials condition. FWD deflection testing would also be useful, as the absence of voids in the presence of substantial pumping is suspect.

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 direct comparison can be made. This section was the only one without both transverse and longitudinal cracking. It also had the lowest average joint faulting of any of the sections, along with ONT 1-4. The fact that it had the highest pumping of all of the sections may mean that the drainage layer has become clogged and there is no means of removing moisture from the pavement layers. No filter was provided between this permeable base layer and the subgrade. This section should be studied further to gather more information about its drainage characteristics.

Finally, it has been noted that the nondoweled joint design is inadequate for the conditions experienced.(15) The aggregate interlock that was expected to provide load transfer ultimately proved unreliable after a very short period of time and the pavement, as designed and constructed, is probably inadequate to carry the unanticipated heavy traffic loads.

CHAPTER 22 HIGHWAY 427 — TORONTO, ONTARIO

1. INTRODUCTION

Highway 427 is an urban principal arterial highway located in Toronto, Ontario. It is a principal access route into downtown Toronto. Highway 427 is a multi-lane route, with four lanes in the direction of survey, and a total of eleven lanes. It was constructed and opened to traffic in 1971.

2. DESIGN

The cross-section design is 9 in (229 mm) of JPCP on 6 in (152 mm) of cement-treated base. The shoulders are asphalt concrete, 4 in (102 mm) thick. The outer shoulder is 10 ft (3 m) wide. Transverse joints are skewed, with a random joint spacing of 12-13-19-18 ft (3.7-4.0-5.8-5.5 m). Painted and greased dowel bars 1-in (25 mm) in diameter, spaced at 12-in (305 mm) intervals provide load transfer at the transverse joints. Preformed joint sealant is used on the project and it is still in excellent condition. The subgrade type is unknown, but this pavement is located in an area of dense urban development and the highway was most likely built on imported fill. Longitudinal drains, 4-in (102 mm) in diameter, with lateral outlets spaced at approximately 300-ft (91 m) intervals, were placed in 1982. Additional project information is contained in the summary tables in chapter 32.

3. CLIMATE

This section is located in the wet-freeze environmental zone. It has a Corps of Engineers Freezing Index of 1000 and a Thornthwaite Moisture Index of 13. The annual average precipitation is 30 in (762 mm). The highest average monthly maximum temperature is 82 ^{o}F (28 ^{o}C) and the lowest average monthly minimum temperature is 16 ^{o}C (-9 ^{o}C).

4. TRAFFIC

The 1987 two-way ADT was 228,350 vehicles, including 10 percent commercial vehicles. It is estimated that this pavement has sustained roughly 36 million 18-kip (80 kN) Equivalent Single-Axle Load applications in the outer lane.

5. DRAINABILITY AND OTHER PHYSICAL TESTING RESULTS

This highway was one of several on which no physical testing was performed. Therefore, no strength estimates of the various layers are available. From the limited amount of soil and construction information available, the drainability of the section appears to be poor. An AASHTO drainage coefficient of 0.80 was assigned to this section.

6. MAINTENANCE AND REHABILITATION

From records provided by the Ontario Ministry of Transportation and Communication, a subdrainage system was added under the shoulder at the edge of the CTB in 1982. Other rehabilitation performed on the project include resurfacing of the AC shoulders several times over the life of the pavement, most recently in 1984.

7. PAVEMENT PERFORMANCE

Due to the excessive traffic on this pavement and the unavailability of lane closures, physical measurements were only recorded on the outer lane. These results are summarized in table 86.

Table 86. Summary of performance variables at ONT 2.

Variable	Outer Lane
ESAL's (millions)	36
Joint Spalling, %	0
Joint Faulting, in	0.01
Trans. Cracks/mile	5
Long. Cracks, ft/mile	0
Roughness, in/mile	104
PSR	3.9

Transverse cracking was only noted in the 19-ft (5.8 m) slabs. No other distress of any significance was identified in the condition survey.

8. CONCLUSIONS

This pavement is performing very well after 17 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. The excellent performance of the preformed joint sealant should also be noted.

CHAPTER 23 ROUTES 66 AND 422 — KITTANNING, PENNSYLVANIA

1. INTRODUCTION

An experimental project was constructed on Route 66 near Kittanning, Pennsylvania, to investigate the feasibility of several alternative base designs to Pennsylvania Department of Transportation's existing base design. While PennDOT's current base design was intended to provide both adequate strength and adequate drainability, problems with premature pavement and shoulder distress had occurred, with the primary cause being attributed to excess water in the pavement. Permeability tests performed in the lab on the base material showed it to be a relatively impermeable material. Field permeability tests showed similar results. Included in this project were pavement sections with an impermeable cement-treated base, an asphalt-treated permeable base, a permeable aggregate layer of relatively uniform gradation, a more well-graded high permeability aggregate layer, and a control section consisting of PennDOT's typical base design at the time.

The mainline pavement, constructed in 1980, was paved 24 ft (7.3 m) wide in each direction, with a 10-in (254 mm) jointed reinforced concrete pavement (JRCP) slab. The transverse joints were perpendicular, with 1.25-in (32 mm) diameter epoxy-coated dowels. The transverse joint spacing was 46.5 ft (14.2 m). All transverse joints were sealed with a rubberized asphalt sealant. The shoulder for all sections was an AC surface on a 4-in (102 mm) aggregate base.

Each of the different designs was constructed for a length of between 1000 and 1700 ft (305 and 518 m) on adjacent sides of the highway. The only section replicated (aside from the repeat of each design on the adjacent two lanes) was the control section, which was constructed three times on Route 66, and once on an intersecting highway, Route 422. The section with a cement-treated base was only constructed on Route 422.

No design feature other than the base type varies over the length of the project. Thus, only the effect of the base type can be determined. More specifically, the permeability of the different bases is the design variable evaluated in this project. As no other design features are changed, the analysis of this variable is not confounded by other factors, with one exception. As mentioned above, the cement-treated aggregate base section is located on Route 422, a highway adjacent to Route 66. Thus traffic loading is not constant across all of the sections.

One of the purposes of the project was to compare the costs and constructability of the different base sections. They were also subjected to field measures of their permeabilities. Since these can be evaluated without reference to the traffic loadings, for the purpose of the project this was not a limiting factor. However, in order to compare the performance of these different designs and eventually make certain conclusions regarding life-cycle costs, the traffic loadings on each roadway will have to be taken into account.

Five sections, designated PA 1-1 through 1-5, were surveyed and tested in 1987. Other testing results were made available by PennDOT from earlier years of study. Section PA 1-1 was the section with the cement-treated base (containing 6 percent Portland cement) and was located on Route 422.

The remaining sections were all constructed on Route 66. Sections PA 1-2 and PA 1-3, shared the same uniform aggregate gradation, but PA 1-2 was treated with asphalt cement. Sections PA 1-4 and PA 1-5 were well-graded aggregates. There were far fewer fines in PA 1-4; PA 1-5 was the control, consisting of the typical PennDOT base design at the time. A more complete listing of the original design and construction information, along with other project information, is provided in the summary tables of chapter 32.

2. CLIMATE

The pavement sections are located in the west-central part of Pennsylvania, in a wet-freeze environmental zone. The project site has a Thornthwaite Moisture Index of 53, a Corps of Engineers Freezing Index of 300, and receives an average of 41 in (1041 mm) of precipitation annually. The highest average monthly maximum temperature is 83 $^{\circ}$F (28 $^{\circ}$C) and the lowest average monthly minimum temperature is 15 $^{\circ}$F (-9 $^{\circ}$C).

3. TRAFFIC

Both roadways are four-lane highways (two lanes in each direction) and have a functional classification of Urban Minor Arterial. Estimates were made of the number of 18-kip (80 kN) Equivalent Single-Axle Load (ESAL) applications sustained by each section (through 1987). This information, along with other traffic information for each roadway, is provided in table 87.

Table 87. Traffic summary of PA 1 sections.

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

Weigh-in-Motion (WIM) data collected on Route 66 in 1987 provided a traffic count of 15,211 vehicles per day, of which 4.4 percent were heavy trucks.

4. MAINTENANCE AND REHABILITATION

According to records provided by PennDOT, no significant maintenance or rehabilitation has been performed on the surveyed sections.

5. PHYSICAL TESTING RESULTS

Center slab cores were retrieved from the pavement sections in 1987. Joint cores were not retrieved because of the lack of significant distress and the relatively recent construction of these pavements. The center cores were tested in split tensile to provide an indication of the concrete strength. The split tensile strength values were used to obtain an estimate of the modulus of rupture, as provided in chapter 32. A mean modulus of rupture of 747 psi (5.2 MPa) was calculated from the center slab cores.

Deflection testing was performed on the sections in 1987 for the purpose of layer moduli characterization, determination of load transfer efficiencies, and void detection. The elastic modulus of the concrete (E), the composite k-value (on top of the base), and the load transfer efficiency values are summarized for each section in chapter 32.

The slab E-values averaged 3,800,000 psi (26,200 MPa) for the five sections. The composite k-values for the sections are shown in table 88.

Table 88. Summary of composite k-value by base type.

Section	Base Type	k-value, pci
PA 1-1	CTB	731
PA 1-2	PATB	1040
PA 1-3	AGG	538
PA 1-4	AGG	747
PA 1-5	AGG	540

All of the effective k-values were high, indicating stiff support. The permeable ATB and the CTB had especially high k-values. Boring tests and county soil reports showed a variation in the subgrade from an AASHTO A-2-4 to an AASHTO A-4.

The falling weight deflectometer (FWD) results are presented in table 89.

Table 89. FWD deflection testing results at PA 1.

Section	Base Type	Approach Corner Average Deflection, mils	% Load Transfer	% Corners Exhibiting Voids
PA 1-1	CTB	12.9	84	45
PA 1-2	PATB	41.4	100	100
PA 1-3	AGG	40.0	98	95
PA 1-4	AGG	11.1	39	15
PA 1-5	AGG	33.4	98	95

The approach joint corner deflections for PA 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.

Using deflection-based void detection procedures, voids, or loss of support, were detected at slab corners. These are summarized in table 89. 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, PA 1-1 and 1-2, had both low and high percentages of voids. The mid-slab deflection of PA 1-2 was the lowest, and that of PA 1-3 was the highest, but their permeabilities were essentially identical. Well-drained sections had both high LTE's (PA 1-2 and 1-3) and low LTE's (PA 1-4). The LTE's of the poor-draining sections were consistently good (84 to 98 percent).

6. DRAINABILITY OF PAVEMENT SECTIONS

As the major purpose of this project was to evaluate base materials of varying permeabilities, drainage design was an important part of the design of the project. In addition to the base layers described previously, a longitudinal drain was installed underneath the shoulder, 13 in (330 mm) from the lane-shoulder joint, on all of the pavements. A trench was excavated to a depth 6 in (152 mm) below the bottom of the base. Then a perforated plastic drain pipe, 4.7 in (119 mm) in diameter, was placed in the trench to the depth of the base layer. The entire trench was backfilled with pea gravel. Drainage outlets were placed on an average of every 300 ft (91 m).

A drainage analysis was performed on each section to evaluate the section's overall ability to remove water from the pavement structure. Layer permeabilities were calculated for both the base and subbase layers in each section, using the base material recovered from the boring operation. PennDOT also calculated both field and lab permeabilities for the base courses at the time of their construction, and have periodically monitored outflow through devices which they installed. These permeabilities are summarized in table 90. For the analysis, permeabilities estimated from the material collection were used. The calculated field permeability, combined with other inputs from the field survey, have been used to estimate the overall drainability characteristics of these five sections. As this project's intent was to evaluate the performance of bases of different drainability, data from PennDOT's study of these sections is also available.

Table 90. Drainage summary for PA 1 sections.

		PERMEABILITIES, FT/HR				
		PennDOT			AASHTO Drainage	Overall
Section	Layer	Lab	Field	Estimated	Coefficient	Drainability
PA 1-1	CTB	0	0	0		
	AGG	0.05	1.18	0.06	0.80	V.Poor-Fair
PA 1-2	PATB	283	236	N/A		
	AGG	0.05	1.18	0.06	1.05	Good-Exc.
PA 1-3	AGG	898	780	1107		
	AGG	0.05	1.18	0.06	1.05	Good-Exc.
PA 1-4	AGG	756	732	197		
	AGG	0.05	1.18	0.06	1.00	V.Poor-Good
PA 1-5	AGG	0.05	1.18	0.06		
	NONE	---	---	---	1.00	V.Poor-Fair

In this limited sample, the estimated permeabilities are reasonably close to the measured lab and field values from PennDOT's study.

7. DETERIORATION OF PAVEMENT SECTIONS

The primary performance indicators are tabulated in figure 20 and figure 21 for the outer and inner lanes, respectively. The following discussions of performance refer to the outer lanes only.

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 20. Outer lane performance data for Pennsylvania 1.

PENNSYLVANIA 1 RTE. 422 & RTE. 66 KITTANNING

INNER LANE PERFORMANCE DATA

			10 in JRCP 46.5 ft JOINTS
BASE TYPE	CTB		PA 1-1
		PSR	4.0
		ROUGHNESS, in/mi	88
		FAULTING, in	0.01
		T. CKS./mi	0
		LONG. CKS., ft/mi	0
		% JT. SPALL.	0
	PATB		PA 1-2
		PSR	3.8
		ROUGHNESS, in/mi	71
		FAULTING, in	0.01
		T. CKS./mi	0
		LONG. CKS., ft/mi	0
		% JT. SPALL.	0
	AGG (UNIFORM-GRADED)		PA 1-3
		PSR	3.7
		ROUGHNESS, in/mi	93
		FAULTING, in	0.01
		T. CKS./mi	0
		LONG. CKS., ft/mi	0
		% JT. SPALL.	0
	AGG (WELL-GRADED)		PA 1-4
		PSR	4.1
		ROUGHNESS, in/mi	103
		FAULTING, in	0.02
		T. CKS./mi	0
		LONG. CKS., ft/mi	0
		% JT. SPALL.	0
	AGG (CONTROL)		PA 1-5
		PSR	4.2
		ROUGHNESS, in/mi	103
		FAULTING, in	0.03
		T. CKS./mi	0
		LONG. CKS., ft/mi	0
		% JT. SPALL.	0
	AVG.	PSR	4.0
		ROUGHNESS, in/mi	92
		FAULTING, in	0.02
		T. CKS./mi	0
		LONG. CKS., ft/mi	0
		% JT. SPALL.	0

Figure 21. Inner lane performance data for Pennsylvania 1.

Joint Spalling

No joint spalling (medium- and high-severity) was observed in any of the pavement sections.

Joint Faulting

Joint faulting for the sections was extremely low. The average joint faulting was about 0.03 in (0.8 mm), with a range of only 0.02 in (0.5 mm) to 0.03 in (0.8 mm).

Transverse Cracking

No transverse cracks of any severity were observed within any of the surveyed sections.

Longitudinal Cracking

No longitudinal cracking of any severity was observed in any of the pavement sections.

Present Serviceability Rating (PSR) and Roughness

Table 91 summarizes the PSR and roughness measurements for each section:

Table 91. Roughness and present serviceability at PA 1.

Section	Base Type	Roughness, in/mi	PSR
PA 1-1	CTB	75	4.2
PA 1-2	PATB	91	3.8
PA 1-3	AGG	113	3.7
PA 1-4	AGG	109	4.0
PA 1-5	AGG	82	4.0

The smoothest riding sections, as supported by both roughness and PSR, were PA 1-1 and PA 1-5 (control). These sections both contained bases which were much less permeable than the other sections. The fact that they were among the smoothest may reflect the fact that they are more "conventional" designs with which construction contractors are more familiar.

8. EFFECT OF DESIGN FEATURES ON PAVEMENT PERFORMANCE

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. Since very little differentiation is noted for the various reported distresses, 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 around 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.

9. COMPARISON OF OUTER AND INNER LANE PERFORMANCE

In addition to differences in traffic loadings between lanes, there was also a difference in traffic loadings between PA 1-1 and the other sections. PA 1-1 was located on Route 422 and had been subjected to about twice as many ESAL applications than the sections on Route 66. This information and the differences in outer and inner lane performance, are presented in table 92.

Table 92. Comparison of performance by lane at PA 1.

SECTION PA 1-1 (RTE 422)

Variable	Inner Lane	Outer Lane
ESAL's (millions)	0.08	0.60
Joint Spalling, %	0	0
Joint Faulting, in	0.01	0.03
Trans. Cracks/mile	0	0
Long. Cracks, ft/mile	0	0
Roughness, in/mile	88	75
PSR	4.0	4.2

SECTIONS PA 1-2 THROUGH PA 1-5 (RTE 66)

Variable	Inner Lane	Outer Lane
ESAL's (millions)	0.03	0.27
Joint Spalling, %	0	0
Joint Faulting, in	0.02	0.03
Trans. Cracks/mile	0	0
Long. Cracks, ft/mile	0	0
Roughness, in/mile	92	99
PSR	3.9	3.9

It is observed that the traffic levels are low enough that they have not resulted in any substantial difference in performance between the sections. It is also observed that both the inner and the outer lanes displayed excellent performance; no significant difference exists between the two traffic lanes. Again, this is believed to be due to the very low traffic loadings over the 7-year life of the pavement.

10. SUMMARY AND 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 the traffic level is so low that no trends have begun to develop in the performance of the different designs.

Several factors probably contribute 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. This interface 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, 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.

The presence of voids at over 95 percent of the slab corners and the high corner deflections on three of the five sections are causes for concern about the future performance of these particular designs.

11. ADDITIONAL READING

Hoffman, G. L., "Subbase Permeability and Pavement Performance," Interim Report, Pennsylvania Department of Transportation Research Project 79-3, January 1982.

Malesheskie, G. J., "Open-Graded Subbase to Provide All-Season Pavement Subdrainage," 1986.

Highlands, K. L. and G. L. Hoffman, "Subbase Permeability and Pavement Performance," Paper presented at the 67th Annual Meeting of the Transportation Research Board, 1987.

CHAPTER 24 ROUTE 130 — YARDVILLE, NEW JERSEY

1. INTRODUCTION

Route 130 is a four-lane Principal Arterial highway located in a heavily developed part of central New Jersey, east of Trenton. This pavement was constructed in 1951, over the original alignment of Route 25 (Section 1C, Yardville Bypass).

2. DESIGN

This pavement consists of a 10-in (254 mm) JRCP slab resting on a 5-in (127 mm) granular base and a 7-in (178 mm) granular subbase. The subgrade is an AASHTO A-4. The slab lengths are 78.5 ft (24 m) and are constructed with expansion joints located at the end of each slab, rather than contraction joints. These joints contain a compressible filler material and are sealed with a rubberized asphalt joint sealant cap. Load transfer across the joints is provided by stainless steel clad, 1.25-in (32 mm) diameter dowels, 18 in (457 mm) long. The outer shoulders are 10 ft (3 m) wide, and paved with a 2-in (51 mm) AC surfacing. No subsurface drainage pipes are provided in this design. This design is typical of the concrete pavements constructed in New Jersey over the past 40 or more years.

Additional project design information is contained in the summary tables presented in chapter 32.

3. CLIMATE

Located in the wet-freeze environmental zone, this area has a Corps of Engineers Freezing Index of 0 and a Thornthwaite Moisture Index of 37. The annual average precipitation is 43 in (1092 mm). The highest average monthly maximum temperature is 85 ^{o}F (29 ^{o}C) and the lowest average monthly minimum temperature is 25 ^{o}F (-4 ^{o}C).

4. TRAFFIC

The one-way ADT in the northbound lanes was 11,895 vehicles in 1986. This included 21 percent trucks, according to information supplied by the New Jersey Department of Transportation (NJDOT). Since being opened to traffic in 1951, the pavement has sustained approximately 35 million 18-kip (80 kN) Equivalent Single-Axle Load applications in the outer lane and nearly 7.2 million ESAL applications in the inner lane. These represent very high truck loadings.

5. DRAINABILITY AND OTHER PHYSICAL TESTING RESULTS

A drainage analysis was performed on this section using data obtained during the field survey. The calculated permeabilities for the base and subgrade layers are 0.95 ft/hr (0.29 m/hr) and 0.11 ft/hr (0.03 m/hr) respectively. These are indicative of fairly well-draining materials. The calculated drainage coefficient is 1.05, and the pavement received an overall drainability rating of "good."

Deflection data was obtained using a FWD. 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 base of 234 pci (63.5 kPa/mm). The average center slab deflection was 3.4 mils. The adjusted load transfer efficiency at the transverse joints was 76 percent. Deflections taken at the slab corners showed no voids under any of the approach or leave corners.

6. MAINTENANCE AND REHABILITATION

Over the years this project has undergone routine maintenance, consisting mainly of resealing of the joints. The last recorded maintenance work was performed in 1986, at which time the joints were resealed and minor partial- and full-depth repairs were placed. Patches within the section's boundaries are placed using bituminous materials. The shoulders along this pavement were also replaced in 1986.

7. PAVEMENT PERFORMANCE

The primary distresses for both the outer and inner lane are summarized in table 93.

Table 93. Comparison of performance by lane at NJ 2.

Variable	Inner Lane	Outer Lane
ESAL's (millions)	7.2	35.0
Joint Spalling, %	20	20
Joint Faulting, in	0.03	0.06
Trans. Cracks/mile	5	24
Long. Cracks, ft/mile	0	10
Roughness, in/mile	75	63
PSR	3.5	3.8

No other significant distresses were identified in the condition survey.

8. CONCLUSIONS

This section is 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 less 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 these expansion joints. At the time of the survey, the average joint opening in the outer lane was 1.27 in (32 mm).

9. ADDITIONAL READING

Unpublished paper provided by NJDOT which discusses experimental pavements constructed during the period from 1946 to 1955 using the expansion joint.

Van Breemen, W., "Current Design of Concrete Pavement in New Jersey," Highway Research Board, Proceedings of the Twenty-Eighth Annual Meeting, December 1948, pp. 77-91.

CHAPTER 25 INTERSTATE 676 — CAMDEN, NEW JERSEY

1. INTRODUCTION

This experimental project, located on I-676 near Camden, New Jersey, was constructed in 1979. The experiment was conducted to evaluate the construction and performance of permeable bases under jointed reinforced concrete pavements (JRCP). Two different base types were included: a nonstabilized open-graded aggregate and an asphalt-treated open-graded aggregate. Conventional New Jersey JRCP was placed on each base type. The only experimental design variable that can be directly compared is the two different permeable base types.

The setup of the experiment included two sections located in the southbound lanes (one with each base type) that were included as part of this experiment. A filter cloth was used full-width under the open-graded layer and around the edge drain which was placed along the longitudinal edge joint. Two other sections in the northbound lanes had the same design, except that no filter cloth was used between the base and subbase layers, and the top 3 in (76 mm) of the subbase were stabilized with lime-fly ash. The two sections that were surveyed were designated as NJ 3-1 (open-graded aggregate base) and NJ 3-2 (asphalt-treated permeable base).

The JRCP pavement represents the New Jersey DOT's traditional design: a 9-in (229 mm) slab constructed with doweled expansion joints placed at 78 ft (23.8 m) intervals. This design calls for 0.16 percent reinforcing steel. No transverse contraction joints are included. The 1.25-in (32 mm) diameter dowel bars are placed in a stainless steel sleeve to prevent corrosion. A complete listing of the original design and construction information for each section, as well as other pertinent project information, is provided in the summary tables of chapter 32.

2. CLIMATE

The pavement sections are located in the central part of New Jersey, east of Philadelphia, in a wet-freeze environmental zone. The project site has a Thornthwaite Moisture Index of 44, a Corps of Engineers Freezing Index of 0, and receives an average of 43 in (1092 mm) of precipitation annually. The highest average monthly maximum temperature is 86 ^{o}F (30 ^{o}C) and the lowest average monthly minimum temperature is 23 ^{o}F (-5 ^{o}C).

3. TRAFFIC

The roadway's functional classification is Urban Interstate. It is a six-lane divided highway, with three lanes in each direction. Since opening to traffic in 1979, the project has sustained an estimated 4.2 million 18-kip (80 kN) Equivalent Single-Axle Load (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. These ESAL estimates include calculations through 1987.

The two-way Average Daily Traffic (ADT) in 1986 was approximately 77,000 vehicles, including 5 percent heavy trucks. Weigh-in-motion data collected at the project site in 1987 provided a two-way traffic count of 50,565 vehicles per day, including 5.6 percent heavy trucks.

4. MAINTENANCE AND REHABILITATION

The only maintenance and rehabilitation performed on these sections is the resealing of joints with a rubberized asphalt sealant. This is done on an annual basis, as needed.

5. PHYSICAL TESTING RESULTS

Cores from slab centers were retrieved from the pavement sections in 1987. The center cores were tested in split tensile to provide an indication of the concrete strength. The split tensile strength values were used to obtain an estimate of the modulus of rupture, as shown in chapter 32. A mean modulus of rupture of 704 psi (4.8 MPa) was obtained. No joint cores were retrieved from this relatively new pavement.

A Falling Weight Deflectometer (FWD) was used to perform deflection testing on these sections in 1987. The collected data was analyzed to provide layer moduli characterization, to determine load transfer efficiencies, and to aid in the detection of voids under the slab corners. The elastic modulus of the concrete (E), the effective k-value (on top of the base), and the load transfer values are summarized for each section in chapter 32.

The slab E-values averaged 5,400,000 psi (37,230 MPa) for the two sections. The effective k-values for the sections varied by base type, as shown in table 94.

Table 94. Summary of composite k-values at NJ 3.

Section	Base Type	Subgrade Classification	k-value, pci
NJ 3-1	Open-graded aggregate	A-1-a	356
NJ 3-2	Asphalt-treated aggregate	A-2-4	210

The open-graded aggregate base appears to provide much greater slab support than the asphalt-treated permeable base. However, the subgrade for the aggregate base is much stiffer than the subgrade for the asphalt-treated base, as indicated by its soil classification. This can also contribute to the higher k-value. The center slab deflections were higher for the open-graded aggregate base than for the permeable asphalt-treated base. These results reflect the relationship between k-value results. 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.

Using deflection-based void detection procedures, 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.

6. DRAINABILITY OF PAVEMENT SECTIONS

In order to isolate the effects of surface runoff, these sections are located on a substantial fill. This eliminates groundwater infiltration as a potential source of water in the pavement system. Longitudinal edge drains (6 in [152 mm] diameter) were placed in backfilled trenches, 1.5 ft (0.5 m) from the pavement edge, at the edge of the base. The trenches were separated from the subbase by a filter cloth, placed to prevent intrusion of fines into the drainable base layers. Transverse drainage outlets were placed at approximately 500-ft (152 m) intervals. A summary of the drainage information is presented in table 95.

Table 95. Drainage summary for NJ 3.

Drainage Factor	SECTION 4-1	SECTION 4-2
Base Type	Perm. AGG	Perm. ATB
Base Permeability, ft/hr		
Construction Records	N/A	0.86
1987 Field Tests	60	68
Subbase Type	Lime-Flyash Stab.	Lime-Flyash Stab.
Subbase Permeability, ft/hr		
Construction Records	N/A	N/A
1987 Field Tests	1.02	0.66
AASHTO Drainage Coefficient	1.10	1.10
Overall Drainability	Fair-Good	Fair-Good

7. DETERIORATION OF PAVEMENT SECTIONS

In order to better present the results of the extensive distress survey conducted on each pavement section, the primary performance indicators are tabulated in figure 22 for the both the outer lane and the middle lane. The relative performance of each pavement section with respect to the primary performance indicators are discussed below for the outer lane only.

Joint Spalling

Joint design for both sections included doweled expansion joints at 78-ft (23.8 m) intervals. This is the normal practice in New Jersey and has been followed for many years. After 8 years of service, the section with permeable aggregate-treated 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. This type of construction requires forms instead of slip form paving, and may have resulted in excessive hand-finishing at the transverse joints. Joint spalling often occurs where corrosion of the load transfer devices has taken place. The permeability of the stabilized layer was much lower, which may have also contributed to more joint spalling. Over 85 percent of the joints on that section already had low severity joint spalling.

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

		MIDDLE LANE PERFORMANCE DATA	
		PERMEABLE AGG BASE	PERMEABLE ATB
		NJ 3-1	NJ 3-2
9 in JRCP 78.5 ft JT. SPACING	PSR	N/A	N/A
	ROUGHNESS, in/mi	N/A	N/A
	FAULTING, in	0.02	0.01
	T. CKS./mi	0	0
	LONG. CKS., ft/mi	0	0
	% JT. SPALL.	0	14

Figure 22. Outer and middle lane performance data for New Jersey 3.

Joint Faulting

Joint faulting in the outer lane was similar for the two sections, as is shown in table 96. These levels of faulting are not considered very significant for long-jointed, doweled JRCP.

Table 96. Transverse joint faulting at NJ 3.

Base Type	Joint Spacing, ft	Dowel Bars	Joint Faulting, in
Open-graded Aggregate	78	Yes	0.05
Asphalt-treated Aggregate	78	Yes	0.06

Transverse Cracking

No transverse cracking of any severity occurred on either section. This is surprising in consideration of the long-jointed slabs on this project. Recall that the transverse joints are actually expansion joints, constructed with corrosion-resistant dowel bars, which may contribute to the performance of these designs.

Longitudinal Cracking

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 one of the most common causes of longitudinal cracking, insufficient depth of sawcut of the longitudinal joint.

Present Serviceability Rating (PSR) and Roughness

The serviceability and rideability of the sections after 8 years of service are shown in table 97. These results indicate a significant roughness problem, with minimal difference between the two designs. The reason for the high roughness is not readily apparent, since there exists very little surface distress. Roughness may have been built in during construction, especially considering that the pavements were constructed with side forms, as is discussed above. In addition, the wide expansion joints utilized in the design may contribute to roughness as well.

Table 97. Roughness and present serviceability at NJ 3.

Base Type	Roughness, in/mi	PSR
Open-graded Aggregate	134	3.6
Asphalt-treated Aggregate	153	3.5

8. EFFECT OF DESIGN FEATURES ON PAVEMENT PERFORMANCE

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. The high roughness values of both sections is probably not related to the performance of the sections. It is not likely that the permeable base type had anything to do with the spalling of the joints, as the concrete aggregate is generally sound. In general, both of the permeable base designs have performed similarly well after 8 years and 4.2 million ESAL loadings.

9. COMPARISON OF OUTER AND MIDDLE LANE PERFORMANCE

Although being subjected to far fewer traffic loadings than the outer lane, the middle lane sections provide an interesting comparison. Although faulting is very low for the outer lane, faulting in the middle lane is even lower than the outer lane for both sections. Despite the presence of the permeable bases and doweled joints, there is still some effect of heavy truck loads on erosion to create some faulting. The measured load transfer was low, indicating that the dowels may be loose. Table 98 summarizes the performance of the two sections.

Table 98. Comparison of performance by lane at NJ 3.

JRCP WITH OPEN-GRADED AGGREGATE BASE

Variable	Middle Lane	Outer Lane
ESAL's (millions)	2.5	4.2
Joint Spalling, %	0	0
Joint Faulting, in	0.02	0.05
Trans. Cracks/mile	0	0
Long. Cracks, ft/mile	0	0
Roughness, in/mile	—	134
PSR	—	3.6

JRCP WITH PERMEABLE, ASPHALT-TREATED BASE

Variable	Middle Lane	Outer Lane
ESAL's (millions)	2.5	4.2
Joint Spalling, %	14	43
Joint Faulting, in	0.01	0.06
Trans. Cracks/mile	0	0
Long. Cracks, ft/mile	0	0
Roughness, in/mile	—	153
PSR	—	3.5

More joint spalling occurred in the outer lane than in the inner lane for the permeable, asphalt-treated base section. The cause of this spalling is not known. No cracking has occurred in either lane.

10. SUMMARY AND CONCLUSIONS

The most significant conclusion that can be drawn from this experimental project is that both sections with permeable base courses, regardless of type, 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.

Both of these sections provide rapid subdrainage to water entering the pavement section, so that moisture is not available to erode the base or saturate the concrete slab. Thus, joint faulting is minimal.

11. ADDITIONAL READING

Kozlov, G. S., V. Mottola, and G. Mehalchick, "Improved Drainage and Frost Action Criteria for New Jersey Pavement Design, Volume 1-Investigations for Subsurface Drainage Design," FHWA/NJ-84/003, June 1983.

CHAPTER 26 U.S. 101 — GEYSERVILLE, CALIFORNIA

1. INTRODUCTION

In 1975, an experimental project was constructed on U.S. 101 near Geyserville, California. The experiment was conducted to examine the effects of different shoulder types on the performance of jointed plain concrete pavement (JPCP) performance in California. It was also conducted to investigate the effect of joint sealing (which California traditionally has not done) on pavement performance.

The project contained seven experimental sections. Two of these sections were constructed with tied concrete shoulders, two were constructed with nontied concrete shoulders, and three sections were constructed with various asphalt shoulders. The transverse joints on the sections with the AC shoulders were not sealed, whereas one section with tied concrete shoulders and one section with nontied concrete shoulders contained sealed transverse joints.

Three of the seven possible sections were surveyed in 1987. These include a section with a tied concrete shoulder and unsealed transverse joints (designated CA 3-1), a section with a tied concrete shoulder and sealed transverse joints (designated CA 3-2) and a section with nontied concrete shoulders and unsealed transverse joints (designated CA 3-5). The concrete shoulders were 10 ft (3 m) wide, 9 in (229 mm) thick at the pavement edge and tapering to 6 in (152 mm) thick at the outer shoulder edge. When tied, #4 (0.50-in [13 mm] diameter) bars, 22 in (559 mm) long were used at a lateral spacing of 30 in (762 mm).

Each section was constructed as a 9-in (229 mm) JPCP over a 5.4-in (137 mm) cement-treated base (CTB) and a 6-in (152 mm) aggregate subbase. The sections also had a skewed, random joint spacing of 12-13-19-18 ft (3.7-4.0-5.8-5.5 m). The subgrade for the project ranged from AASHTO classification A-4 to A-6. Where applicable, the transverse joints were sealed with a hot-poured asphalt sealant, with a shape factor of 0.42. A complete listing of the original design and construction information for each section is provided in the summary tables of chapter 32.

2. CLIMATE

The pavement sections are located in central California in the wet-nonfreeze environmental zone. The project site has a Thornthwaite Moisture Index of 49, a Corps of Engineers Freezing Index of 0, and receives 43.7 in (1110 mm) of precipitation annually. The highest average monthly maximum temperature is 91 $^{\circ}$F (33 $^{\circ}$C) and the lowest average monthly minimum temperature is 37 $^{\circ}$F (3 $^{\circ}$C).

3. TRAFFIC

The roadway has two lanes in each direction and has a functional classification of Principal Arterial. Since being opened to traffic in 1975, the pavement has sustained approximately 3.6 million 18-kip (80 kN) Equivalent Single-Axle Load (ESAL) applications in the outer lane and nearly 0.7 million ESAL applications in the inner lane. These values represent calculations through 1987. The two-way Average Daily Traffic (ADT) in 1985 was 15,400 vehicles, including 14 percent heavy trucks.

4. MAINTENANCE AND REHABILITATION

Two of the surveyed sections have required maintenance. On CA 3-1, an 80-ft (24.4 m) segment was damaged by flooding and received a substantial amount of bituminous patching. In addition, partial-depth spall repairs were placed at selected joints of CA 3-5. CA 3-2 has not required any maintenance or rehabilitation.

5. PHYSICAL TESTING RESULTS

Cores from slab centers and from typical transverse joints were retrieved from the pavement sections in 1987. The center cores were tested in split tensile to provide an indication of the concrete strength. The split tensile strength values were used to obtain an estimate of the modulus of rupture, as provided in chapter 32. A mean modulus of rupture of 819 psi (5.6 MPa) was obtained, which is indicative of sound concrete.

The joint cores were visually inspected for signs of deterioration beneath the joint or for any signs of material problems. The joint cores taken from the pavement sections indicated that there was no deterioration beneath the joints. However, microcracking in the aggregate was observed in all of the cores. It ranged from low- to high-severity along the length of the project.

Deflection testing was performed on the sections in 1987 for purposes of layer moduli characterization, determination of load transfer efficiencies, and void detection. The elastic modulus of the concrete (E), the composite k-value (on top of the base), and the load transfer values are summarized for each section in chapter 32.

The slab E values averaged 4,027,000 psi (27,770 MPa) for all three sections. The k-value on top of the base course for each section is shown in table 99.

Table 99. Composite k-values at CA 3.

Section	k-value, pci
CA 3-1	286
CA 3-2	312
CA 3-5	397

The k-value on top of the base course is typical for a CTB over a fine-grained subgrade. The subgrade varied along the length of the project, and was classified as an A-4 or an A-6 material for all three projects.

The center slab deflections were fairly consistent along the project. The average center slab deflection ranged from 3.4 mils to 4.3 mils.

The transverse joint load transfer efficiencies ranged from 74 to 86 percent for the three sections. The average load transfer efficiency was also calculated across the longitudinal lane-shoulder joint. The average longitudinal lane-shoulder and transverse joint load transfer efficiencies, as well as the average corner deflections, are tabulated in table 100.

Table 100. FWD deflection testing results at CA 3.

Section	Shoulder Type	Avg. Corner Deflection, mils	Transverse Joint Load Transfer, %	Lane-Shoulder Joint Load Transfer, %
CA 3-1	Tied PCC	3.8	79	70
CA 3-2	Tied PCC	3.1	86	85
CA 3-5	Nontied PCC	6.0	74	55

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.

6. DRAINABILITY OF PAVEMENT SECTIONS

A drainage analysis was performed on each section to evaluate the section's ability to remove water from the pavement structure. It was found that the subgrade material for all three sections was well-drained. The sections are in an area with a potential for the pavement slabs to be saturated approximately 17 percent of the time. The results of the drainage analysis are presented in table 101.

Table 101. Drainage summary for CA 3.

Section	Subbase Permeability, ft/hr	AASHTO Drainage Coefficient	Overall Drainability
CA 3-1	13.40	0.90	Fair-Good
CA 3-2	33.57	1.00	Fair-Good
CA 3-5	2.05	0.90	Fair-Good

The CTB is assumed to be impermeable, but the aggregate subbase material beneath the base had extremely good drainability. This, in combination with the well-drained subgrade, provided the sections with better than average drainability.

7. DETERIORATION OF PAVEMENT SECTIONS

In order to better present the results, the primary performance indicators are tabulated in figure 23 (outer lane) and Figure 24 (inner lane). The relative performance of each pavement section with respect to the primary performance indicators are discussed below for the outer lane only.

Joint Spalling

Any evaluation of joint spalling must consider the fact that the aggregate materials were observed to exhibit reactive aggregate distress. The presence of reactive aggregate can, over time, result in large amounts of joint spalling.

All three sections surveyed exhibited some degree of medium- to high-severity joint spalling. Table 102 summarizes the spalling for the three sections.

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	AVG.
JOINTS SEALED		CA 3-1		
	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 23. Outer lane performance data for California 3.

CALIFORNIA 3 U.S. 101 GEYSERVILLE

INNER LANE PERFORMANCE DATA

		9 in JPCP 12-13-19-18 ft JOINTS		
		TIED PCC SHOULDER	NON TIED PCC SHOULDER	AVG.
JOINTS SEALED		CA 3-1		
	PSR	3.8		3.8
	ROUGHNESS, in/mi	111		111
	FAULTING, in	N/A		N/A
	T. CKS./mi	5		5
	LONG. CKS., ft/mi	0		0
	% JT. SPALL.	0		0
JOINTS UNSEALED		CA 3-2	CA 3-5	
	PSR	4.1	3.9	4.0
	ROUGHNESS, in/mi	82	101	92
	FAULTING, in	N/A	N/A	N/A
	T. CKS./mi	0	28	14
	LONG. CKS., ft/mi	0	0	0
	% JT. SPALL.	0	2	1
AVG.	PSR	4.0	3.9	
	ROUGHNESS, in/mi	96	101	
	FAULTING, in	N/A	N/A	
	T. CKS./mi	2	28	
	LONG. CKS., ft/mi	0	0	
	% JT. SPALL.	0	2	

Figure 24. Inner lane performance data for California 3.

Table 102. Transverse joint spalling at CA 3.

Section	Transverse Joint Sealant	Joint Spalling, %
CA 3-1	Yes	1.6
CA 3-2	No	3.3
CA 3-5	No	6.4

The section which exhibited the least amount of spalling was CA 3-1, whose joints were sealed. The joint sealant appears to keep the incompressibles out of the joints, thus reducing joint spalling. However, the amount of spalling that has occurred for all three sections is so small that it can be considered insignificant.

Joint Faulting

Joint faulting for the three sections is shown in the table 103. Although the faulting values for all three sections are relatively close

Table 103. Transverse joint faulting at CA 3.

Section	Shoulder Type	Joint Sealant	Average Faulting, in	Highest Faulting, in
CA 3-1	Tied	Yes	0.08	0.20
CA 3-2	Tied	No	0.11	0.25
CA 3-5	Nontied	No	0.10	0.18

to one another, the section with sealed transverse joints appeared to perform best in terms of faulting. The sections without transverse joint sealant had more faulting. This may be attributed to the joint sealant keeping the water out of the pavement system, thus reducing the amount of free water available for pumping. The average faulting for these sections is quite high after only 12 years, especially for the sections with tied shoulders.

Transverse Cracking

Very little transverse cracking occurred on sections with the tied PCC shoulders. The section with the nontied PCC shoulders showed considerably more transverse cracking. The amount of cracking for each section is shown in table 104.

Table 104. Transverse cracking at CA 3.

Section	Shoulder Type	Transverse Cracks/mile
CA 3-1	Tied	27
CA 3-2	Tied	6
CA 3-5	Nontied	123

It is believed that the tied concrete shoulders provided more support to the slabs than the nontied concrete shoulders as illustrated by higher load transfer and lower corner deflections. The tied shoulder apparently decreases the edge stresses caused by the traffic loading and thereby reduces the amount of transverse cracking. The transverse cracking occurred in all slab lengths, not just the longer slab lengths.

Longitudinal Cracking

Only one section, CA 3-5, displayed longitudinal cracking. However, the amount exhibited by this section (34 ft [10.4 m]) was not substantial. The longitudinal joint was formed with an insert to a depth of 2 in (51 mm).

Present Serviceability Rating (PSR) and Roughness

Roughness was measured on the pavement sections with a Mays Ridemeter. In conjunction with this, the pavements' rideability was evaluated with a Present Serviceability Rating (PSR). The rideability of the sections varied as shown in table 105.

Table 105. Roughness and present serviceability at CA 3.

Section	Shoulder Type	Average Faulting, in	Transverse Cracks/mile	Roughness, in/mi	PSR
CA 3-1	Tied	0.08	27	134	3.6
CA 3-2	Tied	0.11	6	104	3.9
CA 3-5	Nontied	0.10	123	102	3.8

The smoothest riding section after 12 years is the section with the nontied PCC shoulders which, surprisingly, had high faulting and the most cracking. However, there is really no discernible difference between CA 3-2 and CA 3-5. It is believed that the higher roughness index (and lower panel rating) for section CA 3-1 can be attributed to the substantial amount of bituminous patching that had been performed within an 80-ft (24.4 m) segment of that section.

Other Pavement Distress

Low-severity reactive aggregate distress was observed in all three sections.

8. EFFECT OF DESIGN FEATURES ON PAVEMENT PERFORMANCE

Two design features can be evaluated from this experiment. These are the effect of tied and nontied shoulders, and the effect of transverse joint sealing on pavement performance. The relative effect of these features is discussed in the following sections.

Tied Concrete Shoulders

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. The information collected for the shoulder is presented in table 106.

Table 106. Outer shoulder performance at CA 3.

Section	Longitudinal Joint Sealant Condition	Average Drop-off, in	Average Separation, in	Shoulder Condition
CA 3-1	Excellent	0.1	0.3	Good
CA 3-2	Poor	0.0	0.2	Good
CA 3-5	Poor	0.0	0.3	Excellent

While the overall condition of the nontied shoulder was excellent (compared to "good" condition for each of the tied shoulders), the sealant in the longitudinal lane-shoulder joint was in poor condition. Also, recall that the nontied section had poor load transfer across that longitudinal lane-shoulder joint and also exhibited high corner deflections.

Joint Sealant

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 faulting) and will prevent the infiltration of incompressibles (thereby reducing joint spalling).

9. COMPARISON OF OUTER AND INNER LANE PERFORMANCE

Large traffic volumes prevented the determination of roughness and PSR in the inner lane. Faulting was also not obtained in the inner lane for the same reason. Nevertheless, other distresses occurring in the inner lane were noted. This information is presented in table 107, which summarizes the outer and inner lane distresses for all sections.

Table 107. Comparison of performance by lane at CA 3.

CA 3-1

Variable	Inner Lane	Outer Lane
ESAL's (millions)	0.7	3.6
Joint Spalling, %	0	2
Joint Faulting, in	N/A	0.08
Trans. Cracks/mile	0	27
Long. Cracks, ft/mile	5	0
Roughness, in/mi	111	134
Pumping	None	None
PSR	3.8	3.6

Table 107. Comparison of performance by lane at CA 3. (Cont'd)

CA 3-2

Variable	Inner Lane	Outer Lane
ESAL's (millions)	0.7	3.6
Joint Spalling, %	0	3
Joint Faulting, in	N/A	0.11
Trans. Cracks/mile	0	6
Long. Cracks, ft/mile	0	0
Roughness, in/mi	82	104
Pumping	None	None
PSR	4.1	3.9

CA 3-5

Variable	Inner Lane	Outer Lane
ESAL's (millions)	0.7	3.6
Joint Spalling, %	2	6
Joint Faulting, in	N/A	0.10
Trans. Cracks/mile	28	123
Long. Cracks, ft/mile	0	34
Roughness, in/mi	101	102
Pumping	None	None
PSR	3.9	3.8

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 panel rating 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 was 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.

10. SUMMARY AND 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 part of concrete shoulder construction that should not be omitted.

Conclusions in a report pertaining to this project include: (17)

- The best performance was demonstrated by the sections with tied PCC shoulders and sealed transverse joints. Some separation was observed on the sections with nontied PCC shoulders.
- Sections with full-depth asphalt shoulders are also performing well.

11. ADDITIONAL READING

Wells, G. K., "Evaluation of Edge Drain Performance," Report No. FHWA/CA/TL-85/15, California Department of Transportation, November 1987.

Neal, B. F. and J. H. Woodstrom, "Performance of PCC Pavements in California," Report No. FHWA/CA/TL-78/06, California Department of Transportation, February 1978.

CHAPTER 27 INTERSTATE 95 — ROCKY MOUNT, NORTH CAROLINA

1. INTRODUCTION

This experimental project, located on I-95 near Rocky Mount, North Carolina, was constructed in 1967. The experiment was conducted to evaluate the effects of base type, slab thickness, and load transfer on pavement performance. Design variables include: aggregate, asphalt-treated, cement-treated, and soil-cement base courses; 8-in (203 mm) JRCP with 60-ft (18.2 m) joint spacing and 9-in (229 mm) JPCP with 30-ft (9.1 m) joint spacing; skewed and nonskewed joints; and doweled and nondoweled joints. The cement-treated base incorporated 8 percent portland cement and the soil-cement base incorporated 6 percent portland cement. The subgrade for the project was an AASHTO Classification A-6 fine-grained soil.

Eight sections, designated as NC 1-1 through NC 1-8, were surveyed and tested in 1987. A complete listing of the original design and construction information for each section, as well as other pertinent project information, is provided in the summary tables of chapter 32.

2. CLIMATE

The pavement sections are located in northeastern North Carolina, in the wet-nonfreeze environmental zone. The project site has a Thornthwaite Moisture Index of 29, a Corps of Engineers Freezing Index of 0, and receives an average of 46 in (1168 mm) of rainfall annually. The highest average monthly maximum temperature is 89 $^{\circ}$F (32 $^{\circ}$C) and the lowest average monthly daily minimum temperature is 29 $^{\circ}$F (-2 $^{\circ}$C).

3. TRAFFIC

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) Equivalent Single-Axle Load (ESAL) applications in the outer lane and over 1.9 million ESAL applications in the inner lane. These values represent calculations through 1987. The two-way Average Daily Traffic (ADT) in 1987 was approximately 19,100 vehicles, including 9 percent heavy trucks.

4. MAINTENANCE AND REHABILITATION

According to records furnished by the North Carolina Department of Transportation, no major rehabilitation has been performed on the pavement sections to date. However, the joints on the project have been periodically resealed with a cutback asphalt.

5. PHYSICAL TESTING RESULTS

Cores from slab centers and from typical transverse joints were retrieved from the pavement sections in 1987. The center cores were tested in split tensile to provide an indication of the concrete strength. The joint cores were visually inspected for signs of deterioration beneath the joint or for any signs of material durability distress.

The split tensile strength values were used to obtain an estimate of the modulus of rupture. These values are reported in chapter 32. A mean modulus of rupture of 676 psi (4.7 MPa) was obtained for all sections.

The joint cores taken from the pavement indicated that no significant deterioration was present directly beneath the joint. However, a small amount of microcracking in the coarse aggregate was present for a few of the pavement sections.

Deflection testing was performed on the sections in 1987 for purposes of layer moduli characterization, determination of load transfer efficiencies, and void detection. The elastic modulus of the concrete (E), the composite k-value (on top of the base), and the load transfer values are summarized for each section in chapter 32.

The slab E values averaged 4,744,000 psi (32,710 MPa) for all of the sections. The k-values and the average load transfer efficiencies for the sections varied by base type, as shown in table 108.

Table 108. Composite k-values and load transfer efficiency at NC 1.

Base Type	k-value, pci	Joint Load Transfer, %
Aggregate	437	50
Asphalt-Treated	672	58
Cement-Treated	628	92
Soil-Cement	420	100

Consistent with the k-values reported above, the center slab deflections were largest for the soil-cement base course sections and smallest for the asphalt-treated base course sections. As can be observed, the load transfer efficiencies were very dependent upon base type and less dependent 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. These are shown in table 109. It appears that some erosion is occurring beneath the aggregate base course sections.

Table 109. Percent corners with voids at NC 1.

Base Type	Dowels	% Corners Exhibiting Voids
Aggregate	Yes	15
Aggregate	No	20
Soil-Cement	Yes	5
Soil-Cement	No	0
CTB	No	15
ATB	No	0

6. DRAINABILITY OF PAVEMENT SECTIONS

To evaluate each section's ability to remove water from the pavement structure, a drainage analysis was performed on each section. The sections were determined to be in an area that has the potential for the pavement to be saturated 22 percent of the time. The subgrade drainability ranged from poorly-drained (NC 1-1 and NC 1-4) to well-drained (NC 1-7). The drainability of the other sections fall somewhere between these two extremes. The results of the drainage analysis are summarized in table 110.

Table 110. Drainage summary for NC 1 sections.

Section	Base Permeability, ft/hr	AASHTO Drainage Coefficient	Overall Drainability
NC 1-1	0.12	0.75	V. Poor-Poor
NC 1-2	0.00	0.80	Poor-Fair
NC 1-3	0.00	0.80	Fair
NC 1-4	0.09	0.75	V. Poor-Poor
NC 1-5	0.00	0.75	Poor-Fair
NC 1-6	0.00	0.90	Fair
NC 1-7	0.13	0.90	Poor-Fair
NC 1-8	0.14	0.80	Poor-Fair

It is observed that 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.

7. DETERIORATION OF PAVEMENT SECTIONS

In order to better present the results of the extensive distress survey conducted on each pavement section, as well as to provide the factorial design of the experiment, the primary performance indicators are tabulated in figure 25 (outer lane) and figure 26 (inner lane). Although the experimental layout does not represent a full factorial design, several performance trends can still be identified. These trends are discussed below relative to the primary performance indicators for the outer lane only.

Joint Spalling

Of the eight sections evaluated, only one showed any joint spalling. The 3 percent observed on that section is not significant.

Joint Faulting

For the JPCP sections with 30-ft (9.1 m) perpendicular joints, direct comparisons can be made for the different base types shown in table 111.

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 25. Outer lane performance data for North Carolina 1.

NORTH CAROLINA 1
I-95 ROCKY MOUNT, NC
INNER 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				0.05	0.05
		T. CKS./mi				0	0
		LONG. CKS., ft/mi				0	0
		% JT. SPALL.				5	5
9 in PCC	AGG		NC 1-1	NC 1-4	NC 1-8		
		PSR	3.8	3.4	3.3		3.5
		ROUGHNESS, in/mi	74	85	69		76
		FAULTING, in	0.02	0.03	0.07		0.04
		T. CKS./mi	0	0	5		2
		LONG. CKS., ft/mi	0	0	0		0
		% JT. SPALL.	0	0	0		0
	SOIL CEM			NC 1-2	NC 1-3		
		PSR		3.4	3.4		3.4
		ROUGHNESS, in/mi		65	70		68
		FAULTING, in		0.04	0.09		0.06
		T. CKS./mi		0	0		0
		LONG. CKS., ft/mi		0	0		0
		% JT. SPALL.		0	0		0
	CTB				NC 1-5		
		PSR			3.4		3.4
		ROUGHNESS, in/mi			84		84
		FAULTING, in			0.08		0.08
		T. CKS./mi			0		0
		LONG. CKS., ft/mi			0		0
		% JT. SPALL.			0		0
	ATB				NC 1-6		
		PSR			3.4		3.4
		ROUGHNESS, in/mi			74		74
		FAULTING, in			0.04		0.04
		T. CKS./mi			0		0
		LONG. CKS., ft/mi			0		0
		% JT. SPALL.			0		0
AVG.		PSR	3.8	3.4	3.4	3.4	
		ROUGHNESS, in/mi	74	75	74	85	
		FAULTING, in	0.02	0.04	0.07	0.05	
		T. CKS./mi	0	0	1	0	
		LONG. CKS., ft/mi	0	0	0	0	
		% JT. SPALL.	0	0	0	5	

Figure 26. Inner lane performance data for North Carolina 1.

Table 111. Transverse joint faulting by load transfer type.

	FAULTING, in	
Base Type	No Dowels	Dowels
Aggregate	0.22	0.13
Soil-Cement	0.13	0.16
CTB	0.16	—
ATB	0.05	—

The faulting is lowest for the asphalt-treated base section, while the other treated base types have approximately the same amount of faulting. The faulting for the aggregate base types was the highest. With regards to load transfer, there seems to be some benefit in doweling joints for the aggregate base course sections, but this does not appear to be the case for the soil-cement base course sections. However, it should be noted that the dowel bars were only 1 in (25 mm) in diameter and they received no corrosion protection. It is likely that the 1-in (25 mm) dowel bars are of insufficient size to adequately provide load transfer. Moreover, it is possible that the "effective diameter" of the dowels has been reduced due to corrosion of the devices.

There appears to be some benefit gained by the skewing of transverse joints. The aggregate base course section with skewed, nondoweled joints had approximately half the amount of faulting that the aggregate base course section with perpendicular, nondoweled joints had.

A comparison of jointed reinforced and jointed plain pavements can be made, although confounded by joint spacing and slab thickness. Nevertheless, the doweled JRCP aggregate base course section and the doweled JPCP aggregate base course section had approximately the same amount of faulting.

Best Performance

Based on the results from the limited factorial, the pavement section with the best performance, in terms of joint faulting, had the following design features:

- 9-in (229 mm) slab on asphalt-treated base course with perpendicular joints and no load transfer devices.

Worst Performance

The pavement sections displaying the worst performance, in terms of joint faulting, had the following characteristics:

- Any section with an aggregate base course.

Transverse Cracking

Only four sections displayed transverse cracking. For JPCP, all severity levels of transverse cracking are counted. It was observed that the JPCP section with perpendicular joints, no load transfer devices, and an aggregate base exhibited 64 deteriorated cracks per mile, approaching the critical value of 70 cracks per mile where rehabilitation is needed. Two of the remaining sections displaying transverse cracking were constructed on soil-cement base course. However, the total number of deteriorated transverse cracks per mile for these sections averaged only 7.5. The last section exhibiting cracking was the JPCP with skewed joints and an aggregate base course, with only five cracks per mile. The absence of transverse cracking in the 30-ft (9.1 m) slabs, and the 60-ft (18.2 m) JRCP slabs is surprising, given the heavy traffic levels and the large k-values.

Longitudinal Cracking

Longitudinal cracking, measured in feet per mile, was not observed in the inner lane but was observed in four sections of the outer lane, as shown in table 112.

Table 112. Longitudinal cracking at NC 1.

Pavement Type	Base Type	Load Transfer Devices	Longitudinal Cracking, ft/mi
JPCP	Soil-Cement	No	3068
JPCP	Soil-Cement	Yes	1451
JPCP	CTB	No	179
JRCP	Aggregate	Yes	295

As can be observed, a substantial amount of cracking occurred on the soil-cement base course sections, while the other sections exhibited far less longitudinal cracking. It is interesting to note that two of the three treated base courses exhibited longitudinal cracking. The longitudinal joint was sawed to a depth of 2.75 in (70 mm), or 34 percent of the 8-in (203 mm) slab thickness and 31 percent of the 9-in (229 mm) slab thickness.

Best Performance

The pavement sections with the best performance, in terms of longitudinal cracking, had the following characteristics:

- o Any 9-in (229 mm) slab on either aggregate or asphalt-treated base course.

Worst Performance

The worst performance, in terms of longitudinal cracking, was recorded at pavement sections with the following design features:

- o Any 9-in (229 mm) slab on soil cement base course.

Present Serviceability Rating (PSR) and Roughness

The pavement roughness and PSR values for the sections were all close to one another. The largest PSR (and smallest roughness value) was for the asphalt-treated base course section, which was also the section with the least faulting. The smallest PSR (and largest roughness value) was for the cement-treated base course section, which had significant faulting and transverse cracking. The remaining PSR and roughness values were roughly identical.

8. EFFECT OF DESIGN FEATURES ON PAVEMENT PERFORMANCE

The relative effect of the various design features on the performance of the pavement are summarized below.

Base Type

The pavement section constructed over the asphalt-treated base displayed superior performance in every distress category over all of the other base types. 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. As has previously been mentioned, the longitudinal joint was sawed to a depth of 2.75 in (70 mm).

Joint Load Transfer

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.

It bears repeating that the 1-in (25 mm) dowel bars were probably too small to provide adequate long-term load transfer for the heavy truck loads sustained by the pavement. In addition, the dowel bars are probably corroded to some extent, thus reducing the "effective diameter" of the devices.

Pavement Type/Joint Spacing

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 (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 (reduced curling stresses) and the presence of the less rigid aggregate base course.

Joint Orientation

A direct comparison of skewed and perpendicular joints for JPCP constructed over aggregate base without load transfer devices is shown in table 113.

Table 113. Comparison of performance by transverse joint orientation.

Performance Indicator	Skewed Joints	Perpendicular Joints
Joint Spalling, %	3	0
Joint Faulting, in	0.12	0.22
Trans. Cracks/mile	5	64
Long. Cracks, ft/mile	0	0
PSR	3.4	3.7
Roughness, in/mile	83	95

For nondoweled joints at least, it appears that skewed joints offer some advantages over perpendicular joints, particularly in the level of faulting and measured roughness.

9. COMPARISON OF OUTER LANE AND INNER LANE PERFORMANCE

In addition to the outer lane, performance data was collected for the inner lane as well. Many of the same trends that were observed in the outer lane were also observed in the inner lane. An overall comparison of the two traffic lanes is presented by base type in table 114.

Table 114. Comparison of performance by lane and base type at NC 1.

AGGREGATE BASE COURSE (JPCP and JRCP)

Variable	Inner Lane	Outer Lane
ESAL's (millions)	1.9	9.1
Joint Spalling, %	1	1
Joint Faulting, in	0.04	0.16
Trans. Cracks/mile	1	18
Long. Cracks, ft/mile	0	74
PSR	3.5	3.5
Roughness, in/mile	78	87

SOIL CEMENT BASE COURSE

Variable	Inner Lane	Outer Lane
ESAL's (millions)	1.9	9.1
Joint Spalling, %	0	0
Joint Faulting, in	0.06	0.15
Trans. Cracks/mile	0	8
Long. Cracks, ft/mile	0	2260
PSR	3.4	3.6
Roughness, in/mile	68	78

Table 114. Comparison of performance by lane and base type at NC 1. (Cont'd)

CEMENT-TREATED BASE COURSE

Variable	Inner Lane	Outer Lane
ESAL's (millions)	1.9	9.1
Joint Spalling, %	0	0
Joint Faulting, in	0.08	0.16
Trans. Cracks/mile	0	0
Long. Cracks, ft/mile	0	179
PSR	3.4	3.2
Roughness, in/mile	84	105

ASPHALT-TREATED BASE COURSE

Variable	Inner Lane	Outer Lane
ESAL's (millions)	1.9	9.1
Joint Spalling, %	0	0
Joint Faulting, in	0.04	0.05
Trans. Cracks/mile	0	0
Long. Cracks, ft/mile	0	0
PSR	3.4	3.8
Roughness, in/mile	74	69

The primary differences in performance that exist between the inner and outer lane are in the amount of cracking and faulting. Most of the transverse cracking within the project occurred on the outer lane, as did all of the longitudinal cracking. The inner lane faulting is less than half that of the outer lane. Given these observations, it is surprising to note the closeness of the two lanes in terms of rideability, as evidenced by both the PSR and the roughness values.

The doweled aggregate base course sections had less faulting in the inner lane than the nondoweled aggregate base course section. The same is true for the asphalt-treated base course sections. The inner lane section with skewed joints also displayed superior performance over its counterpart with perpendicular joints.

10. SUMMARY AND CONCLUSIONS

The performance of the section constructed over the asphalt-treated base course was superior to all other designs within this experiment. This is contrary to what was observed in Minnesota, and may be due in part to the milder North Carolina climate. 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 also 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.

It is unclear why a substantial amount of longitudinal cracking occurred on the sections constructed over soil-cement base courses. The cause of the longitudinal cracking is probably related to the amount of friction between the base and slab. The soil-cement base courses must 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 or insufficient depth of sawing of the longitudinal joint.

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 of insufficient diameter to keep faulting to a reasonable level. The use of adequate dowel bars must be observed to achieve the desired levels of load transfer.

11. ADDITIONAL READING

"Experimental Project - Concrete Pavement, Interstate 95, Nash and Halifax County, North Carolina," Internal Project Reports, North Carolina Department of Transportation, 1968-1984.

CHAPTER 28 INTERSTATE 85 — GREENSBORO, NORTH CAROLINA

1. INTRODUCTION

This pavement is a six-lane Rural Interstate highway located on I-85 in the West Central part of the State. Designated as NC 2, this section was constructed and opened to traffic in 1982.

2. DESIGN

This section is an 11-in (279 mm) JPCP. It has perpendicular joints placed in a random pattern of 18-25-23-19 ft (5.5-7.6-7.0-5.8 m). The transverse joints contain 1.38-in (35 mm) diameter dowels bars and are sealed with a silicone sealant utilizing a shape factor of 0.13.

A 5-in (127 mm) lean concrete base is located beneath the slab. The subgrade is an AASHTO Classification A-4 material. The section has tied concrete shoulders that are 10 in (254 mm) thick and rest on the 5-in (127 mm) lean concrete base. No subdrainage was provided on the section.

3. CLIMATE

This section is located in a wet-nonfreeze climatic zone. The area has a Thornthwaite Moisture Index of 40, a Freezing Index of 0, and receives an average annual precipitation of 42 in (1067 mm). The highest average monthly maximum temperature is 87 ^{O}F (31 ^{O}C) while the lowest average monthly minimum temperature is 27 ^{O}F (-3 ^{O}C).

4. TRAFFIC

Traffic information provided by the North Carolina Department of Transportation indicated that in 1987 this section had a two-way ADT of 26,000 vehicles, including 17 percent trucks. At the time of survey, the pavement was 5 years old and had carried 5.8 million 18-kip (80 kN) ESAL's in the outer lane, 1.5 million ESAL's in the center lane, and 0.4 million ESAL's in the inner lane.

5. DRAINABILITY AND OTHER PHYSICAL TESTING RESULTS

The pavement consists of an impermeable lean concrete base and contains no positive subsurface drainage. However, the subgrade is somewhat well-draining, which should assist in the removal of excess moisture. The pavement section was assigned an AASHTO drainage coefficient of 1.00, indicating an overall drainability of fair to good.

From deflection testing, the backcalculated E of the slab was determined to be 5,890,000 psi (40,610 MPa). The modulus of rupture estimated from slab cores was 712 psi (4.9 MPa). The composite dynamic k-value on the top of the lean concrete base was 293 pci (80 kPa/mm).

The load transfer measured across the doweled transverse joints was 100 percent. The mean slab corner deflection (under a 9-kip (40 kN) load) was measured to be 7.5 mils. Slab corner deflection tests showed that only 5 percent of the corners were exhibiting loss of support.

6. MAINTENANCE AND REHABILITATION

Longitudinal drains were installed in the pavement in 1985. Hydraway drainage mats were placed at the outer edge of the shoulder. The mats were placed such that their top protruded 1.5 in (38 mm) above the shoulder surface-base interface. No other rehabilitation measures have been performed on the pavement.

7. PAVEMENT PERFORMANCE

After 5 years of service, this pavement section is beginning to show signs of distress. The major problem is the significant amount of pumping that is occurring in the outer traffic lane. Large stains are clearly evident throughout the project, occurring at the corners of transverse joints on both the mainline pavement and the shoulder. Pumping is also occasionally observed at the mid-slab edge between the mainline pavement and the tied shoulder. Performance data for the section is summarized in table 115.

Table 115. Comparison of performance by lane for NC 2.

Distress	Inner Lane	Center Lane	Outer Lane
ESAL's, millions	0.4	1.5	5.8
Joint Spalling, %	N/A	0	0
Joint Faulting, in	N/A	N/A	0.02
Pumping	N/A	None	High
Transverse Cracks/mile	0	5	10
Longitudinal Cracks, ft/mile	0	0	0
Roughness, in/mile	N/A	N/A	64
PSR	N/A	N/A	4.2

Due to high traffic volumes, certain distress items were not able to be determined in the inner two lanes.

The concrete shoulders were generally in good condition. However, as noted above, the transverse joints in the shoulder were exhibiting pumping, which was also noted along the lane-shoulder joint. Load transfer at the longitudinal lane-shoulder joint was determined to be 70 percent. The shoulders were tied with 0.62-in (16 mm) diameter, 30-in (762 mm) long tiebars, spaced at 30-in (762 mm) intervals.

8. CONCLUSIONS

Overall, this pavement section is performing fairly well. However, the extensive amount of pumping occurring on the section is a cause of concern. It is interesting that such high levels of pumping are observed over a section with a lean concrete base. The pumping is believed to be occurring between the lean concrete base and the concrete slab, as the pumped material displays the reddish-brown tint of the curing compound placed on the base.

There was a small amount of transverse cracking observed within the pavement section. These transverse cracks occurred only in the longer 25 ft (7.6 m) and 23-ft (7.0 m) slabs.

Even though this pavement is exhibiting extensive pumping, this distress has not yet translated into any other deterioration. It is believed that the substantial amount of pumping coupled with the heavy traffic loading will soon result in significant deterioration. The 1.38-in (35 mm) dowel bars have apparently minimized the amount of faulting occurring on this pavement section to date.

This pavement section exhibited an extensive amount of pumping soon after construction. Studies by the North Carolina Department of Transportation on this project have concluded that the longitudinal joints and the concrete-soil interface at the edge of the shoulders were the major sources of water infiltration. Further, the cross-sectional design for the pavement structure was determined to be a "trench" design that trapped water in the pavement structure. The pumping is apparently occurring from moisture trapped between the lean concrete base and the concrete slab.

9. ADDITIONAL READING

Wu, Shie-Shin, and T. M. Hearne, Jr., "Performance of Concrete Pavement With Econocrete Base," Paper Prepared for the Fourth International Conference on Concrete Pavement Design and Rehabilitation, Purdue University, April 1989.

CHAPTER 29 I-75 — TAMPA, FLORIDA (HILLSBOROUGH COUNTY)

1. INTRODUCTION

This pavement section, located in Hillsborough County near Tampa, Florida, is a six-lane divided Urban Interstate highway that was constructed in 1986. The pavement was not part of any experimental project and is designated as FL 2.

2. DESIGN

The section is JPCP, with a slab thickness of 13 in (330 mm) and a random skewed joint spacing of 12-18-19-13 ft (4.9-5.5-5.8-4.0 m). Transverse joint load transfer is provided by 1.25-in (32 mm) diameter, plastic-coated dowel bars. The transverse joints are sealed with a silicone sealant utilizing a shape factor of 0.17.

The base is 6 in (152 mm) of sand over an AASHTO Classification A-3 (sand) subgrade. The section has tied PCC shoulders that have the same slab and base materials and thickness as the traffic lanes. No subsurface drainage was provided to the pavement.

3. CLIMATE

This section is located in the wet-nonfreeze climatic zone. The area has a Thornthwaite Moisture Index of 4, a Freezing Index of 0, and receives an average of 47 in (1186 mm) of annual precipitation. The highest average monthly maximum temperature is 90 ^{o}F (32 ^{o}C) while the lowest average monthly minimum temperature is 50 ^{o}F (10 ^{o}C).

4. TRAFFIC

In 1987, this section had a two-way ADT of 28,700 vehicles, including 20 percent heavy trucks. The pavement has sustained 2 million 18-kip (80 kN) Equivalent Single-Axle Load (ESAL) applications in the outer lane, 0.8 million ESAL's in the center lane, and 0.2 million ESAL's in the inner lane.

5. DRAINABILITY AND OTHER PHYSICAL TESTING RESULTS

From field data, the permeability of the base was calculated to be 0.06 ft/day (0.02 m/day). The condition survey indicated no signs of poor drainage. The presence of the A-3 subgrade, being somewhat drainable, probably contributes to provide some drainability to the pavement structure.

Deflection measurements were obtained from the pavement sections using a Falling Weight Deflectometer. From FWD deflections, the E of the slab was backcalculated to be 5,550,000 psi (38,270 MPa), and the modulus of rupture estimated from cores was 664 psi (4.6 MPa). The composite dynamic k-value on top of the base was 378 pci (180 kPa/mm).

The load transfer efficiency of the transverse joints was extremely low, averaging 29 percent. The mean slab corner deflection under a 9-kip (40 kN) load was 7.6 mils, which is fairly low considering the poor load transfer. Slab corner deflection tests showed that only 10 percent of the corners had loss of support.

6. MAINTENANCE AND REHABILITATION

According to records provided by the Florida Department of Transportation, no maintenance or rehabilitation has been performed on this pavement.

7. PAVEMENT PERFORMANCE

As this pavement section was only 1 year old at the time of survey, it had essentially no deterioration. Faulting averaged 0.01 in (0.3 mm) and no signs of pumping were evident. No joint spalling or transverse and longitudinal cracks were found. The tied PCC shoulder was in excellent condition, with the load transfer across the lane-shoulder longitudinal joint calculated to be 100 percent. However, the relatively low PSR (3.7) may be an indication that some roughness was built in at construction. The performance information for this section is summarized in table 116.

Table 116. Pavement performance at FL 2.

Item	Center Lane	Outer Lane
ESAL's (millions)	0.8	2.0
Joint Spalling, %	0	0
Joint Faulting, in	N/A	0.01
Transverse Cracks/mile	0	0
Longitudinal Cracks, ft/mile	0	0
Roughness, in/mile	N/A	64
PSR	N/A	3.7

No deterioration was identified in the center lane. Due to very high traffic volumes, no faulting measurements were obtained in the center lane.

8. CONCLUSIONS

This 1-year-old pavement showed no deterioration at all and is in excellent condition. However, while faulting is minimal, it is expected that the low load transfer, combined with heavy traffic loading, will result in increased faulting of the transverse joints.

CHAPTER 30 INTERSTATE 75 — TAMPA, FLORIDA (MANATEE COUNTY)

1. INTRODUCTION

This pavement is a six-lane Rural Interstate highway constructed and opened to traffic in 1982. The project is four miles long and is located between Highways 70 and 64, in Manatee County near Tampa, Florida. The section is designated as FL 3.

2. DESIGN

The section is JPCP having a slab thickness of 9 in (229 mm), and a random skewed joint spacing of 16-17-23-22 ft (4.9-5.2-7.0-6.7 m). The transverse joints contain 1.00-in (25 mm) diameter, epoxy-coated dowels, and are sealed with a silicone sealant utilizing a shape factor of 0.17.

The base consists of lean concrete, 6 in (152 mm) thick. The subgrade is an AASHTO Classification A-3 sandy material. The section has tied lean concrete shoulders that have an average slab thickness of 7.5 in (191 mm), and a 6 in (152 mm) thick sand base. Complete design and construction information is provided in chapter 32 of this appendix.

3. CLIMATE

This section is located in a wet-nonfreeze climate zone. The area has a Thornthwaite Moisture Index of 16, a freezing index of 0, and receives an average annual precipitation of 58.7 in (1491 mm). The highest average monthly maximum temperature is 91 ^{O}F (33 ^{O}C) while the lowest average monthly minimum temperature is 50 ^{O}F (10 ^{O}C).

4. TRAFFIC

In 1987, this section had a two-way ADT of 32,700 vehicles, including 20 percent trucks. At the time of survey, the pavement was 5 years old and had carried 4.1 million 18-kip (80 kN) ESAL's in the outer lane, 1.7 million in the center lane, and 0.4 million in the inner lane.

5. DRAINABILITY AND OTHER PHYSICAL TESTING RESULTS

The pavement contains no positive subsurface drainage. A strip of filter fabric, 58 in (1473 mm) wide, was placed under the lean concrete shoulder, apparently to improve the flow of water beneath the lean concrete, which is assumed to have no permeability. The subgrade had very poor drainability, which resulted in the assignment of a drainage coefficient of 0.75.

FWD deflections and cores were taken on the section. The backcalculated E of the slab was determined to be 4,160,000 psi (28,680 MPa). The modulus of rupture estimated from cores was 599 psi (4.1 MPa). The composite dynamic k-value on the top of the lean concrete base was 529 pci (144 kPa/mm). The load transfer measured on the doweled transverse joints averaged only 19 percent, while the mean slab corner deflection (under a 9-kip (40 kN) load) was an extremely high 23.9 mils. Slab corner deflection tests showed that 73 percent of the corners were exhibiting loss of support.

6. MAINTENANCE AND REHABILITATION

Due to the large amounts of deterioration that has occurred on this project, extensive rehabilitation has been performed on the pavement. Primarily, this has included full- and partial-depth patching and joint and crack sealing. This was performed in 1984 and again in 1986.

7. PAVEMENT PERFORMANCE

For only being 5 years old, this pavement section is displaying a large amount of deterioration. This is summarized in the table 117. Due to high traffic volumes, certain distress items were not able to be determined in the inner two lanes.

Table 117. Comparison of performance by lane at FL 3.

Distress	Inner Lane	Center Lane	Outer Lane
ESAL's, millions	0.4	1.7	4.1
Joint Spalling, %	N/A	3	2
Joint Faulting, in	N/A	N/A	0.08
Pumping	Low	Low	Medium
Transverse Cracks/mile	80	250	310
Longitudinal Cracks, ft/mile	0	460	440
Roughness, in/mile	N/A	N/A	84
PSR	N/A	N/A	3.2

The major distress that has occurred is deteriorated transverse cracking, which has reached an excessive amount (generally 70 transverse cracks/mile is believed the 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 18-kip (80 kN) ESAL's, as seen from table 118.

Table 118. Transverse cracking at FL 3.

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

The slab lengths in the random joint spacing pattern ranged from a low of 16 ft (4.9 m) to a high of 23 ft (7.0 m). The effect of joint spacing on cracking for each traffic lane is shown in table 119.

Combined, the results from these two tables 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 (22 and 23 ft [6.7 and 7.0 m]) consistently had higher amounts of transverse cracking, and it should be noted that many of those larger slabs actually had two transverse cracks.

Table 119. Slab cracking as a function of slab length at FL 3.

Slab Length	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 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 slabs cracked. A general reduction in transverse cracking is observed as lower traffic loadings are endured.

The longer slab lengths produce more transverse cracking because of larger thermal curling and shrinkage stresses induced in the slab. This is particularly true for sections constructed over a stiff base, such as the lean concrete used here. The results show that for the heavy trafficked outer and center lanes, even the shorter 16- and 17-ft (4.9 and 5.2 m) slabs and 9 in (229 mm) slab thickness were inadequate to control cracking. When traffic was much lower, as in the inner traffic lane, no cracking was observed for the 16- and 17-ft (4.9 and 5.2 m) slabs.

Studies by Florida DOT indicate that no provision was made for positive drainage of subsurface moisture.(18) This resulted in water being trapped between the slab and the lean concrete base. The joints started to pump under heavy traffic loads, which resulted in erosion of the base material and, ultimately, slab cracking.

There also existed a fair amount of longitudinal cracking within the project. It also varied across traffic lanes, showing 440 ft/mi for the outer, 456 ft/mi for the center, and 0 ft/mi for the inner lane. Some of these cracks were diagonal, somewhat similar to corner breaks.

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 (fines on the surface). This is apparently from the lean concrete base, which may be eroding away. The 1-in (25.4 mm) diameter dowels may be insufficient to handle the heavy traffic loadings that are present on the project. Nearly three out of four 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 deflections were very high, indicating extensive pumping and loss of support exists along the outer lane. The net result of these factors was a low PSR in the outer lane of 3.2.

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.

8. 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. 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), and traffic loading stresses. There still remains 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.

9. ADDITIONAL READING

"Appropriateness of Design, Construction and Rehabilitation Actions on Concrete Pavement Sections of Interstate 75 in Sarasota and Manatee Counties," Office of Inspector General, Florida Department of Transportation, July 1986.

CHAPTER 31 KEY TO PROJECT SUMMARY TABLES

This chapter provides a key to the summary tables which are presented in chapter 32. Each category and columnar heading is described, a key to the abbreviations used in the tables are presented, and, where appropriate, sources of information used to complete the tables are provided. The summary tables in chapter 32 document and summarize all important project data, including environmental data, original design and construction data, traffic information, drainage information, and performance data.

1. GENERAL AND ENVIRONMENTAL DATA

The following is an explanation of the headings in the table "General and Environmental Data." Where applicable, the source of the entries in the columns are provided. All climatic data are, of necessity, obtained from the nearest data collection source.

ZONE. The U.S. is divided into climatic zones based on temperature and annual rainfall, from work done by Thornthwaite.(19) The divisions used are those proposed for use in SHRP.(20) The project sections are grouped according to their location in these zones to highlight the effect that climate has on pavement performance.

PROJECT LOCATION. Project sections are identified by their highway classification and number. They are further located by the nearest city.

TOTAL NUMBER OF EXPERIMENTAL SECTIONS. Many of the sections selected for this project are part of experiments in which one or several design features were evaluated. This column shows how many experimental sections were constructed at any given location. If the entry in this column is one, then there was no experiment conducted at this location. In that case, the section is included either as a control section for use with the experimental sections or as an example of new construction incorporating unique experimental design features.

NUMBER OF SECTIONS EVALUATED. The entry in this column provides the number of sections at each project location actually included in the study.

PROJECT AVERAGE SECTION LENGTH. Generally, an attempt was made to select sections 0.2 mi (1056 ft [322 m]) long. However, this was not always possible, due to a number of conditions which existed at the sites. The major limitation was that some sections were constructed less than 1056 ft (322 m).

THORNTHWAITE MOISTURE INDEX. This value indicates the amount of free moisture available in a particular area. A value of 0 indicates that, on average, there is no free moisture available. Positive values represent an excess of free moisture and negative values represent a high evapotranspiration potential. The Thornthwaite Moisture Index values are based on work described in reference 19.

CORPS OF ENGINEERS FREEZING INDEX. This index represents the number of degree days with a mean air temperature below freezing. Values can be obtained from reference 21.

HIGHEST AVERAGE MONTHLY MAXIMUM TEMPERATURE, °F. A substantial amount of climatological data are collected by the National Oceanic and Atmospheric Administration (NOAA) at locations in each state and summarized in the publication, "Climatography of the United States No. 81, Monthly Normals of Temperature, Precipitation, and Heating and Cooling Degree Days, 1951-1980."(22) The temperature data presented in this publication are the normal maximum, minimum, and average temperatures, summarized by month. In this column, the value from the month with the highest maximum temperature is reported.

LOWEST AVERAGE MONTHLY MINIMUM TEMPERATURE, °F. This data comes from the same source described above. The data presented in this column is the value from the month with the lowest minimum temperature.

AVERAGE MAXIMUM TEMPERATURE - AVERAGE MINIMUM TEMPERATURE. This temperature change is the difference between the highest average monthly maximum temperature and the lowest average monthly minimum temperature.

AVERAGE NUMBER OF DAYS OF PRECIPITATION PER YEAR. This is an average of the number of days during the year in which precipitation in the vicinity of the section is recorded. It represents an average over a 30-year period. This information is obtained from county soil reports, which are prepared by the United States Department of Agriculture, Soil Conservation Service (SCS).

ANNUAL AVERAGE PRECIPITATION. This is the average precipitation based on the climatic information available in reference 22.

DESIGN FEATURES EVALUATED. Project sections were selected for one of three reasons: they were experimental projects, they were a "control" section which could be used to compare performance with the experimental sections, or they were an example of a new design which included some experimental feature(s) being evaluated. This column identifies the reason for inclusion of the section in the study. Where the section was initially constructed as an experiment, the design features evaluated as part of the experiment are presented. If the section is included as new construction, the experimental features incorporated in its design are noted.

AVERAGE NUMBER OF ANNUAL FREEZE-THAW CYCLES. The average number of annual freeze-thaw cycles data at over 1300 sites was collected and recorded by the National Weather Bureau. This information was obtained from reference 23.

2. DESIGN DATA

The following is an explanation of the headings for the design tables. This includes the slab, base, subbase and subgrade design data, the outer shoulder data, and the joint information.

SLAB DESIGN DATA

PROJECT LOCATION. Project sections are identified by their highway designation and number, the nearest city, and the year of construction. The direction of the survey is also given. If the survey was conducted in one direction only, that is shown as EB, WB, NB, or SB. If sections on both sides of a four-lane (or greater) highway were surveyed, both directions are shown as EB&WB or NB&SB. If the section was located on a two-lane highway, the lane in each direction was surveyed. This is shown as E/W or N/S. Sections included as controls or as representative of new construction incorporating innovative design features are identified with their year of construction.

PROJECT SECTION ID. To aid in the overall organization of this project, each different project location within a State was identified by that State's abbreviation and a number. Each different design within that project was further identified by a number sequentially assigned as a suffix, thereby differentiating between sections.

PAVEMENT TYPE. Two types of pavements are included in this study, Jointed Plain Concrete Pavement (JPCP) and Jointed Reinforced Concrete Pavement (JRCP). The appropriate pavement type for each section is entered in this column.

PCC SURFACE

Thickness, in

The design thickness is obtained from plans, drawings, or reports provided by the State or reporting agency. Core thicknesses come from the coring performed as part of the field survey. The core thickness reported represents one to two center slab cores. This is not a valid sample size and it is not intended to suggest that the actual thickness of the slab is that obtained from the cores. The design thickness has been used in backcalculation procedures and ESAL calculations. for some projects, the core thickness was unavailable and an entry of N/A is made.

Joint Spacing, ft

The transverse joint spacing of the section is entered in this column. Joint spacings are either uniform or random. Random joint spacings are a sequence of four slab lengths that are repeated in a consistent pattern. All slab lengths are reported to the nearest foot.

% Steel, JRCP

Jointed reinforced concrete pavements have a small percentage of steel whose purpose is to keep transverse shrinkage cracks tight. The percent steel is calculated as follows:

$$\%\ STEEL = \frac{A_S * n}{t * 12}$$

where: A_s = cross-sectional area of the longitudinal steel, in
n = number of pieces of longitudinal steel per foot
t = thickness of the slab, in

This actually gives a value of percent steel/foot, which is very close to the percent steel calculated on the slab's entire cross section.

Skewed Joints, Yes/No

Skewed joints are transverse joints which are not constructed perpendicular to the longitudinal centerline. The standard practice is to have an offset of 2 ft in 12 and to construct the skew counterclockwise. Sections with skewed joints are identified by a "Y".

Load Transfer Devices (LTD's)

The most commonly encountered LTD's are dowels. If dowels are used, the dowel diameter, in inches, is shown. If no LTD is used, a diameter of 0.0 is entered. Several projects in New York used LTD's other than dowels. These are indicated in the summary tables as "ACME" for the two-part malleable iron load transfer devices or "I-BEAM" for steel I-beam bars.

Coating

Dowels are often placed with a coating to inhibit corrosion and to facilitate movement. The coatings most commonly used include paint and/or grease (P/G), epoxy, a plastic coating, stainless steel (ST STL), and liquid asphalt (LA). If there was no dowel, a series of three dashes is entered.

E, ksi

Young's modulus of elasticity (E) is estimated using backcalculation procedures. The FWD deflection basin and radius of relative stiffness are used to characterize the strength of the surface in terms of the dynamic modulus of elasticity. This figure is rounded to the nearest 10 ksi. The dynamic E is not the same as the E calculated with other correlations.

M_r, psi

The modulus of rupture reported here is an estimate of the value obtained from third point loading. It is calculated from the correlation $M_r = 1.02*F_t + 210$, where F_t is the split tensile strength from the cores collected during the field survey. This correlation is developed in work by Foxworthy.(24)

BASE DESIGN DATA

The first three columns in the table (Project Location, Project Section ID, and Pavement Type) are repeats of categories discussed in section 2. The base data begins in column 4.

Type

The base layer is the layer in the pavement system directly beneath the surface. Many different materials are used in the construction of base layers. The following list includes the abbreviations used for both base and subbase materials:

AGG:	gravel or crushed stone
SC:	cement-treated soil
ATB:	asphalt-treated base, usually a dense-graded hot-mix
SAND:	sand
CTB:	cement-treated base
LCB:	Lean concrete base (also econocrete)
PATB:	permeable asphalt-treated base
HMAC:	hot-mix asphalt concrete
PAGG:	permeable (nonstabilized) open-graded base
LFAS:	lime-fly ash stabilized base
LTSG:	lime-treated subgrade
NONE:	no base; slab is constructed directly on the subgrade

Thickness

The design thickness is obtained from plans, drawings, or reports provided by the State or reporting agency. Core thicknesses are measured from the coring performed as part of the field survey. The values represent one to two center slab cores. As with the slab core measurements, this is not a valid sample size and its inclusion here is not intended to imply that the actual thickness of the base is that obtained from the cores.

If there is not base layer, the design and core thicknesses are entered as three dashes. As is noted above, some core thicknesses were not available and are noted as N/A.

Estimated Permeability

The coefficient of permeability, k, is reported in units of ft/hr. The procedure used in this study to estimate the permeability of porous base and subbase layers follows that outlined in reference 25. The permeability equation in that reference has been worked into a computerized solution, DRAINIT.(26) The inputs to this solution include the effective grain size, D_{10}, the specific gravity, G_s, and the percent passing the No. 200 sieve. The types of fines and the general material type are also needed. All of the material properties were obtained from the coring and boring performed as part of the field surveys, with the exception of G_s. In some cases, that is available from project records. Where it is not available it is estimated. The estimated permeability of stabilized, nondraining layers (ATB, CTB, CAM, SC, LTSG, LFAS) are entered as zero. Several projects had in situ or laboratory drainage evaluations performed and, where available, these results are also presented. The study permeability value is presented first, separated by an in/in from the estimated permeability. If there is no base, the estimated permeability is recorded as three dashes. If insufficient material was obtained to estimate the permeability, an entry of N/A is made.

K_{eff}, (Dynamic)

The effective dynamic modulus of subgrade reaction on the base is backcalculated using a closed-form numerical procedure which evaluates the stiffness of the surface and subsurface layers in terms of dynamic loading. The FWD deflection basin and radius of relative stiffness are used to determine the dynamic k-value on top of the base. The dynamic k-value is approximately 50 percent higher than the static k-value due to the stress state induced by the dynamic load.

SUBBASE

The subbase is the layer of the pavement system located beneath the base. The descriptions of the categories and the entries are identical to those used in the section on bases.

AASHTO Subgrade Soil Type

The subgrade is the lowest layer of the pavement system. It is the existing material upon which the pavement system is constructed. The pavement may be constructed on the existing soil or it may rest on fill material. The AASHTO soil type is determined in accordance with AASHTO M-145. Results were obtained from the boring operations, construction reports, or county soil surveys.

OUTER SHOULDER DESIGN DATA

Type

Two surface types were observed on the outer shoulders in this project. They are asphalt concrete (AC) and portland cement concrete (PCC). In most cases, the PCC shoulders are tied to the mainline pavement with regularly spaced rebar.

Thickness

The thickness of the surface and base courses are provided for the outer shoulder. In certain cases, the cross section of the surface layer tapers from the pavement edge to the outer edge of the shoulder. Then the thickness provided is an average of the thickest and thinnest part of the cross section and is noted by an asterisk.

PAVEMENT TRANSVERSE JOINT DATA

Again there is some repetition of column headings for sake of clarity. These have been described elsewhere.

TRANSVERSE JOINT.

Dowel Diameter, in

This category has been previously described elsewhere.

Calculated Average Joint Opening, in

Joint movement is a function of slab length, temperature change, the thermal coefficient of expansion of the slab material, and the friction between the slab and the base. The calculated mean joint opening can be estimated from the following equation:

$$\Delta L = C * L * \alpha * \Delta T$$

where:
- ΔL = mean joint opening, inches
- C = an adjustment factor for base friction; 0.80 for granular material and 0.65 for stabilized material
- L = slab length, inches
- α = thermal coefficient of expansion of PCC, $5.5 * 10^{-6}$ in/in/^{o}F
- ΔT = design temperature change, from table 1, General and Environmental Data

Skewed Joints, Y/N

This category has been previously described.

Joint Sealant Shape Factor

The joint shape factor is the ratio of the joint reservoir width to the joint reservoir depth. This is based on the design and not on actual field measurements. If the joint was not sealed at construction and remained unsealed, the shape factor is 0.0. Note that the joint reservoir may be different than the initial sawcut of the transverse joint.

Joint Sealant

Type

The types of sealants used in the transverse joints and their abbreviations are presented below:

- PREF: preformed elastomeric compound
- HP: hot-poured bituminous material
- AC: asphalt cement
- SIL: silicone sealant
- RA: rubberized asphalt

Age

The age of the sealant at the time of the survey is recorded. This is not always the same as the age of the pavement, as some sections have been resealed.

Condition

The joint sealant condition was evaluated by severity during the field survey. The condition reported is the average condition, or the condition of the sealant in the majority of the joints. Only the outer lane is included in this rating. The following rating scheme is used:

SEVERITY LEVEL	CONDITION
NONE	EXCELLENT
LOW	GOOD
MODERATE	FAIR
HIGH	POOR

DEPTH OF LONGITUDINAL JOINT, IN. The depth of the longitudinal joint between lanes is obtained from construction records or other information supplied by the States. It was not measured in the field. If the lanes were placed at separate times and no joint was sawed or formed by an insert, this is recorded as N/A.

3. MONITORING DATA

This section describes selected monitoring information. Such items as deflection data, outer shoulder information, drainage information, and traffic information are included here.

DEFLECTION DATA — OUTER LANE

DEFLECTION, MILS.

Mid-Slab Deflections

A Falling Weight Deflectometer (FWD) was used to measure pavement deflections under a dynamic load. The test patterns used are illustrated in chapter 1 of appendix B. A series of four loads in a range from 7 kips (31 kN) to 17 kips (76 kN) were applied to the center of the slab and the resultant deflections were recorded by a set of sensors in thousandths of an inch. The deflections at the load closest to 9 kips (40 kN) were then "normalized" to 9 kips (40 kN) by plotting load vs. deflection and obtaining a deflection for each test. The results presented here for each section are the normalized high and low deflections recorded from the load plate sensor (D_o), and the average of all of the mid-slab deflections for the section.

Loaded Corner

As the above-mentioned figures show, FWD testing was performed at the corners of the slabs. Data collected at this location is used to determine load transfer at the joint and to determine the existence of voids under the slab corners. The loaded corner deflection is the deflection recorded by the sensor directly under the load plate (D_o). The value presented here is an average of all of the loaded corner deflections from the section.

Unloaded Corner

When deflection testing is performed in the corner of the slab, a sensor is placed opposite the loaded corner, on the unloaded corner. The deflections recorded from this sensor (D_1 or D_6) represent the unloaded corner deflections. This value is also an average of all of the unloaded corner deflections from the section.

ADJUSTED PERCENT LOAD TRANSFER EFFICIENCY. The general definition of load transfer (percent) is the deflection of the unloaded corner divided by the deflection of the loaded corner multiplied by 100. The adjusted load transfer is a corrected value to take into account the fact that the slab deflects under loading; the natural bending of the slab under load must be accounted for to more accurately model the deflection of the corner. The correction factor used is the average of D_0/D_1 for the section. The load transfer is multiplied by the correction factor to obtain the adjusted load transfer efficiency.

PERCENT LOAD TRANSFER ACROSS SHOULDER. The load transfer across the shoulder can be calculated from deflections measured across this joint, if the shoulder is PCC. The method of calculation is the same as that described above. If the shoulder is AC or AGG, the entry in this column is N/A.

AVERAGE NDT TEST TEMPERATURE, ^{o}F. FWD test results are somewhat sensitive to the temperature of the slab being tested. The average ambient temperature over the course of the testing is presented here.

PERCENT CORNERS WITH VOIDS. Using procedures developed under NCHRP 1-21 (reference 27), deflection measurements obtained at the slab corners can be used to identify the presence of voids under those corners. The percent of the corners tested which had voids is presented here.

OUTER SHOULDER INFORMATION

TYPE-THICKNESS, IN.

Surface

The layer type and its thickness are given for the surface. This information is obtained from plans, specifications or other sources made available by the States. The surface types are asphalt concrete (AC) and portland cement concrete (PCC).

Base

The base type and its thickness are given. The base types are the same as previously described. The thicknesses followed by an asterisk (*) indicate an average thickness. An average is given when the shoulder thickness changes from the pavement edge to the outer edge.

OVERALL SHOULDER CONDITION. The shoulder condition rating is a subjective evaluation made at the time of the field survey. The ratings used were excellent, good, fair, and poor. They are based on the amount of distress recorded on the shoulder.

SHOULDER JOINT SEAL CONDITION. As part of the drainage survey, the condition of the lane-shoulder joint sealant was evaluated for each section. The severity levels for the observed sealant distress were NONE, LOW, MODERATE, and HIGH. These correspond to an overall shoulder joint seal condition of excellent, good, fair, and poor.

DRAINAGE INFORMATION

PERMEABILITY, FT/HR.

Base

The calculation of the estimated permeability, k, of the base has been previously described.

Subbase

The calculation of the estimated permeability, k, of the subbase has been previously described.

Subgrade

The determination of the estimated permeability, k, of the subgrade is presented when made available from other sources.

AASHTO DRAINAGE COEFFICIENT, C_d. This parameter is an overall estimate of the drainability of the entire section or its ability to remove water from the pavement structure. It is based on a number of factors, including environment, layer permeabilities, time of saturation, longitudinal and transverse slopes, and material characteristics.

DOWELS, Y/N. This is the same as information presented earlier. It is included again as an aid to understanding the other data on this page.

SUBDRAINAGE TYPE. If the only pavement drainage is positive flow from the slope of the surface, this is recorded as NONE. Types of subdrainage included here are: a drainage blanket (DB) or draining layer, longitudinal edge drains (EDG DRNS), transverse drains, and a drainage blanket with longitudinal edge drains (DB/ED).

DEPTH TO DITCH, FT. The depth from the pavement edge to the bottom of the ditch line was estimated during the field survey. This number is only an estimate. If a value of 0 is entered, there is no drainage curbs and gutters or storm drains.

AVERAGE TRANSVERSE SLOPE, PERCENT. The average transverse slope of the outer lane of the pavement was measured at the beginning, middle, and end of the section, using a bubble level with a slope indicator.

The three values are averaged and converted from in/ft to a percentage. A negative value indicates that the outer lane sloped down toward the outer shoulder when facing in the direction of traffic.

AVERAGE LONGITUDINAL GRADE, PERCENT. The average longitudinal grade was also measured three times and the readings were averaged. A negative slope indicates that the pavement slopes down in the direction of the survey. In some cases the slope changed signs during the section. In those instances, the three readings are still averaged.

TRAFFIC INFORMATION

ORIGINAL DESIGN TRAFFIC

Very little original design traffic was available. The design traffic that was available is presented here.

ESAL's
This is the number of 18-kip (80 kN) Equivalent Single-Axle Load (ESAL) applications used for the design of the pavement.

Average Daily Traffic (ADT)
This is the two-way ADT used for the design of the pavement.

Percent Trucks
This is the percent of heavy trucks used for the design of the pavement.

AGE AT SURVEY The age of the pavement at the time of the survey is the number of years passed from the time of construction until the summer of 1987.

1987 ESTIMATED.

ADT
This is the 1987 two-way ADT obtained from the participating State agencies.

Percent Trucks
This is the 1987 truck percentage (excluding panels and pickups) obtained from the participating State agencies.

OUTER LANE

1987 ESAL Estimated From ADT and Percent Trucks
Using ADT and percent truck information provided by the State agencies, and truck weight information, the ESAL applications for 1987 were calculated and entered here. The outline of the traffic computations is given in chapter 3 of appendix B.

Estimated Accumulated ESAL's Through 1987
Using historical ADT and percent truck information provided by the State agencies, and truck weight information, the cumulative ESAL applications (from the date of opening to

traffic through 1987) were calculated and entered here. The outline of the traffic computations is given in chapter 3 of appendix B.

LANE #2

Lane #2 is the lane adjacent to the outer lane.

1987 ESAL Estimated From ADT and Percent Trucks
This is the same as described above for the outer lane.

Estimated Accumulated ESAL's Through 1987
This is the same as described above for the outer lane.

4. PERFORMANCE DATA

Key elements of each pavement section's performance are summarized for both the outer lane (lane 1) and the adjacent lane (lane 2). This data was collected during the field surveys conducted over a 6-month period during 1987. It is not possible to include all of the performance data in these tables; that information is available in the computerized database. Instead, key performance indicators are provided. The distress identification, rating of severity levels, and recording of quantities were all performed in accordance with guidelines presented in reference 28. Sections with more than two lanes in the direction of the survey were only visually surveyed for condition and distress in all lanes other than the outer lane due to safety considerations.

OUTER LANE PERFORMANCE DATA

LOAD TRANSFER DEVICE (LTD) DIAMETER, IN. This category of data is described elsewhere and is included here for reference only.

AVERAGE PRESENT SERVICEABILITY RATING. The PSR was recorded by two people while running the Mays Roughness survey. The PSR is a rating assigned to the pavement by the survey crew after driving over the pavement at the posted speed limit. The rating scale ranges from 0 (considered an "impassible" pavement) to 5 (considered a "perfect" pavement). The PSR is a highly subjective rating given the small sample size and is included for reference purposes only.

MAYS ROUGHNESS, IN/MI. A 1985 Buick Le Sabre equipped with a Mays Roughness Meter was used to perform a roughness survey on every section of the project. The vehicle, loaded to a fairly constant weight, made two passes over each section at 50 mi/hr (80 km/hr). The roughness readings from both passes were averaged to obtain the value presented in this table. If there were two lanes in the direction of the survey, roughness information was gathered for both lanes. However, if there were three or more lanes in the direction of the survey, PSR and roughness information were only collected in the outermost lane (lane 1).

AVERAGE TRANSVERSE FAULTING, IN. Hand measurements of the faulting of each transverse joint were recorded in the outer wheel path of each lane, for sections with fewer than three lanes in the direction of the survey, and in the outer lane only, for sections with three or more lanes in the direction of the survey. The average of the faulting measurements is given in this column.

DETERIORATED TRANSVERSE CRACKS/MI. The occurrence of transverse cracks and their severity was recorded during the field survey. For JPCP pavements, transverse cracks of low, moderate, and high severity are counted together and summed as deteriorated cracks per mile. For JRCP pavements, only transverse cracks of moderate and high severity are counted.

LONGITUDINAL CRACKING, LINEAR FT/MI. Longitudinal cracks of all severities were measured and recorded. The totals are summarized as a number of linear feet of longitudinal cracking per mile of pavement.

PUMPING. The entire section is given a rating for pumping based on the presence of the highest severity level of pumping noted during the field survey.

PERCENT OF TRANSVERSE JOINTS SPALLED. Three severity levels of transverse joint spalling are recognized and were recorded. Only medium- and high-severity joint spalling are summarized together and shown here. They are recorded as a percent of the total number of joints in the section.

MATERIALS DURABILITY DISTRESS. Materials problems usually occur over the entire length of the section. Typical problems noted are "D" cracking (DCRK) and reactive aggregate (RAG). Where no durability problem exists, this is noted as NONE.

LANE 2 PERFORMANCE DATA

The column headings for this table are the same as the ones previously described for "Outer Lane Performance Data."

Table 120. General information and design data for projects included in study.

ZONE	PROJECT LOCATION	Total Number of Exptl. Sects.	Number of Sects. Eval.	Project Average Section Length, FT	Thornthwaite Moisture Index	Corps of Engr. Freez. Index	No. of Freeze/Thaw Cycles/Year	Highest Average Daily Maximum Temp, °F	Lowest Average Daily Minimum Temp, °F	Average Max - Average Min, °F	Average Number of Days Precip/Year	Annual Average Precip, IN	Design Features Evaluated (See Codes)
DRY FREEZE	I-94 Rothsay, MN	24	12	729	0	2188	92	84	-3	87	109	23.4	1, 3, 6, 7
	I-94 Rothsay, MN	1	1	800	7	2188	92	84	-3	87	109	23.4	CONTROL
	I-90 Albert Lea, MN	4	4	1053	16	1688	90	85	6	79	106	29.7	1, 2, 4, 6, 7
	I-90 Austin, MN	1	1	1053	21	1250	90	83	2	81	110	31.2	NEW- 7
	TH 15 New Ulm, MN	1	1	1243	0	1800	80	86	2	84	105	28.1	NEW- 1,4,7,8
	TH 15 Truman, MN	1	1	1053	17	1800	88	83	1	82	105	30.2	NEW- 1,3,7,8
DRY NONFREEZE	RT 360 Phoenix, AZ	7	6	1059	-47	0	40	105	36	69	34	8.0	1, 3, 7, 8
	I-10 Phoenix, AZ	1	1	1041	-49	0	20	105	39	66	34	6.9	NEW- 3,4,6
	I-5 Tracy, CA	9	5	1056	-42	0	40	94	37	57	61	9.7	1, 3, 4
	I-5 Sacramento, CA	1	1	1069	-31	0	40	93	38	55	58	17.2	CONTROL (8)
	I-210 Los Angeles, CA	2	2	988	-29	0	40	89	41	48	40	15.6	3, 8
	US 101 1000 Oaks, CA	1	1	1056	-25	0	40	76	43	33	40	14.6	NEW-1,3,4,7,8
	RT 14 Solemint, CA	1	1	1072	-43	0	40	98	32	66	40	7.3	NEW- 3,4,8
WET FREEZE	US 10 Clare, MI	10	8	644	29	875	100	82	10	72	135	32.3	2,3,4,5,6,8
	I-69 Charlotte, MI	12	2	4068	22	563	100	84	13	71	144	31.8	7
	I-94 Marshall, MI	1	1	1066	27	563	100	83	15	68	144	34.0	NEW- 3,8
	I-94 Paw Paw, MI	1	1	1066	40	563	100	85	17	68	86	38.2	NEW- 3,8
	RT 23 Catskill, NY	30	6	599	54	500	80	82	9	73	136	38.9	2, 3, 4, 5, 6
	I-88 Otego, NY	4	4	1000	53	500	88	83	13	70	164	40.5	2, 4, 7
	RT 23 Chillicothe, OH	15	7	266	33	25	90	86	20	66	141	39.3	3, 4
	SR 2 Vermillion, OH	104	2	500	19	300	100	83	19	64	146	33.8	7
	HWY 3N Ruthven, ONT	4	4	1054	22	1000	100	80	19	61	128	32.3	1, 3, 7, 8
	HWY 427 Toronto, ONT	1	1	1054	13	1000	100	82	16	66	112	29.8	CONTROL
	RT 422 Kittanning, PA	5	5	965	53	300	100	83	15	68	162	41.2	3, 8
	RT 676 Camden, NJ	4	2	471	44	0	75	86	23	63	131	44.3	3, 8
	RT 130 Yardville, NJ	1	1	1099	37	0	80	85	25	60	131	42.5	CONTROL
WET NONFREEZE	US 101 Geyserville, CA	6	3	933	49	0	50	91	37	54	75	43.7	7
	I-95 Rocky Mount, NC	9	8	1065	29	0	63	89	29	60	122	46.4	1,2,3,4,5,6
	I-85 Greensboro, NC	1	1	1057	25	0	70	87	27	60	120	42.0	NEW- 1,3,4
	I-75 Tampa, FL (Hill.)	1	1	1078	4	0	0	90	50	40	115	46.7	NEW- 1,4,7
	I-75 Tampa, FL (Man.)	1	1	1057	16	0	0	91	50	41	115	58.7	NEW- 3,4,8
	TOTAL	264	95										

Design Features Codes:

1 = Slab Thickness
2 = Slab Type
3 = Base Type
4 = Joint Spacing
5 = Joint Configuration
6 = Load Transfer
7 = PCC Shoulder/ Widened Lanes
8 = Subdrainage

Table 121. Slab design data for projects in dry-freeze environmental zone.

Project Location (Year Constructed)	Project Section ID	Pvt. Type	PCC SURFACE: THICKNESS, IN: Design	THICKNESS, IN: Core	Joint Spacing, FT	% Steel JRCP	Skewed Joints Y/N	LTD'S: Dia., IN	LTD'S: Coating	E, KSI from FWD	Mr, PSI (from cores)
I-94 Rothsay, MN WB (1970)	MN 1-1	JRCP	9.0	N/A	27	0.08	Y	0.0	---	7090	---
	MN 1-2	JRCP	9.0	N/A	27	0.08	Y	1.0	P/G	7750	801
	MN 1-3	JRCP	8.0	N/A	27	0.09	Y	0.0	---	6660	---
	MN 1-4	JRCP	8.0	N/A	27	0.09	Y	1.0	P/G	6920	552
	MN 1-5	JRCP	8.0	N/A	27	0.09	Y	0.0	---	9130	---
	MN 1-6	JRCP	8.0	N/A	27	0.09	Y	1.0	P/G	9360	587
	MN 1-7	JRCP	9.0	N/A	27	0.08	Y	0.0	---	8300	---
	MN 1-8	JRCP	9.0	N/A	27	0.08	Y	1.0	P/G	7880	689
	MN 1-9	JRCP	9.0	N/A	27	0.08	Y	0.0	---	6670	---
	MN 1-10	JRCP	9.0	N/A	27	0.08	Y	1.0	P/G	6740	735
	MN 1-11	JRCP	8.0	N/A	27	0.09	Y	0.0	---	8030	---
	MN 1-12	JRCP	8.0	N/A	27	0.09	Y	1.0	P/G	7790	763
I-94 Rothsay, MN WB (1969- control)	MN 5	JRCP	9.0	N/A	39	0.04	N	1.0	P/G	7560	---
I-90 Albert Lea, MN EB (1977)	MN 2-1	JPCP	9.0	N/A	13-16-14-19	N/A	Y	1.0*	P/G	6780	682
	MN 2-2	JPCP	8.0	N/A	13-16-14-19	N/A	Y	1.0*	P/G	8010	---
	MN 2-3	JRCP	9.0	N/A	27	0.09	Y	1.0	P/G	7320	743
	MN 2-4	JRCP	9.0	N/A	27	0.09	Y	1.0	P/G	6620	---
I-90 Austin, MN EB (1984- new)	MN 3	JRCP	9.0	N/A	27	0.06	Y	1.0	EPOXY	8810	---
TH 15 New Ulm, MN N/S (1986- new)	MN 4	JPCP	7.5	N/A	13-16-14-17	N/A	Y	1.0	P/G	6300	832
TH 15 Truman, MN N/S (1983- new)	MN 6	JRCP	8.0	N/A	27	0.06	Y	1.0	P/G	6570	616

* Outer Lane Only

Table 122. Base, subbase, subgrade, and outer shoulder design data for projects in dry-freeze environmental zone.

Project Location (Year Constructed)	Project Section ID	Pvt. Type	BASE Type	BASE THICKNESS, IN Design	BASE THICKNESS, IN Core	BASE Estimated Perm., FT/HR	BASE Keff, (Dyn.) PCI	SUBBASE Type	SUBBASE THICKNESS, IN Design	SUBBASE THICKNESS, IN Core	SUBBASE Estmtd. Perm., FT/HR	AASHTO SUBGRADE Soil Type	OUTER SHOULDER Type	OUTER SHOULDER THICKNESS, IN Surface	OUTER SHOULDER THICKNESS, IN Base
I-94 Rothsay, MN WB (1970)	MN 1-1	JRCP	AGG	6.0	N/A	0.63	191	NONE	---	---	---	A-6	PCC	6.0	9.0
	MN 1-2	JRCP	AGG	6.0	N/A	0.17	172	NONE	---	---	---	A-6	AC	2.0	13.0
	MN 1-3	JRCP	AGG	6.0	N/A	0.73	217	NONE	---	---	---	A-6	PCC	6.0	8.0
	MN 1-4	JRCP	AGG	6.0	N/A	0.77	222	NONE	---	---	---	A-6	AC	2.0	12.0
	MN 1-5	JRCP	ATB	5.0	N/A	0.00	304	NONE	---	---	---	A-6	AC	2.0	11.0
	MN 1-6	JRCP	ATB	5.0	N/A	0.00	314	NONE	---	---	---	A-6	AC	2.0	11.0
	MN 1-7	JRCP	ATB	5.0	N/A	0.00	287	NONE	---	---	---	A-6	AC	2.0	12.0
	MN 1-8	JRCP	ATB	5.0	N/A	0.00	278	NONE	---	---	---	A-6	AC	2.0	12.0
	MN 1-9	JRCP	CTB	5.0	N/A	0.00	291	NONE	---	---	---	A-6	PCC	6.0	8.0
	MN 1-10	JRCP	CTB	5.0	N/A	0.00	285	NONE	---	---	---	A-6	AC	2.0	12.0
	MN 1-11	JRCP	CTB	5.0	N/A	0.00	245	NONE	---	---	---	A-6	PCC	6.0	7.0
	MN 1-12	JRCP	CTB	5.0	N/A	0.00	239	NONE	---	---	---	A-6	AC	2.0	11.0
I-94 Rothsay, MN WB (1969- control)	MN 5	JRCP	AGG	3.0	N/A	0.78	156	AGG	3.0	N/A	0.34	A-6	AC	2.0	13.0
I-90 Albert Lea, MN EB (1977)	MN 2-1	JPCP	AGG	5.0	N/A	0.93	128	NONE	---	---	---	A-2-7	PCC	6.0	8.0
	MN 2-2	JPCP	AGG	6.0	N/A	0.08	127	NONE	---	---	---	A-2-6	PCC	6.0	8.0
	MN 2-3	JRCP	AGG	5.0	N/A	0.06	162	NONE	---	---	---	A-2-6	AC	2.0	12.0
	MN 2-4	JRCP	AGG	6.0	N/A	0.40	178	NONE	---	---	---	A-2-6	AC	2.0	13.0
I-90 Austin, MN EB (1984- new)	MN 3	JRCP	AGG	4.0	N/A	N/A	256	AGG	10.0	N/A	N/A	A-4	AC*	3.0	10.0
TH 15 New Ulm, MN N/S (1986- new)	MN 4	JPCP	AGG	5.0	N/A	0.34	222	NONE	---	---	---	A-2-6	AC*	3.0	9.5
TH 15 Truman, MN N/S (1983- new)	MN 6	JRCP	PATB	4.0	N/A	N/A	199	AGG	4.0	N/A	0.42	A-2-4	AC*	4.0	12.0

* Widened outer lane

Table 123. Pavement joint data for projects in dry-freeze environmental zone.

Project Location (Year Constructed)	Project Section ID	Slab T, IN	Pvt. Type	Joint Spacing, FT	Base Type	TRANSVERSE JOINT: Dowel Dia., IN	Calc Avg Jt Open, IN	Skewed Joints Y/N	Jt Seal Shape Factor	JOINT SEALANT: Type	Age	Cond.	Depth of Long. Joint IN
I-94 Rothsay, MN WB (1970)	MN 1-1	9.0	JRCP	27	AGG	0.0	0.12	Y	0.17	PREF	3	GOOD	2.75
	MN 1-2	9.0	JRCP	27	AGG	1.0	0.12	Y	0.17	PREF	3	GOOD	2.75
	MN 1-3	8.0	JRCP	27	AGG	0.0	0.12	Y	0.17	PREF	3	GOOD	2.75
	MN 1-4	8.0	JRCP	27	AGG	1.0	0.12	Y	0.17	PREF	3	GOOD	2.75
	MN 1-5	8.0	JRCP	27	ATB	0.0	0.10	Y	0.17	PREF	3	GOOD	2.75
	MN 1-6	8.0	JRCP	27	ATB	1.0	0.10	Y	0.17	PREF	3	GOOD	2.75
	MN 1-7	9.0	JRCP	27	ATB	0.0	0.10	Y	0.17	PREF	3	GOOD	2.75
	MN 1-8	9.0	JRCP	27	ATB	1.0	0.10	Y	0.17	PREF	3	GOOD	2.75
	MN 1-9	9.0	JRCP	27	CTB	0.0	0.10	Y	0.17	PREF	3	GOOD	2.75
	MN 1-10	9.0	JRCP	27	CTB	1.0	0.10	Y	0.17	PREF	3	FAIR	2.75
	MN 1-11	8.0	JRCP	27	CTB	0.0	0.10	Y	0.17	PREF	3	FAIR	2.75
	MN 1-12	8.0	JRCP	27	CTB	1.0	0.10	Y	0.17	PREF	3	FAIR	2.75
I-94 Rothsay, MN WB (1969- control)	MN 5	9.0	JRCP	39	AGG	1.0	0.18	N	0.32	PREF	18	GOOD	2.75
I-90 Albert Lea, MN EB (1977)	MN 2-1	9.0	JPCP	13-16-14-19	AGG	1.0*	0.08	Y	0.25	HP	10	FAIR	2.75
	MN 2-2	8.0	JPCP	13-16-14-19	AGG	1.0*	0.08	Y	0.25	HP	10	FAIR	2.75
	MN 2-3	9.0	JRCP	27	AGG	1.0	0.11	Y	0.17	PREF	10	GOOD	2.75
	MN 2-4	9.0	JRCP	27	AGG	1.0	0.11	Y	0.17	PREF	10	GOOD	2.75
I-90 Austin, MN EB (1984- new)	MN 3	9.0	JRCP	27	AGG	1.0	0.12	Y	0.17	PREF	3	GOOD	2.75
TH 15 New Ulm, MN N/S (1986- new)	MN 4	7.5	JPCP	13-16-14-17	AGG	1.0	0.08	Y	0.63	HP	1	EXC	2.75
TH 15 Truman, MN N/S (1983- new)	MN 6	8.0	JRCP	27	PATB	1.0	0.09	Y	0.42	PREF	4	EXC	2.75

* Outer Lane Only

Table 124. Outer lane deflection data of projects in dry-freeze environmental zone.

Project Location (Year Constructed)	Project Section ID	Slab T, IN	Pvt. Type	Joint Spacing, FT	Base Type	DEFLECTION, mils: Mid-Slab High	DEFLECTION, mils: Mid-Slab Low	DEFLECTION, mils: Mid-Slab Ave.	DEFLECTION, mils: Loaded Corner	DEFLECTION, mils: Unloaded Corner	Adjusted Percent LTE	% LT Across Shldr	Avg. NDT Test Temp, °F	Percent Corners with Voids
I-94 Rothsay, MN WB (1970)	MN 1-1	9.0	JRCP	27	AGG	3.6	2.9	3.2	15.5	4.0	28	100	57	0
	MN 1-2	9.0	JRCP	27	AGG	3.5	3.0	3.3	12.3	5.5	48	N/A	57	0
	MN 1-3	8.0	JRCP	27	AGG	4.0	3.5	3.8	11.5	4.8	47	100	57	7
	MN 1-4	8.0	JRCP	27	AGG	3.7	3.4	3.5	12.9	6.5	56	N/A	53	0
	MN 1-5	8.0	JRCP	27	ATB	2.9	2.5	2.7	9.8	4.4	51	N/A	53	0
	MN 1-6	8.0	JRCP	27	ATB	2.9	2.3	2.6	8.5	4.2	54	N/A	44	0
	MN 1-7	9.0	JRCP	27	ATB	2.7	2.2	2.5	9.3	3.6	42	N/A	44	0
	MN 1-8	9.0	JRCP	27	ATB	2.7	2.1	2.4	7.3	5.7	87	N/A	39	0
	MN 1-9	9.0	JRCP	27	CTB	2.8	2.3	2.7	14.4	5.8	45	100	39	14
	MN 1-10	9.0	JRCP	27	CTB	2.6	2.1	2.3	11.9	6.9	65	N/A	41	0
	MN 1-11	8.0	JRCP	27	CTB	3.7	2.9	3.0	15.5	8.4	62	100	41	29
	MN 1-12	8.0	JRCP	27	CTB	3.9	2.6	3.1	11.8	8.0	75	N/A	68	0
I-94 Rothsay, MN WB (1969- control)	MN 5	9.0	JRCP	39	AGG	4.5	3.0	3.5	10.6	5.9	62	N/A	68	0
I-90 Albert Lea, MN EB (1977)	MN 2-1	9.0	JPCP	13-16-14-19	AGG	5.0	4.2	4.6	16.0	11.2	79	100	66	7
	MN 2-2	8.0	JPCP	13-16-14-19	AGG	7.2	4.7	5.5	13.7	11.1	86	96	66	7
	MN 2-3	9.0	JRCP	27	AGG	4.9	3.6	4.0	9.4	8.2	99	N/A	66	0
	MN 2-4	9.0	JRCP	27	AGG	5.5	3.2	3.6	7.8	6.0	86	N/A	62	0
I-90 Austin, MN EB (1984- new)	MN 3	9.0	JRCP	27	AGG	3.4	2.6	2.9	9.6	8.2	93	N/A	48	17
TH 15 New Ulm, MN N/S (1986- new)	MN 4	7.5	JPCP	13-16-14-17	AGG	4.5	3.9	4.2	7.9	5.9	86	N/A	51	0
TH 15 Truman, MN N/S (1983- new)	MN 6	8.0	JRCP	27	PATB	7.4	3.1	4.1	7.2	5.2	80	N/A	57	0

Sections MN 1-1, 1-3, 1-9, and 1-11 had tied and dowelled shoulders added in 1984.

Table 125. Outer shoulder and drainage information for projects in dry-freeze environmental zone.

Project Location (Year Constructed)	Project Section ID	Slab T, IN	Pvt. Type	Joint Spacing, FT	Base Type	Outer Shoulder Type- Thickness Inches: Surface	Outer Shoulder Type- Thickness Inches: Base	Overall Shoulder Condition	Shoulder Joint Sealant Condition	Permeability FT/HR: Base	Permeability FT/HR: Sub-base	AASHTO Drain. Coeff., Cd	Dowel Y/N	Sub-Drainage Type	Depth to Ditch FT	Average Trans. Slope, %	Average Longit. Grade, %
I-94 Rothsay, MN WB (1970)	MN 1-1	9.0	JRCP	27	AGG	PCC- 6*	AGG- 9	GOOD	EXC	0.62	---	1.05	N	NONE	4	-1.04	-0.52
	MN 1-2	9.0	JRCP	27	AGG	AC- 2	AGG- 13	FAIR	POOR	0.17	---	1.05	Y	NONE	4	-1.04	-1.04
	MN 1-3	8.0	JRCP	27	AGG	PCC- 6*	AGG- 8	GOOD	EXC	0.72	---	1.05	N	NONE	3	-1.04	-1.04
	MN 1-4	8.0	JRCP	27	AGG	AC- 2	AGG- 12	POOR	POOR	0.78	---	1.05	Y	NONE	6	-1.04	-1.04
	MN 1-5	8.0	JRCP	27	ATB	AC- 2	AGG- 11	POOR	POOR	0.00	---	0.05	N	NONE	7	-1.04	-1.04
	MN 1-6	8.0	JRCP	27	ATB	AC- 2	AGG- 11	POOR	GOOD	0.00	---	0.05	Y	NONE	4	-1.04	0.35
	MN 1-7	9.0	JRCP	27	ATB	AC- 2	AGG- 12	FAIR	POOR	0.00	---	0.85	N	NONE	4	2.08	0.69
	MN 1-8	9.0	JRCP	27	ATB	AC- 2	AGG- 12	FAIR	FAIR	0.00	---	0.85	Y	NONE	6	-1.04	0.00
	MN 1-9	9.0	JRCP	27	CTB	PCC- 6*	AGG- 0	GOOD	GOOD	0.00	---	0.00	N	NONE	3	-1.04	-1.04
	MN 1-10	9.0	JRCP	27	CTB	AC- 2	AGG- 13	FAIR	FAIR	0.00	---	0.80	Y	NONE	4	-1.04	0.35
	MN 1-11	8.0	JRCP	27	CTB	PCC- 6*	AGG- 7	EXC	GOOD	0.00	---	0.80	N	NONE	4	-5.21	0.00
	MN 1-12	8.0	JRCP	27	CTB	AC- 2	AGG- 11	POOR	POOR	0.00	---	0.80	Y	NONE	4	-5.21	0.00
I-94 Rothsay, MN WB (1969- control)	MN 5	9.0	JRCP	39	AGG	AC- 2	AGG- 13	FAIR	POOR	0.78	0.34	0.95	Y	NONE	10	-0.87	1.39
I-90 Albert Lea, MN EB (1977)	MN 2-1	9.0	JPCP	13-16-14-19	AGG	PCC- 6	AGG- 8	EXC	EXC	0.93	---	0.05	Y	NONE	7	-1.04	0.00
	MN 2-2	8.0	JPCP	13-16-14-19	AGG	PCC- 6	AGG- 0	EXC	EXC	0.08	---	0.75	Y	NONE	6	-1.04	0.00
	MN 2-3	9.0	JRCP	27	AGG	AC- 2	AGG- 12	EXC	POOR	0.06	---	0.00	Y	NONE	12	-1.04	0.00
	MN 2-4	9.0	JRCP	27	AGG	AC- 2	AGG- 13	POOR	POOR	0.41	---	0.95	Y	NONE	6	-1.04	0.00
I-90 Austin, MN EB (1984- new)	MN 3	9.0	JRCP	27	AGG	AC-3**	AGG- 10	GOOD	POOR	N/A	N/A	0.80	Y	NONE	6	-1.04	-0.35
TH 15 New Ulm, MN N/S (1986- new)	MN 4	7.5	JPCP	13-16-14-17	AGG	AC-3**	AGG-9.5	GOOD	POOR	0.34	---	0.90	Y	EDGE DRN	10	-1.56	0.00
TH 15 Truman, MN N/S (1983- new)	MN 6	8.0	JRCP	27	PATB	AC-4**	AGG- 12	FAIR	POOR	N/A	0.42	1.05	Y	DRN LAYR	10	-1.73	0.00

*Tied and dowelled PCC shoulders added, outer lane diamond ground in 1984.

**Has widened concrete outer lane.

Table 126. Traffic information for projects in dry-freeze environmental zone.

Project Location (Year Constructed)	Project Section ID	ORIGINAL DESIGN TRAFFIC: ESAL's, (million)	ORIGINAL DESIGN TRAFFIC: ADT*, thous.	ORIGINAL DESIGN TRAFFIC: % Trucks	Age at Survey	1987 ESTIMATED: ADT, thous.	1987 ESTIMATED: % Trucks	OUTER LANE (1): 1987 ESAL from ADT, % Trucks	OUTER LANE (1): Estimated ESAL's to Date	LANE # 2: 1987 ESAL from ADT, % Trucks	LANE # 2: Estimated ESAL's to Date
I-94 Rothsay, MN	MN 1-1		2.0		17	5.0	21.0	301700	5518900	26000	550900
WB (1970)	MN 1-2		2.0		17	5.0	21.0	301700	5518900	26000	550900
	MN 1-3		2.0		17	5.0	21.0	301700	5518900	26000	550900
	MN 1-4		2.0		17	5.0	21.0	301700	5518900	26000	550900
	MN 1-5		2.0		17	5.0	21.0	301700	5518900	26000	550900
	MN 1-6		2.0		17	5.0	21.0	301700	5518900	26000	550900
	MN 1-7		2.0		17	5.0	21.0	301700	5518900	26000	550900
	MN 1-8		2.0		17	5.0	21.0	301700	5518900	26000	550900
	MN 1-9		2.0		17	5.0	21.0	301700	5518900	26000	550900
	MN 1-10		2.0		17	5.0	21.0	301700	5518900	26000	550900
	MN 1-11		2.0		17	5.0	21.0	301700	5518900	26000	550900
	MN 1-12				17	5.0	21.0	301700	5518900	26000	550900
I-94 Rothsay, MN WB (1969- control)	MN 5				18	5.0	21.0	301700	5518900	26000	550900
I-90 Albert Lea, MN	MN 2-1				10	3.9	20.0	222400	2785500	13881	215100
EB (1977)	MN 2-2				10	3.9	20.0	222400	2785500	13881	215100
	MN 2-3				10	3.9	20.0	222400	2785500	13881	215100
	MN 2-4				10	3.9	20.0	222400	2785500	13881	215100
I-90 Austin, MN EB (1984- new)	MN 3				3	10.6	15.0	368800	1466000	60700	243700
TH 15 New Ulm, MN N/S (1986- new)	MN 4				1	2.9	13.5	111000	218800	N/A	N/A
TH 15 Truman, MN N/S (1983- new)	MN 6				4	3.9	17.0	188800	845000	N/A	N/A

* All ADT's are two-way

Table 127. Outer lane performance data for projects in dry-freeze environmental zone.

Project Location (Year Constructed)	Project Section ID	Slab T, IN	Pvt. Type	Joint Spacing, FT	Base Type	Dowel Dia., IN	Ave. PSR*	Mays Rough. IN/MI	1984 Pre-Grind Fault, IN.	Avg Trans. Fault, IN.	Deter. Trans. Crack/Mile	Longit. Cracking, LIN FT/MI	Pumping N/L/M/H	Percent Joints Spalled	Mtls Dur. Dist
I-94 Rothsay, MN WB (1970)	MN 1-1	9.0	JRCP	27	AGG	0.0	3.7	53	0.31	0.07	8	0	L	4	DCRK
	MN 1-2	9.0	JRCP	27	AGG	1.0	3.3	104	0.06	0.10	23	0	N	14	DCRK
	MN 1-3	8.0	JRCP	27	AGG	0.0	3.4	44	0.31	0.12	33	301	L	8	DCRK
	MN 1-4	8.0	JRCP	27	AGG	1.0	3.3	110	0.06	0.08	47	0	L	15	DCRK
	MN 1-5	8.0	JRCP	27	ATB	0.0	3.4	46	0.37	0.11	41	1776	L	24	DCRK
	MN 1-6	8.0	JRCP	27	ATB	1.0	3.4	107	0.00	0.08	0	2073	N	31	DCRK
	MN 1-7	9.0	JRCP	27	ATB	0.0	3.6	32	0.31	0.05	0	1760	M	15	DCRK
	MN 1-8	9.0	JRCP	27	ATB	1.0	3.4	93	0.06	0.09	102	2509	N	69	DCRK
	MN 1-9	9.0	JRCP	27	CTB	0.0	3.7	50	0.37	0.08	0	0	L	4	DCRK
	MN 1-10	9.0	JRCP	27	CTB	1.0	3.3	108	0.13	0.00	39	0	L	6	DCRK
	MN 1-11	8.0	JRCP	27	CTB	0.0	3.7	50	0.50	0.11	0	1306	L	37	DCRK
	MN 1-12	8.0	JRCP	27	CTB	1.0	3.5	110	0.06	0.10	40	177	M	29	DCRK
I-94 Rothsay, MN WB (1969- control)	MN 5	9.0	JRCP	39	AGG	1.0	3.3	100	---	0.09	53	1261	N	36	DCRK
I-90 Albert Lea, MN EB (1977)	MN 2-1	9.0	JPCP	13-16-14-19	AGG	1.0	3.8	72	---	0.06	0	0	N	3	NONE
	MN 2-2	8.0	JPCP	13-16-14-19	AGG	1.0	3.9	82	---	0.06	0	150	N	9	NONE
	MN 2-3	9.0	JRCP	27	AGG	1.0	4.0	76	---	0.05	0	0	N	3	NONE
	MN 2-4	9.0	JRCP	27	AGG	1.0	4.0	96	---	0.06	5	0	N	0	NONE
I-90 Austin, MN EB (1984- new)	MN 3	9.0	JRCP	27	AGG	1.0	3.8	44	---	0.02	0	0	N	0	NONE
TH 15 New Ulm, MN N/S (1986- new)	MN 4	7.5	JPCP	13-16-14-17	AGG	1.0	4.8	42	---	0.01	0	0	N	0	NONE
TH 15 Truman, MN N/S (1983- new)	MN 6	8.0	JRCP	27	PATB	1.0	4.5	51	---	0.01	0	0	N	0	NONE

* Project sections 1-1, 1-3, 1-5, 1-7, 1-9, and 1-11 were diamond ground in the outer lane in 1984; tied and dowelled PCC shoulders were added at same time.

Table 128. Lane 2 performance data for projects in dry-freeze environmental zone.

Project Location (Year Constructed)	Project Section ID	Slab T, IN	Pvt. Type	Joint Spacing, FT	Base Type	Dowel Dia., IN	PSR	Mays Rough. IN/MI	Average Trans. Faulting IN	Deter. Trans. Cracks/Mile	Longit. Cracking, LIN FT/MI	Pumping N/L/M/H	Percent Joints Spalled	Mtls Dur. Dist
I-94 Rothsay, MN WB (1970)	MN 1-1	9.0	JRCP	27	AGG	0.0	3.4	124	0.03	0	0	N	8	DCRK
	MN 1-2	9.0	JRCP	27	AGG	1.0	3.5	86	0.03	0	0	N	4	DCRK
	MN 1-3	8.0	JRCP	27	AGG	0.0	3.3	108	0.06	0	0	N	24	DCRK
	MN 1-4	8.0	JRCP	27	AGG	1.0	3.6	100	0.02	47	0	N	8	DCRK
	MN 1-5	8.0	JRCP	27	ATB	0.0	3.4	98	0.07	16	880	N	16	DCRK
	MN 1-6	8.0	JRCP	27	ATB	1.0	3.3	92	0.03	0	1299	N	23	DCRK
	MN 1-7	9.0	JRCP	27	ATB	0.0	3.5	95	0.04	0	1190	N	15	DCRK
	MN 1-8	9.0	JRCP	27	ATB	1.0	3.5	80	0.04	55	2644	N	81	DCRK
	MN 1-9	9.0	JRCP	27	CTB	0.0	3.4	118	0.03	0	0	N	15	DCRK
	MN 1-10	9.0	JRCP	27	CTO	1.0	3.4	08	0.03	35	0	N	10	DCRK
	MN 1-11	8.0	JRCP	27	CTB	0.0	3.2	50	0.04	0	2874	N	74	DCRK
	MN 1-12	8.0	JRCP	27	CTB	1.0	3.4	104	0.02	40	843	N	50	DCRK
I-94 Rothsay, MN WB (1969- control)	MN 5	9.0	JRCP	39	AGG	1.0	3.7	94	0.04	33	0	N	23	DCRK
I-90 Albert Lea, MN EB (1977)	MN 2-1	9.0	JPCP	13-16-14-19	AGG	0.0	3.0	02	0.03	0	746	N	1	NONE
	MN 2-2	8.0	JPCP	13-16-14-19	AGG	0.0	3.9	72	0.03	0	0	N	0	NONE
	MN 2-3	9.0	JRCP	27	AGG	1.0	4.0	62	0.03	0	0	N	3	NONE
	MN 2-4	9.0	JRCP	27	AGG	1.0	4.0	74	0.02	0	0	N	5	NONE
I-90 Austin, MN EB (1984- new)	MN 3	9.0	JRCP	27	AGG	1.0	4.0	51	0.01	0	0	N	0	NONE
TH 15 New Ulm, MN N/S (1986- new)	MN 4	7.5	JPCP	13-16-14-17	AGG	1.0	4.7	37	0.01	0	0	N	0	NONE
TH 15 Truman, MN N/S (1983- new)	MN 6	8.0	JRCP	27	PATB	1.0	4.4	51	0.01	0	0	N	0	NONE

Table 129. Slab design for projects in dry-nonfreeze environmental zone.

Project Location (Year Constructed)	Project Section ID	Pvt. Type	PCC SURFACE THICKNESS, IN Design	Core	Joint Spacing, FT	% Steel JRCP	Skewed Joints Y/N	LTD's Dia., IN	Coating	E, KSI from FWD	Mr, PSI (from Cores)
RT 360 Phoenix, AZ ('72)	AZ 1-1	JPCP	9.0	N/A	13-15-17-15	---	Y	0.00	---	3140	687
WB&EB ('75)	AZ 1-2	JPCP	13.0	N/A	13-15-17-15	---	Y	0.00	---	3440	649
('79)	AZ 1-4	JPCP	13.0	13.0	13-15-17-15	---	Y	0.00	---	3490	702
('79)	AZ 1-5	JPCP	11.0	11.0	13-15-17-15	---	Y	0.00	---	3290	761
('81)	AZ 1-6	JPCP	9.0	N/A	13-15-17-15	---	Y	0.00	---	3090	853
('81)	AZ 1-7	JPCP	9.0	N/A	13-15-17-15	---	Y	0.00	---	3690	868
I-10 Phoenix, AZ EB (1983- new)	AZ 2	JPCP	10.0	N/A	13-15-17-15	---	Y	1.25	EPOXY	5560	725
I-5 Tracy, CA	CA 1-1	JPCP	8.4	8.5	8-11-7-5	---	Y	0.00	---	6610	N/A
NB (1971)	CA 1-3	JPCP	8.4	8.8	12-13-19-18	---	Y	0.00	---	5240	723
	CA 1-5	JPCP	11.4	11.9	12-13-19-18	---	Y	0.00	---	5280	918
	CA 1-7	JPCP	8.4	8.5	12-13-19-18	---	Y	0.00	---	6480	781
	CA 1-9	JPCP	8.4	8.5	12-13-19-18	---	Y	0.00	---	6950*	802
I-5 Sacramento, CA NB (1979- control)	CA 7	JPCP	10.2	10.2	12-13-19-18	---	Y	0.00	---	6260	552
I-210 Los Angeles, CA	CA 2-2	JPCP	8.4	8.5	12-13-19-18	---	Y	0.00	---	7040	730
EB (1980)	CA 2-3	JPCP	8.4	10.0	12-13-19-18	---	Y	0.00	---	4980	644
US 101 1000 Oaks, CA NB (1983- new)	CA 8	JPCP	10.2	10.8	12-13-15-14	---	Y	0.00	---	6420	727
RT 14 Solemint, CA SB (1980- new)	CA 6	JPCP	9.0	9.0	12-13-15-14	---	Y	0.00	---	6710	791

*High-strength (7.5 sack) mix.

Table 130. Base, subbase, subgrade, and outer shoulder design data for projects in dry-nonfreeze environmental zone.

Project Location (Year Constructed)	Project Section ID	Pvt. Type	BASE Type	BASE THICKNESS, IN Design	BASE THICKNESS, IN Core	BASE Estimated Perm., FT/HR	Keff, (Dyn.) PCI	SUBBASE Type	SUBBASE THICKNESS, IN Design	SUBBASE THICKNESS, IN Core	SUBBASE Estimated Perm., FT/HR	SUBGRADE AASHTO Soil Type	SUBGRADE Estimated Perm., FT/HR	OUTER SHOULDER Type	OUTER SHOULDER THICKNESS, IN Surface	OUTER SHOULDER THICKNESS, IN Base
RT 360 Phoenix, AZ ('72) WB&EB	AZ 1-1	JPCP	CTB	6.0	N/A	0.00	546	AGG	4.0	N/A	3.78	A-4	0.11	AC	3.0	6.0
('75)	AZ 1-2	JPCP	NONE	---	---	---	492	NONE	---	---	---	A-6	0.11	PCC	9.5*	0.0
('79)	AZ 1-4	JPCP	NONE	---	---	---	344	NONE	---	---	---	A-6	0.11	PCC	13.0	0.0
('79)	AZ 1-5	JPCP	NONE	---	---	---	439	NONE	---	---	---	A-6	0.11	PCC	11.0	0.0
('81)	AZ 1-6	JPCP	LCB	4.0	N/A	0.00	621	NONE	---	---	---	A-6	0.04	PCC	9.0	4.0
('81)	AZ 1-7	JPCP	LCB	4.0	N/A	0.00	584	NONE	---	---	---	A-6	0.04	PCC	9.0	4.0
I-10 Phoenix, AZ EB (1983- new)	AZ 2	JPCP	LCB	5.0	N/A	0.00	174	NONE	---	---	---	A-6	0.03	PCC	10.0	5.0
I-5 Tracy, CA NB (1971)	CA 1-1	JPCP	CTB	5.4	N/A	0.00	232	AGG	24.0	N/A	26.34	A-1-a		AC	3.0	6.0
	CA 1-3	JPCP	CTB	5.4	5.5	0.00	349	AGG	24.0	9.7	0.75	A-2-4		AC	3.0	6.0
	CA 1-5	JPCP	CTB	5.4	4.8	0.00	335	AGG	24.0	N/A	2.32	A-1-a		AC	3.0	6.0
	CA 1-7	JPCP	LCB	5.4	6.0	0.00	433	AGG	24.0	N/A	1.65	A-1-a		AC	3.0	6.0
	CA 1-9	JPCP	CTB	5.4	4.6	0.00	298	AGG	24.0	N/A	2.72	A-1-a		AC	3.0	6.0
I-5 Sacramento, CA NB (1979- control)	CA 7	JPCP	CTB	5.4	5.5	0.00	326	LTSG	5.4	N/A	0.00	A-2-4		AC	3.6	12.0
I-210 Los Angeles, CA EB (1980)	CA 2-2	JPCP	PCTB	5.4	N/A		1423	AGG	3.0	N/A	0.00	A-4	0.11	AC	3.0	6.0
	CA 2-3	JPCP	CTB	5.4	N/A	0.00	572	AGG	6.0	4.6	0.00	A-4	0.11	AC	3.0	6.0
US 101 1000 Oaks, CA NB (1983- new)	CA 8	JPCP	HMAC	5.4	5.5	0.00	339	AGG	9.0	N/A	2.04	A-7	0.01	AC	4.2	7.2
RT 14 Solemint, CA SB (1980- new)	CA 6	JPCP	LCB	4.2	5.0	0.78	294	HMAC	1.8	N/A	0.00	A-2-4		AC	3.6	5.4

* Average

Table 131. Pavement joint data for projects in dry-nonfreeze environmental zone.

Project Location (Year Constructed)		Project Section ID	Slab T, IN	Pvt. Type	Joint Spacing, FT	Base Type	TRANSVERSE JOINT: Dowel Dia., IN	Calc Avg Jt Open, IN	Skewed Joints Y/N	Jt Seal Shape Factor	JOINT SEALANT: Type	Age	Cond.	Depth of Long. Joint, IN
RT 360 Phoenix, AZ	('72)	AZ 1-1	9.0	JPCP	13-15-17-15	CTB	0.00	0.05	Y	0.67	RA	1	GOOD	2.25
WB&EB	('75)	AZ 1-2	13.0	JPCP	13-15-17-15	NONE	0.00	0.06	Y	0.67	RA	1	EXC	3.25
	('79)	AZ 1-4	13.0	JPCP	13-15-17-15	NONE	0.00	0.06	Y	0.67	RA	1	GOOD	3.25
	('79)	AZ 1-5	11.0	JPCP	13-15-17-15	NONE	0.00	0.06	Y	0.67	RA	1	FAIR	2.75
	('81)	AZ 1-6	9.0	JPCP	13-15-17-15	LCB	0.00	0.05	Y	0.67	RA	1	GOOD	2.25
	('81)	AZ 1-7	9.0	JPCP	13-15-17-15	LCB	0.00	0.05	Y	0.67	RA	1	FAIR	2.25
I-10 Phoenix, AZ EB (1983- new)		AZ 2	10.0	JPCP	13-15-17-15	LCB	1.25	0.05	Y	0.67	RA	4	GOOD	2.5
I-5 Tracy, CA NB (1971)		CA 1-1	8.4	JPCP	8-11-7-5	CTB	0.00	0.03	Y	0.0	NONE	---	---	2.0
		CA 1-3	8.4	JPCP	12-13-19-18	CTB	0.00	0.05	Y	0.0	NONE	---	---	2.0
		CA 1-5	11.4	JPCP	12-13-19-18	CTB	0.00	0.05	Y	0.0	NONE	---	---	3.0
		CA 1-7	8.4	JPCP	12-13-19-18	LCB	0.00	0.05	Y	0.0	NONE	---	---	2.0
		CA 1-9	8.4	JPCP	12-13-19-18	CTB	0.00	0.05	Y	0.0	NONE	---	---	2.0
I-5 Sacramento, CA NB (1979- control)		CA 7	10.2	JPCP	12-13-19-18	CTB	0.00	0.04	Y	0.0	NONE	---	---	3.0
I-210 Los Angeles, CA EB (1980)		CA 2-2	8.4	JPCP	12-13-19-18	PCTB	0.00	0.04	Y	0.0	NONE	---	---	2.0
		CA 2-3	8.4	JPCP	12-13-19-18	CTB	0.00	0.04	Y	0.0	NONE	---	---	2.0
US 101 1000 Oaks, CA NB (1983- new)		CA 8	10.2	JPCP	12-13-15-14	HMAC	0.00	0.02	Y	0.0	NONE	---	---	3.0
RT 14 Solemint, CA SB (1980- new)		CA 6	9.0	JPCP	12-13-15-14	LCB	0.00	0.04	Y	0.0	NONE	---	---	2.0

Table 132. Outer lane deflection data of projects in dry-nonfreeze environmental zone.

Project Location (Year Constructed)		Project Section ID	Slab T, IN	Pvt. Type	Joint Spacing, FT	Base Type	DEFLECTION, mils Mid-Slab High	Low	Ave.	Loaded Corner	Unloaded Corner	Adjusted Percent LTE	% LT Across Shldr	Avg. NDT Test Temp, °F	Percent Corners with Voids
RT 360 Phoenix, AZ	('72)	AZ 1-1	9.0	JPCP	13-15-17-15	CTB	3.9	2.5	3.3	8.6	6.3	94	N/A	73	30
WB&EB	('75)	AZ 1-2	13.0	JPCP	13-15-17-15	NONE	3.2	2.0	2.7	4.6	4.0	100	86	69	0
	('79)	AZ 1-4	13.0	JPCP	13-15-17-15	NONE	3.0	1.8	2.3	6.3	5.5	100	79	74	0
	('79)	AZ 1-5	11.0	JPCP	13-15-17-15	NONE	3.6	2.5	3.2	12.5	9.9	100	72	70	36
	('81)	AZ 1-6	9.0	JPCP	13-15-17-15	LCB	2.4	1.0	2.1	5.0	4.4	100	100	68	0
	('81)	AZ 1-7	9.0	JPCP	13-15-17-15	LCB	3.2	2.5	2.8	6.9	5.1	94	95	74	24
I-10 Phoenix, AZ EB (1983- new)		AZ 2	10.0	JPCP	13-15-17-15	LCB	3.3	2.1	2.6	11.1	6.1	72	100	80	31
I-5 Tracy, CA NB (1971)		CA 1-1	8.4	JPCP	8-11-7-5	CTB	4.6	3.4	3.9	15.9	12.1	85	N/A	68	60
		CA 1-3	8.4	JPCP	12-13-19-18	CTB	3.5	2.8	3.2	13.7	10.4	87	N/A	63	50
		CA 1-5	11.4	JPCP	12-13-19-18	CTB	2.6	1.6	2.2	13.2	10.5	89	N/A	63	60
		CA 1-7	8.4	JPCP	12-13-19-18	LCB	3.4	2.1	2.7	8.2	6.1	88	N/A	64	10
		CA 1-9	8.4	JPCP	12-13-19-18	CTB	3.9	2.8	3.5	15.0	11.5	86	N/A	70	55
I-5 Sacramento, CA NB (1979- control)		CA 7	10.2	JPCP	12-13-19-18	CTB	4.9	1.9	2.4	9.8	3.2	35	N/A	69	23
I-210 Los Angeles, CA EB (1980)		CA 2-2	8.4	JPCP	12-13-19-18	PCTB	1.9	1.0	1.4	14.7	1.2	10	N/A	67	65
		CA 2-3	8.4	JPCP	12-13-19-18	CTB	5.2	2.0	3.1	24.4	4.1	19	N/A	69	90
US 101 1000 Oaks, CA NB (1983- new)		CA 8	10.2	JPCP	12-13-15-14	HMAC	2.7	2.1	2.4	3.2	2.6	92	N/A	73	0
RT 14 Solemint, CA SB (1980- new)		CA 6	9.0	JPCP	12-13-15-14	LCB	4.4	2.5	3.1	5.9	4.2	79	N/A	72	0

Table 133. Outer shoulder and drainage information for projects in dry-nonfreeze environmental zone.

Project Location (Year Constructed)	Project Section ID	Slab T, IN	Pvt. Type	Joint Spacing, FT	Base Type	Outer Shoulder Type- Thickness Inches: Surface	Outer Shoulder Type- Thickness Inches: Base	Overall Shoulder Condition	Shoulder Joint Sealant Condition	Permeability FT/HR: Base	Permeability FT/HR: Sub-base	AASHTO Drain. Coeff. Cd	Dowel Y/N	Sub-Drainage Type	Depth to Ditch FT	Average Trans. Slope, %	Average Longit. Grade, %
RT 360 Phoenix, AZ ('72)	AZ 1-1	9.0	JPCP	13-15-17-15	CTB	AC-3	AGG-6	FAIR	GOOD	0.00	N/A	1.00	N	NONE	0	-0.52	0.78
WB&EB ('75)	AZ 1-2	13.0	JPCP	13-15-17-15	NONE	PCC-9*	NONE	EXC	FAIR	0.00	---	1.10	N	NONE	0	-1.04	0.87
('79)	AZ 1-4	13.0	JPCP	13-15-17-15	NONE	PCC-13	NONE	EXC	FAIR	0.00	---	1.10	N	NONE	0	-2.08	0.00
('79)	AZ 1-5	11.0	JPCP	13-15-17-15	NONE	PCC-11	NONE	EXC	POOR	0.00	---	1.10	N	NONE	0	-2.08	1.39
('81)	AZ 1-6	9.0	JPCP	13-15-17-15	LCB	PCC-9	LCB-4	EXC	FAIR	0.00	---	1.10	N	NONE	0	-2.08	0.00
('81)	AZ 1-7	9.0	JPCP	13-15-17-15	LCB	PCC-9	LCB-4	EXC	POOR	0.00	---	1.15	N	NONE	0	-2.08	0.00
I-10 Phoenix, AZ EB (1983- new)	AZ 2	10.0	JPCP	13-15-17-15	LCB	PCC-10	LCB-5	EXC	GOOD	0.00	---	1.05	Y	NONE	2	-3.39	1.04
I-5 Tracy, CA NB (1971)	CA 1-1	8.4	JPCP	8-11-7-5	CTB	AC-3	AGG-6	GOOD	POOR	0.00	26.34	1.10	N	NONE	5	-1.21	0.00
	CA 1-3	8.4	JPCP	12-13-19-18	CTB	AC-3	AGG-6	GOOD	POOR	0.00	0.75	1.00	N	NONE	6	-1.39	0.00
	CA 1-5	11.4	JPCP	12-13-19-18	CTB	AC-3	AGG-6	GOOD	POOR	0.00	2.32	1.10	N	NONE	7	-1.39	0.00
	CA 1-7	8.4	JPCP	12-13-19-18	LCB	AC-3	AGG-6	EXC	POOR	0.00	1.65	1.15	N	NONE	10	-1.21	1.04
	CA 1-9	8.4	JPCP	12-13-19-18	CTB	AC-3	AGG-6	EXC	POOR	0.00	2.72	1.10	N	NONE	5	-1.04	0.00
I-5 Sacramento, CA NB (1979- control)	CA 7	10.2	JPCP	12-13-19-18	CTB	AC-4	PATB-12	EXC	EXC	0.00	0.00	0.85	N	LONG DRN	5	-2.08	0.00
I-210 Los Angeles, CA EB (1980)	CA 2-2	8.4	JPCP	12-13-19-18	PCTB	AC-3	AGG-6	EXC	POOR	0.00	0.00	1.10	N	DB/LD	N/A	-1.73	2.78
	CA 2-3	8.4	JPCP	12-13-19-18	CTB	AC-3	AGG-6	EXC	POOR	0.00	0.00	0.90	N	DB	40	-3.12	1.04
US 101 1000 Oaks, CA NB (1983- new)	CA 8	10.2	JPCP	12-13-15-14	HMAC	AC-4	AGG-7	EXC	POOR	0.00	2.04	0.95	N	DB/LD	15	-0.52	1.73
RT 14 Solemint, CA SB (1980- new)	CA 6	9.0	JPCP	12-13-15-14	LCB	AC-4	AGG-5	GOOD	POOR	0.78	0.00	0.60	N	LONG DRN	2	-1.73	2.26

* Average

Table 134. Traffic information for projects in dry- nonfreeze environmental zone.

Project Location (Year Constructed)	Project Section ID	ORIGINAL DESIGN TRAFFIC: ESAL's, (million)	ORIGINAL DESIGN TRAFFIC: ADT*, thous.	ORIGINAL DESIGN TRAFFIC: % Trucks	Age at Survey	1987 ESTIMATED: ADT, thous.	1987 ESTIMATED: % Trucks	OUTER LANE (1): 1987 ESAL from ADT, % Trucks	OUTER LANE (1): Estimated ESAL's to Date	LANE # 2: 1987 ESAL from ADT, % Trucks	LANE # 2: Estimated ESAL's to Date
RT 360 Phoenix, AZ ('72)	AZ 1-1		74.0		15	110.4	3.1	517300	3068600	362400	1981200
WB&EB ('75)	AZ 1-2		74.0		12	118.6	3.1	549700	3414900	395000	1849400
('79)	AZ 1-4				8	93.7	3.1	450100	2387100	297300	1189100
('79)	AZ 1-5				8	106.4	3.1	501600	2801200	346900	1477400
('82)	AZ 1-6				5	97.8	3.1	571400	2012200	274900	809000
('82)	AZ 1-7				5	75.4	3.1	454500	1523000	197900	540000
I-10 Phoenix, AZ EB (1983- new)	AZ 2		40.5		4	50.0	9.0	924600	1622900	484700	810700
I-5 Tracy, CA	CA 1-1				16	13.0	19.0	773000	7621800	145300	1146700
NB (1971)	CA 1-3				16	13.0	19.0	773000	7621800	145300	1146700
	CA 1-5				16	13.0	19.0	773000	7621800	145300	1146700
	CA 1-7				16	13.0	19.0	773000	7621800	145300	1146700
	CA 1-9				16	13.0	19.0	773000	7621800	145300	1146700
I-5 Sacramento, CA NB (1980- control)	CA 7				7	22.0	22.0	1436500	1052500	362800	2364700
I-210 Los Angeles, CA	CA 2-2				7	44.0	9.0	527200	4423800	263200	2120500
EB (1980)	CA 2-3				7	44.0	9.0	527200	4423800	263200	2120500
US 101 Los Angeles, CA NB (1980- new)	CA 8				7	135.0	7.0	1234900	5263400	929100	3864600
RT 14 Los Angeles, CA SB (1980- new)	CA 6				7	46.0	9.0	632700	4434300	321400	2227500

* All ADT's are two-way

Table 135. Outer lane performance data for projects in dry-nonfreeze environmental zone.

Project Location (Year Constructed)		Project Section ID	Slab T, IN	Pvt. Type	Joint Spacing, FT	Base Type	Dowel Dia., IN	Ave. PSR	Mays Rough. IN/MI	Average Trans. Faulting IN	Deter. Trans. Cracks/Mile	Longit. Cracking, LIN FT/MI	Pumping N/L/M/H	Percent Joints Spalled	Mtls Dur. Dist
RT, 360 Phoenix, AZ	('72)	AZ 1-1	9.0	JPCP	13-15-17-15	CTB	0.00	3.4	114	0.08	0	149	N	22	NONE
WB&EB	('75)	AZ 1-2	13.0	JPCP	13-15-17-15	NONE	0.00	3.8	65	0.01	0	0	N	1	NONE
	('79)	AZ 1-4	13.0	JPCP	13-15-17-15	NONE	0.00	3.6	102	0.01	0	0	N	0	NONE
	('79)	AZ 1-5	11.0	JPCP	13-15-17-15	NONE	0.00	3.8	85	0.03	0	0	N	0	NONE
	('81)	AZ 1-6	9.0	JPCP	13-15-17-15	LCB	0.00	3.5	97	0.01	0	0	N	1	NONE
	('81)	AZ 1-7	9.0	JPCP	13-15-17-15	LCB	0.00	3.8	91	0.02	0	0	N	0	NONE
I-10 Phoenix, AZ EB (1983- new)		AZ 2	10.0	JPCP	13-15-17-15	LCB	1.25	3.6	71	0.01	0	0	N	0	NONE
I-5 Tracy, CA		CA 1-1	8.4	JPCP	8-11-7-5	CTB	0.00	2.9	102	0.06	5	0	M	2	RAGG
NB (1971)		CA 1-3	8.4	JPCP	12-13-19-18	CTB	0.00	3.0	94	0.10	30	500	N	3	RAGG
		CA 1-5	11.4	JPCP	12-13-19-18	CTB	0.00	2.7	122	0.11	0	0	L	3	RAGG
		CA 1-7	8.4	JPCP	12-13-19-18	LCB	0.00	2.7	102	0.06	75	230	H	9	RAGG
		CA 1-9	8.4	JPCP	12-13-19-18	CTB	0.00	2.5	116	0.13	190	449	L	3	RAGG
I-5 Sacramento, CA NB (1979- control)		CA 7	10.2	JPCP	12-13-19-18	CTB	0.00	3.8	87	0.06	119	598	N	0	NONE
I-210 Los Angeles, CA		CA 2-2	8.4	JPCP	12-13-19-18	PCTB	0.00	3.8	107	0.11	0	0	N	0	RAGG
EB (1980)		CA 2-3	8.4	JPCP	12-13-19-18	CTB	0.00	4.1	98	0.11	290	0	N	0	RAGG
US 101 1000 Oaks, CA NB (1983- new)		CA 8	10.2	JPCP	12-13-15-14	HMAC	0.00	3.8	92	0.04	0	0	N	0	NONE
RT 14 Solemint, CA SB (1980- new)		CA 6	9.0	JPCP	12-13-15-14	LCB	0.00	3.4	158	0.15	10	0	N	3	RAGG

Table 136. Lane 2 performance data for projects in dry-nonfreeze environmental zone.

Project Location (Year Constructed)		Project Section ID	Slab T, IN	Pvt. Type	Joint Spacing, FT	Base Type	Dowel Dia., IN	PSR	Mays Rough. IN/MI	Average Trans. Faulting IN	Deter. Trans. Cracks/Mile	Longit. Cracking, LIN FT/MI	Pumping N/L/M/H	Percent Joints Spalled	Mtls Dur. Dist
RT 360 Phoenix, AZ	('72)	AZ 1-1	9.0	JPCP	13-15-17-15	CTB	0.00	3.4	102	N/A	0	84	N	26	NONE
WB&EB	('75)	AZ 1-2	13.0	JPCP	13-15-17-15	NONE	0.00	3.8	76	N/A	0	0	N	6	NONE
	('79)	AZ 1-4	13.0	JPCP	13-15-17-15	NONE	0.00	3.4	100	N/A	0	0	N	0	NONE
	('79)	AZ 1-5	11.0	JPCP	13-15-17-15	NONE	0.00	3.8	03	N/A	0	0	N	0	NONE
	('81)	AZ 1-6	9.0	JPCP	13-15-17-15	LCB	0.00	3.8	03	N/A	0	0	N	0	NONE
	('81)	AZ 1-7	9.0	JPCP	13-15-17-15	LCB	0.00	3.0	74	N/A	0	0	N	0	NONE
I-10 Phoenix, AZ EB (1983- new)		AZ 2	10.0	JPCP	13-15-17-15	LCB	1.25	N/A	N/A	N/A	0	0	N	0	NONE
I-5 Tracy, CA		CA 1-1	8.4	JPCP	8-11-7-5	CTB	0.00	3.8	88	N/A	0	0	L	1	RAGG
NB (1971)		CA 1-3	8.4	JPCP	12-13-19-18	CTB	0.00	4.0	86	N/A	0	440	N	0	RAGG
		CA 1-5	11.4	JPCP	12-13-19-18	CTB	0.00	3.3	103	N/A	0	0	N	0	RAGG
		CA 1-7	8.4	JPCP	12-13-19-18	LCB	0.00	3.4	115	N/A	0	0	M	3	RAGG
		CA 1-9	8.4	JPCP	12-13-19-18	CTB	0.00	3.2	120	N/A	0	70	M	1	RAGG
I-5 Sacramento, CA NB (1979- control)		CA 7	10.2	JPCP	12-13-19-18	CTB	0.00	3.9	96	N/A	74	1462	N	0	NONE
I-210 Los Angeles, CA		CA 2-2	8.4	JPCP	12-13-19-18	PCTB	0.00	N/A	N/A	N/A	0	0	N	0	RAGG
EB (1980)		CA 2-3	8.4	JPCP	12-13-19-18	CTB	0.00	N/A	N/A	N/A	162	0	N	0	RAGG
US 101 1000 Oaks, CA NB (1983- new)		CA 8	10.2	JPCP	12-13-15-14	HMAC	0.00	N/A	N/A	N/A	0	0	N	0	NONE
RT 14 Solemint, CA SB (1980- new)		CA 6	9.0	JPCP	12-13-15-14	LCB	0.00	N/A	N/A	N/A	0	0	N	13	RAGG

Table 137. Slab design data for projects in wet-freeze environmental zone.

Project Location (Year Constructed)	Project Section ID	Pvt. Type	PCC SURFACE: THICKNESS, IN: Design	PCC SURFACE: THICKNESS, IN: Core	Joint Spacing, FT	% Steel JRCP	Skewed Joints, Y/N	LTD's: Dia., IN	LTD's: Coating	E, KSI from FWD	Mr, PSI (from Cores)
US 10 Clare, MI	MI 1-1a	JRCP	9.0	9.4	71.2	0.15	N	1.25	LA/EPOXY	5450	745
WB&EB (1975)	MI 1-1b	JRCP	9.0	9.0	71.2	0.15	N	1.25	LA/EPOXY	5790	N/A
	MI 1-4a	JPCP	9.0	9.6	13-19-10-12	N/A	Y	NONE	---	5880	756
	MI 1-7a	JPCP	9.0	9.1	13-17-16-12	N/A	N	1.25	LA/EPOXY	6340	744
	MI 1-7b	JPCP	9.0	9.0	13-17-16-12	N/A	N	1.25	LA/EPOXY	6090	N/A
	MI 1-10a	JPCP	9.0	8.4	13-19-18-12	N/A	Y	NONE	---	6230	N/A
	MI 1-10b	JPCP	9.0	8.9	13-19-18-12	N/A	Y	NONE	---	5280	N/A
	MI 1-25	JPCP	9.0	N/A	13-19-18-12	N/A	Y	NONE	---	---	---
I-94 Marshall, MI	MI 3	JRCP	10.0	9.8	41.0	0.14	N	1.25	LA/EPOXY	4380	810
WB (1986- new)											
I-69 Charlotte, MI	MI 4-1	JRCP	9.0	9.5	71.2	0.15	N	1.25	LA	4830	596
NB (1972)	MI 4-2	JRCP	9.0	9.2	71.2	0.15	N	1.25	LA	4530	756
I-94 Paw Paw, MI	MI 5	JRCP	10.0	10.3	41.0	0.14	N	1.25	LA/EPOXY	4490	671
WB (1984- new)											
RT 23 Catskill, NY	NY 1-1	JPCP	9.0	9.4	20.0	N/A	N	ACME	NONE	3810	N/A
EB&WB (1965)	NY 1-3	JRCP	9.0	8.8	60.8	0.20	N	ACME	NONE	3890	809
	NY 1-4	JRCP	9.0	9.0	60.8	0.20	N	ACME	NONE	3892	658
	NY 1-6	JPCP	9.0	8.8	20.0	N/A	N	ACME	NONE	4020	N/A
	NY 1-8a	JPCP	9.0	9.8	20.0	N/A	N	NONE	---	4100	684
	NY 1-8b	JPCP	9.0	9.4	20.0	N/A	Y	NONE	---	3880	N/A
I-88 Otego, NY	NY 2-3	JPCP	9.0	9.0	20.0	N/A	N	1"I-BEAM	EPOXY	5100	864
EB&WB (1975)	NY 2-9	JPCP	9.0	9.4	20.0	N/A	N	1"I-BEAM	EPOXY	5090	705
	NY 2-11	JPCP	9.0	9.1	26.7	N/A	N	1"I-BEAM	EPOXY	6120	812
	NY 2-15	JRCP	9.0	9.8	63.5	0.20	N	1"I-BEAM	EPOXY	5520	N/A

Table 137. Slab design data for projects in wet-freeze environmental zone (cont'd).

Project Location (Year Constructed)	Project Section ID	Pvt. Type	PCC SURFACE: THICKNESS, IN: Design	THICKNESS, IN: Core	Joint Spacing, FT	% Steel JRCP	Skewed Joints, Y/N	LTD's: Dia., IN	LTD's: Coating	E, KSI from FWD	Mr, PSI (from Cores)
RT 23 Chillicothe, OH	OH 1-1	JRCP	9.0	9.0	40.0	0.09	N	1.25	P/G	4430	686
SB (1973)	OH 1-3	JRCP	9.0	9.2	21.0	0.09	N	1.25	P/G	5310	N/A
	OH 1-4	JRCP	9.0	10.0	40.0	0.09	N	1.25	P/G	5230	N/A
	OH 1-6	JRCP	9.0	9.2	21.0	0.09	N	1.25	PLASTIC	4060	761
	OH 1-7	JRCP	9.0	9.0	40.0	0.09	N	1.25	PLASTIC	4430	848
	OH 1-9	JRCP	9.0	8.8	40.0	0.09	N	1.25	P/G	3400	833
	OH 1-10	JRCP	9.0	8.8	21.0	0.09	N	1.25	P/G	4510	788
SR 2 Vermillion, OH	OH 2-33a	JPCP	15.0	N/A	20.0	N/A	Y	NONE	---	---	N/A
WB (1974)	OH 2-33b	JPCP	15.0	N/A	20.0	N/A	Y	NONE	---	---	N/A
HWY 3N Ruthven, ONT *	ONT 1-1	JPCP	12.0	12.0	13-19-18-12	N/A	Y	NONE	---	see report	see report
E/W (1982)	ONT 1-2	JPCP	8.0	7.9	13-19-18-12	N/A	Y	NONE	---		
	ONT 1-3	JPCP	8.0	7.9	13-19-18-12	N/A	Y	NONE	---	---	N/A
	ONT 1-4	JPCP	7.0	7.1	13-19-18-12	N/A	Y	NONE	---	---	N/A
HWY 427 Toronto, ONT	ONT 2-1	JPCP	9.0	N/A	13-19-18-12	N/A	Y	1.00	P/G	---	N/A
SB (1971- control)											
TR 66 Kittanning, PA	PA 1-1**	JRCP	10.0	7.0	46.5	0.09	N	1.25	EPOXY	4210	731
NB (1980)	PA 1-2	JRCP	10.0	9.5	46.5	0.09	N	1.25	EPOXY	3390	720
	PA 1-3	JRCP	10.0	9.8	46.5	0.09	N	1.25	EPOXY	3230	709
	PA 1-4	JRCP	10.0	10.3	46.5	0.09	N	1.25	EPOXY	4530	870
	PA 1-5	JRCP	10.0	10.0	46.5	0.09	N	1.25	EPOXY	3620	704
RT 130 Yardville, NJ	NJ 2-1	JRCP	10.0	10.3	78.5***	0.14	N	1.25	ST STL	6720	700
NB (1951- control)											
RT 676 Camden, NJ	NJ 3-1	JRCP	9.0	9.2	78.5***	0.16	N	1.25	ST STL	5400	681
SB (1979)	NJ 3-2	JRCP	9.0	9.1	78.5***	0.16	N	1.25	ST STL	5330	726

* Field results from reports.
** RT 422 EB

*** All NJ projects had expansion joints on 78.5-ft centers

Table 138. Base, subbase, subgrade, and outer shoulder design data for projects in wet-freeze environmental zone.

Project Location (Year Constructed)	Project Section ID	Pvt. Type	BASE					SUBBASE				SUBGRADE		OUTER SHOULDER		
			Type	THICKNESS, IN Design	THICKNESS, IN Core	Estimated Perm., FT/HR	Keff, (Dyn.) PCI	Type	THICKNESS, IN Design	THICKNESS, IN Core	Estimated Perm., FT/HR	AASHTO Soil Type	Estimated Perm., FT/HR	Type	THICKNESS, IN Surface	THICKNESS, IN Base
US 10 Clare, MI WB&EB (1975)	MI 1-1a	JRCP	AGG	4.0	N/A	0.01/0.04	353	AGG	10.0	N/A	0.01/0.25	A-2-4	0.50	AC	9.0	14.0
	MI 1-1b	JRCP	AGG	4.0	5.8	0.02/0.01	300	AGG	10.0	N/A	1.03/2.98	A-2-4	0.46	AC	9.0	14.0
	MI 1-4a	JPCP	PATB	4.0	3.0	307/N/A	468	AGG	10.0	N/A	0.60/1.10	A-2-4	0.50	AC	9.0	14.0
	MI 1-7a	JPCP	AGG	4.0	6.0	0.02/1.22	292	AGG	10.0	N/A	0.66/0.01	A-2-4	0.07	AC	9.0	14.0
	MI 1-7b	JPCP	AGG	4.0	N/A	0.01/0.02	269	AGG	10.0	N/A	0.58/2.01	A-2-4	0.11	AC	9.0	14.0
	MI 1-10a	JPCP	ATB	4.0	4.2	0.00	436	AGG	10.0	N/A	0.72/4.48	A-2-4	0.14	AC	9.0	14.0
	MI 1-10b	JPCP	ATB	4.0	6.4	0.00	502	AGG	10.0	N/A	0.87/2.02	A-2-4	0.47	AC	9.0	14.0
	MI 1-25	JPCP	ATB	4.0	N/A	0.00	---	AGG	10.0	N/A	0.32/ N/A	A-2-4	0.62	PCC	9.0	4.0
I-94 Marshall, MI WB (1986- new)	MI 3	JRCP	PAGG	4.0	4.3	4.38	186	AGG	3.0	N/A	3.59	A-2-4		PCC	8.0*	4.0
I-69 Charlotte, MI NB (1972)	MI 4-1	JRCP	AGG	4.0	N/A	0.01	283	AGG	10.0	6.3	0.35	A-4	0.11	PCC	7.5*	4.0
	MI 4-2	JRCP	AGG	4.0	4.2	0.06	189	AGG	10.0	9.0	0.85	A-4	0.11	AC	1.5	4.5
I-94 Paw Paw, MI WB (1984- new)	MI 5	JRCP	PAGG	4.0	4.2	1.85	233	AGG	21.0	22.1	0.85	A-2-4	0.50	PCC	8.0*	4.0
RT 23 Catskill, NY EB&WB (1965)	NY 1-1	JPCP	ATB	3.0	N/A	0.00	549	AGG	8.0	N/A	0.03	A-2-4		AC	4.0	16.0
	NY 1-3	JRCP	ATB	3.0	N/A	0.00	503	AGG	8.0	N/A	0.12	A-2-4		AC	4.0	16.0
	NY 1-4	JRCP	AGG	4.0	5.4	0.11	534	AGG	8.0	N/A	0.09	A-2-4		AC	4.0	16.0
	NY 1-6	JPCP	AGG	4.0	3.8	N/A	619	AGG	0.0	N/A	0.00	A-1-a		AC	4.0	16.0
	NY 1-8a	JPCP	ATB	3.0	N/A	0.00	638	AGG	8.0	N/A	N/A	N/A		AC	4.0	16.0
	NY 1-8b	JPCP	ATB	3.0	N/A	0.00	548	AGG	8.0	N/A	0.01	A-2-4		AC	4.0	16.0
I-88 Otego, NY EB&WB (1975)	NY 2-3	JPCP	AGG	4.0	5.1	N/A	341	AGG	8.0	N/A	0.23	A-1-a		PCC	6.0	15.0
	NY 2-9	JPCP	AGG	6.0	6.6	0.25	471	NONE	---	---	---	A-1-a		PCC	6.0	9.0
	NY 2-11	JPCP	AGG	6.0	5.3	N/A	296	NONE	---	---	---	A-1-a		PCC	6.0	9.0
	NY 2-15	JRCP	AGG	4.0	9.6	0.17	273	AGG	8.0	N/A	0.41	A-1-a		AC	4.0	9.0

* Average

Table 138. Base, subbase, subgrade, and outer shoulder design data for projects in wet-freeze environmental zone (cont'd).

Project Location (Year Constructed)	Project Section ID	Pvt. Type	BASE Type	BASE THICKNESS, IN Design	BASE THICKNESS, IN Core	BASE Estimated Perm., FT/HR	BASE Keff, (Dyn.) PCI	SUBBASE Type	SUBBASE THICKNESS, IN Design	SUBBASE THICKNESS, IN Core	SUBBASE Estimated Perm., FT/HR	SUBGRADE AASHTO Soil Type	SUBGRADE Estimated Perm., FT/HR	OUTER SHOULDER Type	OUTER SHOULDER THICKNESS, IN Surface	OUTER SHOULDER THICKNESS, IN Base
RT 23 Chillicothe, OH SB (1973)	OH 1-1	JRCP	AGG	7.5	8.0	0.27	449	NONE	---	---	---	A-6	0.11	AC	3.0	6.0
	OH 1-3	JRCP	ATB	4.0	4.0	0.00	440	NONE	---	---	---	A-4	0.11	AC	3.0	6.0
	OH 1-4	JRCP	ATB	4.0	3.5	0.00	525	NONE	---	---	---	A-4	0.11	AC	3.0	6.0
	OH 1-6	JRCP	AGG	7.5	7.0	0.06	431	NONE	---	---	---	A-4	0.11	AC	3.0	6.0
	OH 1-7	JRCP	AGG	7.5	6.2	7.25	340	NONE	---	---	---	A-4	0.11	AC	3.0	6.0
	OH 1-9	JRCP	AGG	7.5	7.6	0.16	351	NONE	---	---	---	A-6	0.11	AC	3.0	6.0
	OH 1-10	JRCP	AGG	7.5	7.6	0.58	405	NONE	---	---	---	A-6	0.11	AC	3.0	6.0
SR 2 Vermillion, OH * WB (1974)	OH 2-33a	JPCP	NONE	---	---	---	---	NONE	---	---	---	A-4	0.11	PCC	8.0*	NONE
	OH 2-33b	JPCP	NONE	---	---	---	---	NONE	---	---	---	A-4	0.11	AC	8.0*	NONE
HWY 3N Ruthven, ONT * E/W (1982)	ONT 1-1	JPCP	NONE	---	---	---	---	NONE	---	---	---	A-7-6		AC	3.0	9.0
	ONT 1-2	JPCP	PATB	4.0	N/A	14.6	---	NONE	---	---	---	A-7-6		AC	3.0	9.0
	ONT 1-3	JPCP	LCB	5.0	N/A	0.00	---	NONE	---	---	---	A-7-6		PCC	8.0	5.0
	ONT 1-4	JPCP	LCB	5.0	N/A	0.00	---	NONE	---	---	---	A-7-6		PCC	7.0	5.0
HWY 427 Toronto, ONT SB (1971- control)	ONT 2-1	JPCP	CTB	6.0	N/A	0.00	---	NONE	---	---	---	N/A		AC	4.0	5.8
TR 66 Kittanning, PA NB (1980)	PA 1-1**	JRCP	CTB	6.0	9.0	0.00	731	AGG	7.0	N/A	0.06	A-2-4	0.33	AC	1.0	4.0
	PA 1-2	JRCP	PATB	5.0	N/A	236/N/A	1040	AGG	8.0	N/A	97.58	A-4	0.09	AC	1.0	4.0
	PA 1-3	JRCP	AGG	8.0	4.8	236/1107	538	AGG	5.0	N/A	85.61	A-4	0.11	AC	1.0	4.0
	PA 1-4	JRCP	AGG	8.0	8.9	732/197	747	AGG	5.0	N/A	0.10	A-2-4	0.33	AC	1.0	4.0
	PA 1-5	JRCP	AGG	13.0	15.0	0.05/0.15	540	NONE	---	---	---	A-4	0.09	AC	1.0	4.0
RT 130 Yardville, NJ NB (1951- control)	NJ 2-1	JRCP	AGG	5.0	5.0	0.95	234	AGG	7.0	7.0	0.95	A-4	0.11	AC	2.0	7.0
RT 676 Camden, NJ SB (1979)	NJ 3-1	JRCP	NSOG	4.0	4.3	61/N/A	356	LFAS	4.0	---	0.00	A-1-a	0.01	AC	4.0	5.0
	NJ 3-2	JRCP	PATB	4.0	4.1	68/0.86	210	LFAS	4.0	---	0.00	A-2-4		AC	4.0	5.0

* Field results from reports.

** RT 422 EB

* Average

Table 139. Pavement joint data for projects in wet-freeze environmental zone.

Project Location (Year Constructed)	Project Section ID	Slab T, IN	Pvt. Type	Joint Spacing, FT	Base Type	TRANSVERSE JOINT: LTD Diameter IN	TRANSVERSE JOINT: Calc Avg Jt Open, IN	TRANSVERSE JOINT: Skewed Joints Y/N	TRANSVERSE JOINT: Jt Seal Shape Factor	TRANSVERSE JOINT: JOINT SEALANT Type	TRANSVERSE JOINT: JOINT SEALANT Age	TRANSVERSE JOINT: JOINT SEALANT Cond.	Depth of Long. Joint IN
US 10 Clare, MI	MI 1-1a	9.0	JRCP	71.2	AGG	1.25	0.27	N	0.20	PREF	12	GOOD	2.75
WB&EB (1975)	MI 1-1b	9.0	JRCP	71.2	AGG	1.25	0.27	N	0.20	PREF	12	EXC	2.75
	MI 1-4a	9.0	JPCP	13-19-18-12	PATB	NONE	0.06	Y	0.20	PREF	12	EXC	2.75
	MI 1-7a	9.0	JPCP	13-17-16-12	AGG	1.25	0.07	N	0.20	PREF	12	EXC	2.75
	MI 1-7b	9.0	JPCP	13-17-16-12	AGG	1.25	0.07	N	0.20	PREF	12	EXC	2.75
	MI 1-10a	9.0	JPCP	13-19-18-12	ATB	NONE	0.06	Y	0.20	PREF	12	EXC	2.75
	MI 1-10b	9.0	JPCP	13-19-18-12	ATB	NONE	0.06	Y	0.20	PREF	12	POOR	2.75
	MI 1-25	9.0	JPCP	13-19-18-12	ATB	NONE	0.06	Y	0.20	PREF	12	GOOD	2.75
I-94 Marshall, MI	MI 3	10.0	JRCP	41.0	PAGG	1.25	0.15	N	0.22	PREF	1	EXC	2.75
WB (1986- new)													
I-69 Charlotte, MI	MI 4-1	9.0	JRCP	71.2	AGG	1.25	0.27	N	0.20	PREF	14	FAIR	2.75
NB (1972)	MI 4-2	9.0	JRCP	71.2	AGG	1.25	0.27	N	0.20	PREF		FAIR	2.75
I-94 Paw Paw, MI	MI 5	10.0	JRCP	41.0	PAGG	1.25	0.15	N	0.22	PREF	3	EXC	2.75
WB (1984- new)													
RT 23 Catskill, NY	NY 1-1	9.0	JPCP	20.0	ATB	ACME	0.06	N	0.16	PREF		EXC	2.0
EB&WB (1965)	NY 1-3	9.0	JRCP	60.8	ATB	ACME	0.06	N	0.16	PREF		EXC	2.0
	NY 1-4	9.0	JRCP	60.8	AGG	ACME	0.08	N	0.16	PREF		EXC	2.0
	NY 1-6	9.0	JPCP	20.0	AGG	ACME	0.08	N	0.16	PREF		EXC	2.0
	NY 1-8a	9.0	JPCP	20.0	ATB	NONE	0.06	N	0.16	PREF		EXC	2.0
	NY 1-8b	9.0	JPCP	20.0	ATB	NONE	0.06	Y	0.16	PREF		EXC	2.0
I-88 Otego, NY	NY 2-3	9.0	JPCP	20.0	AGG	1"I-BEAM	0.07	N	0.32	RA	2	EXC	2.25
EB&WB (1975)	NY 2-9	9.0	JPCP	20.0	AGG	1"I-BEAM	0.07	N	0.32	RA	2	EXC	2.25
	NY 2-11	9.0	JPCP	26.7	AGG	1"I-BEAM	0.10	N	0.32	RA	2	EXC	2.25
	NY 2-15	9.0	JRCP	63.5	AGG	1"I-BEAM	0.23	N	0.32	RA	2	EXC	2.25

Table 139. Pavement joint data for projects in wet-freeze environmental zone (cont'd).

Project Location (Year Constructed)	Project Section ID	Slab T, IN	Pvt. Type	Joint Spacing, FT	Base Type	TRANSVERSE JOINT: LTD Diameter IN	TRANSVERSE JOINT: Calc Avg Jt Open, IN	TRANSVERSE JOINT: Skewed Joints Y/N	TRANSVERSE JOINT: Jt Seal Shape Factor	TRANSVERSE JOINT: JOINT SEALANT Type	TRANSVERSE JOINT: JOINT SEALANT Age	TRANSVERSE JOINT: JOINT SEALANT Cond.	Depth of Long. Joint IN
RT 23 Chillicothe, OH SB (1973)	OH 1-1	9.0	JRCP	40.0	AGG	1.25	0.14	N	0.25	PREF	14	FAIR	2.25
	OH 1-3	9.0	JRCP	21.0	ATB	1.25	0.06	N	0.25	PREF	14	GOOD	2.25
	OH 1-4	9.0	JRCP	40.0	ATB	1.25	0.11	N	0.25	PREF	14	GOOD	2.25
	OH 1-6	9.0	JRCP	21.0	AGG	1.25	0.07	N	0.25	PREF	14	GOOD	2.25
	OH 1-7	9.0	JRCP	40.0	AGG	1.25	0.14	N	0.25	PREF	14	GOOD	2.25
	OH 1-9	9.0	JRCP	40.0	AGG	1.25	0.14	N	0.25	PREF	14	GOOD	2.25
	OH 1-10	9.0	JRCP	21.0	AGG	1.25	0.07	N	0.25	PREF	14	GOOD	2.25
SR 2 Vermillion, OH WB (1974)	OH 2-33a	15.0	JPCP	20.0	NONE	NONE	0.07	Y	0.25	RA	13	FAIR	3.75
	OH 2-33b	15.0	JPCP	20.0	NONE	NONE	0.07	Y	0.25	RA	13	FAIR	3.75
HWY 3N Ruthven, ONT E/W (1982)	ONT 1-1	12.0	JPCP	13-19-18-12	NONE	NONE	0.06	Y	0.50	RA	5	EXC	3.0
	ONT 1-2	8.0	JPCP	13-19-18-12	PATB	NONE	0.05	Y	0.50	RA	5	EXC	2.2
	ONT 1-3	8.0	JPCP	13-19-18-12	LCB	NONE	0.05	Y	0.50	RA	5	EXC	N/A
	ONT 1-4	7.0	JPCP	13-19-18-12	LCB	NONE	0.05	Y	0.50	RA	5	GOOD	N/A
HWY 427 Toronto, ONT SB (1971- control)	ONT 2	9.0	JPCP	13-19-18-12	CTB	1.00	0.05	Y	0.50	PREF		EXC	2.50
TR 66 Kittanning, PA NB (1980)	PA 1-1*	10.0	JRCP	46.5	CTB	1.25	0.14	N	1.00	RA	7	GOOD	2.75
	PA 1-2	10.0	JRCP	46.5	PATB	1.25	0.14	N	1.00	RA	7	GOOD	2.75
	PA 1-3	10.0	JRCP	46.5	AGG	1.25	0.17	N	1.00	RA	7	GOOD	2.75
	PA 1-4	10.0	JRCP	46.5	AGG	1.25	0.17	N	1.00	RA	7	GOOD	2.75
	PA 1-5	10.0	JRCP	46.5	AGG	1.25	0.17	N	1.00	RA	7	GOOD	2.75
RT 130 Yardville, NJ NB (1951- control)	NJ 2	9.0	JRCP	78.5	AGG	1.25	** 0.25	N	0.75	RA		POOR	N/A
RT 676 Camden, NJ SB (1979)	NJ 3-1	9.0	JRCP	78.5	NSOG	1.25	** 0.26	N	0.75	RA	1	GOOD	N/A
	NJ 3-2	9.0	JRCP	78.5	PATB	1.25	** 0.21	N	0.75	RA	1	GOOD	N/A

* RT 422 EB

** All NJ projects had expansion joints on 78.5-ft centers.

Table 140. Outer lane deflection data of projects in wet-freeze environmental zone.

Project Location (Year Constructed)	Project Section ID	Slab T, IN	Pvt. Type	Joint Spacing, FT	Base Type	DEFLECTION, Mils: Mid-Slab High	DEFLECTION, Mils: Mid-Slab Low	DEFLECTION, Mils: Mid-Slab Ave.	DEFLECTION, Mils: Loaded Corner	DEFLECTION, Mils: Non-loaded Corner	Adjusted Percent LTE	% LT Across Shldr	Avg. NDT Test Temp, °F	Percent Corners with Voids
US 10 Clare, MI WB&EB (1975)	MI 1-1a	9.0	JRCP	71.2	AGG	3.2	2.8	3.0	6.7	6.0	98	N/A	82	0
	MI 1-1b	9.0	JRCP	71.2	AGG	4.7	2.8	3.4	10.3	6.4	72	N/A	82	15
	MI 1-4a	9.0	JPCP	13-19-18-12	PATB	2.6	2.1	2.4	21.8	3.7	19	N/A	75	85
	MI 1-7a	9.0	JPCP	13-17-16-12	AGG	3.7	2.8	3.1	26.3	23.0	100	N/A	78	85
	MI 1-7b	9.0	JPCP	13-17-16-12	AGG	3.8	3.3	3.5	31.8	27.2	100	N/A	63	100
	MI 1-10a	9.0	JPCP	13-19-18-12	ATB	2.7	2.2	2.5	13.7	5.1	41	N/A	76	70
	MI 1-10b	9.0	JPCP	13-19-18-12	ATB	3.6	2.3	2.6	20.4	7.2	42	N/A	72	100
	MI 1-25	9.0	JPCP	13-19-18-12	ATB	---	---	---	---	---	---	---	---	---
I-94 Marshall, MI WB (1986- new)	MI 3	10.0	JRCP	41.0	PAGG	7.7	5.2	6.3	5.1	4.1	88	90	85	0
I-69 Charlotte, MI NB (1972)	MI 4-1	9.0	JRCP	71.2	AGG	4.4	3.3	3.7	15.6	7.0	51	35	80	22
	MI 4-2	9.0	JRCP	71.2	AGG	6.0	3.3	5.0	27.1	15.8	67	N/A	75	95
I-94 Paw Paw, MI WB (1984- new)	MI 5	10.0	JRCP	41.0	PAGG	4.7	3.2	3.8	26.9	21.9	94	48	75	95
RT 23 Catskill, NY EB&WB (1965)	NY 1-1	9.0	JPCP	20.0	ATB	3.4	2.2	2.9	16.4	15.7	100	N/A	78	55
	NY 1-3	9.0	JRCP	60.8	ATB	3.9	2.6	3.1	10.0	4.0	47	N/A	82	20
	NY 1-4	9.0	JRCP	60.8	AGG	3.3	2.4	2.9	11.1	8.9	94	N/A	79	25
	NY 1-6	9.0	JPCP	20.0	AGG	3.1	2.2	2.6	16.5	16.1	100	N/A	73	30
	NY 1-8a	9.0	JPCP	20.0	ATB	2.9	2.2	2.5	11.1	10.5	100	N/A	76	20
	NY 1-8b	9.0	JPCP	20.0	ATB	3.1	2.4	2.8	10.6	9.9	100	N/A	80	25
I-88 Otego, NY EB&WB (1975)	NY 2-3	9.0	JPCP	20.0	AGG	4.1	3.0	3.3	28.7	25.0	100	65	71	90
	NY 2-9	9.0	JPCP	20.0	AGG	3.0	2.2	2.7	21.4	18.4	97	64	67	50
	NY 2-11	9.0	JPCP	26.7	AGG	3.7	2.8	3.2	11.0	8.1	81	69	73	5
	NY 2-15	9.0	JRCP	63.5	AGG	4.0	3.0	3.5	8.4	7.3	99	N/A	74	0

Table 140. Outer lane deflection data of projects in wet-freeze environmental zone (cont'd).

Project Location (Year Constructed)	Project Section ID	Slab T, IN	Pvt. Type	Joint Spacing, FT	Base Type	DEFLECTION, Mils: Mid-Slab High	Mid-Slab Low	Mid-Slab Ave.	Loaded Corner	Non-loaded Corner	Adjusted Percent LTE	% LT Across Shldr	Avg. NDT Test Temp, °F	Percent Corners with Voids
RT 23 Chillicothe, OH	OH 1-1	9.0	JRCP	40.0	AGG	3.3	2.7	3.0	13.0	10.6	95	N/A	70	71
SB (1973)	OH 1-3	9.0	JRCP	21.0	ATB	2.9	2.4	2.6	7.8	5.4	79	N/A	75	0
	OH 1-4	9.0	JRCP	40.0	ATB	3.1	2.1	2.6	7.7	4.7	70	N/A	78	0
	OH 1-6	9.0	JRCP	21.0	AGG	3.4	2.6	3.1	20.4	18.6	100	N/A	74	94
	OH 1-7	9.0	JRCP	40.0	AGG	5.4	3.2	3.8	16.4	14.9	100	N/A	73	81
	OH 1-9	9.0	JRCP	40.0	AGG	7.2	3.2	4.1	15.7	11.5	86	N/A	72	100
	OH 1-10	9.0	JRCP	21.0	AGG	3.5	2.6	3.1	9.5	5.8	71	N/A	73	0
SR 2 Vermillion, OH	OH 2-33a	15.0	JPCP	20.0	NONE	---	---	---	---	---	---	---	---	---
WB (1974)	OH 2-33b	15.0	JPCP	20.0	NONE	---	---	---	---	---	---	---	---	---
												---		
HWY 3N Ruthven, ONT	ONT 1-1	12.0	JPCP	13-19-18-12	NONE	---	---	---	---	---	---	---	---	---
E/W (1982)	ONT 1-2	8.0	JPCP	13-19-18-12	PATB	---	---	---	---	---	---	---	---	---
	ONT 1-3	8.0	JPCP	13-19-18-12	LCB	---	---	---	---	---	---	---	---	---
	ONT 1-4	7.0	JPCP	13-19-18-12	LCB	---	---	---	---	---	---	---	---	---
												---		
HWY 427 Toronto, ONT	ONT 2	9.0	JPCP	13-19-18-12	CTB	---	---	---	---	---	---	---	---	---
SB (1971- control)														
TR 66 Kittanning, PA	PA 1-1	10.0	JRCP	46.5	CTB	4.3	1.6	2.1	12.9	9.4	84	N/A	72	45
NB (1980)	PA 1-2	10.0	JRCP	46.5	PATB	2.1	1.7	1.9	41.4	36.0	100	N/A	71	100
	PA 1-3	10.0	JRCP	46.5	AGG	3.4	2.2	2.9	40.0	33.5	98	N/A	72	95
	PA 1-4	10.0	JRCP	46.5	AGG	3.0	1.9	2.4	11.1	3.9	39	N/A	77	15
	PA 1-5	10.0	JRCP	46.5	AGG	3.4	1.8	2.6	33.4	28.5	98	N/A	71	95
RT 130 Yardville, NJ	NJ 2	9.0	JRCP	78.5	AGG	3.7	3.2	3.4	7.2	4.9	76	N/A	79	0
NB (1951- control)														
RT 676 Camden, NJ	NJ 3-1	9.0	JRCP	78.5	NSOG	3.4	2.9	3.0	9.2	4.0	50	N/A	81	0
SB (1979)	NJ 3-2	9.0	JRCP	78.5	PATB	4.5	3.6	4.1	6.7	3.4	57	N/A	85	0

Table 141. Outer shoulder and drainage information for projects in wet-freeze environmental zone.

Project Location (Year Constructed)	Project Section ID	Slab T, IN	Pvt. Type	Joint Spacing, FT	Base Type	Outer Shoulder Type- Thickness Inches: Surface	Outer Shoulder Type- Thickness Inches: Base	Overall Shoulder Condition	Shoulder Jt Seal Condition	PERMEABILITY FT/HR: Base	PERMEABILITY FT/HR: Sub-base	AASHTO Drain. Coeff. Cd	Dowel Y/N	Sub-Drainage Type	Depth to Ditch FT	Average Trans. Slope, %	Average Longit. Grade, %
US 10 Clare, MI WB&EB (1975)	MI 1-1a	9.0	JRCP	71.2	AGG	AC- 9	AGG- 14	GOOD	GOOD	0.04	0.25	1.00	Y	LONG DRN	1	2.08	-0.17
	MI 1-1b	9.0	JRCP	71.2	AGG	AC- 9	AGG- 14	FAIR	GOOD	0.01	2.98	0.90	Y	NONE	4	-1.04	0.35
	MI 1-4a	9.0	JPCP	13-19-18-12	PATB	AC- 9	AGG- 14	FAIR	POOR	N/A	1.10	1.10	N	LONG DRN	3	-1.91	0.00
	MI 1-7a	9.0	JPCP	13-17-16-12	AGG	AC- 9	AGG- 14	GOOD	GOOD	1.22	0.01	0.90	Y	LONG DRN	3	-1.39	0.17
	MI 1-7b	9.0	JPCP	13-17-16-12	AGG	AC- 9	AGG- 14	GOOD	GOOD	0.02	2.01	0.80	Y	NONE	2	-2.08	0.27
	MI 1-10a	9.0	JPCP	13-19-18-12	ATB	AC- 9	AGG- 14	GOOD	EXC	0.00	4.48	0.85	N	LONG DRN	4	2.08	0.00
	MI 1-10b	9.0	JPCP	13-19-18-12	ATB	AC- 9	AGG- 14	GOOD	FAIR	0.00	2.02	0.85	N	NONE	5	-1.39	0.52
	MI 1-25	9.0	JPCP	13-19-18-12	ATB	PCC- 9	ATB- 4	GOOD **	POOR	0.00	0.32	0.70	N	NONE	40	-1.30	0.00
I-94 Marshall, MI WB (1986- new)	MI 3	10.0	JRCP	41.0	PAGG	PCC- 8*	AGG- 4	EXC	EXC	4.30	3.59	1.12	Y	DB/LD	5	-1.04	0.00
I-69 Charlotte, MI NB (1972)	MI 4-1	9.0	JRCP	71.2	AGG	PCC- 7*	AGG- 4	FAIR	FAIR	0.01	0.35	0.75	Y	NONE	5	-1.04	0.69
	MI 4-2	9.0	JRCP	71.2	AGG	AC- 1	ATB- 5	FAIR	POOR	0.06	0.85	0.70	Y	NONE	5	-1.04	0.00
I-94 Paw Paw, MI WB (1984- new)	MI 5	10.0	JRCP	41.0	PAGG	PCC- 8*	AGG- 4	EXC	EXC	1.85	0.85	1.05	Y	DB/LD	2	-2.08	0.00
RT 23 Catskill, NY EB&WB (1965)	NY 1-1	9.0	JPCP	20.0	ATB	AC- 4	AGG- 16	GOOD	POOR	0.0	0.03	0.90	Y	NONE	8	-1.39	0.52
	NY 1-3	9.0	JRCP	60.8	ATB	AC- 4	AGG- 16	GOOD	GOOD	0.0	0.12	0.90	Y	NONE	8	-1.21	-1.04
	NY 1-4	9.0	JRCP	60.8	AGG	AC- 4	AGG- 16	GOOD	POOR	0.11	0.09	0.80	Y	NONE	4	-0.87	0.17
	NY 1-6	9.0	JPCP	20.0	AGG	AC- 4	AGG- 16	GOOD	POOR	N/A	0.00	0.75	Y	NONE	6	-0.87	-0.52
	NY 1-8a	9.0	JPCP	20.0	ATB	AC- 4	AGG- 16	GOOD	POOR	0.0	N/A	0.90	N	NONE	8	1.56	-1.91
	NY 1-8b	9.0	JPCP	20.0	ATB	AC- 4	AGG- 16	GOOD	POOR	0.0	0.01	1.00	N	NONE	8	-0.87	-0.17
I-88 Otego, NY EB&WB (1975)	NY 2-3	9.0	JPCP	20.0	AGG	PCC- 6	AGG- 15	FAIR	GOOD	N/A	0.23	1.00	Y	NONE	8	-2.08	-1.04
	NY 2-9	9.0	JPCP	20.0	AGG	PCC- 6	AGG- 9	POOR	EXC	0.25	---	0.85	Y	NONE	15	1.73	-0.87
	NY 2-11	9.0	JPCP	26.7	AGG	PCC- 6	AGG- 9	POOR	EXC	N/A	---	0.80	Y	NONE	10	-1.04	-1.91
	NY 2-15	9.0	JRCP	63.5	AGG	AC- 4	AGG- 9	FAIR	POOR	0.17	0.41	0.85	Y	NONE	12	-1.56	0.87

* Average

** Next lane is an acceleration ramp

Table 141. Outer shoulder and drainage information for projects in wet-freeze environmental zone (cont'd).

Project Location (Year Constructed)	Project Section ID	Slab T, IN	Pvt. Type	Joint Spacing, FT	Base Type	Outer Shoulder Type- Thickness Inches: Surface	Outer Shoulder Type- Thickness Inches: Base	Overall Shoulder Condition	Shoulder Jt Seal Condition	PERMEABILITY FT/HR: Base	PERMEABILITY FT/HR: Subbase	AASHTO Drain. Coeff. Cd	Dowel Y/N	Sub-Drainage Type	Depth to Ditch FT	Average Trans. Slope, %	Average Longit. Grade, %
RT 23 Chillicothe, OH SB' (1973)	OH 1-1	9.0	JRCP	40.0	AGG	AC- 3	AGG- 4	FAIR **	POOR	0.27	---	0.95	Y	NONE	50	-1.56	-1.04
	OH 1-3	9.0	JRCP	21.0	ATB	AC- 3	AGG- 4	FAIR	POOR	0.0	---	0.90	Y	NONE	50	-2.43	-2.08
	OH 1-4	9.0	JRCP	40.0	ATB	AC- 3	AGG- 4	FAIR	POOR	0.0	---	0.90	Y	NONE	50	-2.25	-1.91
	OH 1-6	9.0	JRCP	21.0	AGG	AC- 3	AGG- 4	FAIR	POOR	0.06	---	0.85	Y	NONE	50	-1.04	-2.08
	OH 1-7	9.0	JRCP	40.0	AGG	AC- 3	AGG- 4	POOR	POOR	7.25	---	1.05	Y	NONE	50	-1.56	0.00
	OH 1-9	9.0	JRCP	40.0	AGG	AC- 3	AGG- 4	FAIR	POOR	0.16	---	0.90	Y	NONE	50	-2.08	-0.17
	OH 1-10	9.0	JRCP	21.0	AGG	AC- 3	AGG- 4	FAIR	POOR	0.58	---	1.05	Y	NONE	50	-1.91	0.00
SR 2 Vermillion, OH WB (1974)	OH 2-33a	15.0	JPCP	20.0	NONE	PCC- 8*	NONE	EXC	EXC	---	---	0.90	N	NONE	8	-1.04	0.00
	OH 2-33b	15.0	JPCP	20.0	NONE	AC- 8*	NONE	FAIR	POOR	---	---	0.95	N	NONE	10	-2.08	-0.17
HWY 3N Ruthven, ONT * E/W (1982)	ONT 1-1	12.0	JPCP	13-19-18-12	NONE	AC- 3	AGG- 9	GOOD	FAIR	---	---	0.95	N	LONG DRN	6	-2.08	0.00
	ONT 1-2	8.0	JPCP	13-19-18-12	PATB	AC- 3	AGG- 9	GOOD	FAIR	14.6	---	1.00	N	LONG DRN	8	-2.08	0.17
	ONT 1-3	8.0	JPCP	13-19-18-12	LCB	PCC- 8	AGG- 5	GOOD	EXC	0.0	---	1.00	N	LONG DRN	6	-2.08	0.00
	ONT 1-4	7.0	JPCP	13-19-18-12	LCB	PCC- 7	AGG- 5	GOOD	GOOD	0.0	---	1.00	N	LONG DRN	8	-1.39	0.00
HWY 427 Toronto, ONT SB (1971- control)	ONT 2	9.0	JPCP	13-19-18-12	CTB	AC- 4	AGG- 6	GOOD	FAIR	0.0	---	0.80	Y	LONG DRN	1	-1.73	0.00
TR 66 Kittanning, PA NB (1980)	PA 1-1**	10.0	JRCP	46.5	CTB	AC- 1	ATB- 4	EXC	POOR	0.0	0.06	0.90	Y	LONG DRN	4	-0.87	1.39
	PA 1-2	10.0	JRCP	46.5	PATB	AC- 1	ATB- 4	EXC	POOR	236	97.6	1.05	Y	LONG DRN	5	-1.04	0.17
	PA 1-3	10.0	JRCP	46.5	AGG	AC- 1	ATB- 4	EXC	POOR	1107	85.6	1.05	Y	LONG DRN	4	-3.30	3.82
	PA 1-4	10.0	JRCP	46.5	AGG	AC- 1	ATB- 4	GOOD	POOR	197	0.10	1.00	Y	LONG DRN	5	-0.70	4.00
	PA 1-5	10.0	JRCP	46.5	AGG	AC- 1	ATB- 4	EXC	POOR	0.15	---	1.00	Y	LONG DRN	4	0.35	3.12
RT 130 Yardville, NJ NB (1951- control)	NJ 2	9.0	JRCP	78.5	AGG	AC- 2	AGG- 7	EXC	EXC	0.95	---	1.05	Y	NONE	N/A	-0.87	0.52
RT 676 Camden, NJ SB (1979)	NJ 3-1	9.0	JRCP	78.5	NSOG	AC- 4	PAM- 5	EXC	POOR	61	1.02	1.10	Y	DB/LD	N/A	-1.21	0.35
	NJ 3-2	9.0	JRCP	78.5	PATB	AC- 4	PAM- 5	EXC	POOR	0.86	0.73	1.10	Y	DB/LD	N/A	-1.21	0.52

* Field results from reports.
** RT 422 EB

* Average

** Next lane is an acceleration ramp

Table 142. Traffic information for projects in wet-freeze environmental zone.

Project Location (Year Constructed)	Project Section ID	ORIGINAL DESIGN TRAFFIC: ESAL's, (million)	ORIGINAL DESIGN TRAFFIC: ADT*, thous.	ORIGINAL DESIGN TRAFFIC: % Trucks	Age at Survey	1987 ESTIMATED: ADT, thous.	1987 ESTIMATED: % Trucks	OUTER LANE (1): 1987 ESAL from ADT, % Trucks	OUTER LANE (1): Estimated ESAL's to Date	LANE # 2: 1987 ESAL from ADT, % Trucks	LANE # 2: Estimated ESAL's to Date
US 10 Clare, MI	MI 1-1a			5.0	12	5.1	8.0	71100	885200	6300	77500
WB&EB (1975)	MI 1-1b			5.0	12	5.1	8.0	71100	885200	6300	77500
	MI 1-4a			5.0	12	5.1	8.0	71100	885200	6300	77500
	MI 1-7a			5.0	12	5.1	8.0	71100	885200	6300	77500
	MI 1-7b			5.0	12	5.1	8.0	71100	885200	6300	77500
	MI 1-10a			5.0	12	5.1	8.0	71100	885200	6300	77500
	MI 1-10b			5.0	12	5.1	8.0	71100	885200	6300	77500
	MI 1-25			5.0	12	5.1	8.0	71100	885200	6300	77500
I-94 Marshall, MI	MI 3		15.7	25.0	1	31.3	22.0	1411400	2768800	423400	830700
WB (1986- new)											
I-69 Charlotte, MI	MI 4-1			5.0	16	13.7	11.0	324800	4367400	63000	747300
NB (1972)	MI 4-2			5.0	16	13.7	11.0	324800	4367400	63000	747300
I-94 Paw Paw, MI	MI 5		18.4	22.0	3	19.3	20.0	832200	3123700	196300	748500
WB (1984- new)											
RT 23 Catskill, NY	NY 1-1				22	7.3	10.0	214600	3136300	26500	368500
EB&WB (1965)	NY 1-3				22	7.3	10.0	214600	3136300	26500	368500
	NY 1-4				22	7.3	10.0	214600	3136300	26500	368500
	NY 1-6				22	7.3	10.0	214600	3136300	26500	368500
	NY 1-8a				22	7.3	10.0	214600	3136300	26500	368500
	NY 1-8b				22	7.3	10.0	214600	3136300	26500	368500
I-88 Otego, NY	NY 2-3				12	8.5	12.0	272600	1428100	38300	144900
EB&WB (1975)	NY 2-9				12	8.5	12.0	272600	1428100	38300	144900
	NY 2-11				12	8.5	12.0	272600	1428100	38300	144900
	NY 2-15				12	8.5	12.0	272600	1428100	38300	144900

* All ADT's are two-way

Table 142. Traffic information for projects in wet-freeze environmental zone (cont'd).

Project Location (Year Constructed)	Project Section ID	ORIGINAL DESIGN TRAFFIC: ESAL's, (million)	ORIGINAL DESIGN TRAFFIC: ADT**, thous.	ORIGINAL DESIGN TRAFFIC: % Trucks	Age at Survey	1987 ESTIMATED: ADT, thous.	1987 ESTIMATED: % Trucks	OUTER LANE (1): 1987 ESAL from ADT, % Trucks	OUTER LANE (1): Estimated ESAL's to Date	LANE # 2: 1987 ESAL from ADT, % Trucks	LANE # 2: Estimated ESAL's to Date
RT 23 Chillicothe, OH	OH 1-1				14	13.0	12.5	305800	3424600	57500	570200
SB (1973)	OH 1-3				14	13.0	12.5	305800	3424600	57500	570200
	OH 1-4				14	13.0	12.5	305800	3424600	57500	570200
	OH 1-6				14	13.0	12.5	305800	3424600	57500	570200
	OH 1-7				14	13.0	12.5	305800	3424600	57500	570200
	OH 1-9				14	13.0	12.5	305800	3424600	57500	570200
	OH 1-10				14	13.0	12.5	305800	3424600	57500	570200
SR 2 Vermillion, OH	OH 2-33a				13	13.5	16.6	391000	3295700	75200	561700
WB (1974)	OH 2-33b				13	13.5	16.6	391000	3295700	75200	561700
HWY 3N Ruthven, ONT	ONT 1-1		4.4	13	5	5.4	14.5	244500	1011100	N/A	N/A
E/W (1982)	ONT 1-2		4.4	13	5	5.4	14.5	244500	1011100	N/A	N/A
	ONT 1-3		4.4	13	5	5.4	14.5	244500	1011100	N/A	N/A
	ONT 1-4		4.4	13	5	5.4	14.5	244500	1011100	N/A	N/A
HWY 427 Toronto, ONT	ONT 2				16	228.4	10.0	3702600	35649300	3349000	28235000
SB (1971- control)											
TR 66 Kittanning, PA	PA 1-1*				7	10.3	6.0	97100	603400	15700	75300
NB (1980)	PA 1-2				7	10.2	4.0	64200	270200	10300	28700
	PA 1-3				7	10.2	4.0	64200	270200	10300	28700
	PA 1-4				7	10.2	4.0	64200	270200	10300	28700
	PA 1-5				7	10.2	4.0	64200	270200	10300	28700
RT 130 Yardville, NJ	NJ 2				36	24.7	22.0	1475800	34813100	394800	7194600
NB (1951- control)											
RT 676 Camden, NJ	NJ 3-1				8	78.7	5.0	641200	4181400	397700	2514300
SB (1979)	NJ 3-2				8	78.7	5.0	641200	4181400	397700	2514300

* RT 422 EB ** All ADT's are two-way

Table 143. Outer lane performance data for projects in wet-freeze environmental zone.

Project Location (Year Constructed)	Project Section ID	Slab T, IN	Pvt. Type	Joint Spacing, FT	Base Type	LTD Dia., IN	Ave. PSR	Mays Rough. IN/MI	Average Trans. Faulting IN	Deter. Trans. Cracks/ Mile	Longit. Cracking, LIN FT/MI	Pumping, N/L/M/H	Percent Joints Spalled	Mtls Dur. Dist
US 10 Clare, MI	MI 1-1a	9.0	JRCP	71.2	AGG	1.25	3.6	96	0.05	0	0	N	0	DCRK
WB&EB (1975)	MI 1-1b	9.0	JRCP	71.2	AGG	1.25	3.3	91	0.08	5	0	N	0	DCRK
	MI 1-4a	9.0	JPCP	13-19-18-12	PATB	0.00	3.9	47	0.03	0	0	N	9	DCRK
	MI 1-7a	9.0	JPCP	13-17-16-12	AGG	1.25	3.6	58	0.04	0	0	N	12	DCRK
	MI 1-7b	9.0	JPCP	13-17-16-12	AGG	1.25	3.7	37	0.04	0	0	N	11	DCRK
	MI 1-10a	9.0	JPCP	13-19-18-12	ATB	0.00	2.9	98	0.14	0	0	L	41	DCRK
	MI 1-10b	9.0	JPCP	13-19-18-12	ATB	0.00	2.8	104	0.19	18	0	L	63	DCRK
	MI 1-25	9.0	JPCP	13-19-18-12	ATB	0.00	2.9	159	0.20	29	0	L	75	DCRK
I-94 Marshall, MI	MI 3	10.0	JRCP	41.0	PAGG	1.25	4.8	37	0.02	0	0	N	0	NONE
WB (1986- new)														
I-69 Charlotte, MI	MI 4-1	9.0	JRCP	71.2	AGG	1.25	2.4	132	0.12	227	40	N	0	DCRK
NB (1972)	MI 4-2	9.0	JRCP	71.2	AGG	1.25	2.4	112	0.06	183	0	N	6	DCRK
I-94 Paw Paw, MI	MI 5	10.0	JRCP	41.0	PAGG	1.25	4.2	36	0.05	144	0	N	0	NONE
WB (1984- new)														
RT 23 Catskill, NY	NY 1-1	9.0	JPCP	20.0	ATB	ACME	4.0	56	0.02	0	0	N	6	NONE
EB&WB (1965)	NY 1-3	9.0	JRCP	60.8	ATB	ACME	3.6	79	0.14	0	0	M	73	NONE
	NY 1-4	9.0	JRCP	60.8	AGG	ACME	3.4	90	0.09	0	0	N	9	NONE
	NY 1-6	9.0	JPCP	20.0	AGG	ACME	3.9	82	0.03	35	0	N	13	NONE
	NY 1-8a	9.0	JPCP	20.0	ATB	0.00	4.1	64	0.01	9	0	N	0	NONE
	NY 1-8b	9.0	JPCP	20.0	ATB	0.00	3.8	65	0.03	0	0	N	0	NONE
I-88 Otego, NY	NY 2-3	9.0	JPCP	20.0	AGG	1" IB	4.2	61	0.01	35	0	N	0	NONE
EB&WB (1975)	NY 2-9	9.0	JPCP	20.0	AGG	1" IB	4.0	66	0.02	25	543	N	0	NONE
	NY 2-11	9.0	JPCP	26.7	AGG	1" IB	4.1	76	0.01	26	0	N	0	NONE
	NY 2-15	9.0	JRCP	63.5	AGG	1" IB	4.0	63	0.02	0	73	N	0	NONE

Table 143. Outer lane performance data for projects in wet-freeze environmental zone (cont'd).

Project Location (Year Constructed)	Project Section ID	Slab T, IN	Pvt. Type	Joint Spacing, FT	Base Type	LTD Dia., IN	Ave. PSR	Mays Rough. IN/MI	Average Trans. Faulting IN	Deter. Trans. Cracks/Mile	Longit. Cracking, LIN FT/MI	Pumping, N/L/M/H	Percent Joints Spalled	Mtls Dur. Dist.
RT 23 Chillicothe, OH	OH 1-1	9.0	JRCP	40.0	AGG	1.25	4.2	109	0.13	0	0	N	0	NONE
SB (1973)	OH 1-3	9.0	JRCP	21.0	ATB	1.25	4.2	106	0.06	0	0	N	13	NONE
	OH 1-4	9.0	JRCP	40.0	ATB	1.25	4.1	123	0.07	29	0	N	0	NONE
	OH 1-6	9.0	JRCP	21.0	AGG	1.25	4.2	104	0.03	31	0	N	0	NONE
	OH 1-7	9.0	JRCP	40.0	AGG	1.25	4.2	115	0.07	235	0	N	0	NONE
	OH 1-9	9.0	JRCP	40.0	AGG	1.25	4.2	109	0.14	106	0	N	0	NONE
	OH 1-10	9.0	JRCP	21.0	AGG	1.25	4.2	122	0.10	0	0	N	0	NONE
SR 2 Vermillion, OH	OH 2-33a	15.0	JPCP	20.0	NONE	0.00	3.4	90	0.11	0	104	L	15	NONE
WB (1974)	OH 2-33b	15.0	JPCP	20.0	NONE	0.00	3.5	85	0.11	11	148	M	23	NONE
HWY 3N Ruthven, ONT	ONT 1-1	12.0	JPCP	13-19-18-12	NONE	0.00	3.8	90	0.05	0	0	L	0	NONE
E/W (1982)	ONT 1-2	8.0	JPCP	13-19-18-12	PATB	0.00	3.8	78	0.05	0	0	L	0	NONE
	ONT 1-3	8.0	JPCP	13-19-18-12	LCB	0.00	3.8	57	0.04	30	0	L	0	NONE
	ONT 1-4	7.0	JPCP	13-19-18-12	LCB	0.00	3.7	65	0.04	45	85	N	0	NONE
HWY 427 Toronto, ONT	ONT 2	9.0	JPCP	13-19-18-12	CTB	1.00	3.9	104	0.01	5	0	N	0	NONE
SB (1971- control)														
TR 66 Kittanning, PA	PA 1-1*	10.0	JRCP	46.5	CTB	1.25	4.2	75	0.03	0	0	N	0	NONE
NB (1980)	PA 1-2	10.0	JRCP	46.5	PATB	1.25	3.8	91	0.02	0	0	N	0	NONE
	PA 1-3	10.0	JRCP	46.5	AGG	1.25	3.7	113	0.03	0	0	N	0	NONE
	PA 1-4	10.0	JRCP	46.5	AGG	1.25	4.0	109	0.03	0	0	N	0	NONE
	PA 1-5	10.0	JRCP	46.5	AGG	1.25	4.0	82	0.03	0	0	N	0	NONE
RT 130 Yardville, NJ	NJ 2	9.0	JRCP	78.5	AGG	1.25	3.8	63	0.06	24	10	N	20	NONE
NB (1951- control)														
RT 676 Camden, NJ	NJ 3-1	9.0	JRCP	78.5	NSOG	1.25	3.6	134	0.05	0	0	N	0	NONE
SB (1979)	NJ 3-2	9.0	JRCP	78.5	PATB	1.25	3.5	153	0.06	0	0	N	43	NONE

* RT 422 EB

Table 144. Lane 2 performance data for projects in wet-freeze environmental zone.

Project Location (Year Constructed)	Project Section ID	Slab T, IN	Pvt. Type	Joint Spacing, FT	Base Type	LTD Dia., IN	PSR	Mays Rough. IN/MI	Average Trans. Faulting IN	Deter. Trans. Cracks/Mile	Longit. Cracking, LIN FT/MI	Pumping, N/L/M/H	Percent Joints Spalled	Mtls Dur. Dist
US 10 Clare, MI	MI 1-1a	9.0	JRCP	71.2	AGG	1.25	3.9	76	0.04	0	0	N	0	DCRK
WB&EB (1975)	MI 1-1b	9.0	JRCP	71.2	AGG	1.25	3.2	74	0.04	10	0	N	0	DCRK
	MI 1-4a	9.0	JPCP	13-19-18-12	PATB	0.00	4.0	41	0.00	0	0	N	2	DCRK
	MI 1-7a	9.0	JPCP	13-17-16-12	AGG	1.25	3.8	50	0.02	0	0	N	0	DCRK
	MI 1-7b	9.0	JPCP	13-17-16-12	AGG	1.25	4.0	44	0.03	0	0	N	0	DCRK
	MI 1-10a	9.0	JPCP	13-19-18-12	ATB	0.00	3.0	78	0.03	0	0	L	32	DCRK
	MI 1-10b	9.0	JPCP	13-19-18-12	ATB	0.00	3.6	80	0.03	0	0	L	43	DCRK
	MI 1-25	9.0	JPCP	13-19-10-12	ATB	0.00	3.4	103	0.01	0	0	L	25	DCRK
I-94 Marshall, MI	MI 3	10.0	JRCP	41.0	PAGG	1.25	4.0	39	0.01	0	0	N	0	NONE
WB (1906- new)														
I-69 Charlotte, MI	MI 4-1	9.0	JRCP	71.2	AGG	1.25	2.6	114	0.05	69	0	N	0	DCRK
NB (1972)	MI 4-2	9.0	JRCP	71.2	AGG	1.25	2.6	103	0.06	54	143	N	0	DCRK
I-94 Paw Paw, MI	MI 5	10.0	JRCP	41.0	PAGG	1.25	4.4	50	0.03	5	0	N	0	NONE
WB (1984- new)														
RT 23 Catskill, NY	NY 1-1	9.0	JPCP	20.0	ATB	ACME	4.0	57	0.01	0	0	N	3	NONE
EB&WB (1965)	NY 1-9	9.0	JRCP	60.8	ATB	ACME	4.0	76	0.03	0	0	M	73	NONE
	NY 1-4	9.0	JRCP	60.8	AGG	ACME	4.0	66	0.02	9	0	N	9	NONE
	NY 1-6	9.0	JPCP	20.0	AGG	ACME	3.6	79	0.01	9	264	N	10	NONE
	NY 1-8a	9.0	JPCP	20.0	ATB	0.00	4.0	64	0.01	9	290	N	0	NONE
	NY 1-8b	9.0	JPCP	20.0	ATB	0.00	4.1	59	0.01	0	0	N	0	NONE
I-88 Otego, NY	NY 2-3	9.0	JPCP	20.0	AGG	1" IB	4.0	78	0.01	35	0	N	0	NONE
EB&WB (1975)	NY 2-9	9.0	JPCP	20.0	AGG	1" IB	4.1	62	0.01	10	0	N	0	NONE
	NY 2-11	9.0	JPCP	26.7	AGG	1" IB	4.0	58	0.01	13	0	N	0	NONE
	NY 2-15	9.0	JRCP	63.5	AGG	1" IB	4.2	57	0.00	0	161	N	6	NONE

Table 144. Lane 2 performance data for projects in wet-freeze environmental zone (cont'd).

Project Location (Year Constructed)	Project Section ID	Slab T, IN	Pvt. Type	Joint Spacing, FT	Base Type	LTD Dia., IN	PSR	Mays Rough. IN/MI	Average Trans. Faulting IN	Deter. Trans. Cracks/Mile	Longit. Cracking, LIN FT/MI	Pumping, N/L/M/H	Percent Joints Spalled	Mtls Dur. Dist
RT 23 Chillicothe, OH	OH 1-1	9.0	JRCP	40.0	AGG	1.25	4.2	113	0.06	22	0	N	0	NONE
SB (1973)	OH 1-3	9.0	JRCP	21.0	ATB	1.25	4.2	113	0.03	0	0	N	0	NONE
	OH 1-4	9.0	JRCP	40.0	ATB	1.25	4.2	115	0.04	29	0	N	0	NONE
	OH 1-6	9.0	JRCP	21.0	AGG	1.25	4.4	88	0.04	0	0	N	0	NONE
	OH 1-7	9.0	JRCP	40.0	AGG	1.25	4.4	105	0.06	73	0	N	0	NONE
	OH 1-9	9.0	JRCP	40.0	AGG	1.25	4.4	108	0.07	40	0	N	0	NONE
	OH 1-10	9.0	JRCP	21.0	AGG	1.25	4.2	108	0.07	0	0	N	0	NONE
SR 2 Vermillion, OH	OH 2-33a	15.0	JPCP	20.0	NONE	0.00	3.3	86	0.04	0	264	L	3	NONE
WB (1974)	OH 2-33b	15.0	JPCP	20.0	NONE	0.00	3.5	75	0.03	0	0	M	11	NONE
HWY 3N Ruthven, ONT	ONT 1-1	12.0	JPCP	13-19-18-12	NONE	0.00	3.0	62	0.07	0	40	L	0	NONE
E/W (1982)	ONT 1-2	8.0	JPCP	13-19-18-12	PATB	0.00	3.7	70	0.06	0	0	M	1	NONE
	ONT 1-3	8.0	JPCP	13-19-18-12	LCB	0.00	3.8	66	0.09	25	490	L	1	NONE
	ONT 1-4	7.0	JPCP	13-19-18-12	LCB	0.00	3.8	72	0.07	50	260	N	0	NONE
HWY 427 Toronto, ONT	ONT 2	9.0	JPCP	13-19-18-12	CTB	1.00	---	---	---	---	---	---	---	NONE
SB (1971- control)														
TR 66 Kittanning, PA	PA 1-1*	10.0	JRCP	46.5	CTB	1.25	4.0	88	0.01	0	0	N	0	NONE
NB (1980)	PA 1-2	10.0	JRCP	46.5	PATB	1.25	3.8	71	0.01	0	0	N	0	NONE
	PA 1-3	10.0	JRCP	46.5	AGG	1.25	3.7	93	0.01	0	0	N	0	NONE
	PA 1-4	10.0	JRCP	46.5	AGG	1.25	4.1	103	0.02	0	0	N	0	NONE
	PA 1-5	10.0	JRCP	46.5	AGG	1.25	4.2	103	0.03	0	0	N	0	NONE
RT 130 Yardville, NJ	NJ 2	9.0	JRCP	78.5	AGG	1.25	3.5	75	0.03	10	0	N	20	NONE
NB (1951- control)														
RT 676 Camden, NJ	NJ 3-1	9.0	JRCP	78.5	NSOG	1.25	---	---	0.02	0	0	N	0	NONE
SB (1979)	NJ 3-2	9.0	JRCP	78.5	PATB	1.25	---	---	0.01	0	0	N	14	NONE

* RT 422 EB

Table 145. Slab design data for projects in wet-nonfreeze environmental zone.

Project Location (Year Constructed)	Project Section ID	Pvt. Type	PCC SURFACE THICKNESS, IN Design	PCC SURFACE THICKNESS, IN Core	Joint Spacing, FT	% Steel JRCP	Skewed Joints Y/N	LTD's Dia, IN	LTD's Coating	E, KSI from FWD	Mr, PSI (from Cores)
I-95 Rocky Mount, NC NB&SB (1967)	NC 1-1	JPCP	9.0	9.5	30	0.00	Y	0.00	---	4630	736
	NC 1-2	JPCP	9.0	9.5	30	0.00	N	1.00	NONE	5190	674
	NC 1-3	JPCP	9.0	9.1	30	0.00	N	0.00	---	3970	705
	NC 1-4	JPCP	9.0	9.0	30	0.00	N	1.00	NONE	4210	709
	NC 1-5	JPCP	9.0	9.0	30	0.00	N	0.00	---	5500	674
	NC 1-6	JPCP	9.0	9.5	30	0.00	N	0.00	---	5140	559
	NC 1-7	JRCP	8.0	8.0	60	0.17	N	1.00	NONE	5090	644
	NC 1-8	JPCP	9.0	8.9	30	0.00	N	0.00	---	4220	705
I-85 Greensboro, NC NB (1982- new)	NC 2	JPCP	11.0	11.0	18-25-23-19	0.00	N	1.38	P/G	5890	712
I-75 Tampa, FL NB (1986- new)	FL 2	JPCP	13.0	13.6	12-18-19-13	0.00	Y	1.25	P	5550	664
I-75 Tampa, FL SB (1982- new)	FL 3	JPCP	9.0	9.5	16-17-23-22	0.00	Y	1.00	EPOXY	4160	599
US 101, Geyserville, CA NB (1975)	CA 3-1	JPCP	9.0	8.5	12-13-19-18	0.00	Y	0.00	N/A	3530	N/A
	CA 3-2	JPCP	9.0	8.5	12-13-19-18	0.00	Y	0.00	N/A	4170	796
	CA 3-5	JPCP	9.0	9.0	12-13-19-18	0.00	Y	0.00	N/A	4380	842

Table 146. Base, subbase, subgrade, and outer shoulder design data for projects in wet-nonfreeze environmental zone.

Project Location (Year Constructed)	Project Section ID	Pvt. Type	BASE Type	BASE THICKNESS, IN Design	BASE THICKNESS, IN Core	BASE Estimated Perm., FT/HR	BASE Keff, (Dyn.) PCI	SUBBASE Type	SUBBASE THICKNESS, IN Design	SUBBASE THICKNESS, IN Core	SUBBASE Estimated Perm., FT/HR	SUBGRADE AASHTO Soil Type	SUBGRADE Estimated Perm., FT/HR	OUTER SHOULDER Type	OUTER SHOULDER THICKNESS, IN Surface	OUTER SHOULDER THICKNESS, IN Base
I-95 Rocky Mount, NC NB&SB (1967)	NC 1-1	JPCP	AGG	4.0	2.0	0.12	538	NONE	---	---	---	A-2-4	0.50	AC	3.0	5.0
	NC 1-2	JPCP	SC	6.0	6.0	0.00	347	NONE	---	---	---	A-2-4	0.50	AC	3.0	5.0
	NC 1-3	JPCP	SC	6.0	6.0	0.00	494	NONE	---	---	---	A-2-6	0.33	AC	3.0	5.0
	NC 1-4	JPCP	AGG	4.0	3.5	0.09	570	NONE	---	---	---	A-2-6	0.50	AC	3.0	5.0
	NC 1-5	JPCP	CTB	4.0	4.6	0.00	620	NONE	---	---	---	A-4	0.11	AC	3.0	5.0
	NC 1-6	JPCP	ATB	4.0	4.0	0.00	672	NONE	---	---	---	A-2-6	0.50	AC	3.0	5.0
	NC 1-7	JRCP	AGG	4.0	4.0	0.13	128	NONE	---	---	---	A-2-4	0.50	AC	3.0	5.0
	NC 1-8	JPCP	AGG	4.0	4.7	0.14	513	NONE	---	---		A-2-6	0.50	AC	3.0	5.0
I-85 Greensboro, NC NB (1982- new)	NC 2	JPCP	LCB	5.0	5.0	0.00	293	NONE	---	---	---	A-4	0.33	PCC	10.0	5.0
I-75 Tampa, FL NB (1986- new)	FL 2	JPCP	SAND	6.0	1.6	0.06	378	NONE	---	---	---	A-3	0.52	PCC	13.0	6.0
I-75 Tampa, FL SB (1982- new)	FL 3	JPCP	LCB	6.0	5.7	0.00	529	NONE	---	---	---	A-3	1.10	PCC	* 7.5	6.0
US 101, Geyserville, CA NB (1975)	CA 3-1	JPCP	CTB	5.4	5.0	0.00	286	AGG	6.0	5.5	13.40	A-4	0.11	PCC	* 7.5	11.0
	CA 3-2	JPCP	CTB	5.4	5.5	0.00	312	AGG	6.0	18.0	33.57	A-4	0.11	PCC	* 7.5	11.0
	CA 3-5	JPCP	CTB	5.4	5.5	0.00	397	AGG	6.0	3.5	2.05	A-4	0.11	PCC	* 7.5	11.0

* Average

Table 147. Pavement joint data for projects in wet-nonfreeze environmental zone.

Project Location (Year Constructed)	Project Section ID	Slab T, IN	Pvt. Type	Joint Spacing, FT	Base Type	TRANSVERSE JOINT Dowel Dia., IN	Calc Avg Jt Open, IN	Skewed Joints, Y/N	Jt Seal Shape Factor	JOINT SEALANT Type	Age	Cond	Depth of Long. Joint, IN
I-95 Rocky Mount, NC	NC 1-1	9.0	JPCP	30.0	AGG	0.00	0.10	Y	0.5	AC	20	GOOD	2.75
NB&SB (1967)	NC 1-2	9.0	JPCP	30.0	SC	1.00	0.08	N	0.5	AC	20	GOOD	2.75
	NC 1-3	9.0	JPCP	30.0	SC	0.00	0.08	N	0.5	AC	20	GOOD	2.75
	NC 1-4	9.0	JPCP	30.0	AGG	1.00	0.10	N	0.5	AC	20	GOOD	2.75
	NC 1-5	9.0	JPCP	30.0	CTB	0.00	0.08	N	0.5	AC	20	GOOD	2.75
	NC 1-6	9.0	JPCP	30.0	ATB	0.00	0.08	N	0.5	AC	20	EXC	2.75
	NC 1-7	8.0	JRCP	60.0	AGG	1.00	0.19	N	0.5	AC	20	FAIR	2.75
	NC 1-8	9.0	JPCP	30.0	AGG	0.00	0.10	N	0.5	AC	20	GOOD	2.75
I-85 Greensboro, NC NB (1982- new)	NC 2	11.0	JPCP	18-23-25-19	LCB	1.38	0.06	N	0.13	SIL	5	GOOD	3.5
I-75 Tampa, FL NB (1986- new)	FL 2	13.0	JPCP	12-19-18-13	SAND	1.25	0.04	Y	0.17	SIL	1	EXC	
I-75 Tampa, FL SB (1982- new)	FL 3	9.0	JPCP	16-17-23-22	LCB	1.00	0.03	Y	0.17	SIL	2	GOOD	
US 101 Geyserville, CA	CA 3-1	9.0	JPCP	12-13-19-18	CTB	0.00	0.04	Y	0.42	RA	12	GOOD	2.0
NB (1975)	CA 3-2	9.0	JPCP	12-13-19-18	CTB	0.00	0.04	Y	N/A	NONE	---	---	2.0
	CA 3-5	9.0	JPCP	12-13-19-18	CTB	0.00	0.04	Y	N/A	NONE	---	---	2.0

Table 148. Outer lane deflection data of projects in wet-nonfreeze environmental zone.

Project Location (Year Constructed)	Project Section ID	Slab T, IN	Pvt. Type	Joint Spacing, FT	Base Type	DEFLECTION, mils: Mid-Slab High	Mid-Slab Low	Mid-Slab Ave.	Loaded Corner	Unloaded Corner	Adjusted Percent LTE	% LT Across Shldr	Avg. NDT Test Temp, °F	Percent Corners with Voids
I-95 Rocky Mount, NC NB&SB (1967)	NC 1-1	9.0	JPCP	30.0	AGG	3.5	2.3	2.7	7.6	3.3	50	N/A	73	0
	NC 1-2	9.0	JPCP	30.0	SC	3.5	2.5	3.3	5.7	5.1	100	N/A	74	0
	NC 1-3	9.0	JPCP	30.0	SC	4.3	2.3	3.1	4.0	3.6	100	N/A	76	0
	NC 1-4	9.0	JPCP	30.0	AGG	2.9	2.4	2.6	8.0	4.5	59	N/A	67	15
	NC 1-5	9.0	JPCP	30.0	CTB	3.9	1.7	2.3	4.5	3.4	92	N/A	69	0
	NC 1-6	9.0	JPCP	30.0	ATB	2.4	1.9	2.2	4.6	2.3	58	N/A	70	0
	NC 1-7	8.0	JRCP	60.0	AGG	8.9	5.8	6.8	8.3	5.8	77	N/A	74	0
	NC 1-8	9.0	JPCP	30.0	AGG	3.3	2.5	2.9	11.8	1.7	16	N/A	61	40
I-85 Greensboro, NC NB (1982- new)	NC 2	11.0	JPCP	18-23-25-19	LCB	2.7	1.6	2.2	7.5	7.0	100	61	70	5
I-75 Tampa, FL NB (1986- new)	FL 2	13.0	JPCP	12-19-18-13	SAND	2.5	1.6	1.9	7.6	2.0	29	100	74	10
I-75 Tampa, FL SB (1982- new)	FL 3	9.0	JPCP	16-17-23-22	LCB	3.9	2.1	2.8	23.4	3.7	19	43	75	73
US 101 Geyserville, CA NB (1975)	CA 3-1	9.0	JPCP	12-13-19-18	CTB	5.0	3.9	4.3	3.8	2.7	79	70	68	0
	CA 3-2	9.0	JPCP	12-13-19-18	CTB	4.1	2.8	3.6	3.1	2.4	86	85	74	0
	CA 3-5	9.0	JPCP	12-13-19-18	CTB	5.5	2.0	3.4	6.0	3.9	74	55	62	12

Table 149. Outer shoulder and drainage information for projects in wet-nonfreeze environmental zone.

Project Location (Year Constructed)	Project Section ID	Slab T, IN	Pvt. Type	Joint Spacing, FT	Base Type	Outer Shoulder Type- Thickness Inches: Surface	Outer Shoulder Type- Thickness Inches: Base	Overall Shoulder Condition	Shoulder Jt Seal Condition	PERMEABILITY FT/HR: Base	PERMEABILITY FT/HR: Sub-base	AASHTO Drain. Coeff. Cd	Dowel Y/N	Sub-Drainage Type	Depth to Ditch FT	Average Trans. Slope, %	Average Longit. Grade, %
I-95 Rocky Mount, NC NB&SB (1967)	NC 1-1	9.0	JPCP	30.0	AGG	AC-3	AGG-6	GOOD	POOR	0.12	---	0.75	N	NONE	5	-1.39	0.17
	NC 1-2	9.0	JPCP	30.0	SC	AC-3	AGG-8	FAIR	POOR	0.00	---	0.80	Y	NONE	5	-0.70	0.00
	NC 1-3	9.0	JPCP	30.0	SC	AC-3	AGG-8	GOOD	GOOD	0.00	---	0.98	N	NONE	5	-1.21	0.69
	NC 1-4	9.0	JPCP	30.0	AGG	AC-3	AGG-6	GOOD	POOR	0.09	---	0.75	Y	NONE	5	-1.21	0.69
	NC 1-5	9.0	JPCP	30.0	CTB	AC-3	AGG-6	GOOD	POOR	0.00	---	0.75	N	NONE	5	-1.21	0.69
	NC 1-6	9.0	JPCP	30.0	ATB	AC-3	AGG-6	FAIR	POOR	0.00	---	0.90	N	NONE	5	-1.56	0.00
	NC 1-7	8.0	JRCP	60.0	AGG	AC-3	AGG-6	GOOD	POOR	0.13	---	0.90	Y	NONE	5	-1.91	0.00
	NC 1-8	9.0	JPCP	30.0	AGG	AC-3	AGG-6	GOOD	POOR	0.14	---	0.80	N	NONE	5	-1.56	0.35
I-85 Greensboro, NC NB (1982- new)	NC 2	11.0	JPCP	18-23-25-19	LCB	PCC-10	LCB-5	EXC	GOOD	0.00	---	0.95	N	NONE	3	-2.08	-1.39
I-75 Tampa, FL NB (1906- new)	FL 2	13.0	JPCP	12-19-18-13	SAND	PCC-12	SAND-6	EXC	EXC	0.06	---	0.80	Y	NONE	5	-1.56	0.00
I-75 Tampa, FL SB (1982- new)	FL 3	9.0	JPCP	16-17-23-22	LCB	PCC-7 *	ECC-6	FAIR	GOOD	0.00	---	0.75	Y	FAB/EDRN	5	-2.08	0.00
US 101 Geyserville, CA NB (1975)	CA 3-1	9.0	JPCP	12-13-19-18	CTB	PCC-7 *	AGG-11	GOOD	GOOD	0.00	13.4	0.90	N	NONE	N/A	0.69	2.08
	CA 3-2	9.0	JPCP	12-13-19-18	CTB	PCC-7 *	AGG-11	GOOD	POOR	0.00	33.57	1.00	N	NONE	N/A	-1.73	-2.43
	CA 3-5	9.0	JPCP	12-13-19-18	CTB	PCC-7 *	AGG-11	EXC	POOR	0.00	2.05	0.90	N	NONE	N/A	-1.91	1.04

* Average

Table 150. Traffic information for projects in wet-nonfreeze environmental zone.

Project Location (Year Constructed)	Project Section ID	ORIGINAL DESIGN TRAFFIC: ESAL's, (million)	ADT*, thous.	% Trucks	Age at Survey	1987 ESTIMATED: ADT, thous.	% Trucks	OUTER LANE (1): 1987 ESAL from ADT, % Trucks	Estimated ESAL's to Date	INNER LANE (2): 1987 ESAL from ADT, % Trucks	Estimated ESAL's to Date
I-95 Rocky Mount, NC	NC 1-1	2.4	8.1	16.0	20	19.1	9.0	495500	9137400	116200	1883500
NB&SB (1967)	NC 1-2	2.4	8.1	16.0	20	19.1	9.0	495500	9137400	116200	1883500
	NC 1-3	2.4	8.1	16.0	20	19.1	9.0	495500	9137400	116200	1883500
	NC 1-4	2.4	8.1	16.0	20	19.1	9.0	495500	9137400	116200	1883500
	NC 1-5	2.4	8.1	16.0	20	19.1	9.0	495500	9137400	116200	1883500
	NC 1-6	2.4	8.1	16.0	20	19.1	9.0	495500	9137400	116200	1883500
	NC 1-7	2.4	8.1	16.0	20	19.1	9.0	495500	9137400	116200	1883500
	NC 1-8	2.4	8.1	16.0	20	19.1	9.0	495500	9137400	116200	1883500
I-85 Greensboro, NC	NC 2		33.2	15.0	5	26.0	17.0	1105000	5755300	303400	1532100
NB (1982- new)											
I-75 Tampa, FL	FL 2		17.8		1	28.7	20.0	1085800	2005800	458000	824600
NB (1986- new)											
I-75 Tampa, FL	FL 3		46.4		5	32.7	20.0	1308600	5967200	581900	2465100
SB (1982- new)											
US 101 Geyserville, CA	CA 3-1				12	17.0	14.0	485700	3641200	106900	714500
NB (1975)	CA 3-2				12	17.0	14.0	485700	3641200	106900	714500
	CA 3-3				12	17.0	14.0	485700	3641200	106900	714500

* All ADT's are two-way

Table 151. Outer lane performance data for projects in wet-nonfreeze environmental zone.

Project Location (Year Constructed)	Project Section ID	Slab T, IN	Pvt. Type	Joint Spacing, FT	Base Type	Dowel Dia., IN	Ave. PSR	Mays Rough. IN/MI	Average Trans. Faulting IN	Deter. Trans. Cracks/Mile	Longit. Cracking, LIN FT/MI	Pumping, N/L/M/H	Percent Joints Spalled	Mtls Dur. Dist
I-95 Rocky Mount, NC NB&SB (1967)	NC 1-1	9.0	JPCP	30.0	AGG	0.00	3.4	83	0.12	5	0	N	0	NONE
	NC 1-2	9.0	JPCP	30.0	SC	1.00	3.5	79	0.16	10	1451	N	0	NONE
	NC 1-3	9.0	JPCP	30.0	SC	0.00	3.6	77	0.13	5	3068	N	0	NONE
	NC 1-4	9.0	JPCP	30.0	AGG	1.00	3.4	85	0.13	0	0	N	0	NONE
	NC 1-5	9.0	JPCP	30.0	CTB	0.00	3.2	105	0.16	0	179	N	0	NONE
	NC 1-6	9.0	JPCP	30.0	ATB	0.00	3.8	69	0.05	0	0	N	0	NONE
	NC 1-7	8.0	JRCP	60.0	AGG	1.00	3.4	85	0.15	0	295	N	0	NONE
	NC 1-8	9.0	JPCP	30.0	AGG	0.00	3.7	95	0.22	64	0	N	0	NONE
I-85 Greensboro, NC NB (1982- new)	NC 2	11.0	JPCP	18-23-25-19	LCB	1.38	4.2	64	0.02	10	0	H	0	NONE
I-75 Tampa, FL NB (1986- new)	FL 2	13.0	JPCP	12-19-18-13	SAND	1.25	3.7	64	0.01	0	0	N	0	NONE
I-75 Tampa, FL SB (1982- new)	FL 3	9.0	JPCP	16-17-23-22	LCB	1.00	3.2	84	0.08	300	440	M	2	NONE
US 101 Geyserville, CA NB (1975)	CA 3-1	9.0	JPCP	12-13-19-18	CTB	0.00	3.6	134	0.08	27	0	N	2	RAGG
	CA 3-2	9.0	JPCP	12-13-19-18	CTB	0.00	3.9	104	0.11	6	0	N	3	RAGG
	CA 3-5	9.0	JPCP	12-13-19-18	CTB	0.00	3.8	102	0.10	123	34	N	6	RAGG

Table 152. Lane 2 performance data for projects in wet-nonfreeze environmental zone.

Project Location (Year Constructed)	Project Section ID	Slab T, IN	Pvt. Type	Joint Spacing, FT	Base Type	Dowel Dia., IN	PSR	Mays Rough. IN/MI	Average Trans. Faulting IN	Deter. Trans. Cracks/Mile	Longit. Cracking, LIN FT/MI	Pumping N/L/M/H	Percent Joints Spalled	Mtls Dur. Dist
I-95 Rocky Mount, NC	NC 1-1	9.0	JPCP	30.0	AGG	0.00	3.8	74	0.02	0	0	N	0	NONE
NB&SB (1967)	NC 1-2	9.0	JPCP	30.0	SC	1.00	3.4	65	0.04	0	0	N	0	NONE
	NC 1-3	9.0	JPCP	30.0	SC	0.00	3.4	70	0.09	0	0	N	0	NONE
	NC 1-4	9.0	JPCP	30.0	AGG	1.00	3.4	85	0.03	0	0	N	0	NONE
	NC 1-5	9.0	JPCP	30.0	CTB	0.00	3.4	04	0.08	0	0	N	0	NONE
	NC 1-6	9.0	JPCP	30.0	ATB	0.00	3.4	74	0.04	0	0	N	0	NONE
	NC 1-7	8.0	JRCP	60.0	AGG	1.00	3.4	85	0.05	0	0	N	5	NONE
	NC 1-8	9.0	JPCP	30.0	AGG	0.00	3.3	69	0.07	5	0	N	0	NONE
I-85 Greensboro, NC	NC 2	11.0	JPCP	18-23-25-19	LCB	1.38	N/A	N/A	N/A	5	0	N	0	NONE
NB (1982- new)														
I-75 Tampa, FL	FL 2	13.0	JPCP	12-19-18-13	SAND	1.25	N/A	N/A	N/A	0	0	N	0	NONE
NB (1986- new)														
I-75 Tampa, FL	FL 3	9.0	JPCP	16-17-23-22	LCB	1.00	N/A	N/A	N/A	250	460	N	2	NONE
SB (1982- new)														
US 101 Geyserville, CA	CA 3-1	9.0	JPCP	12-13-19-18	CTB	0.00	3.8	111	N/A	5	0	N	0	RAGG
NB (1975)	CA 3-2	9.0	JPCP	12-13-19-18	CTB	0.00	4.1	82	N/A	0	0	N	0	RAGG
	CA 3-5	9.0	JPCP	12-13-19-18	CTB	0.00	3.9	101	N/A	28	0	N	2	RAGG

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