Performance of Continuously Reinforced Concrete Pavements Volume II: Field Investigations of CRC Pavements

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FOREWORD

This report is one volume of a seven-volume set presenting the results of a study to provide the state-of-the-art for the design, construction, maintenance and rehabilitation of Continuously Reinforced Concrete Pavements (CRCP). Through a through literature review of current and past research work in CRCP and extensive field and laboratory testing of 23 in-service CRC pavements, the effectiveness of various design and construction features were assessed; performance of CRCP was evaluated; and procedures for improving CRC pavement technology were recommended. The 23 test pavements were located in six states that participated in this national pooled fund study. In addition the data available for 83 CRCPs included in the General Pavement Study (GPS) number 5 of the Long Term Pavement Performance (LTPP) Program was presented and analyzed. A number of CRCP maintenance and rehabilitation techniques that have been used over the years, including joint and crack sealing, cathodic protection of reinforcing bars, full-depth patching, resurfacing, etc., were also evaluated. This report will be of interest to engineers and researchers concerned with the state-of-the-art design, construction, maintenance and rehabilitation of CRCP including predictive models. The study was made possible with the financial support of Arizona, Arkansas, Connecticut, Delaware, Illinois, Iowa, Louisiana, Oklahoma, Oregon, Pennsylvania, South Dakota, Texas and Wisconsin.

Sufficient copies of this report are being distributed to provide two copies to each FHWA regional office and three copies to each FHWA division office and each state highway agency. Direct distribution is being made to the division offices. Additional copies for the public are available from the National Technical Information Service (NTIS), United States Department of Commerce, 5285 Port Royal Road, Springfield, Virginia 22161.

Charles J. Nemmers, P.E.
Director, Office of Engineering
Research and Development

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16. Abstract

This report is one of a series of reports prepared as part of a recent study sponsored by the Federal Highway Administration (FHWA) aimed at updating the state of the art of the design, construction, maintenance, and rehabilitation of continuously reinforced concrete (CRC) pavements. The scope of work of the FHWA study included the following:

1. Conduct a literature review and prepare an annotated bibliography on CRC pavements and CRC overlays.
2. Conduct a field investigation and laboratory testing related to 25 existing inservice pavement sections. This was done to evaluate the effects of various design features on CRC pavement performance, to identify any design or construction related problems, and to recommend procedures to improve CRC pavement technology.
3. Evaluate the effectiveness of various maintenance and rehabilitation strategies for CRC pavements.
4. Prepare a summary report on the current state of the practice for CRC pavements.

Each of the above four items is addressed in a separate report. The following reports have been prepared under this study:

Performance of Continuously Reinforced Concrete Pavements
- Volume I - Summary of Practice and Annotated Bibliography
- Volume II - Field Investigation of CRC Pavements
- Volume III - Analysis and Evaluation of Field Test Data
- Volume IV - Resurfacings for CRC Pavements
- Volume V - Maintenance and Rehabilitation of CRC Pavements
- Volume VI - CRC Pavement Design, Construction, and Performance
- Volume VII - Summary

This report is volume II in the series and presents the details related to the field investigations of selected CRC pavement sections and data analyses.

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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

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

APPROXIMATE CONVERSIONS FROM SI UNITS

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

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

(Revised September 1993)

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CHAPTER 1 - INTRODUCTION

General

Continuously reinforced concrete (CRC) pavement is portland cement concrete (PCC) pavement with continuous longitudinal steel reinforcement with no intermediate transverse expansion or contraction joints. The continuous joint-free length of CRC pavements can extend to several thousand meters with breaks provided only at structures. Terminal anchorage is provided at the ends of the CRC pavement to restrain length changes caused by temperature variations and drying shrinkage of concrete. The CRC pavements develop a random cracking pattern with cracks generally spaced at about 0.9 to 2.4 m (3 to 8 ft). The cracking pattern is governed by the environment conditions at the time of construction, the amount of steel, and concrete strength. The steel reinforcement restrains the opening of the cracks. Also, the higher the amount of steel reinforcement, the more closely spaced the cracks will be. Most of the cracks form shortly after construction but additional cracking may develop over the next few years as a result of continued drying shrinkage of concrete, temperature variations, and traffic loading.

Although CRC pavement can be traced back to the late 1930's, the extensive use of CRC pavements began in the early 1960's during the heydays of the U.S. Interstate System construction program. Currently, there are over 45 061 lane km (28,000 lane mi) of CRC pavements in the United States with pavements constructed in at least 35 States. CRC pavements are one of the few pavement types that can truly provide the ideal zero-maintenance pavement if they are designed and constructed properly. Many older CRC pavements are considered to have been under-designed leading to premature failures when subjected to ever increasingly heavy truck traffic.

A major concern with CRC pavement is punchout distress. The definition of punchout distress is the area enclosed by two closely spaced (usually less than 0.6 m (2 ft) transverse cracks, a short longitudinal crack, and the edge of the pavement or a longitudinal joint. It also includes "Y" cracks that exhibit spalling, breakup, and faulting. Other distresses associated with punchouts include spalling along transverse cracks and faulting. Other leading causes of CRC pavement failure are wide (and spalled) transverse cracks caused by steel rupture and spalling of concrete caused by steel corrosion in the presence of heavy deicing salt applications in the northern States. The punchout distress is related to crack spacing, pavement thickness, poor foundation support, and heavy truck loadings. The repair of punchout distress typically consists of full-depth patches. With time, as the number of full-depth patches increase, the pavement may be resurfaced with asphalt concrete (AC) or PCC or it may be reconstructed.

Over the years, many State agencies have conducted research studies to develop better understanding of the effects of various design and construction features on the performance of CRC pavements. A large number of these studies have focused on pavement thickness, concrete aggregate type, amount of steel reinforcement, and base/subbase type. Studies have also been conducted to address the benefits of using epoxy-coated reinforcement and the effectiveness of permeable treated base layers under CRC pavements.

This report is one of a series of reports prepared as part of a recent study administered by the Federal Highway Administration (FHWA) aimed at updating the state of the art of the design, construction, maintenance, and rehabilitation of CRC pavements. The study is a national pooled-fund study with participation by Arizona, Arkansas, Connecticut, Delaware, Illinois, Iowa, Louisiana, Oklahoma, Oregon, Pennsylvania, South Dakota, and Texas. The scope of work of the FHWA study included the following:

1. Conduct a literature review and prepare an annotated bibliography on CRC pavements and CRC overlays.

2. Conduct a field investigation and laboratory testing related to 23 existing in-service pavement sections. This was done to evaluate the effects of various design features on CRC pavement performance, to identify any design or construction related problems, and to recommend procedures to improve CRC pavement technology.

3. Evaluate the effectiveness of various maintenance and rehabilitation strategies for CRC pavements.

4. Prepare a summary report on the current state of the practice for CRC pavements.

Each of the above four items is addressed in a separate report. The following reports have been prepared under this study:

Volume I - Summary of Practice and Annotated Bibliography
Volume II - Field Investigation of CRC Pavements
Volume III - Analysis and Evaluation of Field and Laboratory Test Data
Volume IV - Maintenance and Rehabilitation of CRC Pavements
Volume V - Resurfacings for CRC Pavements
Volume VI - Synthesis of Recommended Practice

This report is volume II in the series and presents the details related to the field investigations of selected CRC pavement sections and data analyses.

Objectives of the Field Investigations

The specific objective of the field investigation was to conduct necessary field investigations and laboratory testing at 20 to 30 existing CRC pavement sections to evaluate the effects of standard new design features and different rehabilitation treatments on CRC pavement performance.

After a detailed evaluation of available project sites in conjunction with participating State highway agencies, 23 project sites were selected as follows:

Illinois	-	5 sites
Iowa	-	3 sites
Oklahoma	-	5 sites
Oregon	-	3 sites
Pennsylvania	-	2 sites
Wisconsin	-	5 sites

At each site, performance of a representative 305-m (1,000-ft) length section was evaluated by performing visual condition surveys, profile measurements, falling weight deflectometer (FWD) testing, and corrosion-related testing. In addition, concrete cores were obtained for strength, stiffness (modulus of elasticity), and coefficient of thermal expansion testing. Samples of base, subbase, and subgrade were also obtained for material characterization. For each project site, available inventory type data related to design, construction, maintenance, performance, and traffic was collected from State highway agencies.

None of the project site included rehabilitated pavement sections. Because of the limited budget for field investigations, it was considered that the most benefits would be obtained by focusing on existing original pavement sections.

The work plan for the field investigation consisted of the appropriate data collection for each test section and conduct of the following type of data analysis:

1. *Project Level Evaluation* - Each project would be examined to identify cause-effect relationship. Also, the performance and characteristics of specific groups of projects would be examined and compared.

2. *Crack Spacing Simulations* - Existing crack spacing models would be used to verify the reasonableness of the models in predicting the crack pattern in CRC pavements.

3. *Structural Analysis* - The FWD data would be used to characterize the structural capacity of the CRC pavements (CRCP). Load transfer effectiveness at cracks would be examined.

4. *Distress Modeling* - The validity of the existing CRCP distress models would be examined using project specific data from the CRCP projects.

5. *Corrosion Analysis* - For projects in the northern States, corrosion-related testing would be performed to determine the level of corrosion and to determine the effects of various design, construction, and climatic features on the level of corrosion.

One focus of the data analysis was to try to identify how specific design and construction features affect pavement performance. It should be noted that the study objectives were not to develop distress or performance models with global applicability as it was realized at the onset that the limited number of projects considered would not provide the necessary foundation for that. It should also be noted that the primary factors that affect the performance of CRC pavements are thickness and crack spacing. Thickness can be controlled as a design factor. However, crack spacing and the consequent crack width cannot be controlled directly and an ideal workable crack spacing can only be attempted by manipulating various design factors and hoping that placement conditions would be favorable. Although, past experience has indicated that crack spacing in the range of 0.9 to 2.4 m (3 to 8 ft) (and possibly around 1.2 to 1.5 m (4 to 5 ft)) are considered acceptable, we are still not able to achieve the desired spacings with certainty. The actual crack spacings also follow a random pattern whereby on a given length of a CRC pavement, crack spacing may range from 0.6 m (2 ft) to over 2.4 m (8 ft) with numerous instances of closely spaced or cluster cracking.

Thus, one of the primary focus in this study was to identify the critical factors that influence crack spacing in CRC pavements. Also, an attempt was made to correlate actual crack spacing to the performance of the pavement in terms of structural capacity, ride, and extent and severity of distress.

Study Details

A technical advisory committee (TAC) consisting of selected State highway agency representatives provided a forum for review of the work plans for the study, specifically for the field investigation portion of the study. The field investigation work plan was presented at a 2-day meeting of the TAC, and the plan, as modified based on TAC comments, was implemented.

The final field investigation plan, as revised, is presented in chapter 2. Chapters 3 to 8 present data for each test section grouped according to the appropriate State. Only a limited section level data is presented in this report. Volume III incorporated a more detailed global data analysis to identify specific factors that affect CRC pavement performance.

CHAPTER 2 - FIELD AND LABORATORY INVESTIGATION PLAN

Introduction

As indicated previously, the funding limitations precluded a very comprehensive field evaluation of CRC pavements. Field investigation and related laboratory testing were completed at 23 projects in several States. It was recognized that ideally a larger number of projects are needed to be evaluated to allow a robust and comprehensive analyses of performance related data. However, the project's funding level allowed only a small scale evaluation of CRC pavements. Therefore, a more focused approach was taken to develop the field and laboratory investigation plan. Specifically, the approach adopted was to evaluate CRC pavements with standard and conventional features and compare them with those incorporating new/improved features. Based on discussions with FHWA, the following key CRC pavement features were identified for possible evaluation in the field:

1. *Thickness* - 279 mm (11 in) or greater thickness.
2. *Reinforcement* - epoxy coated reinforcement; amount of reinforcement.
3. *Drainage Layer* - permeable bases.
4. *Edge Strengthening* - Tied shoulder or widened lane.

The scope of work also included review and analysis of data being collected for the 85 CRC pavement test sections that are being monitored as part of the GPS-5 experiment of the Strategic Highway Research Program (SHRP) initiated Long-Term Pavement Performance (LTPP) program.

For each project evaluated in the field, all available data related to design, construction, maintenance, rehabilitation, and performance were collected. This was done to complement the information and to extend the study data base. A comprehensive data base was developed to incorporate all CRC projects evaluated as part of the study.

Site Identification and Selection

Based on the literature review and contacts with FHWA and several State highway agencies, a large number of candidate CRCP projects were identified for field testing. Site verification and preliminary technical and background information on the projects were provided by State contacts. These projects are listed in table 2.

Test Section Requirements

To aid in the final selection of candidate test sections within the CRCP projects that were identified, the following guidelines were used:

1. The project should be original construction. However, rehabilitated CRC pavements were to be included in the study if they were nearby other core projects. However, no rehabilitated CRCP pavements were identified as being

Table 1. Candidate CRC pavement test sites.

State	Priority	Project Site No.	Route	Layer Thickness (in.) & Type			Year Open	Location, if needed	Features				Comments	Written Approval and Verification
				CRC	Base	Subbase			A	B	C	D		
Arizona*	34	1	SH 101L	9 & 10	ATB		1987	West side of Phoenix-Sun City	X					
California	37	2	I 5	8.4	5.4	12	1971	Tracy					• 3 experimental sections (1 GPS site) – mesh – longitudinal & transverse – longitudinal	
	38	3	I 80	8	4		1949	Fairfield					• Experimental section – 0.5% – 0.65%	
Connecticut*	19	4	I 84	8	Gravel		1980	Vernon to State Lane					• CRC overlay and widening • Outside lane only • SHRP section on different contract • Conn. has located extension historical data performance	
Delaware*	24	5	I 495	9	4 - CTB	4	1978	Wilmington						X
	23	6	US 1	8	4 - CTB		1970	Millford Bypass					• steel wire mesh: D19 @ 4 in. - longitudinal D6 @ 12 in. - transverse	X
Illinois*	2	7	US 50	7, 8, & 9	4 - Econocrete		1986	60 miles east of St. Louis		X			• PCC shoulders tapered to 6 in. • Various CRCP thicknesses – experimental section • WIM installed	X
	12	8	US 36	8	ATB, RCC, & Lime Treated		late 70's	South of Springfield					• Experimental section – various base types	X
	13	9	I 72	8	ATB, RCC, Pozzolanic			Springfield to Champaign					• Various base types – various contracts	X
	25	10	I 55	8	ATB, RCC, Pozzolanic			M.P. 40-217					• WIM installed • Various base types	X
	1	11	US 51					South of LaSalle			X		• Open graded base	X
Iowa*	27	12	I 35	8	4 - ATB		1974	M.P. 143.94 - 177.51					• Good aggregate • ATB shoulder	

Features: **A: D ≥ 11 inches** **B: Tied PCC Shoulders** **C: Epoxy Coated Reinforcement** **D: Permeable Bases** (25.4 mm = 1 in) (1.6 km = 1 mi)

* Pooled Fund Participant

Table 1. Candidate CRC pavement test sites (continued).

State	Priority	Project Site No.	Route	Layer Thickness (in.) & Type			Year Open	Location, if needed	Features				Comments	Written Approval and Verification
				CRC	Base	Subbase			A	B	C	D		
	15	13	I 29	8	4 - CTB		1971	M.P. 0.0 - 19.07					• D-Cracking • ATB shoulder	
	26	14	I 380	8	4 - ATB		1974	M.P. 14.71 - 16.98					• D-Cracking • ATB shoulder	
Louisiana*	3	15	I 10	8	GSB, CTB, ATB		1976	Between Baton Rouge & New Orleans					• CRC concrete shoulders • Shell aggregate	
Maryland	22	16	I 95	9	6 - GSB		1968	Prince George's County					• 6 in. longitudinal underdrains	X
Minnesota	33	17	I 35E	8	3-6 - GSB		1970	Vadnais Heights M.P. 116-117					• Cathodic protection (1980)	X
New Jersey	30	18	US 130				1947	Hightstown					• HRB proceedings 1947	
Ohio	20	19	SH 2	9	4		1974	Station 162+40 - 169+60					• D-Cracking study - various aggregate sizes • Easily accessible	
	31	20	I 270	8	4		1972						• Recent maintenance operations	
Oklahoma*	10	21	I 40	9	4 - Coarse Agg Bit Base		1986	Okfuskee County		X			• Full depth PCC shoulder • May not provide traffic control	X
	9	22	US 69	9	3 - Type "A" AC	12 - GSB	1987	Atoka County		X			• Full depth PCC shoulder • May not provide traffic control	X
	36	23	I 35	8	4 - FABB		1970	Carter County					• May not provide traffic control	X
	9	24	US 69	9	6 - Soil AC	6 - Select Borrow	1985	Bryan County		X			• Full depth PCC shoulder • May not provide traffic control	X
	10	25	I 40	10	4 - Open Graded CTB	12 - Select Borrow	1991	Sequoyah County				X	• Constructed in 1990 • May not provide traffic control	X
Oregon*	4A	26	I 205	11	4 - Treated	6 - Lime Treated Subgrade	1982	M.P. 17.69 - 19.01 Portland	X				• Urban area	X
	11	27	I 5	13	10 - GSB		1984	M.P. 174.73 - 187.85 (SB) Eugene	X				• Outside lane only	X
	11	28	I 5	10	9 - CTB		1986	M.P. 174.73 - 187.85 (NB) Eugene						X
	4B	29	I 205	8	4 - CTB		1975	M.P. 11.34 - 15.02 Portland					• Urban area	X

Features: A: D ≥ 11 inches B: Tied PCC Shoulders C: Epoxy Coated Reinforcement D: Permeable Bases (25.4 mm = 1 in) (1.6 km = 1 mi)

* Pooled Fund Participant

Table 1. Candidate CRC pavement test sites (continued).

State	Priority	Project Site No.	Route	Layer Thickness (in.) & Type			Year Open	Location, if needed	Features				Comments	Written Approval and Verification
				CRC	Base	Subbase			A	B	C	D		
Pennsylvania	7	30	I 180	9 & 10	12		1975-79	Williamsport		X			• Various contracts • Concrete shoulders • May charge for traffic control	X
	8	31	I 78	12	5 - LCB	6	1989	Allentown	X	X			• > 11 in. • Concrete shoulders • May charge for traffic control	X
South Dakota*	16	32	I 29	8	ATB		1972	Brookings					• Alkali-silica reaction • Pavement holding together - no loss of material	X
	17	33	I 90	8	Lime Treated		1971-73	Chamberlain					• Alkali-silica reaction • Pavement holding together - no loss of material	X
Texas*	35	34	SH 6	11	1 - AC Bond Breaker	CTB		Houston					• Various steel percentages • Two types of aggregate • Urban area	
	35	35	SH 6	8			1972	Houston					• Urban area • Spalling	
	18	36	I 45	8			1968						• Possibly overlaid • 0.5% steel	
	14	37	US 290	13	1 - Bond Breaker	CTB	1989						• Two layers of steel • Two aggregate types	
Virginia	21	38	I 64	8	5	6 Soil Cement	1970	Rt 250 to 0.3 mile west of Pauhem Road					• Alkali-silica reaction	
	32	39	I 95	8	6		1982	Greensville County					• Difficult traffic control	
Wisconsin	28	40	US 53	8	GSB		1973	New Auburn to US 8					• Cath. protection recently placed • 0.61% steel placed with tubes	X
	29	41	I 94	8	GSB		1973	Wisconsin River Bridge Approach					• Cath. protection recently placed • 0.61% steel placed with tubes	X
	5	42	I 90/94	8	GSB		~1985	North of Madison			X		• Epoxy coated steel reinforcement • 0.61% steel	X
	6	43	I 43	8	GSB		1978-85				X		• Epoxy coated steel reinforcement • 0.65% steel	X

Features: A: D ≥ 11 inches B: Tied PCC Shoulders C: Epoxy Coated Reinforcement D: Permeable Bases (25.4 mm = 1 in) (1.6 km = 1 mi)

* Pooled Fund Participant

adjacent to the core projects and therefore rehabilitated CRCP pavements were not tested.

2. The test section should have at least 305-m (1,000-ft) length of uniform construction. The subgrade should ideally be uniform over the length of the test sections.

3. The traffic over the test section should preferably be in excess of 200,000 equivalent single axle loads (ESAL)/year.

4. Sufficient project related data would be available from State records to allow rational evaluation of the test sections.

5. The test section was part of an experimental project.

6. For the test sections with new design and construction features, a control section with standard design and construction features was available.

7. The test section exhibited unique performance characteristics, for example, premature development of severe distresses such as punchout.

8. It would be possible to divert traffic from the outside lane of the test section for a period of 12 hours during the day of testing.

Because project scope and funding would allow field investigation at only a limited number of sites (about 25), a scheme was developed to prioritize the identified test sections. CRC pavement sections incorporating improved design features (thicker slab, epoxy-coated reinforcement, drainage layer, and edge strengthening) were given the first priority. Then sections within a state having the first priority sections were given as high a priority to allow cost-effective testing of pavement sections within a State. The remaining sections were prioritized based on the number of test sections available within a State, the number of experimental features at a given project, and the desire to incorporate a broader geographical base.

Table 1 shows the ranking of the identified CRC test sections. Table 2 provides a list of the prioritized sections arranged in order of descending priority. Rehabilitated CRC projects were not considered for the field evaluation. Thus, the field evaluation, was to be limited to original pavements ranging in age from 1 to 23 years. The final list of the test sections selected for field evaluation is given in table 3. As shown in table 3, the selected test sections incorporate a broad range of attributes of interest as follows:

- Thickness - ranging from 203 to 330 mm (8 to 13 in).
- Epoxy coated reinforcement - 3 sections.
- Permeable base - 2 sections.
- Age - ranging from 0.3 to 22 years.

Table 2. Prioritized list of test sections.

Prioritized Section	State	ID	Prioritized Section	State	ID
1	Illinois	11	16	South Dakota	32
2	Illinois	7	17	South Dakota	33
3	Louisiana	15	18	Texas	36
4	Oregon	26, 29	19	Connecticut	4
5	Wisconsin	42	20	Ohio	19
6	Wisconsin	43	21	Virginia	38
7	Pennsylvania	30	22	Maryland	16
8	Pennsylvania	31	23	Delaware	6
9	Oklahoma	22, 24	24	Delaware	5
10	Oklahoma	25, 21	25	Illinois	10
11	Oregon	27, 28	26	Iowa	14
12	Illinois	8	27	Iowa	12
13	Illinois	9	28	Wisconsin	40
14	Texas	37	29	Wisconsin	41
15	Iowa	13	30	New Jersey	18

State	Section ID	Cumulative No. of Sections
Illinois	11, 7, 8, 9, 10	5
Louisiana	15	5
Oregon	26, 29, 27, 28	10
Wisconsin	42, 43, 40, 41	14
Pennsylvania	30, 31	16
Oklahoma	22, 24, 25, 21	20
Texas	37, 36	22
Iowa	13, 14, 12	25
South Dakota	32, 33	27
Connecticut	4	28
Ohio	19	29
Virginia	38	30
Maryland	16	31
Delaware	6, 5	33
New Jersey	18	34

Table 3. Final list of test sections.

Test Section ID	Route	Nearest Mile Post	Direction	No. of Lanes	Age as of Fall 1991 Testing, years	SHRP Climatic Region	Terminal Joint Type	Design Thickness, in.	Subgrade Type (AASHTO)	Base Type	Outside Shoulder Type	Long. Steel Amount, %	Steel Placement Method	Epoxy Coated Steel	1,991 2-Way AADT	Design Lane Cumul. ESALs upto 9/91
IL-1	US51	na	SB	4	0.3	wf	wide flange	10	A-7-6	perm. ctb	pcc	0.7	chair	no	na	180,000
IL-2	I72	45	WB	4	15	wf	lug	8	A-6	ctb	ac	0.59	tube	no	7,500	4,800,000
IL-3	US36	st. 5+30	EB	4	20	wf	lug	8	A-7-5	atb	ac	0.6	chair	no	17,700	4,800,000
IL-4	I55	86	EB	6	20	wf	lug	8	A-7-5	atb	ac	0.6	tube	no	17,700	13,700,000
IL-5	US50	na	WB	4	5	wf	wide flange	8	A-7-5	lcb	pcc	0.7	chair	no	na	300,000
IA-1	I29	18	NB	4	20	wf	lug	8	A-2-6	ctb	ac	0.65	tube	no	7,500	3,700,000
IA-2	I80	15	WB	4	22	wf	lug	8	A-6	atb	ac	0.65	tube	no	12,700	8,850,000
IA-3	I380	15	NB	4	15	wf	lug	8	A-6	atb	pcc	0.65	tube	no	27,700	5,300,000
OK-1	I40	231	WB	4	4	wnf	wide flange	9	A-6	atb	pcc	0.5	chair	no	15,000	na
OK-2	US69	na	NB	4	5	wnf	wide flange	9	A-6	atb	pcc	0.5	chair	no	10,000	na
OK-3	I35	148	NB	4	3	wnf	wide flange	10	A-4	atb	pcc	0.5	chair	yes	30,000	na
OK-4	US69	na	SB	4	7	wnf	wide flange	9	A-6	soil-asphalt	pcc	0.5	chair	no	10,000	na
OK-5	I40	299	EB	4	2	wnf	wide flange	10	A-2-6	perm. ctb	pcc	0.61	chair	no	13,000	na
OR-1	I5	184	SN	4	7	wnf	wide flange	13	A-4	granular	ac	0.6	tube	no	29,700	11,300,000
OR-2	I5	184	NB	4	4	wnf	wide flange	10	A-4	ctb	ac	0.6	tube	no	30,300	3,000,000
OR-3	I205	na	SB	4	20	wnf	lug	8	A-6	ctb	ac	0.54	tube	no	59,000	30,000,000
PA-1	I180	23	EB	4	15	wf	wide flange	9	A-2-4	granular	ac	0.45	tube	no	na	na
PA-2	I81	Rt 11	NB	4	22	wf	lug	9	A-2-4	granular	pcc	0.55	chair	no	na	na
WI-1	I43	31	NB	4	18	wf	lug	8	A-2-4	granular	ac	0.65	chair	no	na	na
WI-2	I90	180	EB	4	6	wf	wide flange	10	A-2-4	granular	pcc	0.67	tube	yes	na	na
WI-3	I90/94	136	NB	6	7	wf	lug	10	A-2-7	granular	pcc	0.67	tube	yes	na	na
WI-4	I90/94	111	WB	6	7	wf	na	10	A-2-6	granular	pcc	0.67	tube	no	na	na
WI-5	I90/94	99	EB	4	16	wf	lug	8	A-2-6	granular	ac	0.61	chair	no	na	na
Average					11.3			9.0				0.6				
Std Dev					7.4			1.2				0.1				
Maximum					22.0			13.0				0.7				
Minimum					0.3			8.0				0.5				

- Subgrade - both coarse and fine grained soils.
- Base - cement treated base (CTB), lean concrete base (LCB), asphalt treated base (ATB), and granular.
- Steel amount - 0.45 to 0.7 percent.
- Shoulder type - 11 AC and 12 PCC.
- Climatic region - wet-freeze and wet-no-freeze.

All testing was performed during the fall of 1991.

Field Data Collection Plan

The field data collection program was aimed at collecting data on the current condition of each test section. The following activities were completed at most of the test sections:

1. Visual condition survey.
2. Nondestructive deflection testing.
3. Profile testing.
4. Corrosion related testing.
5. Coring and shallow borings.
6. Reinforcing steel location survey.
7. Photographic and video imaging.

It should be noted that except for the Oregon testing, all field testing was performed by the contractor staff. In Oregon, the State highway agency performed deflection testing and the coring and boring operations. No profile testing was done for the Oregon sites.

All field testing was accomplished during 1 day of testing with the test crew arriving at the site at dawn and staying at the site until early evening. At a few sites, testing was delayed because of rain.

The crew consisted of one project engineer (also operated the profiler), one FWD operator, and two technicians (for coring and boring and other site activities).

Traffic Control

At most test sections, the State highway agency provided traffic control. At few sites, traffic control activities were subcontracted to a local firm. Traffic control basically consisted of the outside lane closure for about 0.8 km (½ mi) in length incorporating the 305-m (1,000-ft) long test section. Appropriate barriers and traffic cones were placed along the blocked sections. Traffic control was generally put in place by about 8 a.m. on the day of testing and was available until late afternoon.

Data Collection Procedures

The details of field data collection procedures used are given in appendix A.

Data Compilation and Data Analysis

The following chapters present the detailed data for each section. Test sections for each State are grouped within a chapter. For each section, summary data is presented for the following categories of data:

1. Inventory
   - Location and climatic features.
   - Traffic, if available.
   - Structural section.
   - Design and construction, if available.
   - Performance, if available.
   - Maintenance and rehabilitation, if available.

2. Visual Condition Survey
   - Map showing crack spacing within the 305-m (1,000-ft) test section.
   - Crack spacing summaries.
   - Drainage survey summary.
   - Overall windshield survey summary.
   - Terminal joint survey summary.

3. Deflection Testing
   - Basin deflection data for each of the seven sensors along the 305-m (1,000-ft) length of the test section.
   - Slab temperature profile data.
   - Results of load transfer testing at cracks.
   - Average and range of deflections for each sensor for testing conducted at cracks, both at mid-slab and edge locations and for morning and afternoon testing.
   - Backcalculated radius of relative stiffness, ℓ, modulus of subgrade reaction, k, and slab rigidity, D, along the length of the test section for basin, mid-slab crack, and edge crack locations. Backcalculation was performed using program ILLI-BACK[1].

4. Crack Width Measurements
   - Summary of crack width changes between morning and afternoon measurements.

5. Laboratory Testing
   - Concrete test data.

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[1] Ioannides, A.M., "Program ILLI-BACK - A Closed-Form Backcalculation Procedure for Rigid Pavements," Copyright 1988. (Program commercially distributed by A.M. Ioannides, Ph.D., Savoy, Illinois)

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CHAPTER 3 - ILLINOIS TEST SECTIONS

Section IL-1 - US Highway 51, Illinois

Introduction

Section IL-1 was constructed during 1990 and 1991 and was opened to traffic in 1991. IL-1 is a section of US-51 that runs north to south from Cairo to Rockford, Illinois, as shown in figure 1. The length of the highway that was considered is between station 513 to 550. A 305-m (1,000-ft) south bound section was selected for detailed inspection.

The climate in this region is typically continental with warm summers and fairly cold winters. There are no wet and dry seasons. The highest monthly average temperature is 24.7°C (76.5°F) during July, and the lowest monthly average is -4.1°C (24.6°F) during January. US-51 lies within the wet-freeze environmental zone.

Traffic

Since its opening, the pavement has experienced an estimated 0.18 million ESALs in the design lane. The directional split for 1991 was 50 percent. The truck percentage was 30. US-51 was designed for a traffic level of 12,300 vehicles per day.

Structure

US-51, is a four-lane divided highway. The pavement lane width is 3.7 m (12 ft) with an outside shoulder width of 3.0 m (10 ft) and an inside shoulder width of 1.8 m (6 ft). The shoulders are constructed of jointed PCC.

US-51 was designed as a four-layer system comprised of a 254-mm (10-in) thick continuously reinforced concrete pavement (CRCP), a 102-mm (4-in) thick cement treated open graded base course or an econocrete depending on the section, and a 76-mm (3-in) subbase filter layer over a subgrade of clay. The test section was located over the permeable base. The CRC pavement was designed with a steel percentage of 0.70. The reinforcing steel are #6 bars spaced at 159 mm (6.25 in) with no transverse steel. The bars were tube fed during the construction of the pavement and placed as a single layer. The bars are not epoxy coated.

The average concrete pavement thickness for IL-1 was determined to be 259 mm (10.2 in) based on cores taken as part of this study. The concrete is considered well graded but is poorly consolidated. The aggregate type used was a crushed stone with a maximum size of 38 mm (1½ in).

The permeable cement treated base was considered to be uniformly graded and was 99 mm (3.9 in) thick. The subbase was well graded and 152 mm (6 in) thick. Its American Association of State Highway and Transportation Officials (AASHTO) classification is A-2-4.

The subgrade is a clay with the AASHTO classification of A-7-5. The Atterberg Limits of the subgrade were a liquid limit of 43 and a plastic limit of 21.

Design/Construction

The thickness design for US-51 was done by the Illinois Department of Transportation (IDOT) using the 80 percent of the IDOT-AASHTO Jointed Concrete Pavement Thickness Design Method. The design life chosen was 20 years with a design traffic level of 12,300 vehicles per day. The concrete properties assumed were a modulus of elasticity of 27 580 MPa (4,000,000 lbf/in^2) and a minimum modulus of rupture equal to 4.48 MPa (650 lbf/in^2). The subgrade modulus assumed was 51.71 MPa (7,500 lbf/in^2) or a California Bearing Ratio (CBR) of 5. As discussed earlier, drainage was incorporated in the design. The terminal joint design chosen was a wide flange beam system. The bridge approach pavement type was concrete.

Construction began in June 1990 and was finished by August 1991. There were some delays encountered during construction. The delays were caused during placement of the open-graded base course. The use of the open-graded base resulted in some pavement roughness. In these rough areas, the pavement was ground to a smoother profile. The concrete properties specified were 19- to 38-mm (¾- to 1½-in) slump, air content of 5 to 8 percent, and minimum flexural strength of 4.48 MPa (650 lbf/in^2) at 14 days.

Performance

The performance of US-51 had not yet been evaluated because of the young age of the project.

Historical Data Maintenance and Rehabilitation (M&R)

The only maintenance work that has been done was the grinding of the pavement surface during construction to obtain an acceptable surface profile.

Field Investigation Data

Visual Condition Survey

- Total number of cracks = 178
  - Cracks exhibited no distress
- Average crack spacing = 1.56 m (5.11 ft)
- Standard deviation for crack spacing = 1.07 m (3.51 ft)
- Coefficient of variation of the crack spacing = 68 percent
- Figure 2 shows average crack spacing distribution based on the closest five cracks
- Figure 3 shows actual crack spacing
- Number of Y cracks = 4 percent
- Cluster cracking was generally not apparent
- Other distress types - None

Drainage Survey

- Ditches line either side of the pavement
- Ditches were ponded with water because of the rain the previous day

Windshield Survey

- 8.05 km (5 mi) of pavement surveyed was in good condition

Terminal Joint Survey

- Wide flange terminal joints used (two surveyed)
- Joints exhibited no distress

Deflection Testing

- Figure 4 shows normalized/basin (per 4.45 kN (1,000 lb) load) deflections measured along the length of the section (for all seven sensors using the 40.03 kN (9,000 lb) load)
- Load transfer efficiency at cracks at mid-slab
    - 89 to 95 percent in the morning
    - 91 to 95 percent in the afternoon
- Load transfer efficiency at cracks along edge
    - 88 to 94 percent in the morning
    - 90 to 96 percent in the afternoon
- Figure 5 shows the distribution of slab temperature with depth and time of day.
- Figure 6 shows the average deflections and ranges of deflections for each sensor for center and edge loading, for morning and afternoon testing (using the 40.03 kN (9,000 lb) load within the selected 30.5-m (100-ft) subsection).
- Figure 7 shows the backcalculated radius of relative stiffness, ℓ, values for basin testing for each of the three nominal load levels of 40.03, 53.38, and 71.17 kN (9,000, 12,000, and 16,000 lb).
- ℓ value based on 40.03 kN (9,000 lb) load basin testing
    - Average = 1 016 mm (40 in)
    - Standard Deviation = 229 mm (9 in)
- ℓ value at crack locations at mid-slab (morning/afternoon)
    - Average = 762/889 mm (30/35 in)
    - Standard Deviation = 127/127 mm (5/5 in)
- ℓ value at crack locations at edge (morning/afternoon)
    - Average = 686/813 mm (27/32 in)
    - Standard Deviation = 102/102 mm (4/4 in)
- Estimated ℓ value using actual slab thickness and concrete modulus of elasticity and assumed modulus of subgrade reaction of 54.3 MPa/m (200 pci)
    - Value = 991 mm (39 in)
- Figure 8 shows the variation of ℓ in relation to crack spacing (average of the closest five cracks)

- Average deflection under 40.03 kN (9,000 lb) load (0.000001 mm (0.001 in))
    - Basin test - 0.056 (2.2)
    - Mid-slab crack location (morning/afternoon) - 0.079/0.071 (3.1/2.8)
    - Edge crack location (morning/afternoon) - 0.132/0.097 (5.2/3.8)

Crack Width Measurements

- Number of cracks monitored within a 30.5-m (100-ft) subsection = 28
- Morning testing
    - Average temperature 102 mm (4 in) below surface = 6.67°C (44°F)
    - Average crack width = 0.22 mm (0.009 in)
- Afternoon testing
    - Average temperature 102 mm (4 in) below surface = 14.44°C (58°F)
    - Average crack width = 0.16 mm (0.006 in)
- Slab length change, mm/mm/°C (in/in/°F) = 7.0 millionth (3.9 millionth)

Reinforcing Steel Depths and Spacing

- Steel depth measurement offsets = 610, 1 219, 2 438 mm (24, 48, 96 in)
- Range of depth of cover = 91 to 160 mm (3.6 to 6.3 in)
- Average depth of cover = 130 mm (5.1 in)
- Standard deviation for depth of cover = 13 mm (0.5 in)
- Steel spacing was about 203 mm (8 in)

Corrosion Potential Testing (copper-copper sulfate half cell potentiometer)

- Range of measurements = -0.02 to -0.07 volts
- Average measurement = -0.05 volts
- Standard deviation for the measurement = -0.01 volts
- Potential for corrosion was not indicated

Concrete Core Examination for Corrosion

- No steel corrosion was detected

Profile Testing

- Average IRI over the 305-m (1,000-ft) section = 1 469 mm/km (93 in/mi)

Laboratory Testing

The following data were obtained from the testing of the concrete, base/subbase, and subgrade materials:

Concrete

Modulus of Elasticity	-	37 230 MPa (5,400,000 lbf/in^2)
Average Split Tensile Strength	-	3.38 MPa (490 lbf/in^2)
Split Tensile Strength Range	-	2.89 to 3.65 MPa (420 to 530 lbf/in^2)
Coefficient of Thermal Expansion	-	9.20 mm/mm/°C (5.11 in/in/°F)

Coefficient of Thermal Contraction - 6.89 mm/mm/°C (3.83 in/in/°F)

Base

AASHTO Classification - Permeable cement treated base

Subbase

AASHTO Classification - N/A

Subgrade

Liquid Limit - 43 percent
Plastic Limit - 21 percent
Percent Passing #200 Sieve - 87.4 percent
AASHTO Classification - A-7-6

Chloride content testing was conducted on pulverized concrete retrieved at various depths and locations. Total chloride content and water soluble chloride content values are listed below for IL-1.

Location	**Depth, mm (in)**	**Total Chloride Content (% wt.)**	**Water Soluble Chlorides (% wt.)**
No steel below sample	0-51 (0-2)	0.023	0.018
	51-102 (2-4)	0.020	0.019
Steel directly below sample	0-51 (0-2)	0.019	0.010

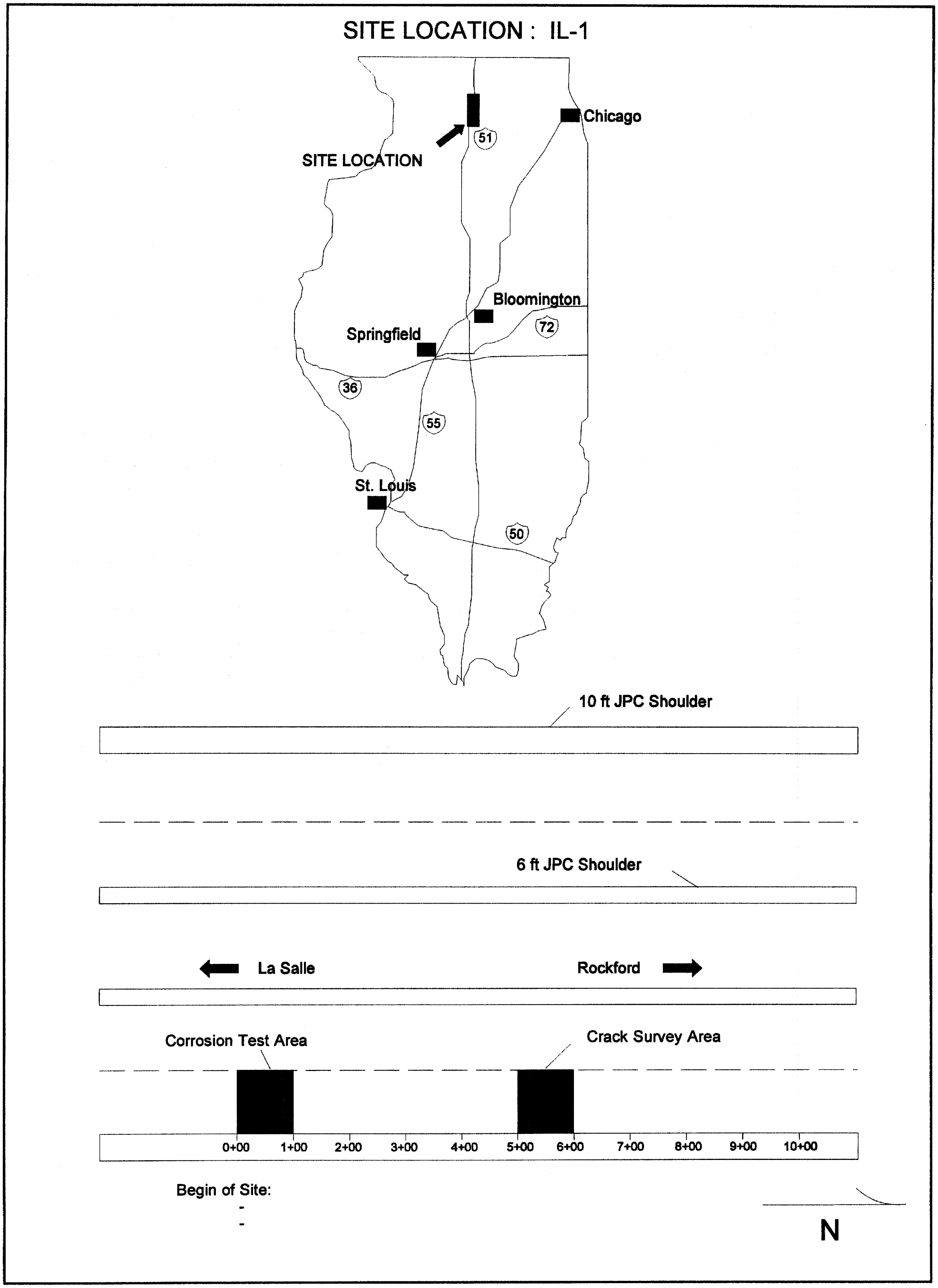
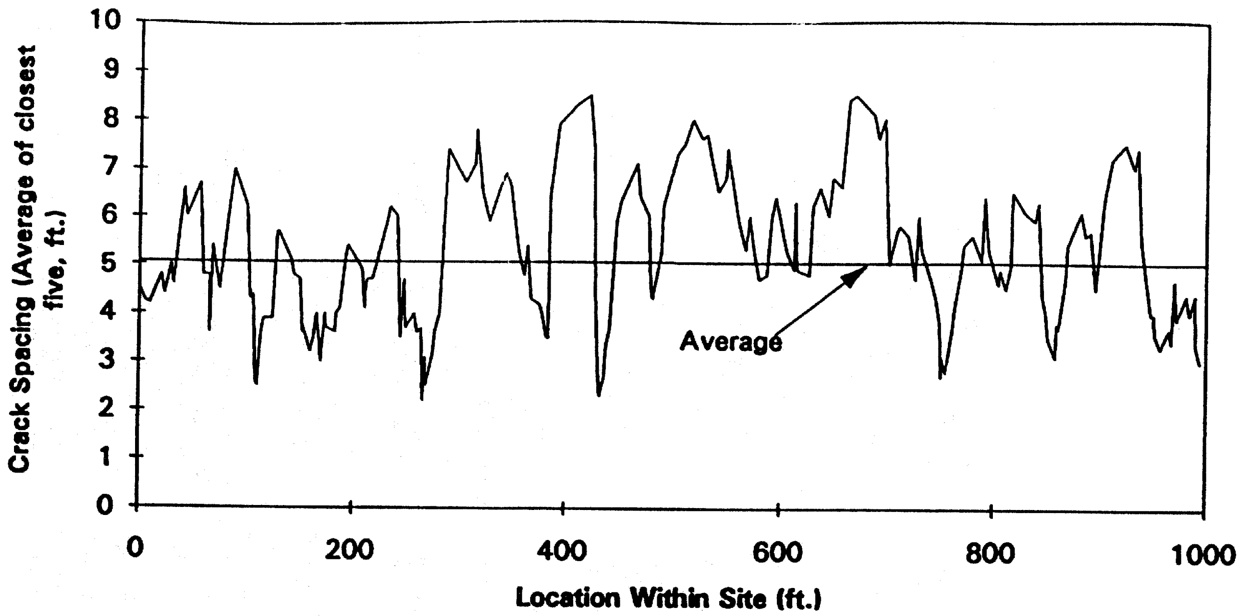


Figure 1. Site details for IL-1.



(0.305 m = 1 ft)

Figure 2. Crack spacing pattern at site IL-1.

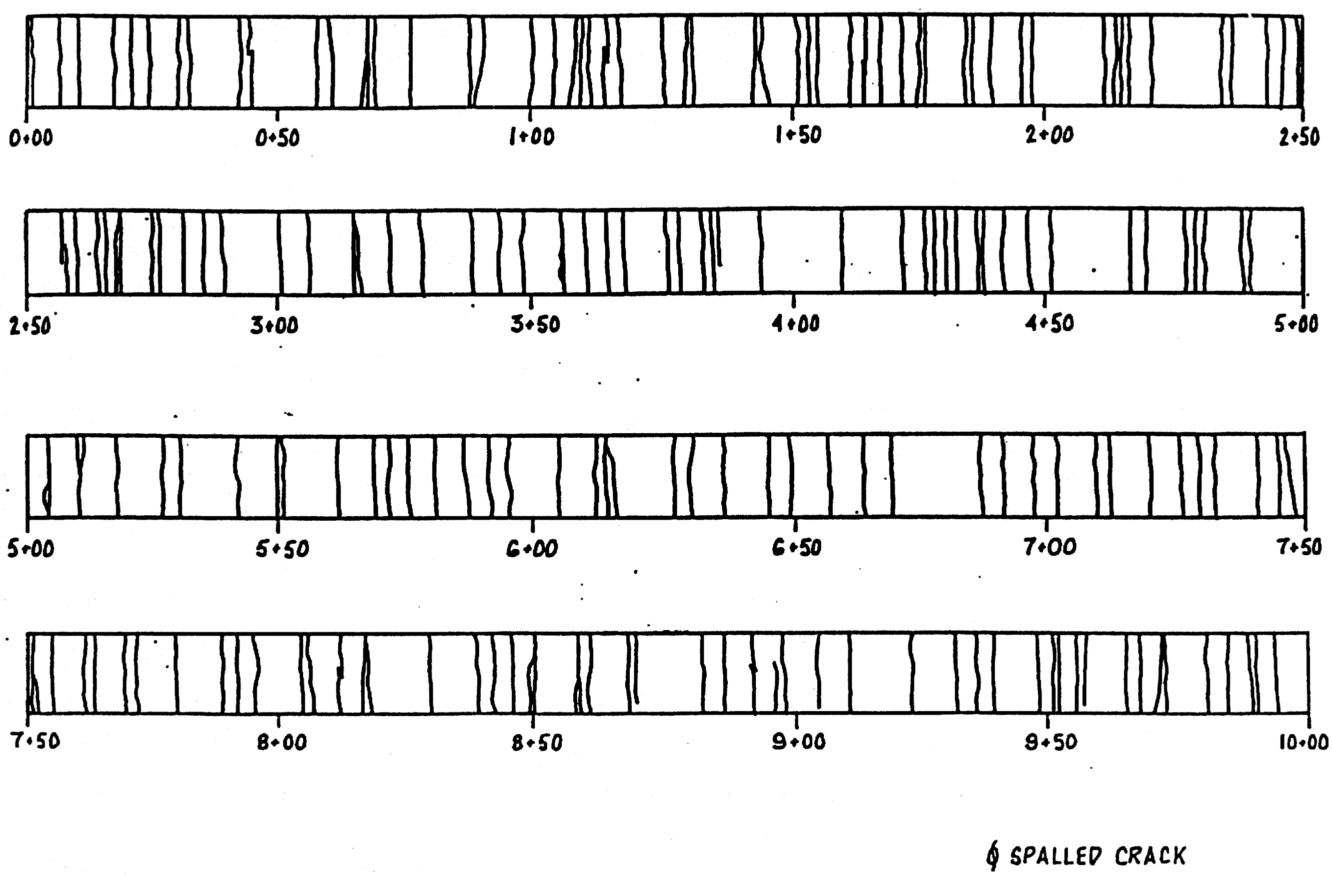
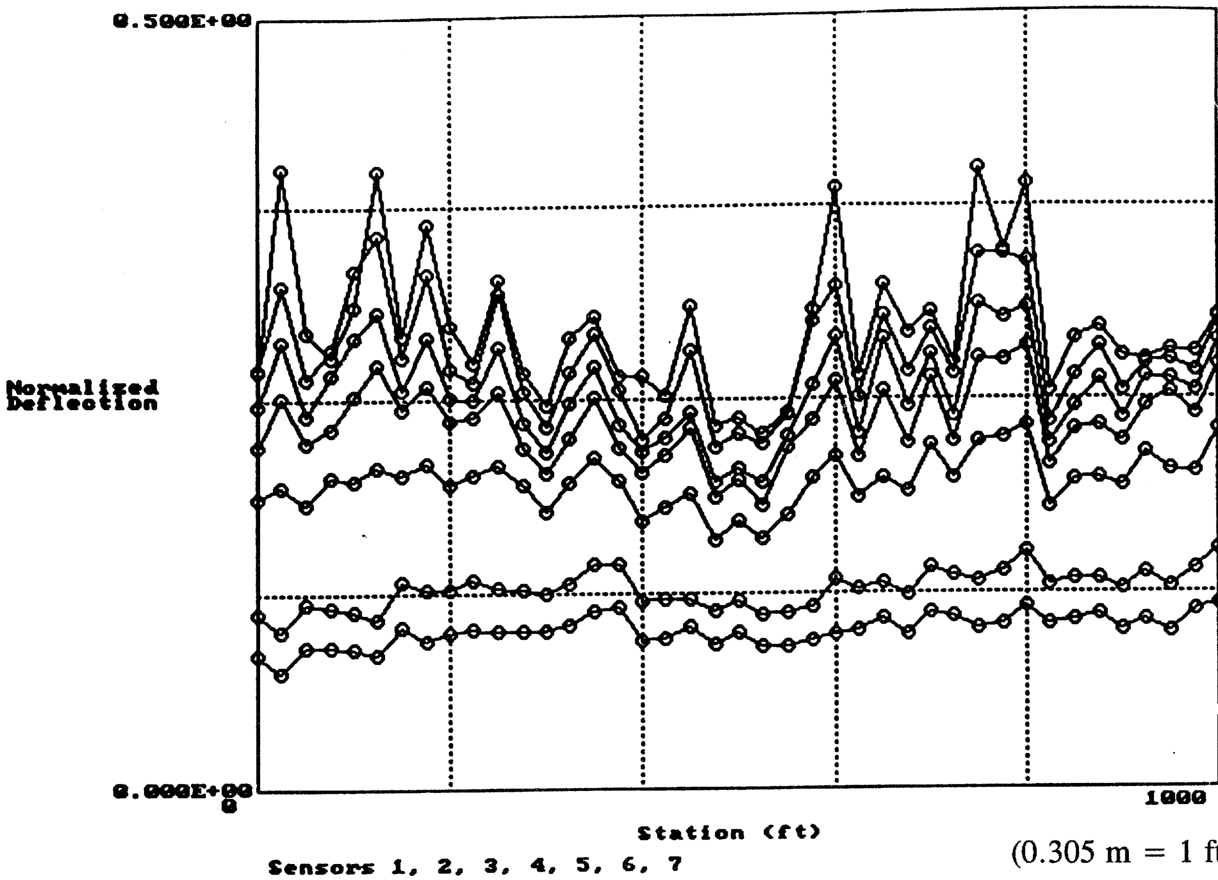
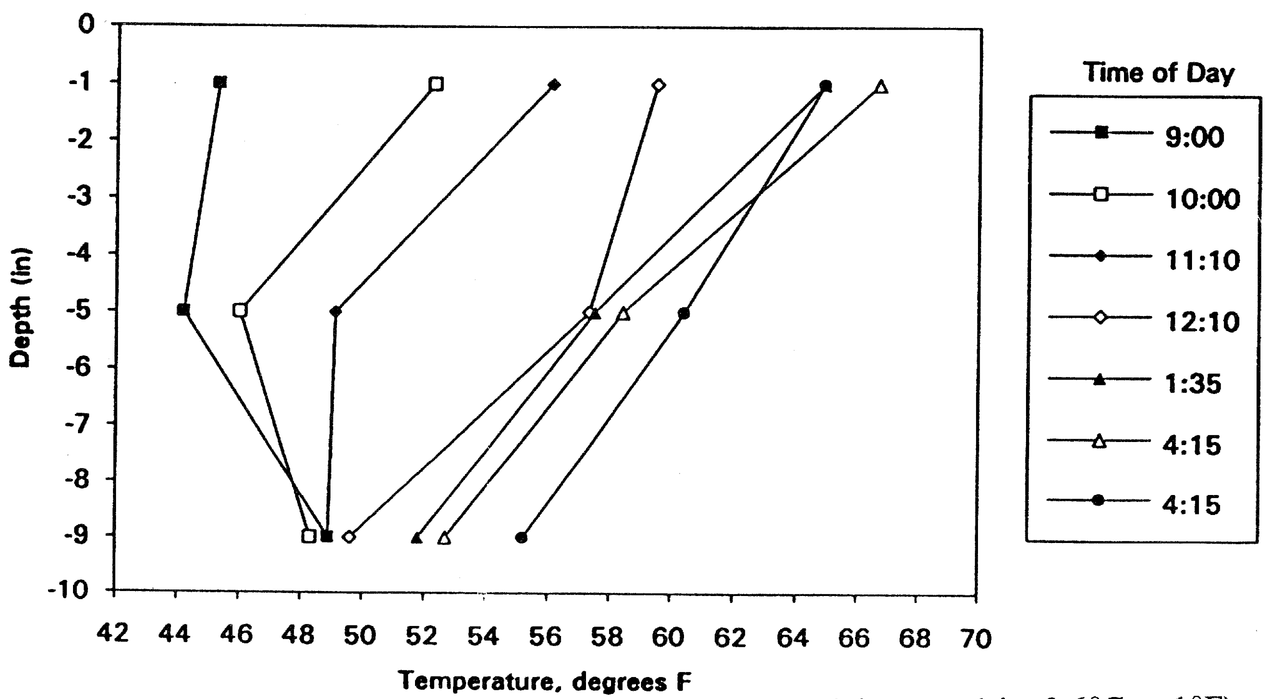


Figure 3. Crack pattern at site IL-1.



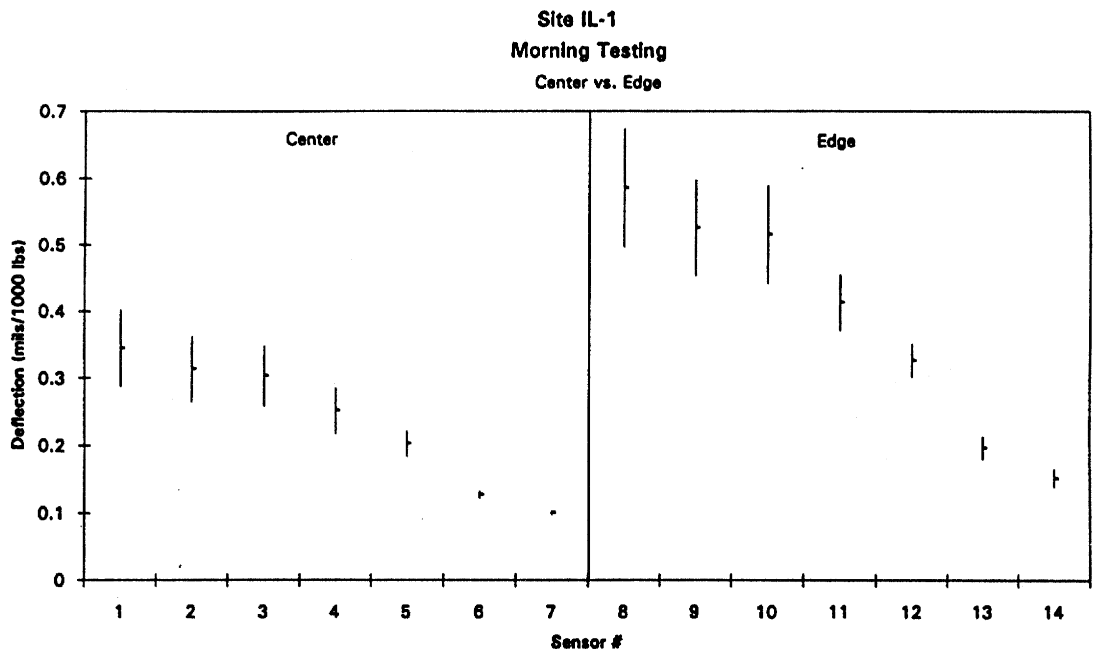
(0.305 m = 1 ft)

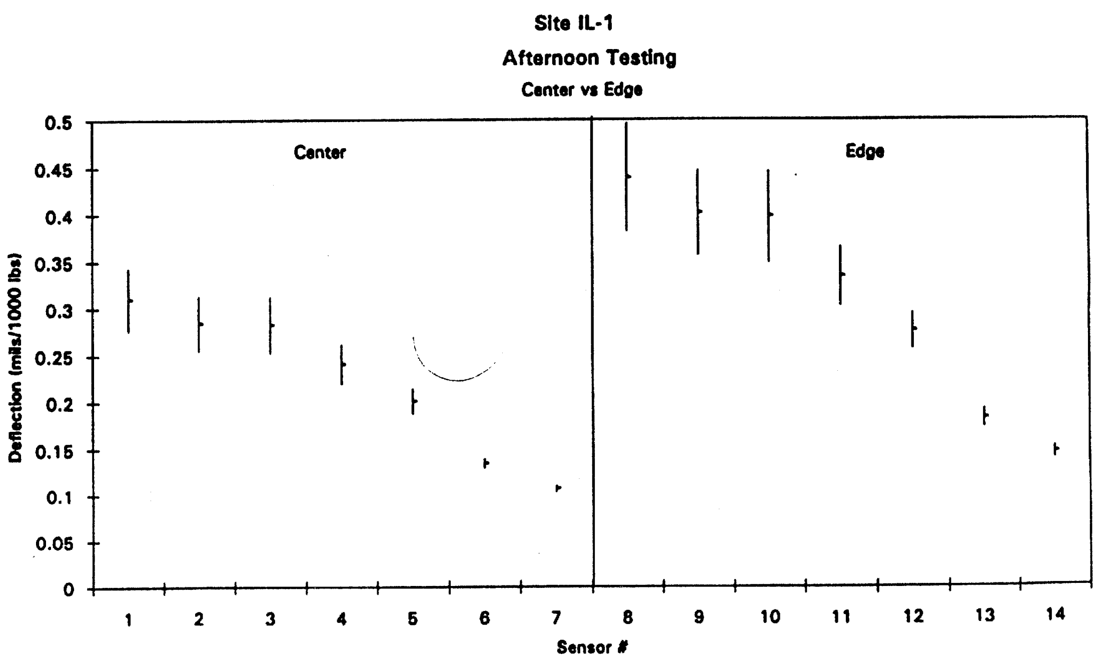
Figure 4. Deflections in inches along the section length at site IL-1.
[normalized per 4.45-kN [1,000-lb] load]



(25.4 mm = 1 in; 0.6°C = 1°F)

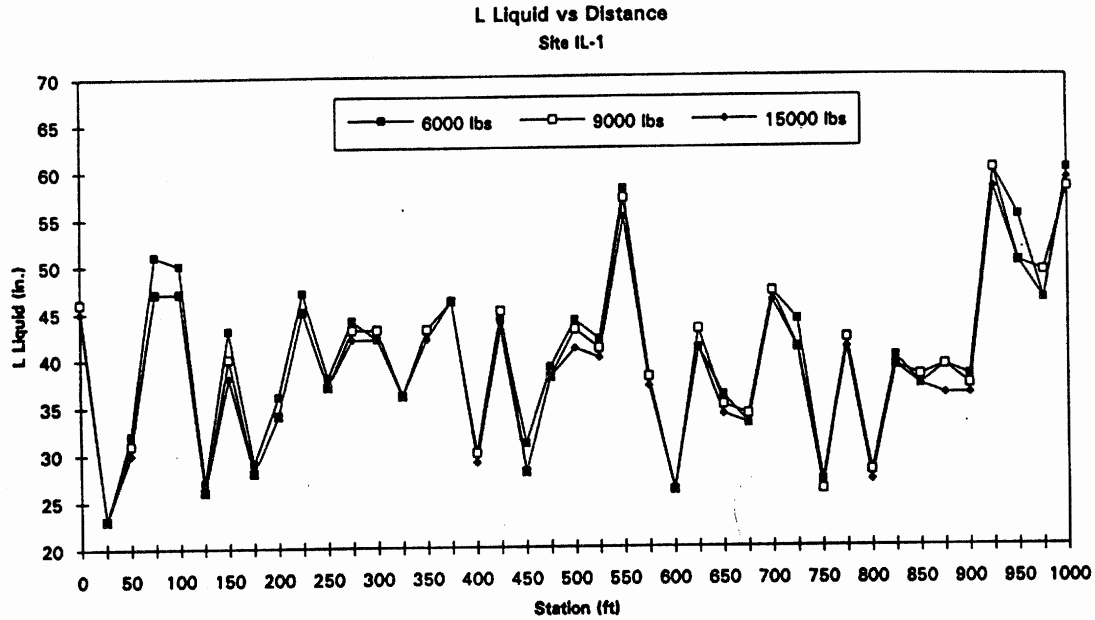
Figure 5. Temperature profile for site IL-1.





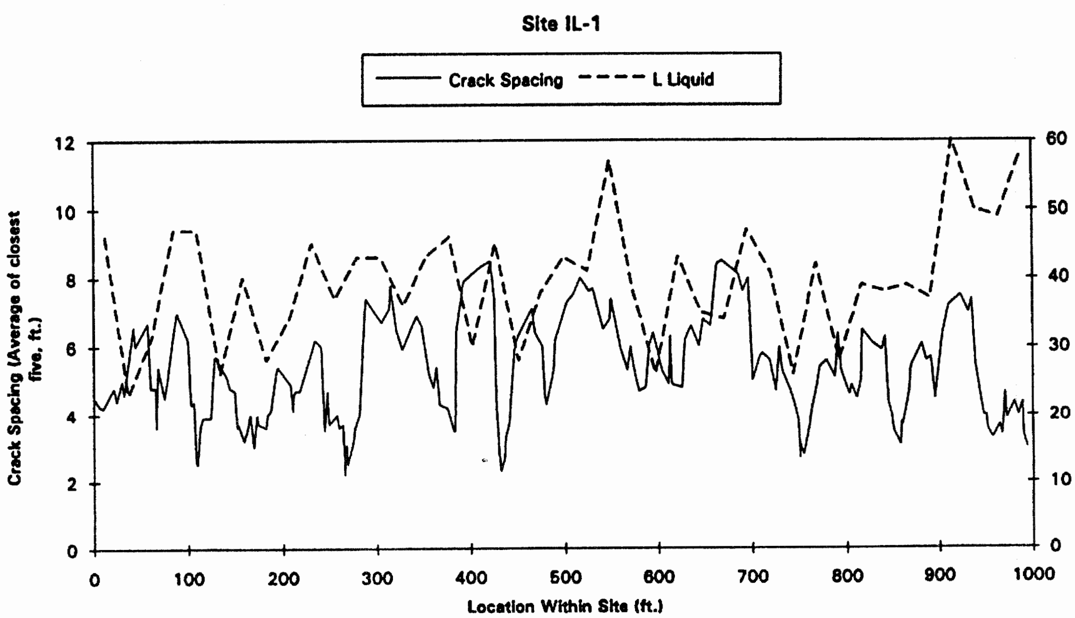
(0.0254 mm = 1 mil; 4.45 kN = 1,000 lb)

Figure 6. Center versus edge deflection for each of the seven sensors at site IL-1.
[40.03 kN (9,000 lb) load]



(25.4 mm = 1 in, 0.305 m = 1 ft)

Figure 7. Backcalculated values of radius of relative stiffness, ℓ (using program ILLI-BACK) for site IL-1.



(0.305 m = 1 ft)

Figure 8. Comparison of radius of relative stiffness values with average crack spacing (nearest five cracks) at site IL-1.

Section IL-2 - Interstate 72, Illinois

Introduction

Section IL-2 was constructed and opened to traffic in 1976. IL-2 is a section of I-72 that runs east to west from Champaign to Springfield, Illinois, as shown in figure 9. A 305 m (1,000 ft) west bound section was selected for detailed inspection between mileposts 45 and 46, as shown in figure 9.

The climate in this region is typically continental with warm summers and fairly cold winters. There are no wet and dry seasons. The highest monthly average temperature is 24.7°C (76.5°F) during July and the lowest monthly average is -4.1°C (24.6°F) during January. I-72 lies within the wet-freeze environmental zone.

Traffic

Since its opening, the pavement has experienced an estimated 4.9 million ESALs in the west bound lane. I-72 was designed for a total of 4.8 million ESALs over 20 years.

1991 Design Lane ESAL	=	390,000
1991 Two-Way ADT	=	7,500 vehicles

Structure

I-72 is a four lane divided highway. The pavement lane width is 3.7 m (12 ft) with an outside shoulder width of 3.0 m (10 ft) and an inside shoulder width of 0.9 m (3 ft). The shoulders are constructed of asphaltic concrete.

I-72 was designed as a three-layer system comprised of a 203-mm (8-in) CRCP and a 102-mm (4-in) CTB over native subgrade. The CRC pavement was designed with a steel percentage of 0.59. The reinforcing steel are #5 bars spaced at 165 mm (6.5 in) with no transverse steel. The bars were tube fed during the construction and placed as a single layer. The bars are not epoxy coated.

The average concrete pavement thickness for IL-2 was determined to be 224 mm (8.8 in) based on cores taken as part of this study. The concrete is considered well graded and is well consolidated. The aggregate type used was a crushed stone with a maximum size of 51 mm (2 in).

The CTB was considered to be well graded and was 89 mm (3.5 in) thick. The subgrade has an AASHTO classification of A-6. The Atterberg Limits of the subgrade were a liquid limit of 26 and a plastic limit of 14.

Design/Construction

The thickness design records for I-72 were not available. It was reported that the method of design was a minimum thickness design. The design life chosen was 20 years with a total design traffic of 4.8 million. The concrete properties assumed were a modulus of elasticity of 27 600 MPa (4,000,000 lbf/in^2) and a minimum modulus of rupture equal to 4.48 MPa (650 lbf/in^2. The bridge approach pavement type was concrete.

Construction began in 1976 and was finished in 1976. There were some delays encountered during construction. The delays were caused by the soft subgrade soil. The concrete properties specified were 19 to 38 mm (¾ to 1½ in) slump, air content of 4 to 7 percent, and minimum flexural strength of 4.48 MPa (650 lbf/in^2) at 14 days.

Performance

The performance of I-72 has been recorded according to the Condition Rating System (CRS) Method, with a rating of 1.0 to 9.0. Nine is excellent for a new pavement, and one is very poor or an unusable pavement. The first value reported was obtained in 1982 and was 8.0. The most current value is 7.2 for 1990. The CRS values are listed below:

Year	CRS Value
1982	8.0
1984	7.8
1986	7.6
1988	7.4
1990	7.2

Historical Data (M&R)

The primary maintenance that has been performed on this section of the highway has been full depth repairs of punchouts. This work was done in 1985, 1986, and 1987.

Field Investigation Data

Visual Condition Survey

- Total number of cracks = 235
    - 5 were at high severity
    - 221 were at medium severity with spalling
    - 9 were at low severity
- Average crack spacing = 1.29 m (4.22 ft)
- Standard deviation for crack spacing = 0.81 m (2.66 ft)
- Coefficient of variation of the crack spacing = 63 percent
- Figure 10 shows average crack spacing distribution based on closest five cracks

- Figure 11 shows actual crack spacing
- Extent of Y cracks = 18 percent
- Cluster cracking was apparent at stations 2+30, 4+50, and 6+50.
- Other distress types included:
  - AC patches (moderate severity)
  - 18.3 m (60 ft) longitudinal cracking
  - 3 PCC patches (low severity)
- Pavement shoulders were in good condition but exhibited both longitudinal and transverse cracking.

Drainage Survey

- Ditches line both sides of the pavement.
- Ditches were lined with vegetation. There was no sign of ponded water or wet ground.

Windshield Survey

- 8.05 km (5 mi) of pavement surveyed was generally in fair to good condition exhibiting 35.7 m (117 ft) of longitudinal cracking condition, 19 AC patches, 1 punchout, and 42 PCC patches.

Terminal Joint Survey

- Terminal joints used = Lug anchor (two joints surveyed)
- Joints exhibited the following distresses:
  - one joint had 0.65 m^2 (7 ft^2) of medium severity spalling
  - both joints had vegetation and incompressibles in the joints.

Deflection Testing

- Figure 12 shows normalized/basin (per 4.45 kN (1,000 lb) load) deflections measured along the length of the section (for all seven sensors using the 40.03 kN (9,000 lb) load).
- Load transfer efficiency at cracks at mid-slab
  - 82 to 96 percent in the morning
  - 88 to 95 percent in the afternoon
- Load transfer efficiency at cracks along edge
  - 44 to 95 percent in the morning
  - 47 to 97 percent in the afternoon
- Figure 13 shows the distribution of slab temperatures with depth and time of day.
- Figure 14 shows the average deflections and ranges of deflections for each sensor for center and edge loading, for morning and afternoon testing (using the 40.03 kN (9,000 lb) load within the selected 30.5-m (100-ft) subsection).
- Figure 15 shows the backcalculated radius of relative stiffness, ℓ, values for basin testing for each of the three nominal load levels of 40.03, 53.38, and 71.17 kN (9,000, 12,000, and 16,000 lb).

- ℓ value based on 40.03 kN (9,000 lb) load basin testing
  - Average = 940 mm (37 in)
  - Standard Deviation = 178 mm (7 in)
- ℓ value at crack locations at mid-slab (morning/afternoon)
  - Average = 635/711 mm (25/28 in)
  - Standard Deviation = 76/51 mm (3/2 in)
- ℓ value at crack locations at edge (morning/afternoon)
  - Average = 610/635 mm (24/25 in)
  - Standard Deviation = 127/127 mm (5/5 in)
- Estimated ℓ value using actual slab thickness and concrete modulus of elasticity and assumed modulus of subgrade reaction
  - Value = 914 mm (36 in)
- Figure 16 show the variation of ℓ in relation to crack spacing (average of closest five cracks)
- Average deflection under 40.03 kN (9,000 lb) load (0.000001 mm (0.001 in))
  - Basin test - 0.11 (4.3)
  - Mid-slab crack location (morning/afternoon) - 0.16/0.14 (6.2/5.4)
  - Edge crack location (morning/afternoon) - 0.30/0.28 (12/10.9)

Crack Width Measurements

- Number of cracks monitored within the 30.5-m (100-ft) subsection = 28
- Morning testing
  - Average temperature 102 mm (4 in) below surface = 14.4°C (58°F)
  - Average crack width = 0.55 mm (0.022 in)
- Afternoon testing
  - Average temperature 102 mm (4 in) below surface = 22.8°C (73°F)
  - Average crack width = 0.44 mm (0.017 in)
- Slab length change, mm/mm/°C (in/in/°F) = 12.1 millionth (6.7 millionth)

Reinforcing Steel Depths and Spacing

- Steel depth measurement offsets = 610, 1 829, 2 438 mm (24, 72, 96 in)
- Range of depth of cover = 36 to 99 mm (1.4 to 3.9 in)
- Average depth of cover = 69 mm (2.7 in)
- Standard deviation for depth of cover = 10 mm (0.4 in)
- Steel spacing was about 203 mm (8 in)

Corrosion Potential Testing (copper-copper sulfate half cell potentiometer)

- Range of measurements = -0.31 to -0.50 volts
- Average measurement = -0.41 volts

- Standard deviation for the measurement = -0.03 volts
- Potential for corrosion was indicated

Concrete Core Examination for Corrosion
- One of the four cores exhibited spot corrosion on the steel bar.

Profile Testing
- Average IRI over 305-m (1,000-ft) section = 2 007 mm/km (127 in/mi)

Laboratory Testing

The following data were obtained from the testing of the concrete, base/subbase, and subgrade materials:

Concrete

Modulus of Elasticity	-	39 300 MPa (5,700,000 lbf/in^2)
Average Split Tensile Strength	-	4.00 MPa (580 lbf/in^2)
Split Tensile Strength Range	-	3.24 to 4.62 MPa (470 to 670 lbf/in^2)
Coefficient of Thermal Expansion	-	8.77 mm/mm/°C (4.87 in/in/°F)

Base

AASHTO Classification	-	CTB

Subbase

AASHTO Classification	-	N/A

Subgrade

Liquid Limit	-	26 percent
Plastic Limit	-	14 percent
Percent Passing #200 Sieve	-	65.2 percent
AASHTO Classification	-	A-6

Chloride content testing was conducted on pulverized concrete retrieved at various depths and locations. Total chloride content and water soluble chloride content values are listed below for IL-2.

Location	Depth, mm (in)	Total Chloride Content (% wt.)	Water Soluble Chlorides (% wt.)
No steel below sample	0-51 (0-2)	0.220	0.183
	51-102 (2-4)	0.076	0.069
Steel directly below sample	0-51 (0-2)	0.190	0.150
	51-102 (2-4)	0.098	0.086

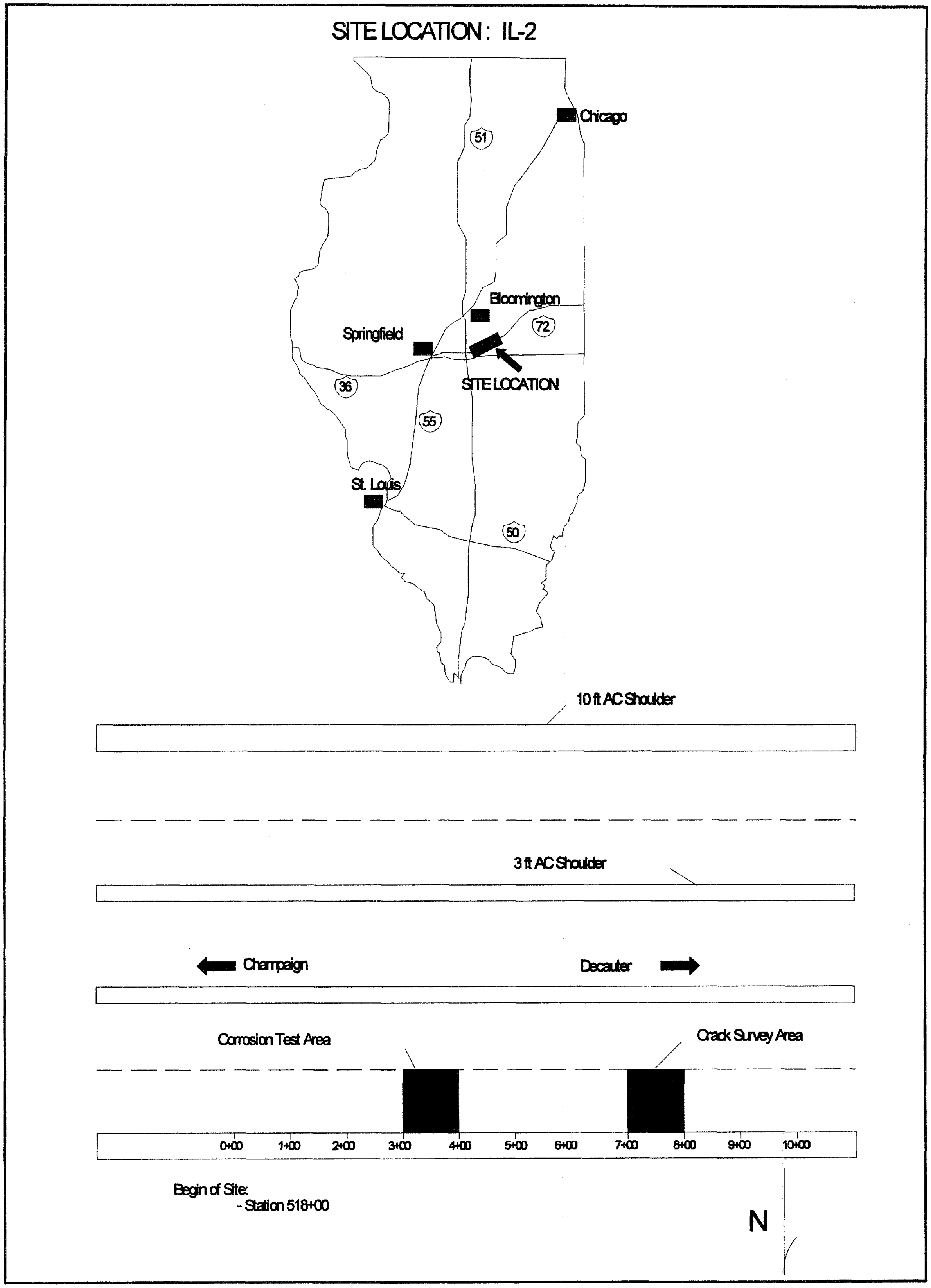


Figure 9. Site details for IL-2.

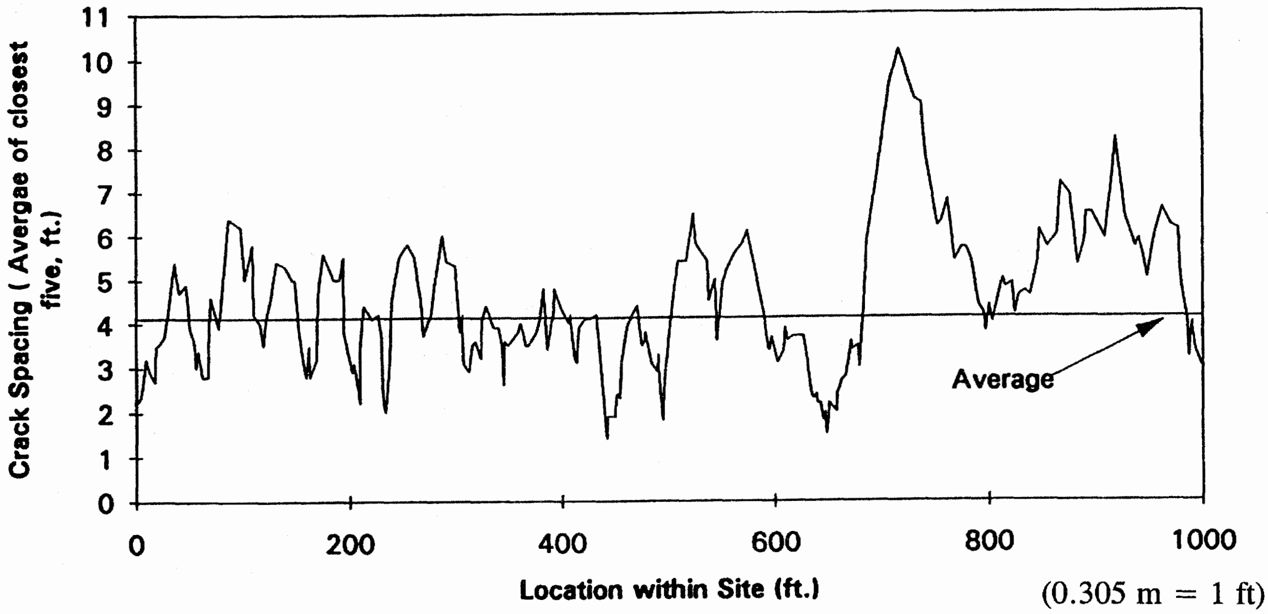


Figure 10. Crack spacing pattern at site IL-2.

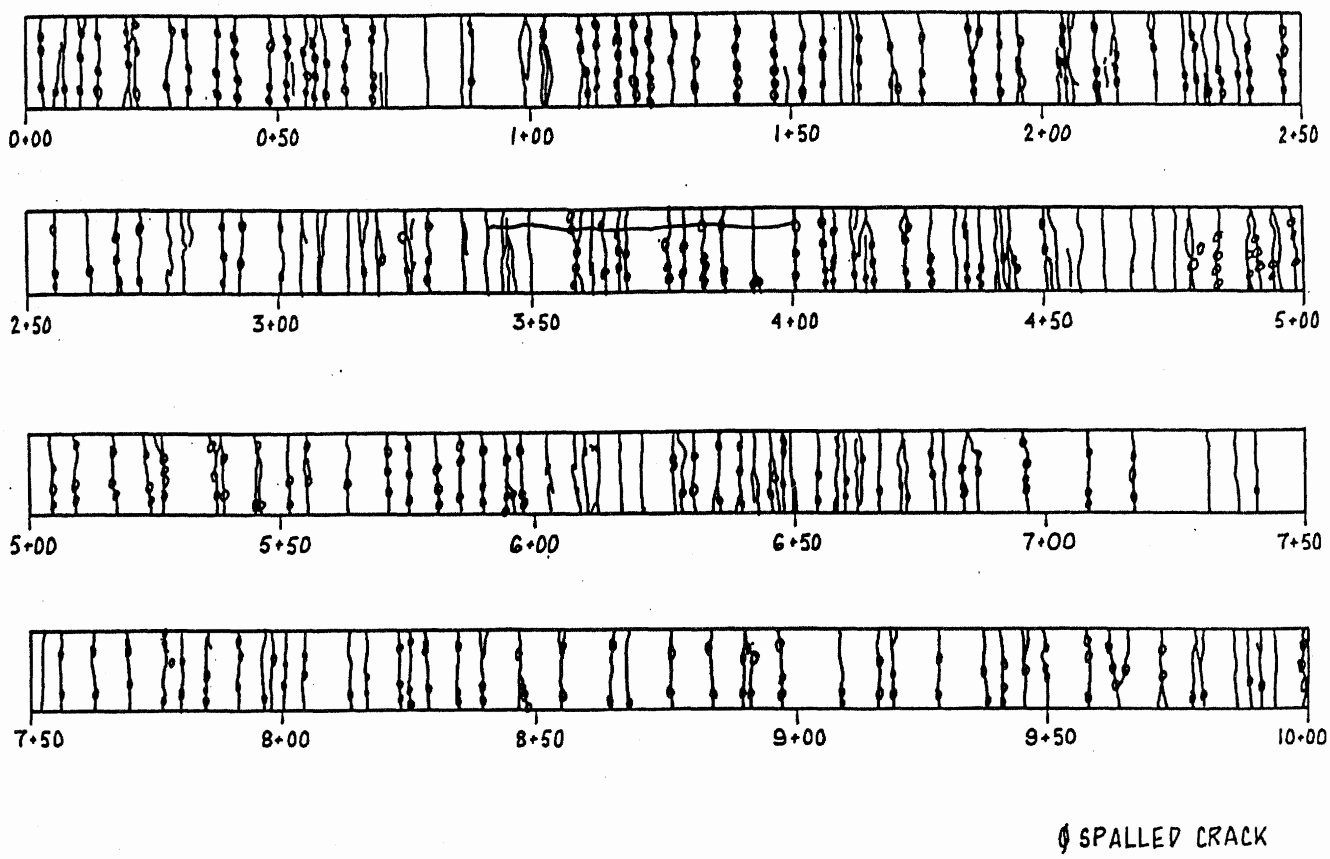
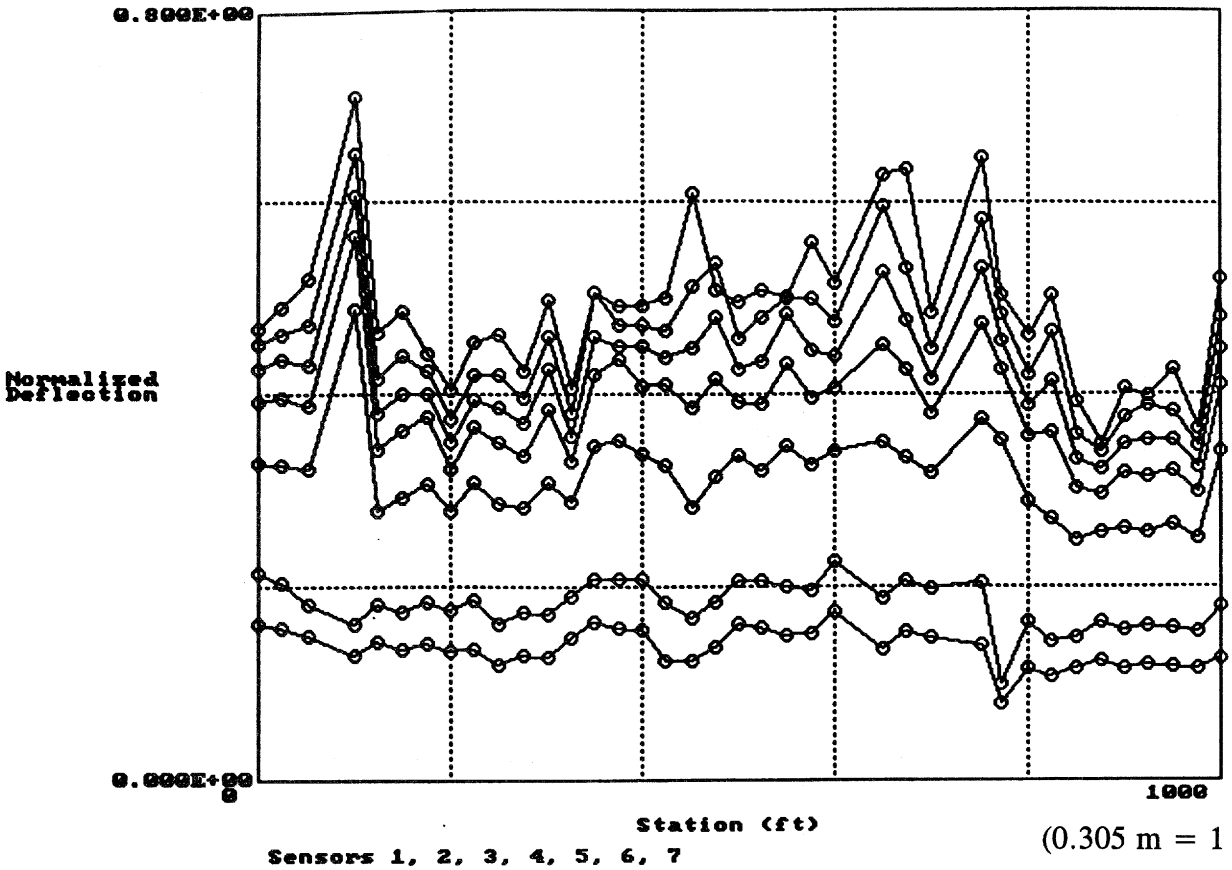
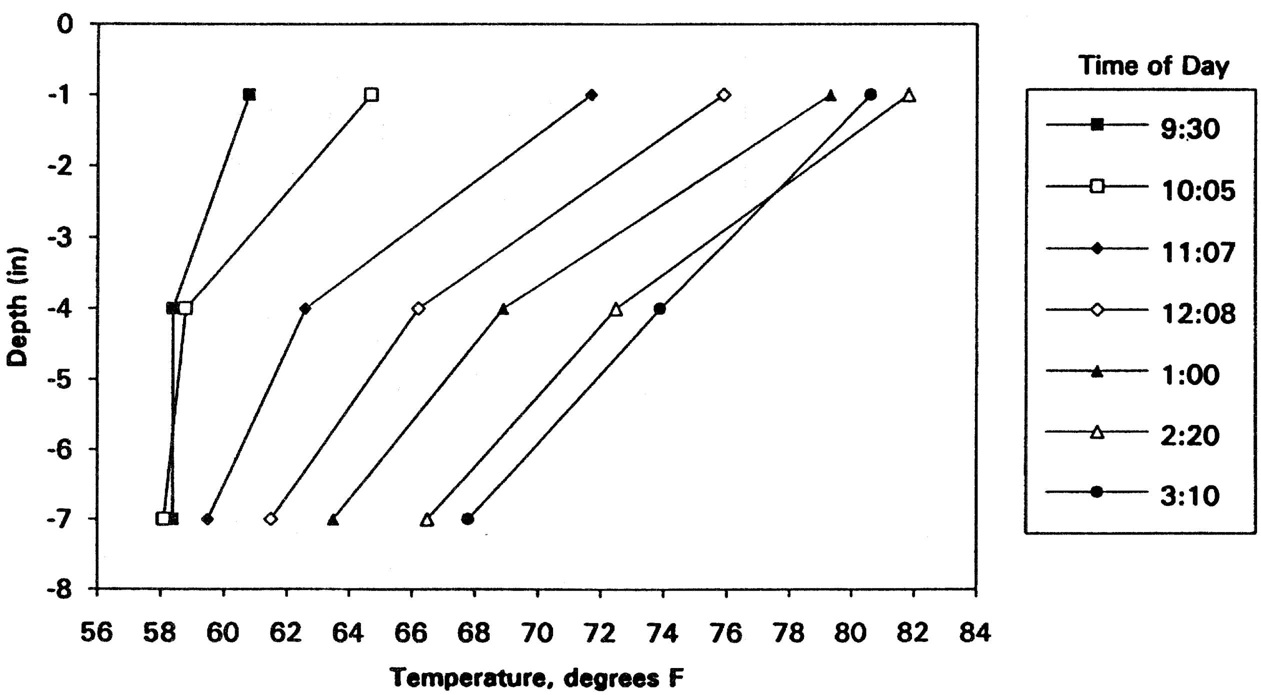


Figure 11. Crack pattern at site IL-2.



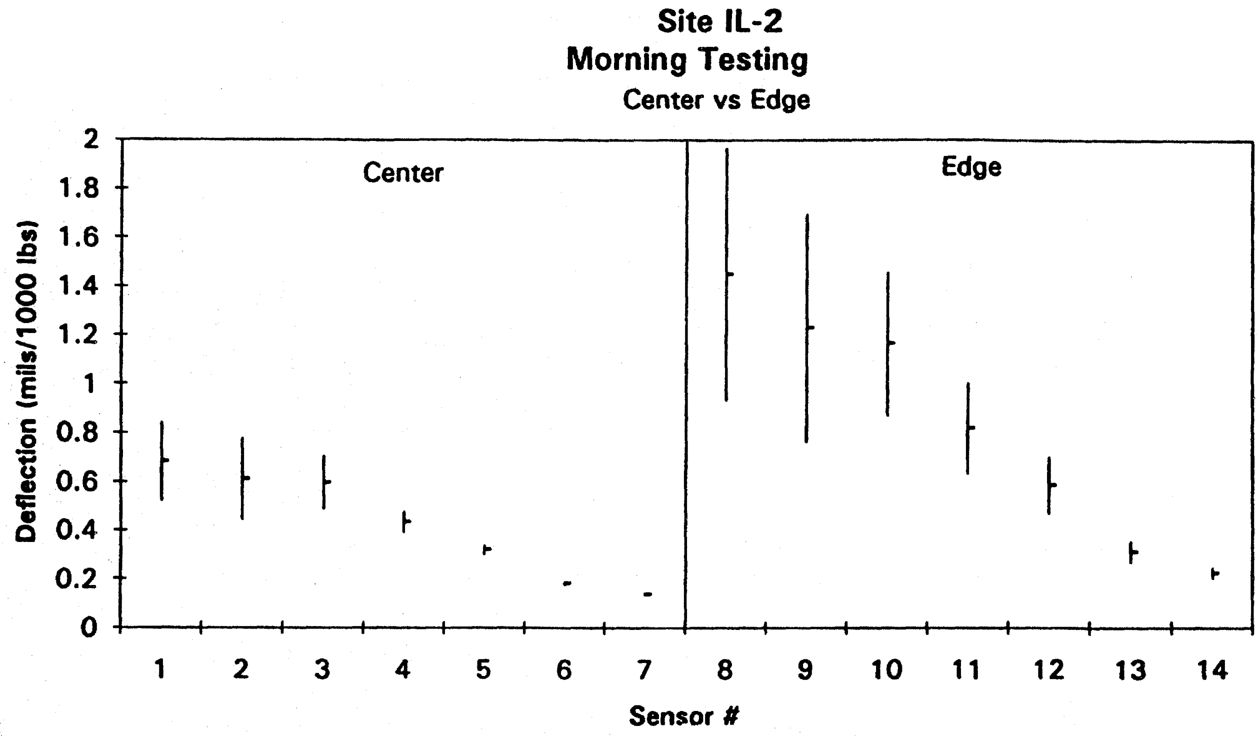
(0.305 m = 1 ft)

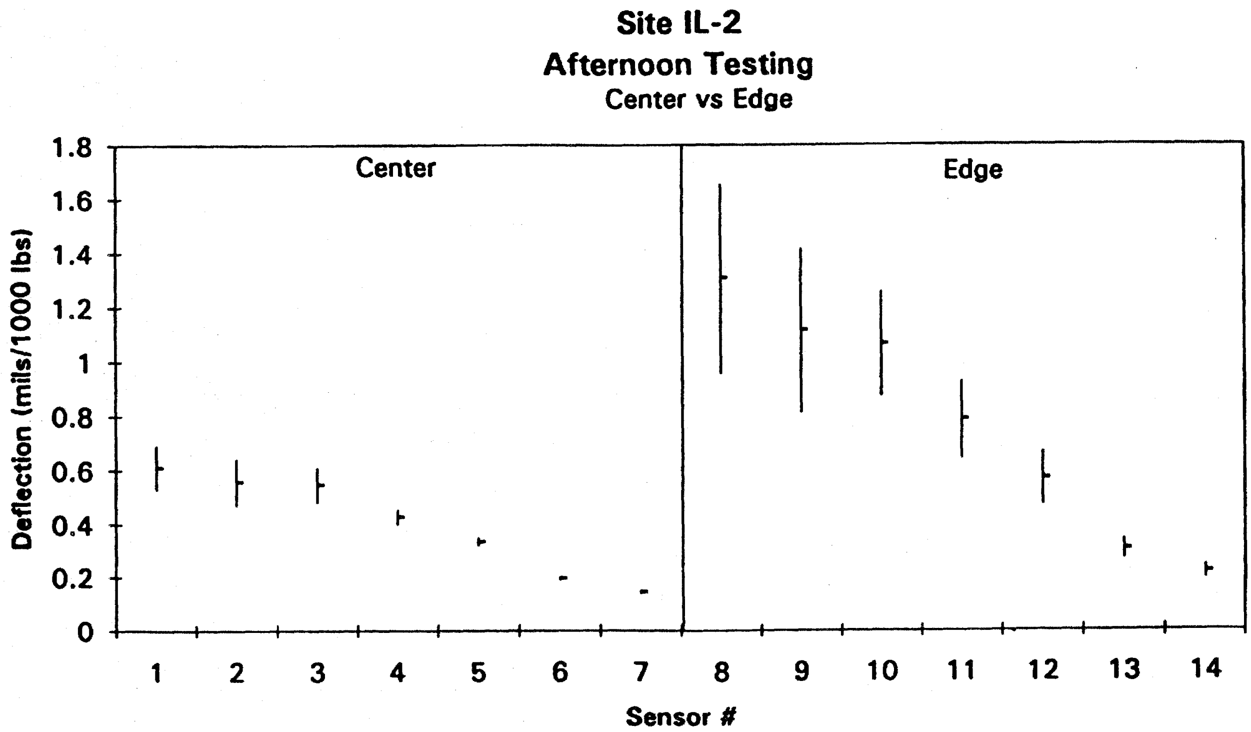
Figure 12. Deflections in inches along the section length at site IL-2.
[normalized per 4.45-kN [1,000-lb] load]



(25.4 mm = 1 in; 0.6°C = 1°F)

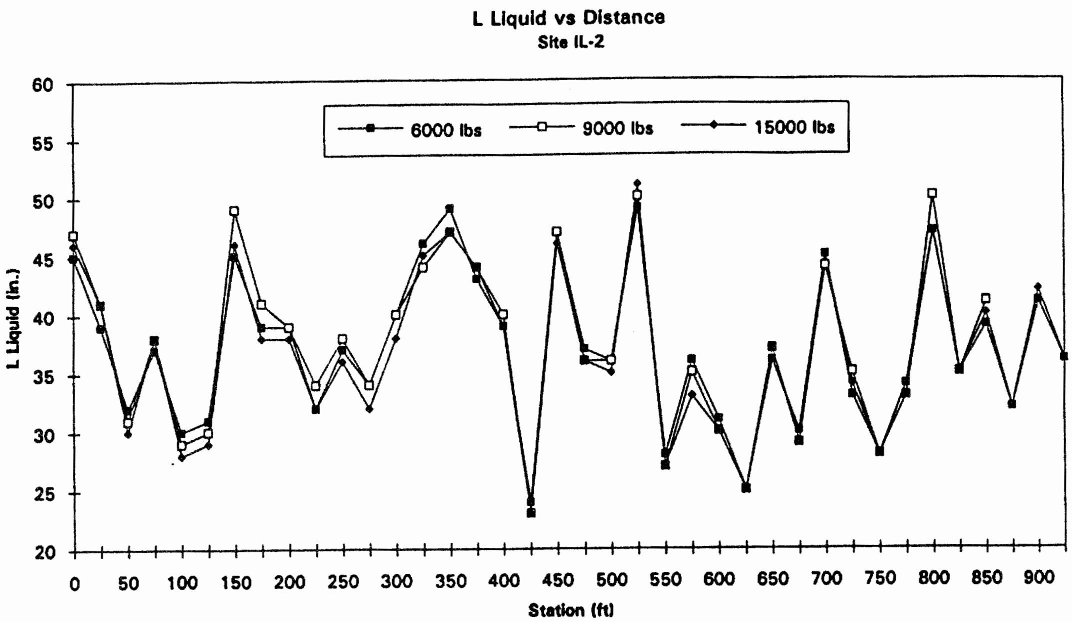
Figure 13. Temperature profile for site IL-2.





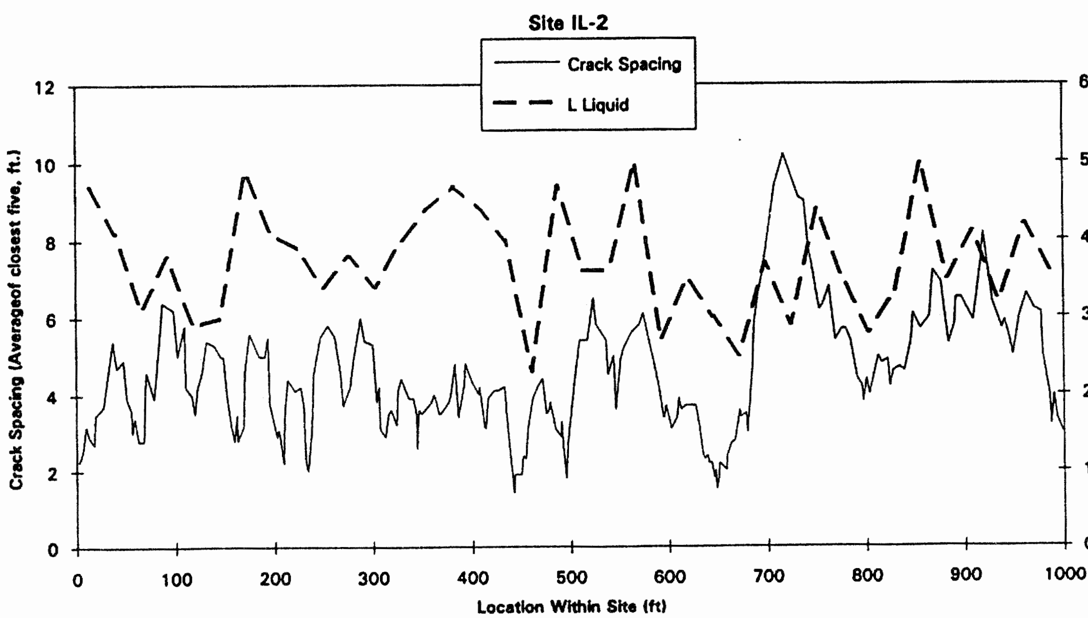
(0.0254 mm = 1 mil; 4.45 kN = 1,000 lb)

Figure 14. Center versus edge deflection for each of the seven sensors at site IL-2.
[40.03 kN (9,000 lb) load]



(25.4 mm = 1 in, 0.305 m = 1 ft)

Figure 15. Backcalculated values of radius of relative stiffness, ℓ (using program ILLI-BACK) for site IL-2.



(0.305 m = 1 ft)

Figure 16. Comparison of radius of relative stiffness values with average crack spacing (nearest five cracks) at site IL-2.

Section IL-3 - US Highway 36, Illinois

Introduction

Section IL-3 was constructed and opened to traffic in 1971. IL-3 is a section of US-36 that runs east to west from Decatur to Champaign, Illinois, as shown in figure 17. The length of the highway that was considered runs from station 379 to 556. A 305-m (1,000-ft) east bound section was selected for detailed inspection at station 530, as shown in figure 18.

The climate in this region is typically continental with warm summers and fairly cold winters. There are no wet and dry seasons. The highest monthly average temperature is 24.7°C (76.5°F) during July, and the lowest monthly average is -4.1°C (24.6°F) during January. US-36 lies within the wet-freeze environmental zone.

Traffic

Since its opening, the pavement has experienced an estimated 4.8 million ESALs in the east bound lane. US-36 was designed for a 20-year design life.

1991 ESAL in Design Lane	= 240,000
1991 Two-Way ADT	= 17,700 vehicles

Structure

US-36 is a four-lane divided highway. The pavement lane width is 3.7 m (12 ft) with an outside shoulder width of 3.0 m (10 ft) and an inside shoulder width of 0.9 m (3 ft). <u>The shoulders are constructed of asphaltic concrete.</u>

This section of US-36 was designed as a three-layer system comprised of a 203-mm (8-in) CRCP and a <u>102 mm (4 in) ATB</u> over a subgrade of clay. The CRC pavement was designed with a steel percentage of 0.6. The reinforcing steel is #5 bars spaced at 165 mm (6.5 in) with transverse steel. The transverse bars, at a steel percentage of 0.06, are #3 bars spaced at 635 mm (25 in). The bars were placed on chairs as a single layer during construction. The bars are not epoxy coated.

The average concrete pavement thickness for IL-3 was determined to be 208 mm (8.2 in) based on cores taken as part of this study. The concrete is considered well graded, and its consolidation is average. The aggregate type used was a crushed stone with a maximum size of 51 mm (2 in).

The ATB was considered to be well graded and was 97 mm (3.8 in) thick. The clay subgrade has an AASHTO classification of A-7-5. The Atterberg Limits of the subgrade were a liquid limit of 49 and a plastic limit of 24.

Design/Construction

The thickness design records for US-36 were not available. It is known that the method of design was a minimum thickness design. The design life chosen was 20 years but the design traffic is not known. The concrete properties that were assumed were a modulus of elasticity of 27 580 MPa (4,000,000 lbf/in^2) and a minimum modulus of rupture equal to 4.48 MPa (650 lbf/in^2). The bridge approach pavement type was concrete.

Performance

The performance data for this section were not available.

Historical Data (M&R)

Maintenance data for this section were not available.

Field Investigation Data

Visual Condition Survey

- Total number of cracks = 275
  - 5 were at high severity
  - 264 were at medium severity
  - 6 were at low severity
- Average crack spacing = 1.09 m (3.57 ft)
- Standard deviation for crack spacing = 0.64 m (2.09 ft)
- Coefficient of variation of the crack spacing = 58 percent
- Figure 19 shows the average crack spacing distribution based on the closest five cracks
- Figure 20 shows actual crack spacing
- Extent of Y cracks = 17 percent
- Cluster cracking was apparent at stations 5+50 and 6+80.
- Other distress types included:
  - One AC patch (moderate severity)
  - One PCC patch (low severity)
  - One PCC patch (moderate severity)
- Pavement shoulders were in fair condition.

Drainage Survey

- Ditches line both sides of the pavement.
- Ditches were lined with vegetation. There was no sign of ponded water or wet ground.

Windshield Survey

- 8.05 km (5 mi) of pavement surveyed was generally in good condition exhibiting about 88.1 m (289 ft) of longitudinal cracking condition, and 14 PCC patches.

Terminal Joint Survey

- Terminal joints used = Lug anchor (two joints surveyed)
- Joints exhibited the following distresses:
    - One joint exhibited 0.93 m^2 (10 ft^2) and the other 0.46 m^2 (5 ft^2) of medium severity spalling. The second joint also had 0.19 m^2 (2 ft^2) of AC patching.

Deflection Testing

- Figure 21 shows normalized/basin (per 4.45 kN (1,000 lb) load) deflections measured along the length of the section (for all seven sensors using the 40.03 kN (9,000 lb) load)
- Load transfer efficiency at cracks at mid-slab
    - 93 to 95 percent in the morning
    - 93 to 94 percent in the afternoon
- Load transfer efficiency at cracks along edge
    - 94 to 97 percent in the morning
    - 93 to 95 percent in the afternoon
- The distribution of slab temperature with depth and time of day was not measured.
- Figure 22 shows the average deflections and ranges of deflections for each sensor for center and edge loading, for morning and afternoon testing (using the 40.03 kN (9,000 lb) load within the selected 30.5-m (100-ft) subsection).
- Figure 23 shows the backcalculated radius of relative stiffness, ℓ, values for basin testing for each of the three nominal load levels of 40.03, 53.38, and 71.17 kN (9,000, 12,000, and 16,000 lb).
- ℓ value based on 40.03 kN (9,000 lb) load basin testing
    - Average = 965 mm (38 in)
    - Standard Deviation = 127 mm (5 in)
- ℓ value at crack locations at mid-slab (morning/afternoon)
    - Average = 889/965 mm (35/38 in)
    - Standard Deviation = 76/51 mm (3/2 in)
- ℓ value at crack locations at edge (morning/afternoon)
    - Average = 991/991 mm (39/39 in)
    - Standard Deviation = 76/51 mm (3/2 in)
- Estimated ℓ value using actual slab thickness and concrete modulus of elasticity and assumed modulus of subgrade reaction
    - Value = 838mm (33 in)
- Figure 24 show the variation of ℓ in relation to crack spacing (average of closest five cracks)

- Average deflection under 40.03 kN (9,000 lb) load (0.000001 mm (0.001 in.))
  - Basin test - 0.124 (4.9)
  - Mid-slab crack location (morning/afternoon) - 0.132/0.127 (5.2/5.0)
  - Edge crack location (morning/afternoon) - 0.254/0.236 (10/9.3)

Crack Width Measurements

- Number of cracks monitored within 30.5 m (100 ft) subsection = 25
- Morning testing
  - Average temperature 102 mm (4 in) below surface = not measured
  - Average crack width = 0.48 mm (0.019 in)
- Afternoon testing
  - Average temperature 102 mm (4 in) below surface = not measured
  - Average crack width = 0.42 mm (0.017 in)
- Slab length change, mm/mm/°C (in/in/°F) = N/A

Reinforcing Steel Depths and Spacing

- Steel depth measurement offsets = 610, 1829, 2438 mm (24, 72, 96 in)
- Range of depth of cover = 79 to 112 mm (3.1 to 4.4 in)
- Average depth of cover = 99 mm (3.9 in)
- Standard deviation for depth of cover = 7.6 mm (0.3 in)
- Steel spacing was about 163 mm (6.4 in)

Corrosion Potential Testing (copper-copper sulfate half cell potentiometer)

- Range of measurements = -0.35 to -0.50 volts
- Average measurement = -0.44 volts
- Standard deviation for the measurement = -0.03 volts
- Potential for corrosion was indicated

Concrete Core Examination for Corrosion

- One of the four cores exhibited spot corrosion on the steel bar.
- Remaining three cores had spread corrosion throughout the surface area of the steel bars.

Profile Testing

- Average IRI over 305-m (1,000-ft) section = 2 402 mm/km (152 in/mi)

Laboratory Testing

The following data were obtained from the testing of the concrete, base/subbase, and subgrade materials:

Concrete

Modulus of Elasticity	-	33 780 MPa (4,900,000 lbf/in^2)
Average Split Tensile Strength	-	4.14 MPa (600 lbf/in^2)
Split Tensile Strength Range	-	2.79 to 4.62 MPa (405 to 670 lbf/in^2)
Coefficient of Thermal Expansion	-	9.49 mm/mm/°C (5.27 in/in/°F)

Base

AASHTO Classification	-	ATB

Subbase

AASHTO Classification	-	N/A

Subgrade

Liquid Limit	-	49 percent
Plastic Limit	-	24 percent
Percent Passing #200 Sieve	-	98.6 percent
AASHTO Classification	-	A-7-5

Chloride content testing was conducted on pulverized concrete retrieved at various depths and locations. Total chloride content and water soluble chloride content values are listed below for IL-3.

Location	**Depth, mm (in)**	**Total Chloride Content (% wt.)**	**Water Soluble Chlorides (% wt.)**
No steel below sample	0-51 (0-2)	0.167	0.152
	51-102 (2-4)	0.046	0.039
Steel directly below sample	0-51 (0-2)	0.129	0.098
	51-102 (2-4)	0.029	0.024

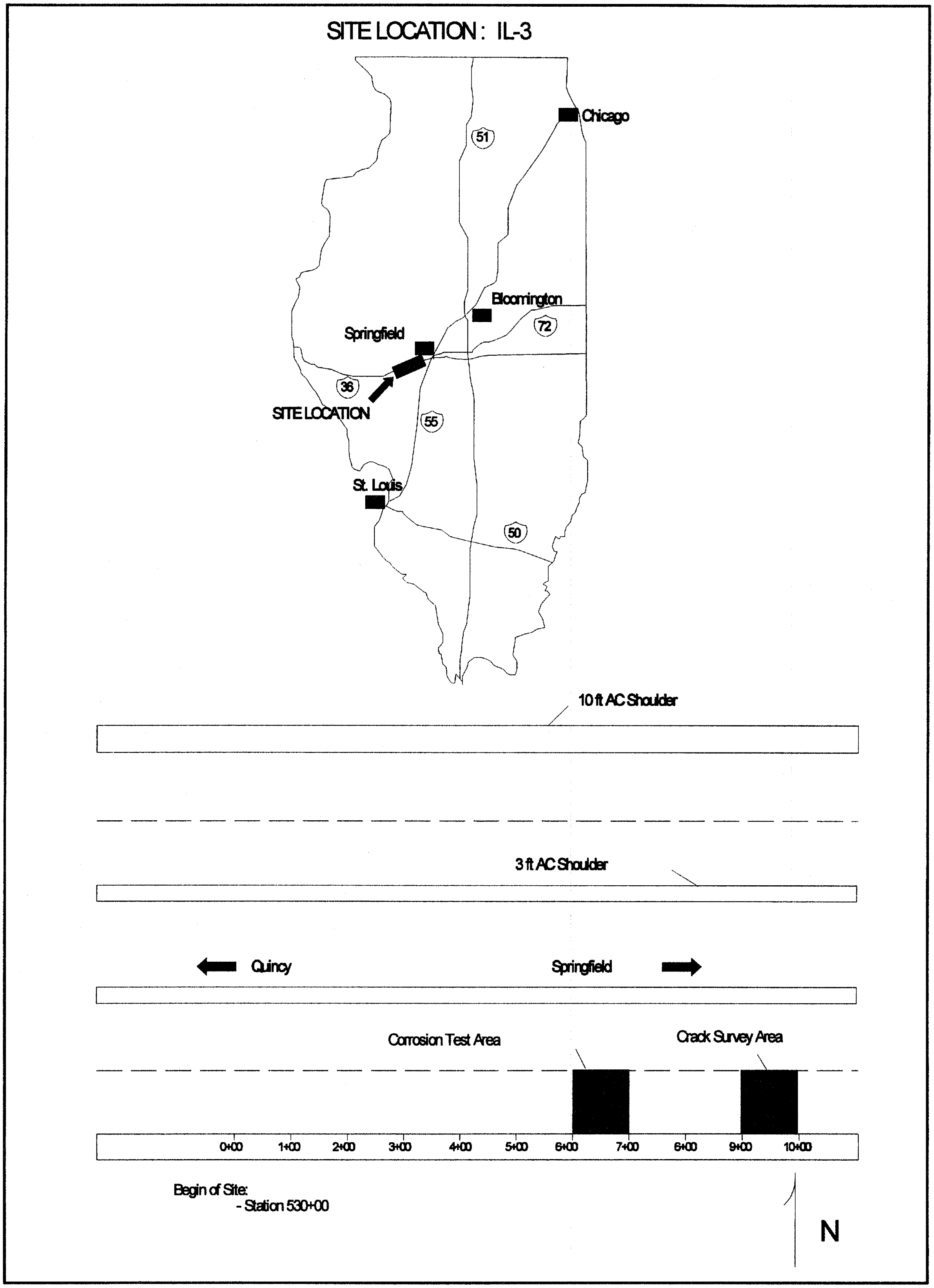
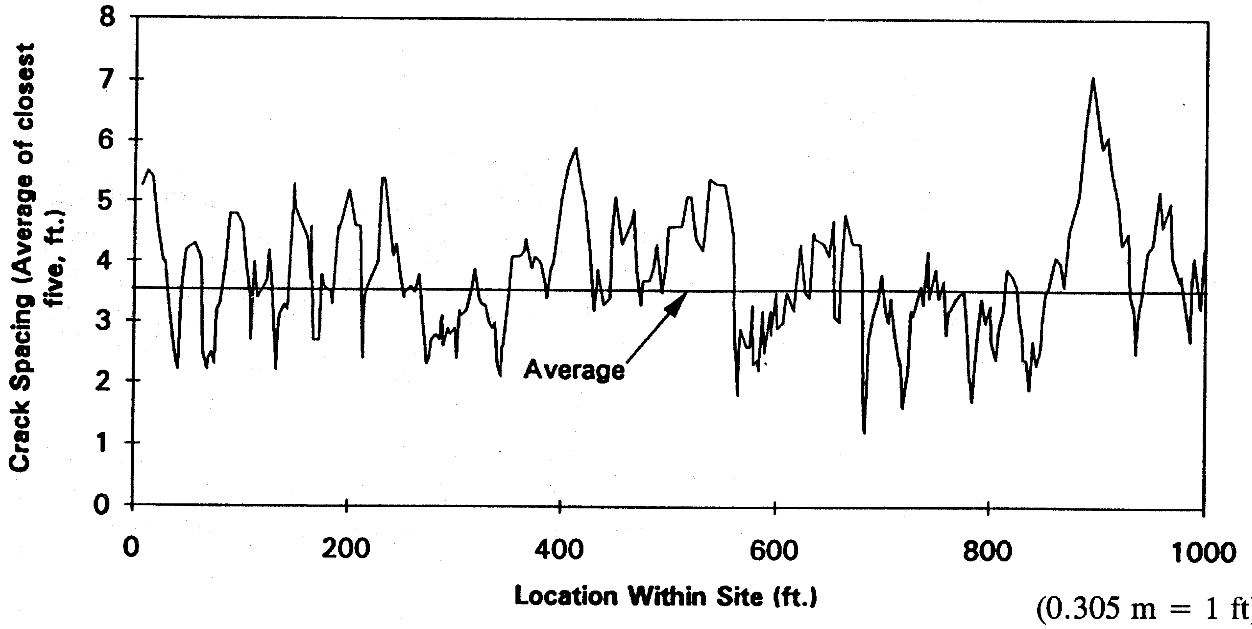


Figure 17. Site details for IL-3.



(0.305 m = 1 ft)

Figure 18. Crack spacing pattern at site IL-3.

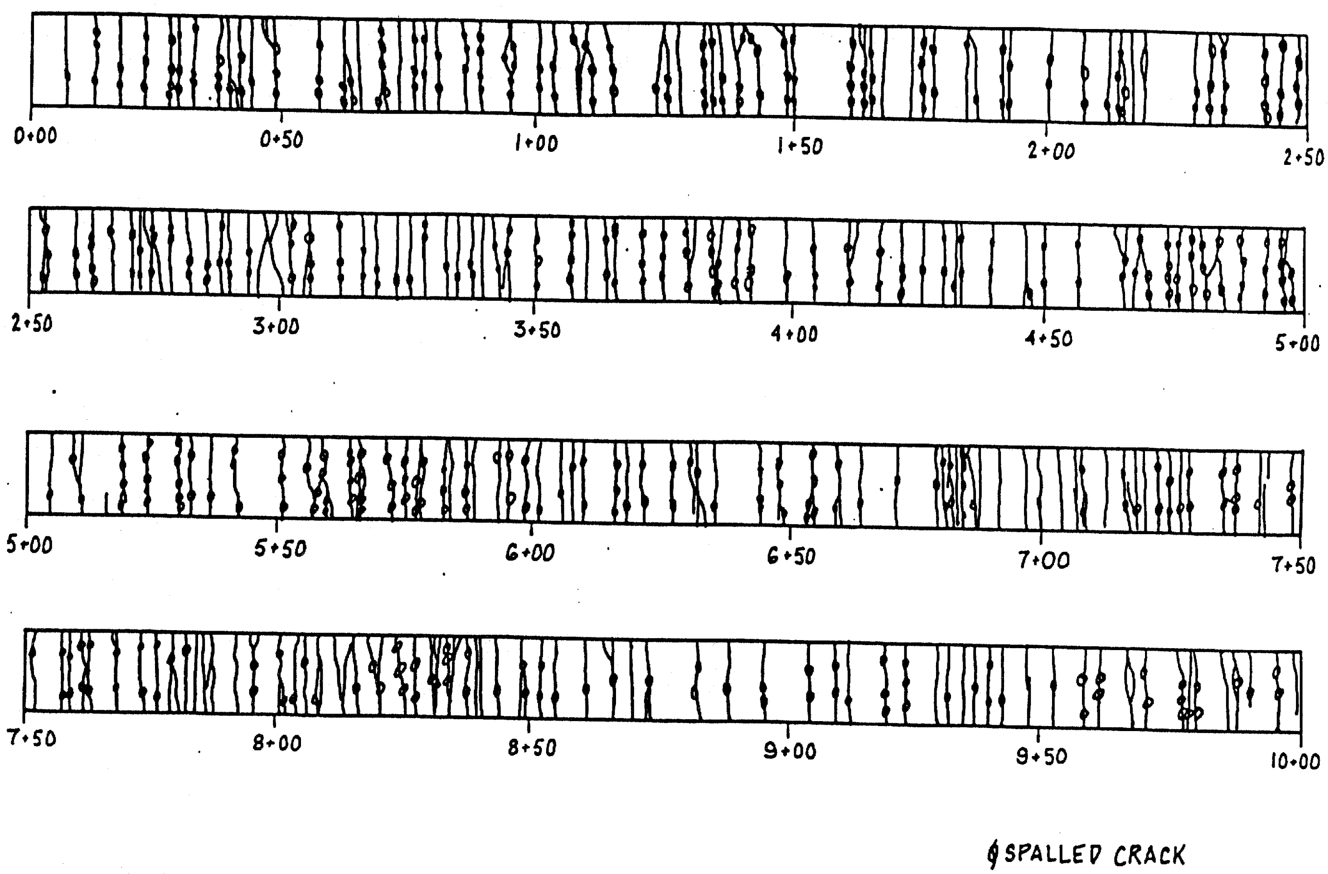


Figure 19. Crack pattern at site IL-3.

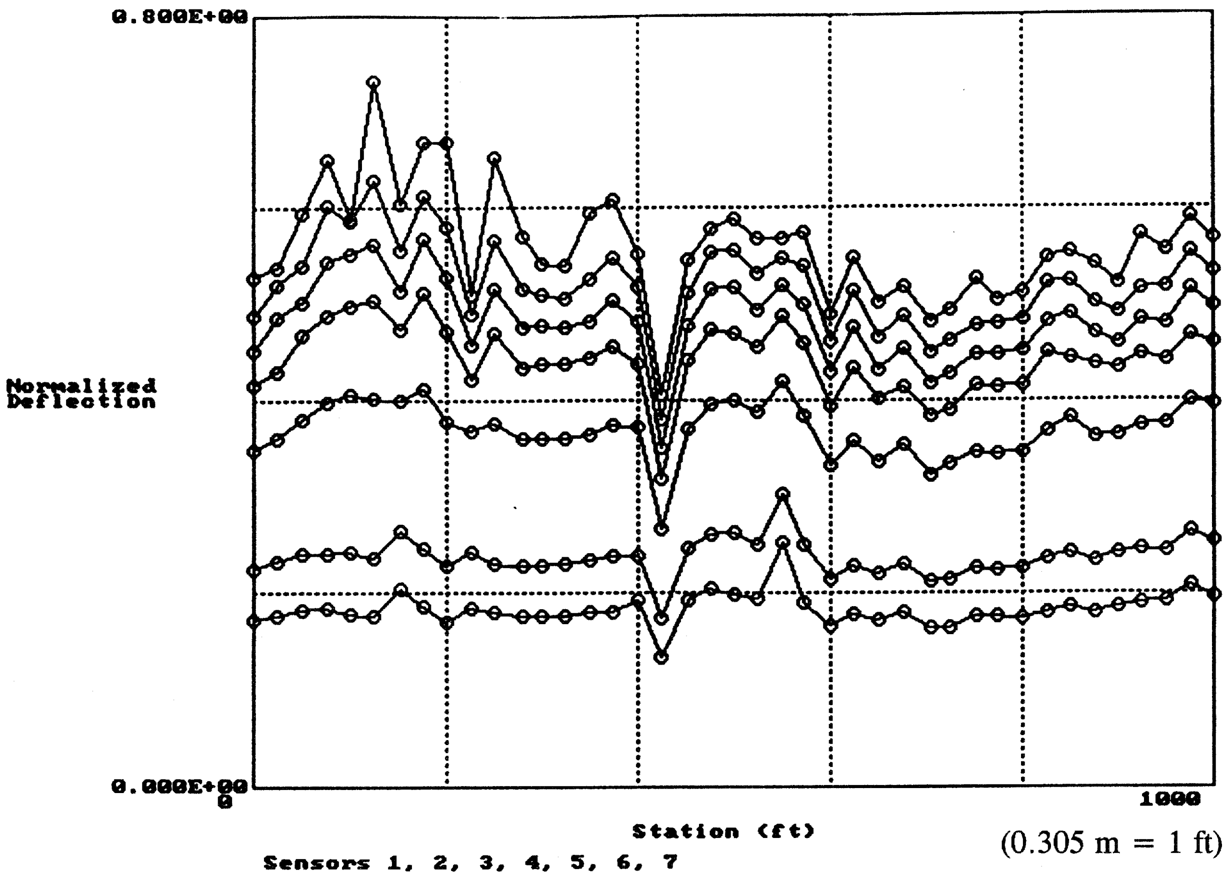
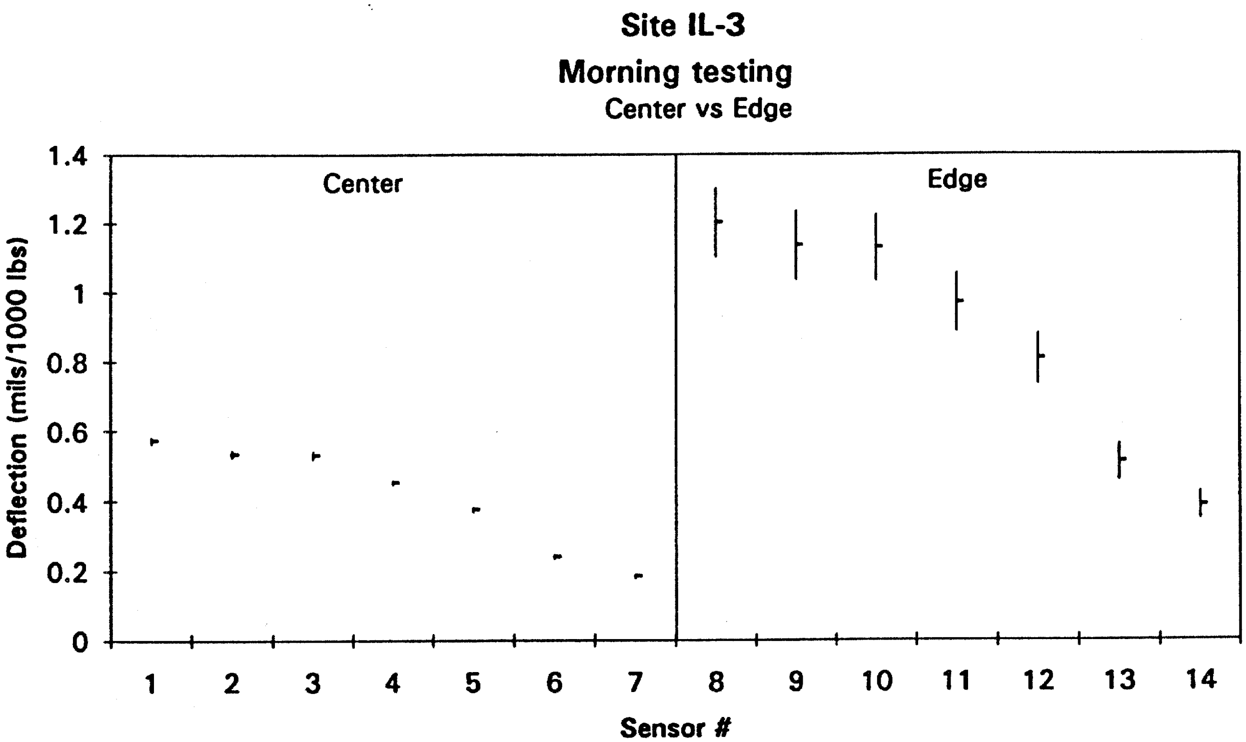


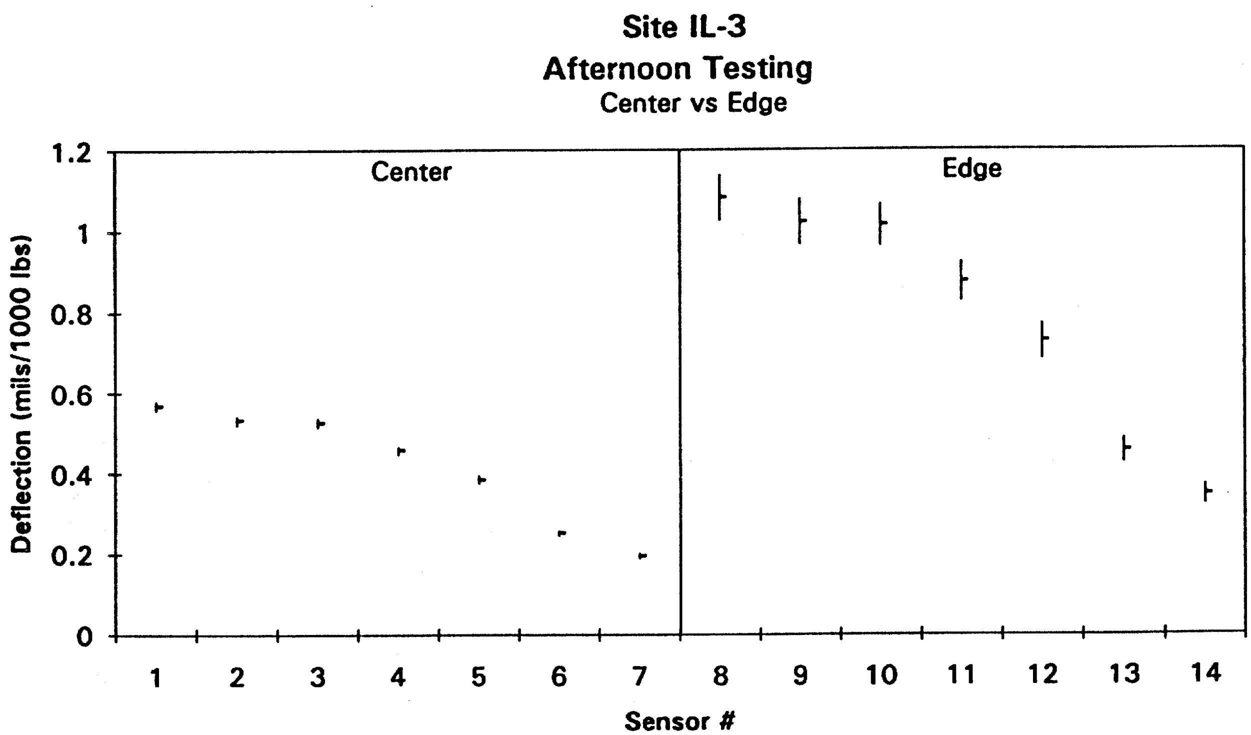
Figure 20. Deflections in inches along the section length at site IL-3.
[normalized per 4.45-kN [1,000-lb] load]

NO TEMPERATURE TESTING WAS PERFORMED AT THIS SITE

(25.4 mm = 1 in; 0.6°C = 1°F)

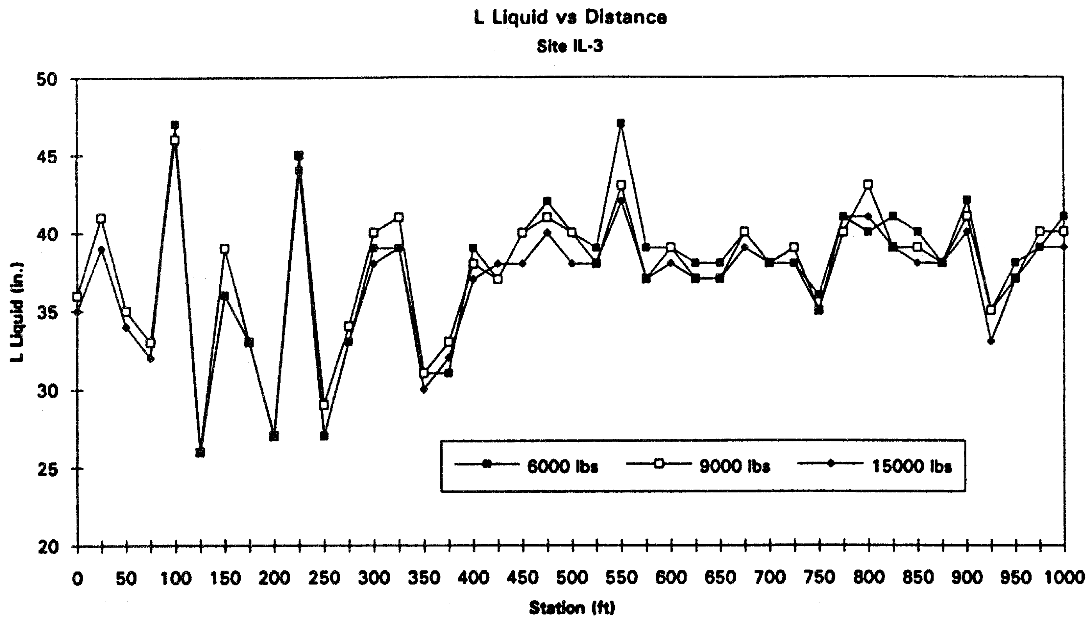
Figure 21. Temperature profile for site IL-3.





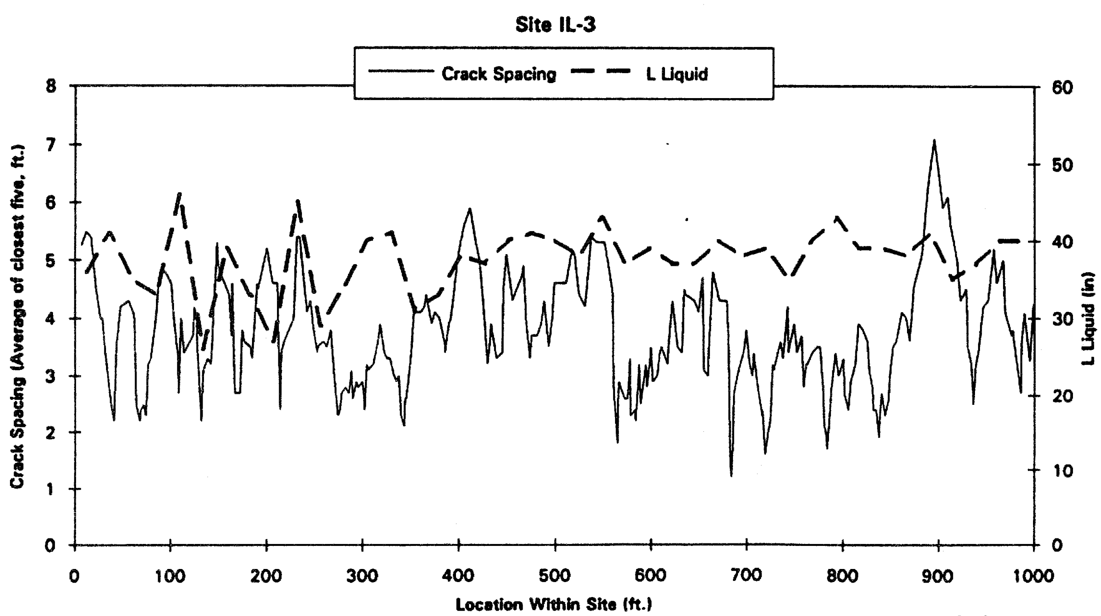
(0.0254 mm = 1 mil; 4.45 kN = 1,000 lb)

Figure 22. Center versus edge deflection for each of the seven sensors at site IL-3.
[40.03 kN (9,000 lb) load]



(25.4 mm = 1 in, 0.305 m = 1 ft)

Figure 23. Backcalculated values of radius of relative stiffness, ℓ (using program ILLI-BACK) for site IL-3.



(0.305 m = 1 ft)

Figure 24. Comparison of radius of relative stiffness values with average crack spacing (nearest five cracks) at site IL-3.

Section IL-4 - Interstate 55, Illinois

Introduction

Section IL-4 was constructed and opened to traffic in 1971. IL-4 is a section of I-55 that runs north to south from St. Louis to Chicago, Illinois, as shown in figure 25. The length of the highway that was considered runs from milepost 40 to 217. A 305-m (1,000-ft) north bound section was selected for detailed inspection between mileposts 86 to 87, as shown in figure 25.

The climate in this region is typically continental with warm summers and fairly cold winters. There are no wet and dry seasons. The highest monthly average temperature is 24.7°C (76.5°F) during July, and the lowest monthly average is -4.1°C (24.6°F) during January. I-55 lies within the wet-freeze environmental zone.

Traffic

Since its opening, the pavement has experienced an estimated 13.7 million ESALs in the design lane. I-55 was designed for a traffic level of 4.8 million ESALs over 20 years.

1991 ESAL in Design Lane	= 950,000
1991 Two-Way ADT	= 17,700 vehicles

Structure

I-55 is a six-lane divided highway. The pavement lane width is 3.7 m (12 ft) with an outside shoulder width of 3.0 m (10 ft) and an inside shoulder width of 1.8 m (6 ft). The shoulders are constructed of asphaltic concrete.

I-55 was designed as a three-layer system comprised of a 203-mm (8-in) CRCP and a 102-mm (4-in) ATB over a subgrade of clay. The CRC pavement was designed with a steel percentage of 0.6. The reinforcing steel is #5 bars spaced at 165 mm (6.5 in) with no transverse steel. The bars were tube fed during the construction of the pavement and placed as a single layer. The bars are not epoxy coated.

The average concrete pavement thickness for IL-4 was determined to be 234 mm (9.2 in) based on cores taken as part of this study. The concrete is considered well graded, and its consolidation is average. The aggregate type used was a crushed stone with a maximum size of 38 mm (1½ in).

The ATB was considered to be well graded and was 97 mm (3.8 in) thick. The clay subgrade has an AASHTO classification of A-7-5. The Atterberg Limits of the subgrade were a liquid limit of 44 and a plastic limit of 21.

Design/Construction

The thickness design records for I-55 were not available. It is known that a minimum thickness design method was used. The design life chosen was 20 years with a design traffic level of 4.8 million ESALs. The concrete properties that were assumed are a modulus of elasticity of 27 580 MPa (4,000,000 lbf/in^2) and a minimum modulus of rupture equal to 4.48 MPa (650 lbf/in^2). The bridge approach pavement type is concrete.

Performance

The performance of I-55 has been evaluated every 2 years since 1982 according to the CRS scale, 1.0 to 9.0. Rating 1.0 is rough and unrideable while rating 9.0 is excellent and considered a new pavement. The 1982 CRS value was 7.4, and the 1990 value was 6.5. The CRS data are given below:

Year	CRS Value
1982	7.4
1984	7.4
1986	7.1
1988	7.1
1990	6.5

Historical Data (M&R)

The only maintenance work that has been reported is CRCP patching done in 1989.

Field Investigation Data

Visual Condition Survey

- Total number of cracks = 472
  - 0 were at high severity
  - 463 were at medium severity with minor spalling
  - 9 were at low severity
- Average crack spacing = 0.65 m (2.12 ft)
- Standard deviation for crack spacing = 0.35 m (1.16 ft)
- Coefficient of variation of the crack spacing = 55 percent
- Figure 26 shows the average crack spacing distribution based on the closest five cracks
- Figure 27 shows actual crack spacing
- Extent of Y cracks = 15 percent
- Cluster cracking was apparent at stations 1+50, 3+00, 5+00, and 8+50
- Other distress types included - None
- Pavement shoulders were in fair condition.

Drainage Survey

- Ditches line both sides of the pavement.
- Ditches were lined with vegetation. There was no sign of ponded water or wet ground.

Windshield Survey

- 8.05 km (5 mi) of pavement surveyed was generally in good condition exhibiting only 9 PCC patches.

Terminal Joint Survey

- Terminal joints used = Lug anchor (two joints surveyed)
- Joints exhibited the following distresses:
    - Both exhibited faulting in excess of 6.4 mm (0.25 in)

Deflection Testing

- Figure 28 shows normalized/basin (per 4.45 kN (1,000 lb) load) deflections measured along the length of the section (for all seven sensors using the 40.03 kN (9,000 lb) load)
- Load transfer efficiency at cracks at mid-slab
    - 92 to 97 percent in the morning
    - 90 to 97 percent in the afternoon
- Load transfer efficiency at cracks along edge
    - 94 to 96 percent in the morning
    - 92 to 95 percent in the afternoon
- Figure 29 shows the distribution of slab temperatures with depth and time of day.
- Figure 30 shows the average deflections and ranges of deflections for each sensor for center and edge loading, for morning and afternoon testing (using the 40.03 kN (9,000 lb) load within the selected 30.5-m (100-ft) subsection).
- Figure 31 shows the backcalculated radius of relative stiffness, ℓ, values for basin testing for each of the three nominal load levels of 40.03, 53.38, and 71.17 kN (9,000, 12,000, and 16,000 lb).
- ℓ value based on 40.03 kN (9,000 lb) load basin testing
    - Average = 1.07 m (42 in)
    - Standard Deviation = 76 mm (3 in)
- ℓ value at crack locations at mid-slab (morning/afternoon)
    - Average = 1 016/991 mm (40/39 in)
    - Standard Deviation = 51/51 mm (2/2 in)
- ℓ value at crack locations at edge (morning/afternoon)
    - Average = 1 041/991 mm (41/39 in)
    - Standard Deviation = 51/76 mm (2/3 in)
- Estimated ℓ value using actual slab thickness and concrete modulus of elasticity and assumed modulus of subgrade reaction
    - Value = 889 mm (35 in)

- Figure 32 show the variation of ℓ in relation to crack spacing (average of the closest five cracks)
- Average deflection under 40.03 kN (9,000 lb) load (0.000001 mm (0.001 in.))
    - Basin test - 0.099 (3.9)
    - Mid-slab crack location (morning/afternoon) - 0.099/0.097 (3.9/3.8)
    - Edge crack location (morning/afternoon) - 0.201/0.175 (7.9/6.9)

Crack Width Measurements

- Number of cracks monitored within the 30.5-m (100-ft) subsection = 43
- Morning testing
    - Average temperature 102 mm (4 in) below surface = 17.8°C (64°F)
    - Average crack width = 0.35 mm (0.014 in)
- Afternoon testing
    - Average temperature 102 mm (4 in) below surface = 24.4°C (76°F)
    - Average crack width = 0.27 mm (0.011 in)
- Slab length change, mm/mm/°C (in/in/°F) = 16.9 millionth (9.4 millionth)

Reinforcing Steel Depths and Spacing

- Steel depth measurement offsets = 610, 1829, 2438 mm (24, 72, 96 in)
- Range of depth of cover = 61 to 89 mm (2.4 to 3.5 in)
- Average depth of cover = 76 mm (3.0 in)
- Standard deviation for depth of cover = 4.6 mm (0.18 in)
- Steel spacing was about 147 mm (5.8 in)

Corrosion Potential Testing (copper-copper sulfate half cell potentiometer)

- Range of measurements = -0.34 to -0.49 volts
- Average measurement = -0.43 volts
- Standard deviation for the measurement = -0.03 volts
- Potential for corrosion was indicated

Concrete Core Examination for Corrosion

- Two cores exhibited spot corrosion on the steel bar
- Two cores exhibited spread corrosion throughout the surface area of the steel bars.

Profile Testing

- Average IRI over 305-m (1,000-ft) section = 2 481 mm/km (157 in/mi)

Laboratory Testing

The following data were obtained from the testing of the concrete, base/subbase, and subgrade materials:

Concrete

Modulus of Elasticity	-	29 650 MPa (4,300,000 lbf/in^2)
Average Split Tensile Strength	-	3.24 MPa (470 lbf/in^2)
Split Tensile Strength Range	-	2.83 to 3.52 MPa (410 to 510 lbf/in^2)
Coefficient of Thermal Expansion	-	9.36 mm/mm/°C (5.20 in/in/°F)

Base

AASHTO Classification	-	ATB

Subbase

AASHTO Classification	-	N/A

Subgrade

Liquid Limit	-	44 percent
Plastic Limit	-	21 percent
Percent Passing #200 Sieve	-	91.1 percent
AASHTO Classification	-	A-7-5

Chloride content testing was conducted on pulverized concrete retrieved at various depths and locations. Total chloride content and water soluble chloride content values are listed below for IL-4.

Location	Depth, mm (in)	Total Chloride Content (% wt.)	Water Soluble Chlorides (% wt.)
No steel below sample	0-51 (0-2)	0.126	0.110
	51-102 (2-4)	0.040	0.024
Steel directly below sample	0-51 (0-2)	0.130	0.107
	51-102 (2-4)	0.082	0.060

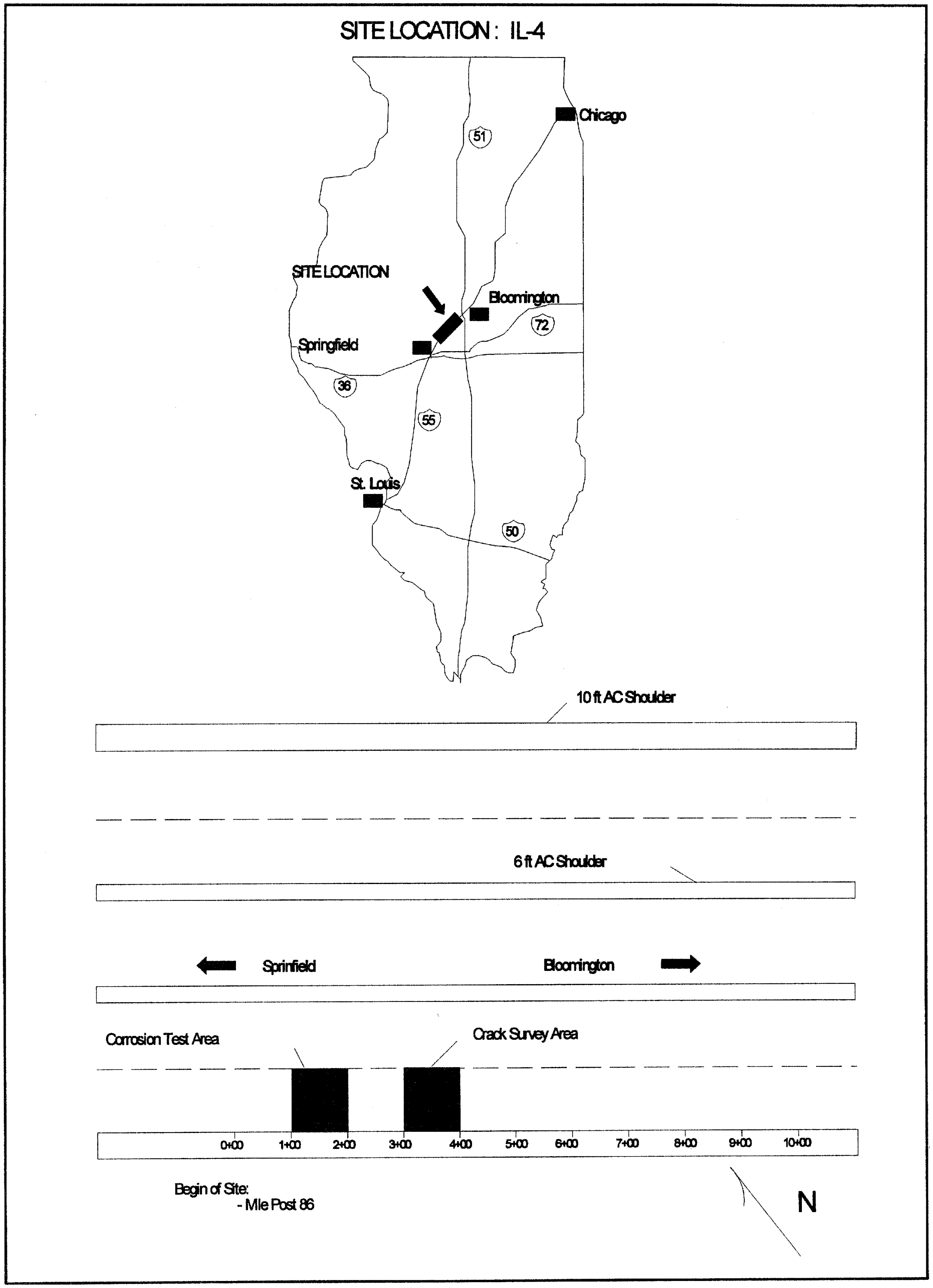
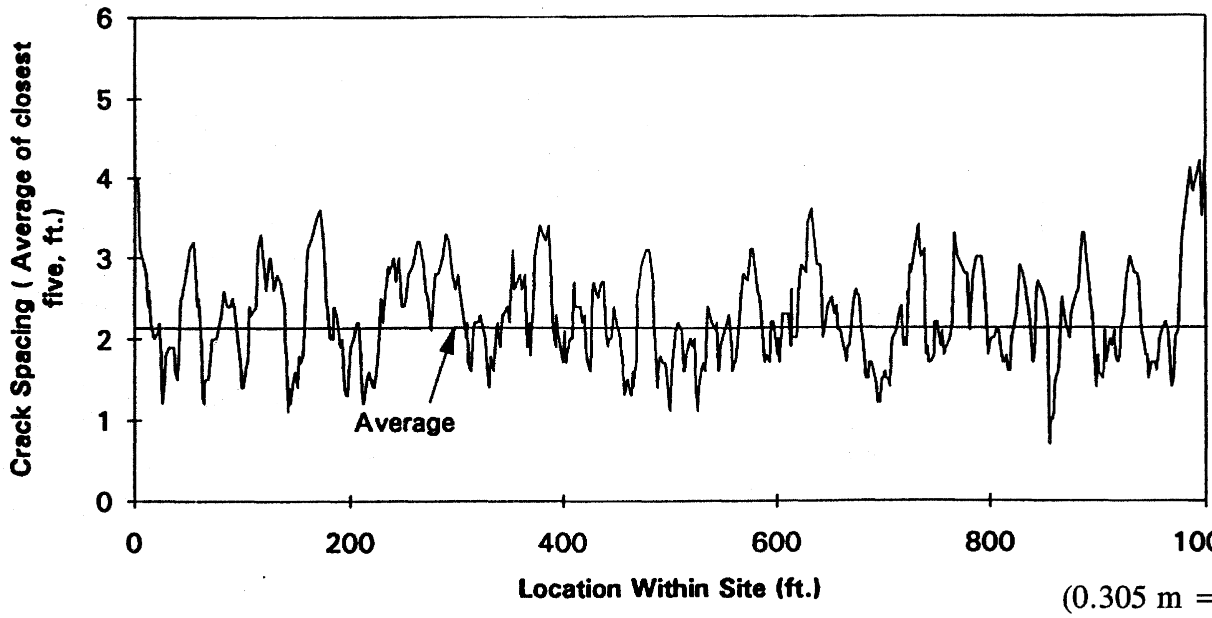


Figure 25. Site details for IL-4.



(0.305 m = 1 ft)

Figure 26. Crack spacing pattern at site IL-4.

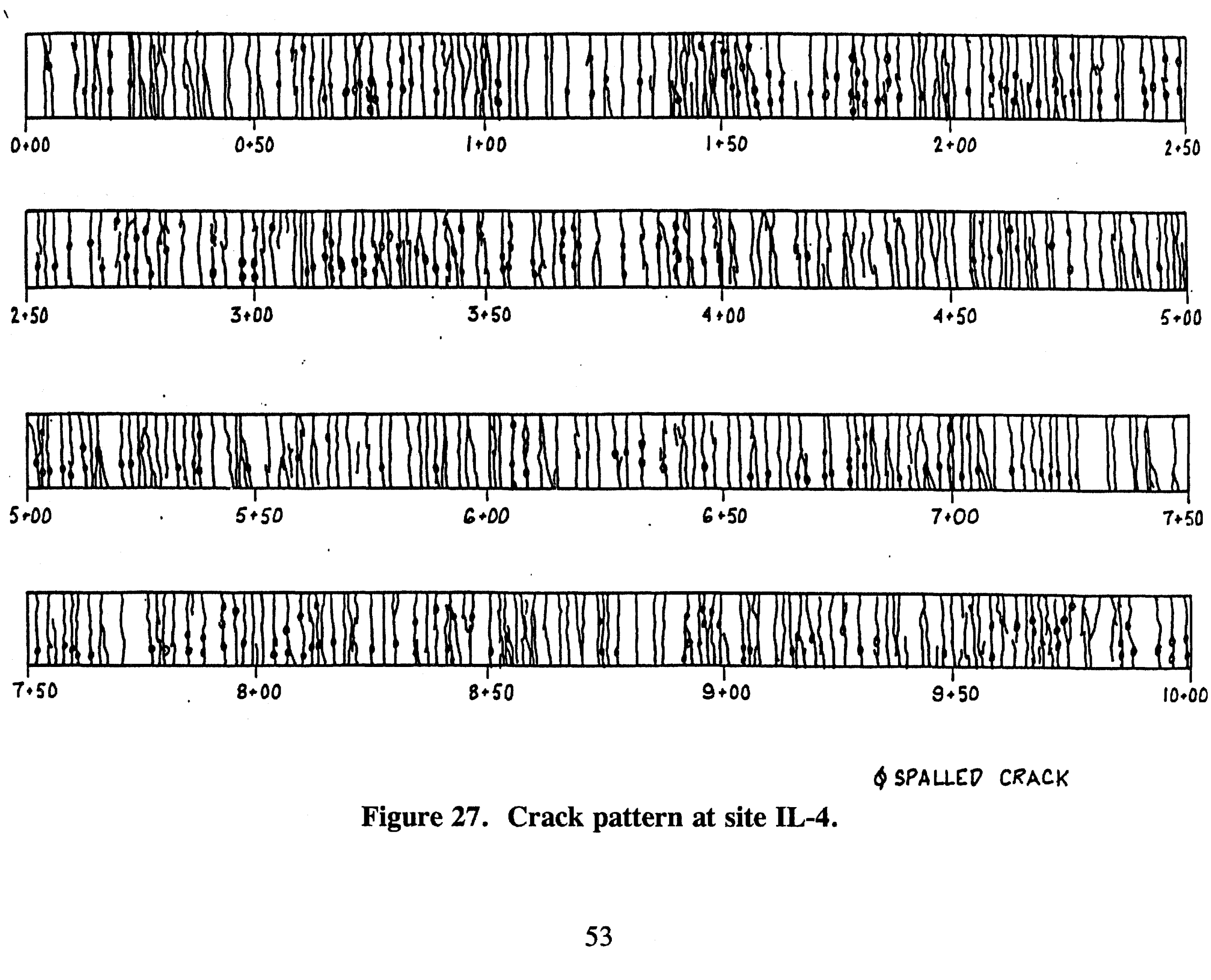
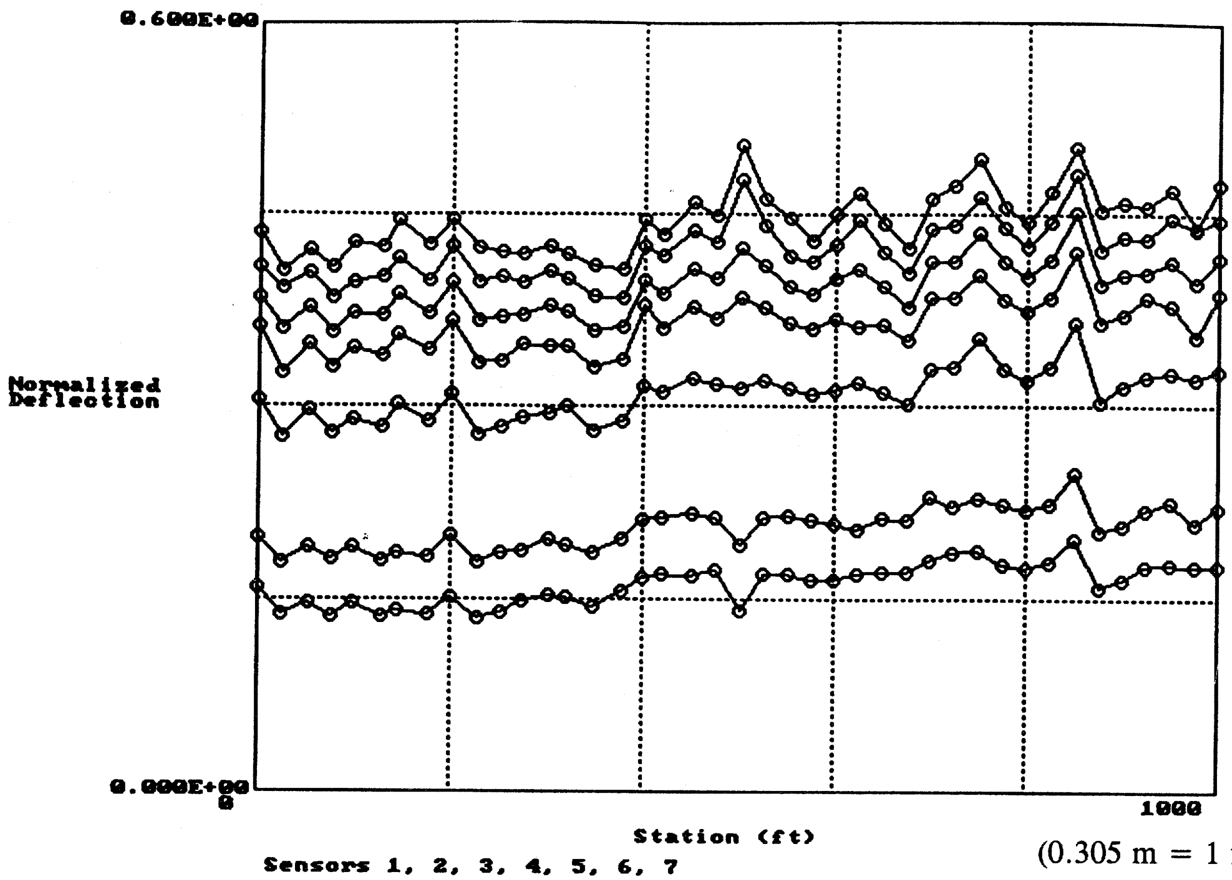
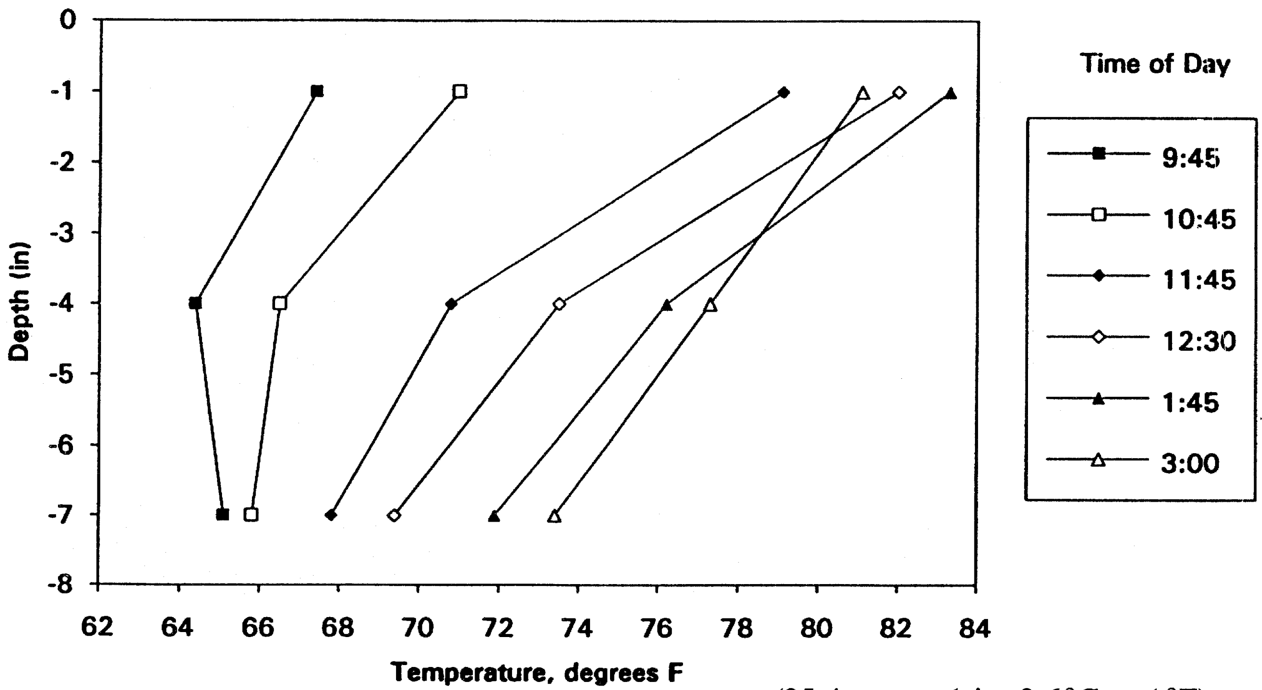


Figure 27. Crack pattern at site IL-4.



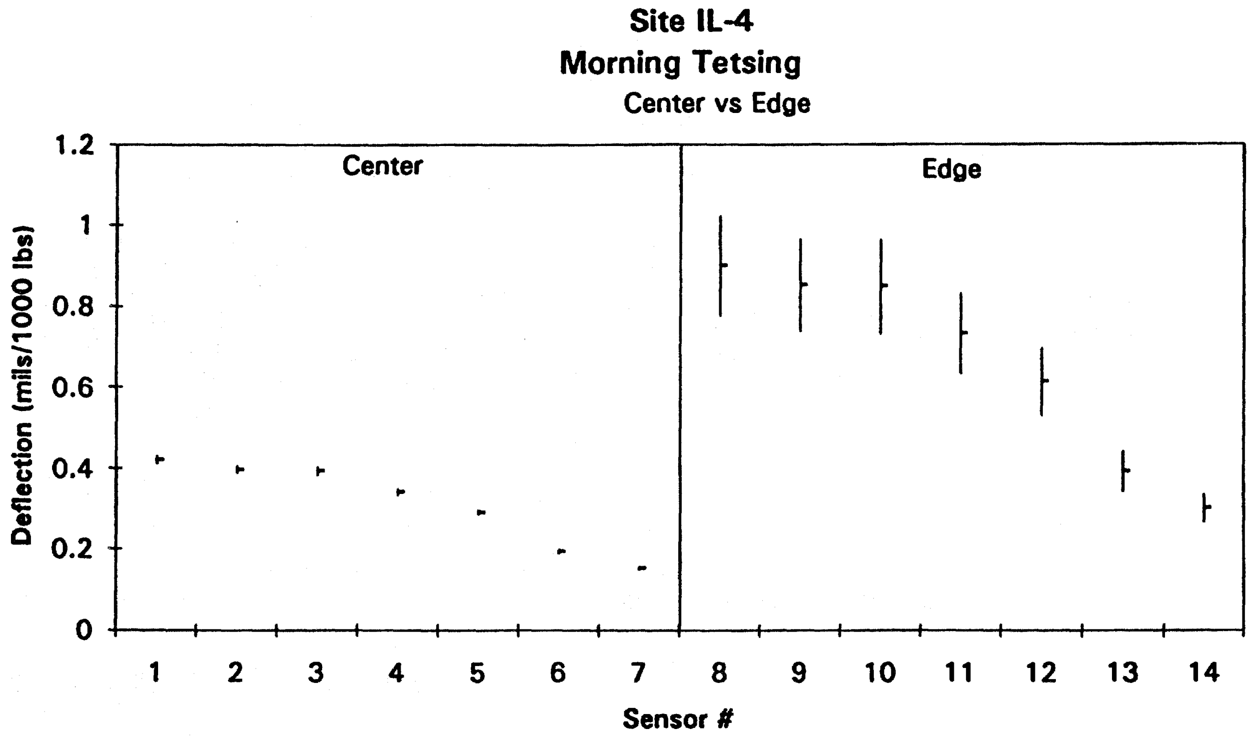
(0.305 m = 1 ft)

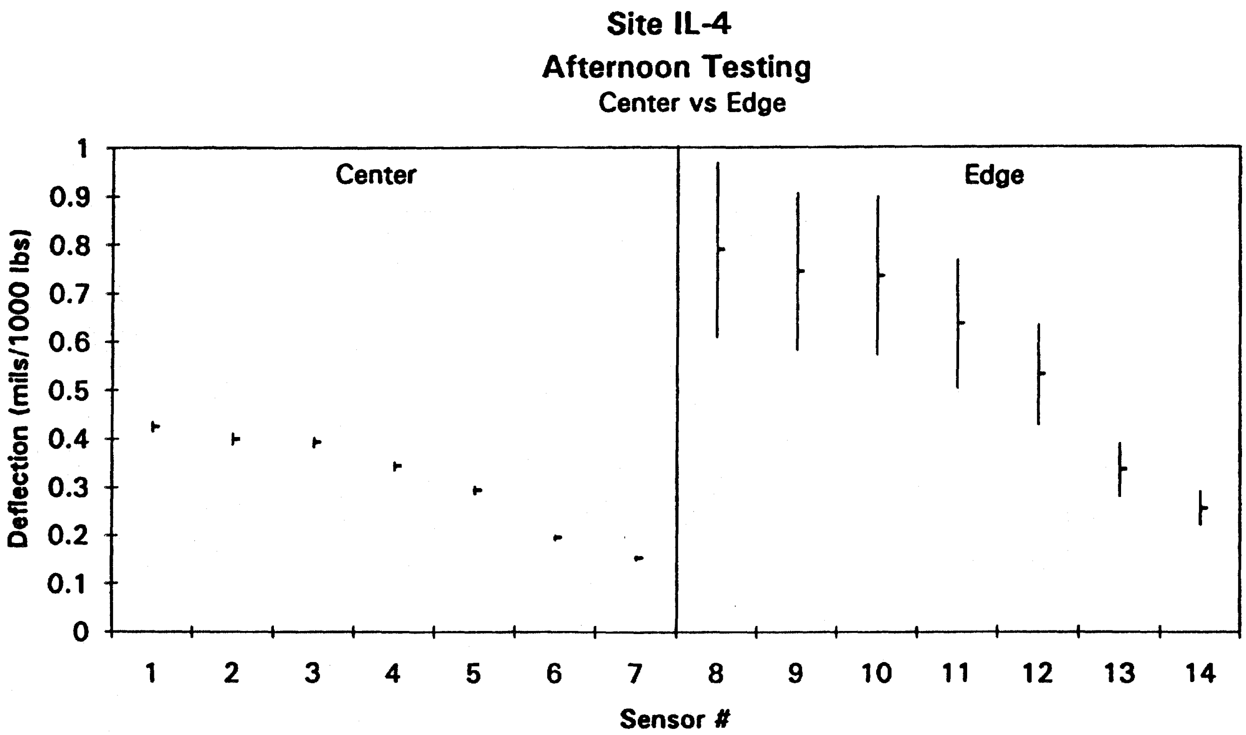
Figure 28. Deflections in inches along the section length at site IL-4.
[normalized per 4.45-kN [1,000-lb] load]



(25.4 mm = 1 in; 0.6°C = 1°F)

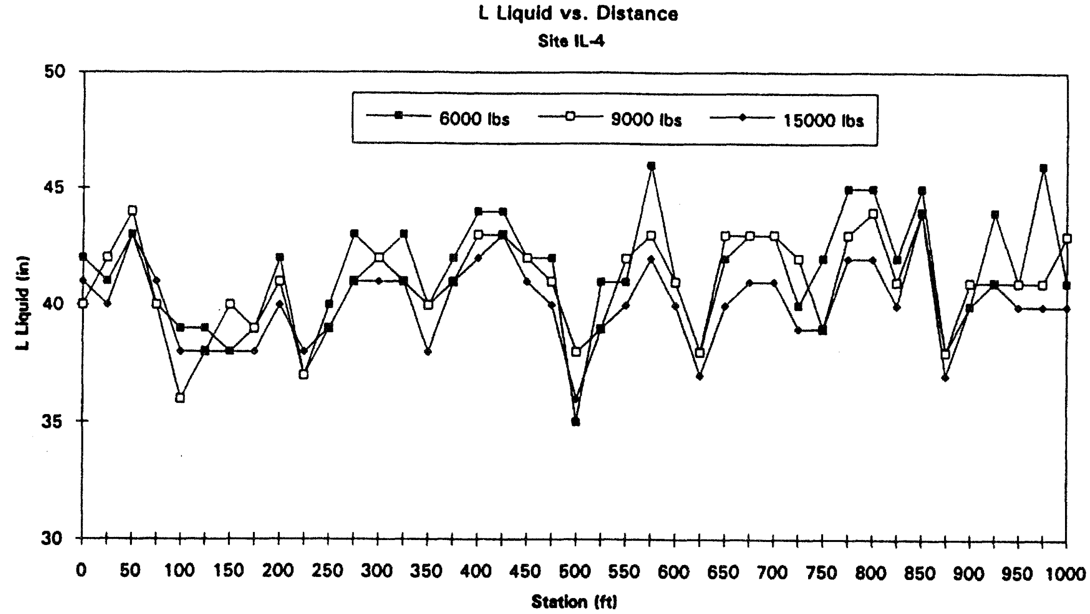
Figure 29. Temperature profile for site IL-4.





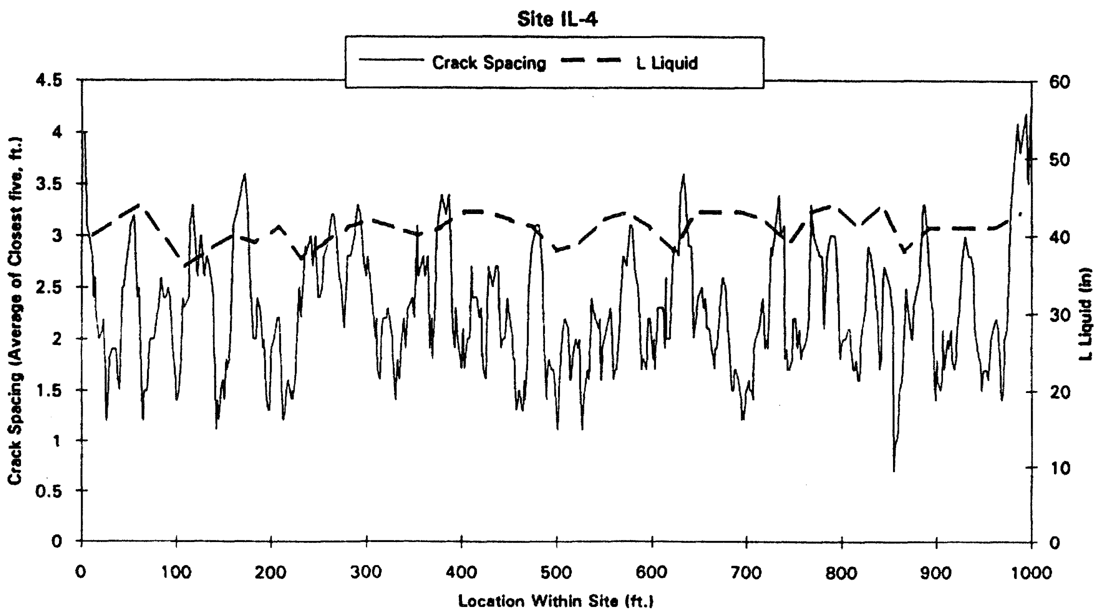
(0.0254 mm = 1 mil; 4.45 kN = 1,000 lb)

Figure 30. Center versus edge deflection for each of the seven sensors at site IL-4.
[40.03 kN (9,000 lb) load]



(25.4 mm = 1 in, 0.305 m = 1 ft)

Figure 31. Backcalculated values of radius of relative stiffness, ℓ (using program ILLI-BACK) at site IL-4.



(0.305 m = 1 ft)

Figure 32. Comparison of radius of relative stiffness values with average crack spacing (nearest five cracks) at site IL-4.

Section IL-5 - US Highway 50, Illinois

Introduction

Section IL-5 was constructed and opened to traffic in 1986. IL-5 is a section of US-50 that runs west to east from East St. Louis to the Illinois-Indiana border as shown in figure 33. A 305-m (1,000-ft) north bound section was selected for detailed inspection between stations 298+80 and 319+90, as shown in figure 33.

The climate in this region is a modified continental climate with warm summers and moderately cold winters. The wet season is from March to May. The dry season is usually during the winter months. The highest monthly average temperature is 26.1°C (78.9°F) during July, and the lowest monthly average is -1.2°C (29.8°F) during January. US-50 lies within the wet-freeze environmental zone.

Traffic

Since its opening, the pavement has experienced an estimated 0.3 million ESALs in the design lane. US-50 was designed for a traffic level of 8,076 ADT per day.

1991 ESAL in Design Lane = 60,000

Structure

US-50 is a four- then a two-lane undivided highway. The pavement lane width is 3.7 m (12 ft) with an outside shoulder width of 2.4 m (8 ft) and an inside shoulder width of 0.9 m (3 ft). <u>The shoulders are constructed of PCC are tied to the CRC mainline.</u>

US-50 was designed as a three-layer system comprised of a 203-m (8-in) CRCP and a <u>102-m (4-in) econocrete base</u> over a subgrade of clay. The CRC pavement was designed with a steel percentage of 0.5. The reinforcing steel is #5 bars spaced at 194 mm (7.625 in) with transverse steel. The transverse steel is #4 bars spaced at 1219 mm (48 in). The bars were placed on chairs as a single layer during the construction of the pavement. The bars are not epoxy coated.

The average concrete pavement thickness for IL-5 was determined to be 216 mm (8.5 in) based on cores taken as part of this study. The concrete is considered well graded, and its consolidation is average. The aggregate type used was a crushed stone with a maximum size of 38 mm (1½ in).

The econocrete base was considered to be well graded and was 130 mm (5.1 in) thick. The clay subgrade has an AASHTO classification of A-7-5. The Atterberg Limits of the subgrade were a liquid limit of 42 and a plastic limit of 19.

Design/Construction

The design for US-50 was experimental. The design was done by the IDOT. The design life chosen was 20 years. The design traffic level of 8,075 ADT per day was used. The concrete properties that were assumed were a modulus of elasticity of 27 600 MPa (4,000,000 lbf/in^2) and a minimum modulus of rupture equal to 4.48 MPa (650 lbf/in^2). The terminal joint design system was a wide flange beam. The bridge approach pavement type is concrete.

The concrete properties specified were slump of 19 to 38 mm (0.75 to 1.5 in), air content of 5 to 8 percent, and a minimum strength of 4.48 MPa (650 lbf/in^2) at 14 days.

Performance

The performance of this section has not been reported.

Historical Data (M&R)

There has been no major maintenance work done on this section.

Field Investigation Data

Visual Condition Survey

- Total number of cracks = 356
    - 0 were at high severity
    - 12 were at medium severity
    - 344 were at low severity
- Average crack spacing = 0.9 m (3.0 ft)
- Standard deviation for crack spacing = 0.64 m (2.1 ft)
- Coefficient of variation of the crack spacing = 70 percent
- Figure 34 shows average crack spacing distribution based on closest five cracks
- Figure 35 shows actual crack spacing
- Extent of Y cracks = 18 percent
- Cluster cracking was apparent at stations 2+00 and 7+00
- Other distress types included - None
- Pavement shoulders were in good condition.

Drainage Survey

- Ditches line both sides of the pavement.
- Ditches were lined with vegetation. There was no sign of ponded water or wet ground.

Windshield Survey

- 4.83 km (3 mi) of pavement surveyed were generally in good condition exhibiting about 104 m (340 ft) of longitudinal cracking condition and three PCC patches.

Terminal Joint Survey

- Terminal joints used = Wide flange (two surveyed)
- Joints exhibited no distresses.

Deflection Testing

- Figure 36 shows normalized/basin (per 4.45 kN (1,000 lb) load) deflections measured along the length of the section (for all seven sensors using the 40.03 kN (9,000 lb) load)
- Load transfer efficiency at cracks at mid-slab
  - 92 to 94 percent in the morning
  - 91 to 94 percent in the afternoon
- Load transfer efficiency at cracks along edge
  - 93 to 96 percent in the morning
  - 91 to 97 percent in the afternoon
- Figure 37 shows the distribution of slab temperatures with depth and time of day.
- Figure 38 shows the average deflections and ranges of deflections for each sensor for center and edge loading, for morning and afternoon testing (using the 40.03 kN (9,000 lb) load within the selected 30.5-m (100-ft) subsection).
- Figure 39 shows the backcalculated radius of relative stiffness, ℓ, values for basin testing for each of the three nominal load levels of 40.03, 53.38, and 71.17 kN (9,000, 12,000, and 16,000 lb).
- ℓ value based on 40.03 kN (9,000 lb) load basin testing
  - Average = 965 mm (38 in)
  - Standard Deviation = 152 mm (6 in)
- ℓ value at crack locations at mid-slab (morning/afternoon)
  - Average = 889/914 mm (35/36 in)
  - Standard Deviation = 102/51 mm (4/2 in)
- ℓ value at crack locations at edge (morning/afternoon)
  - Average = 965/991 mm (38/39 in)
  - Standard Deviation = 178/102 mm (7/4 in)
- Estimated ℓ value using actual slab thickness and concrete modulus of elasticity and assumed modulus of subgrade reaction
  - Value = 864 mm (34 in)
- Figure 40 shows the variation of ℓ in relation to crack spacing (average of the closest five cracks)
- Average deflection under 40.03 kN (9,000 lb) load (0.000001 mm (0.001 in.))
  - Basin test - 0.112 (4.4)

- Mid-slab crack location (morning/afternoon) - 0.109/0.104 (4.3/4.1)
- Edge crack location (morning/afternoon) - 0.236/0.168 (9.3/6.6)

Crack Width Measurements

- Number of cracks monitored within the 30.5-m (100-ft) subsection = 35
- Morning testing
  - Average temperature 102 mm (4 in) below surface = 16.7°C (62°F)
  - Average crack width = 0.29 mm (0.011 in)
- Afternoon testing
  - Average temperature 102 mm (4 in) below surface = 23.9°C (75°F)
  - Average crack width = 0.22 mm (0.009 in)
- Slab length change, mm/mm/°C (in/in/°F) = 11.2 millionth (6.2 millionth)

Reinforcing Steel Depths and Spacing

- Steel depth measurement offsets = 610, 1 829, 2 438 mm (24, 72, 96 in)
- Range of depth of cover = 71 to 99 (2.8 to 3.9 in)
- Average depth of cover = 81 mm (3.2 in)
- Standard deviation for depth of cover = 5.1 mm (0.2 in)
- Steel spacing was about 196 mm (7.7 in)

Corrosion Potential Testing (copper-copper sulfate half cell potentiometer)

- Range of measurements = -0.25 to -0.43 volts
- Average measurement = -0.32 volts
- Standard deviation for the measurement = -0.03 volts
- Potential for corrosion was marginal

Concrete Core Examination for Corrosion

- Two cores exhibited spot corrosion at the steel bar
- Two cores exhibited spread corrosion throughout the surface area of the steel bar

Profile Testing

- Average IRI over the 305-m (1,000-ft) section = 2 212 mm/km (140 in/mi)

Laboratory Testing

Concrete

Modulus of Elasticity	-	33 780 MPa (4,900,000 lbf/in^2)
Average Split Tensile Strength	-	3.34 MPa (485 lbf/in^2)

Split Tensile Strength Range	-	3.31 to 3.45 MPa (480 to 500 lbf/in^2)
Coefficient of Thermal Expansion	-	7.43 mm/mm/°C (4.13 in/in/°F)

Base

AASHTO Classification	-	Econocrete base

Subbase

AASHTO Classification	-	N/A

Subgrade

Liquid Limit	-	42 percent
Plastic Limit	-	19 percent
Percent Passing #200 Sieve	-	94.7 percent
AASHTO Classification	-	A-7-5

Chloride content testing was conducted on pulverized concrete retrieved at various depths and locations. Total chloride content and water soluble chloride content values are listed below for IL-5.

Location	Depth, mm (in)	Total Chloride Content (% wt.)	Water Soluble Chlorides (% wt.)
No steel below sample	0-51 (0-2)	0.093	0.067
	51-102 (2-4)	0.000	0.000
Steel directly below sample	0-51 (0-2)	0.082	0.780
	51-102 (2-4)	0.014	0.010

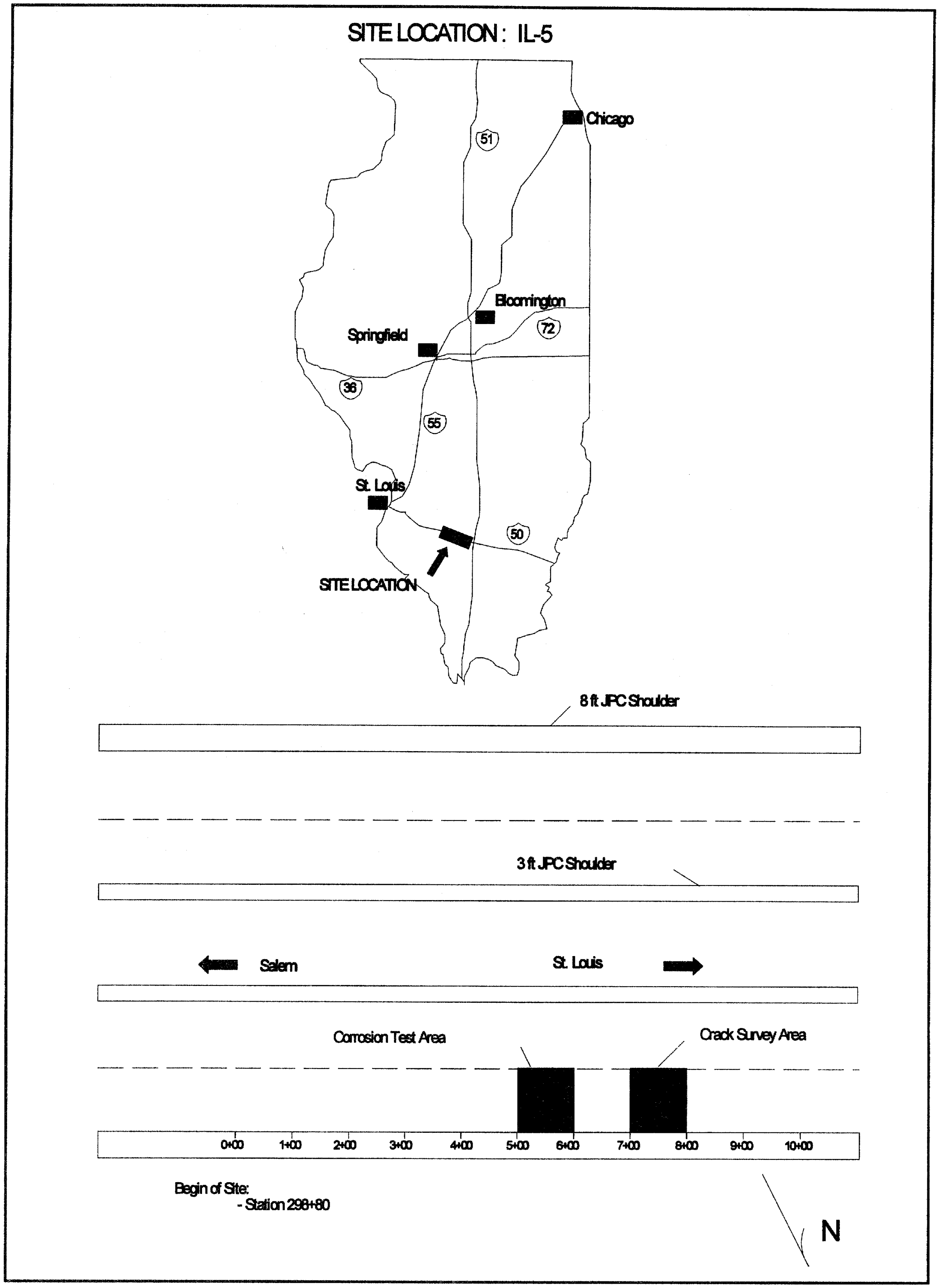
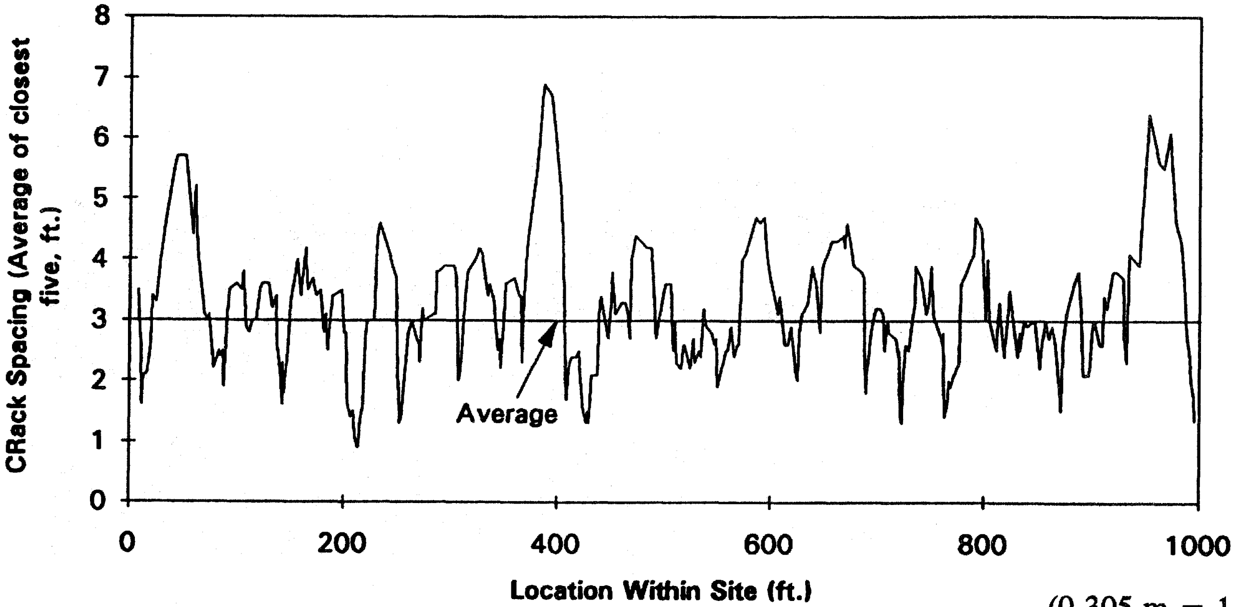


Figure 33. Site details for IL-5.



(0.305 m = 1 ft)

Figure 35. Crack spacing pattern at site IL-5.

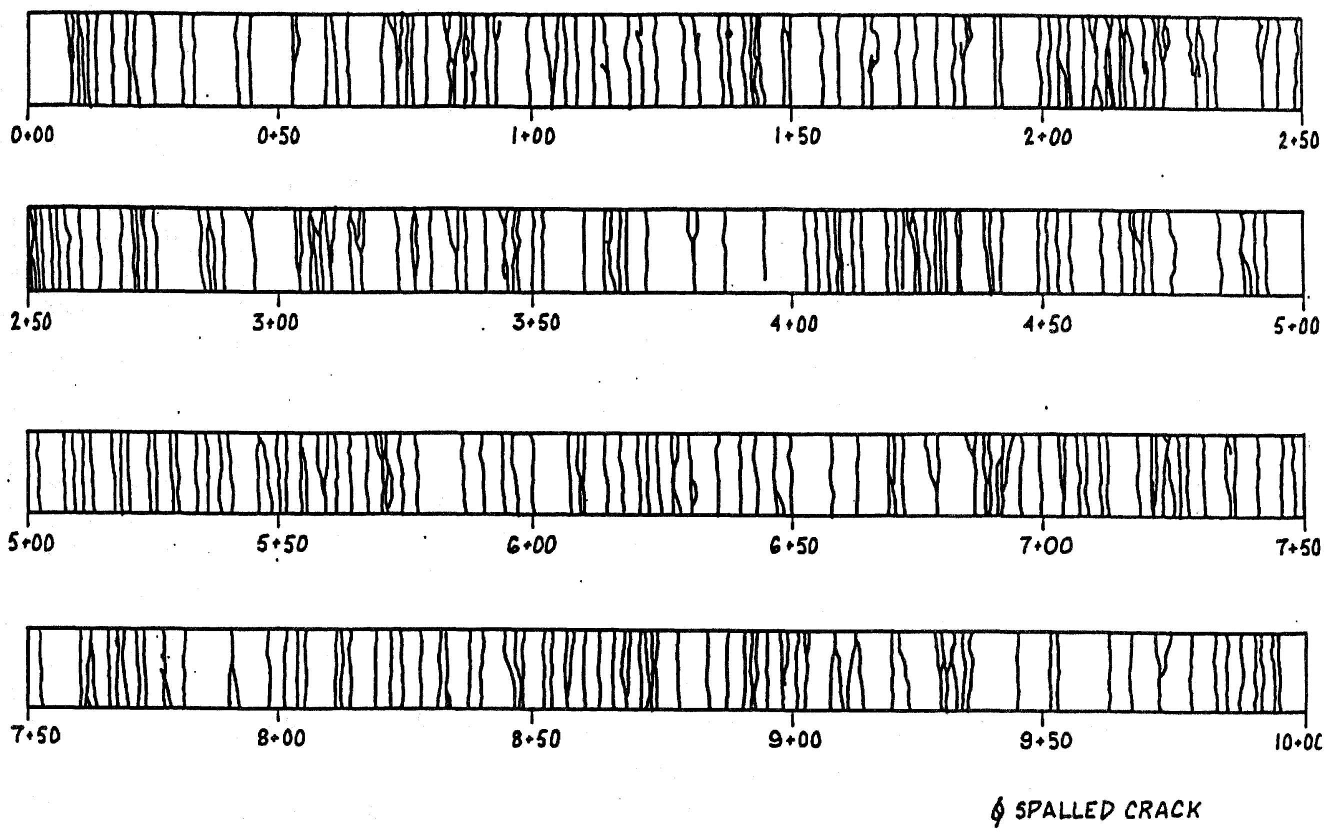
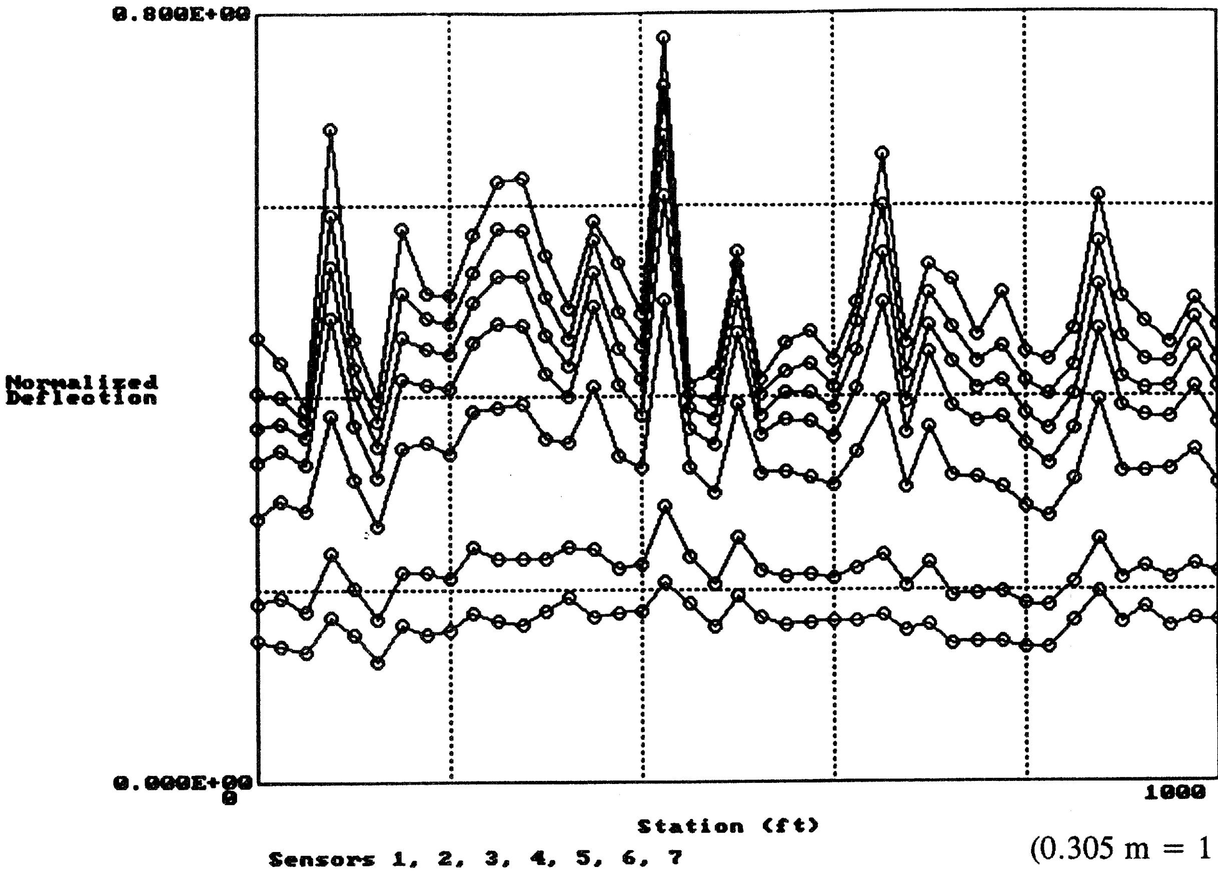
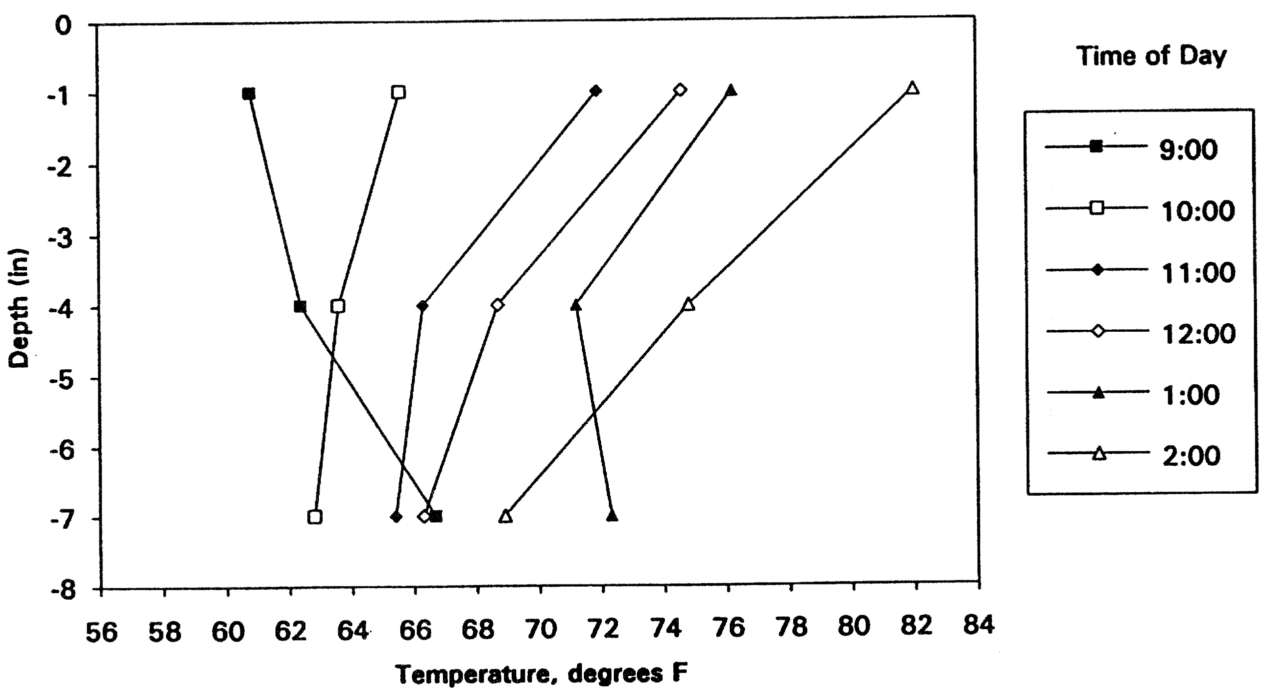


Figure 36. Crack pattern at site IL-5.



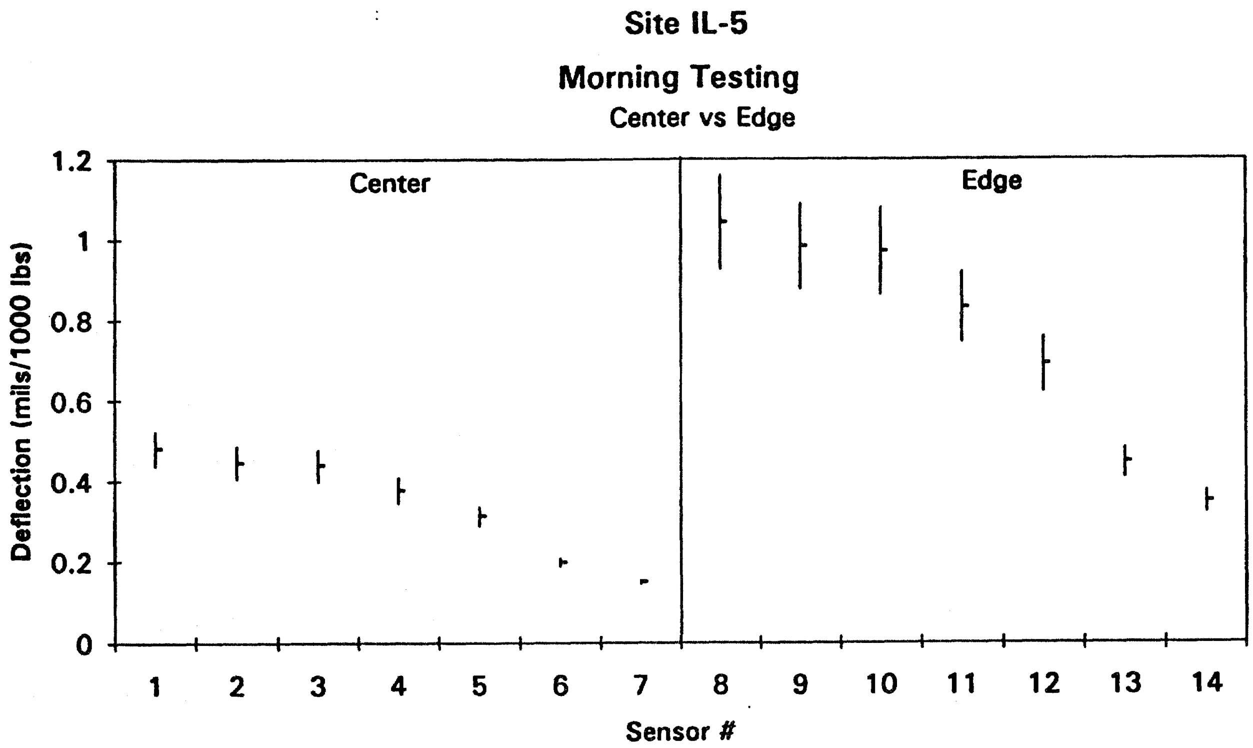
(0.305 m = 1 ft)

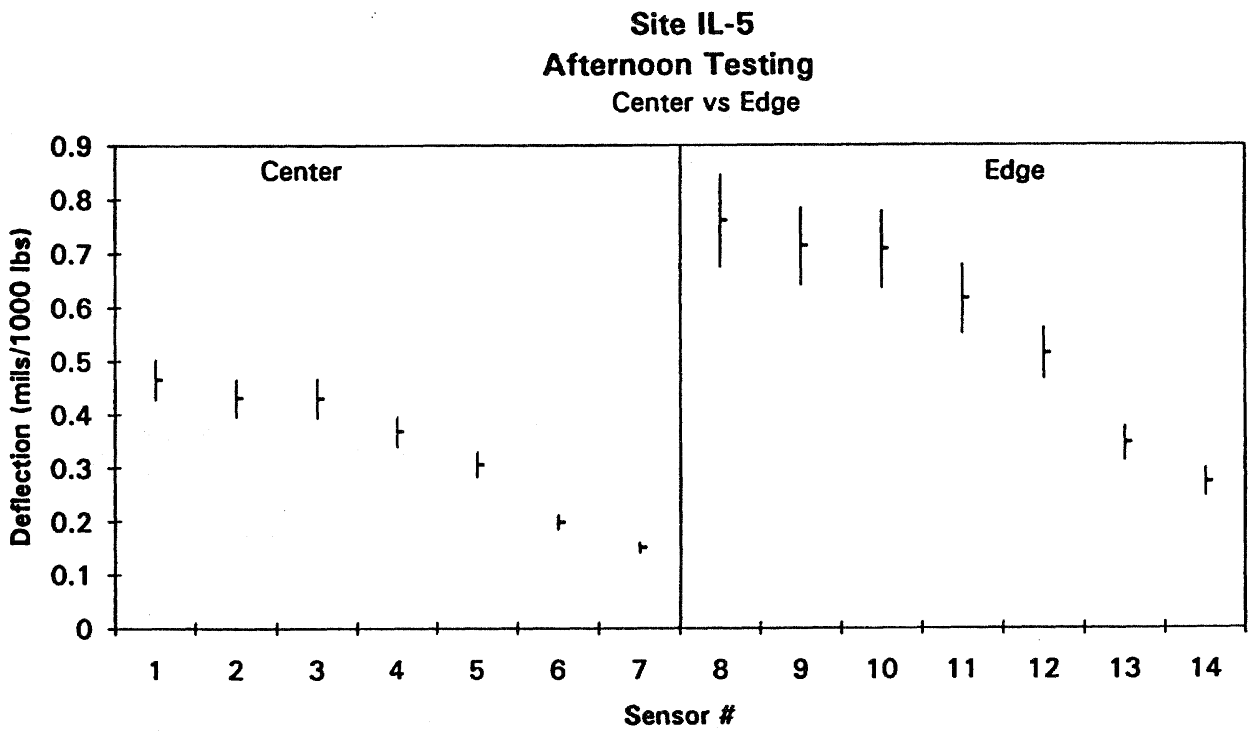
Figure 36. Deflections in inches along the section length at site IL-5.
[normalized per 4.45-kN [1,000-lb] load]



(25.4 mm = 1 in; 0.6°C = 1°F)

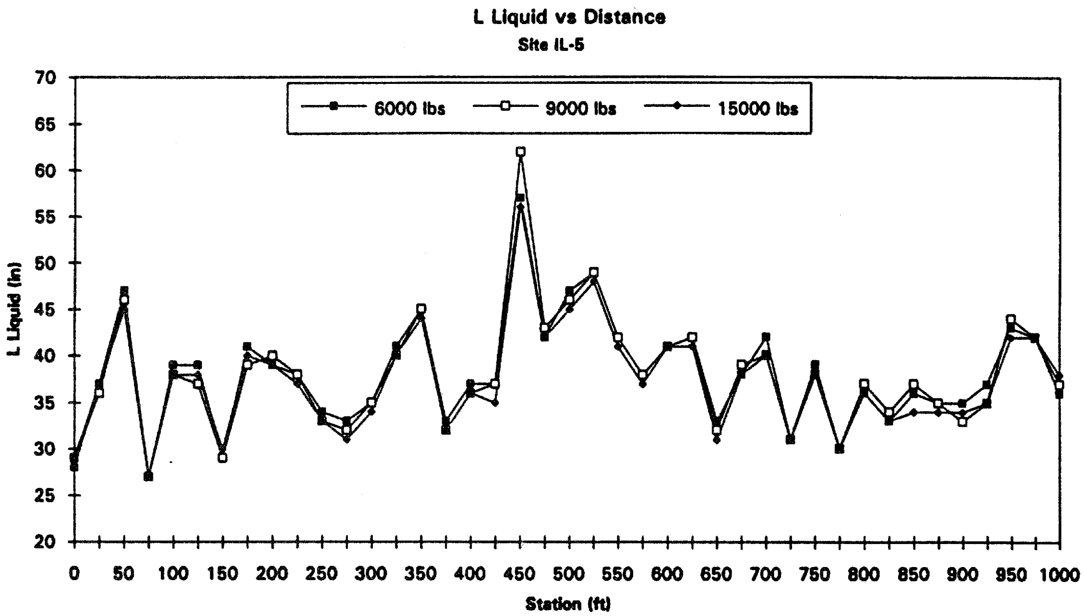
Figure 37. Temperature profile for site IL-5.





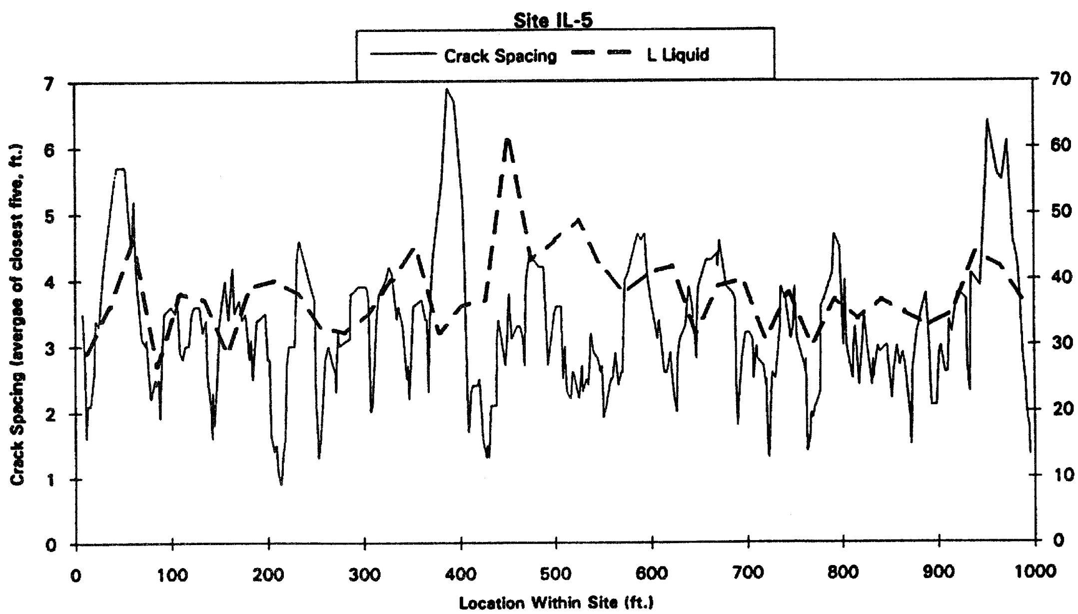
(0.0254 mm = 1 mil; 4.45 kN = 1,000 lb)

Figure 38. Center versus edge deflection for each of the seven sensors at site IL-5.
[40.03 kN (9,000 lb) load]



(25.4 mm = 1 in, 0.305 m = 1 ft)

Figure 39. Backcalculated values of radius of relative stiffness, ℓ (using program ILLI-BACK) for site IL-5.



(0.305 m = 1 ft)

Figure 40. Comparison of radius of relative stiffness values with average crack spacing (nearest five cracks) at site IL-5.

CHAPTER 4 - IOWA TEST SECTIONS

Section IA-1 - Interstate 29

IA-1 or the first section in Iowa was constructed in 1971 and was opened to traffic in 1972. IA-1 is a section of I-29 that runs north to south from Kansas City, Missouri, to Sioux City, Iowa, as shown in figure 41. The length of the highway that was considered runs from milepost 0.0 to 19.07. A 305-m (1,000-ft) north bound section was selected for detailed inspection from milepost 18.00 to 19.00, illustrated in figure 41. I-29 is in the western region of Iowa and runs along the border of Iowa and Nebraska.

The climate in this region is typically continental with relatively warm summers and cold, dry winters. Most of the precipitation, 75 percent, occurs during a 6-month period from April to September. The highest monthly average temperature is 25.4°C (77.7°F) during July, and the lowest monthly average is -6.7°C (20.0°F) during January. I-29 lies within the wet-freeze environmental zone.

Traffic

Since its opening, the pavement has experienced an estimated 3.7 million ESALs in its north bound lanes. The directional split for 1989 was 50.1 percent. The AADT for 1989 was 7,500 with a truck percentage of 29.3. From 1988 to 1989, there was a 6-percent increase in AADT. I-29 was designed for a traffic level of 9,690 vehicles per day.

1991 ESAL in Design Lane = 660,000

Structure

I-29 is a four-lane divided highway located in a rural section of Iowa. The pavement lane width is 3.7 m (12 ft) with an outside shoulder width of 3.0 m (10 ft) and an inside shoulder width of 0.9 m (3 ft). <u>The shoulders are constructed of asphaltic concrete.</u>

I-29 was designed as a three-layer system comprised of a 203-mm (8-in) CRCP and a <u>102-mm (4-in) CTB</u> over a subgrade of sand. The CRC pavement was designed with a steel percentage of 0.65. The reinforcing steel is #6 bars spaced at 216 mm (8.5 in) with no transverse steel. The bars were tube fed during the construction of the pavement and placed as a single layer. The bars are not epoxy coated.

The concrete pavement thickness was determined to be 210 mm (8.25 in) through cores taken as part of this study. The concrete is considered well graded, and its consolidation is average. The aggregate type used was a crushed stone with a maximum size of 38 mm (1½ in).

The CTB was well graded and was about 99 mm (3.9 in) thick. The subgrade is a sandy soil with the AASHTO classification of A-2-6. It had a liquid limit of 15 and a plastic limit of 0.

Design/Construction

The thickness design for I-29 was done in house by the Iowa Department of Transportation using the Portland Cement Association (PCA) method. The design life chosen was 20 years with a design traffic level of 9,690 vehicles per day. A composite modulus of K equal to 40 MPa/m (150 pci) was assumed for subgrade reaction. Drainage was not initially incorporated in the design but was retrofited after or during construction. The terminal joint design used was a lug anchor system. The bridge approach pavement type was concrete.

Performance

The performance of I-29 has been evaluated about every 2 to 3 years since its opening. The performance of I-29 has been recorded according to the present serviceability index (PSI) scale, 0.0 - 5.0, using a South Dakota profilometer. The first PSI value recorded in 1974 was 4.40, and since then has steadily decreased to a value of 3.92 in 1990. The PCI values of I-29 taken over the past 20 years have ranged from 4.40 in 1974 to 3.92 in 1990.

Historical Data (M&R)

Very little maintenance and rehabilitation work has been recorded for I-29. The only maintenance that has been reported is a fog seal of the shoulders in 1983 and 1989.

Field Investigation Data

Visual Condition Survey

- Total number of cracks = 168
  - 0 were at high severity
  - 144 were at medium severity
  - 24 were at low severity
- Average crack spacing = 1.80 m (5.92 ft)
- Standard deviation for crack spacing = 1.12 m (3.66 ft)
- Coefficient of variation of the crack spacing = 62 percent
- Figure 42 shows average crack spacing distribution based on the closest five cracks
- Figure 43 shows actual crack spacing
- Extent of Y cracks = 12 percent
- Cluster cracking was apparent at stations 0+00, 1+80, 4+00, and 6+00
- Other distress types included:
  - 105-m (343-ft) longitudinal cracking
  - Popouts

- Pavement shoulders were in generally good condition but exhibited high severity transverse cracking at approximately every 4.6 m (15 ft).

Drainage Survey
- Ditches line both sides of the pavement.
- Ditches were lined with vegetation. There was no sign of ponded water or wet ground.

Windshield Survey
- 8.05 km (5 mi) of pavement surveyed was generally in good condition exhibiting about 51.8 m (170 ft) of longitudinal cracking condition.

Terminal Joint Survey
- Terminal joints used = Lug anchor (one surveyed)
- Joints exhibited the following distresses:
  - 2.2 m^2 (24 ft^2) of medium severity spalling
  - 0.6 m^2 (6 ft^2) of medium severity AC patch deterioration

Deflection Testing
- Figure 44 shows normalized/basin (per 4.45 kN (1,000 lb) load) deflections measured along the length of the section (for all seven sensors using the 40.03 kN (9,000 lb) load)
- Load transfer efficiency at cracks at mid-slab
  - 92 to 96 percent in the morning
  - 94 to 96 percent in the afternoon
- Load transfer efficiency at cracks along edge
  - 92 to 94 percent in the morning
  - 91 to 94 percent in the afternoon
- Figure 45 shows the distribution of slab temperatures with depth and time of day.
- Figure 46 shows the average deflections and ranges of deflections for each sensor for center and edge loading, for morning and afternoon testing (using the 40.03 kN (9,000 lb) load within the selected 30.5-m (100-ft) subsection).
- Figure 47 shows the backcalculated radius of relative stiffness, ℓ, values for basin testing for each of the three nominal load levels of 40.03, 53.38, and 71.17 kN (9,000, 12,000, and 16,000 lb).
- ℓ value based on 40.03 kN (9,000 lb) load basin testing
  - Average = 1 016 mm (40 in)
  - Standard Deviation = 203 mm (8 in)
- ℓ value at crack locations at mid-slab (morning/afternoon)
  - Average = 1 016/1 092 mm (40/43 in)
  - Standard Deviation = 178/178 mm (7/7 in)
- ℓ value at crack locations at edge (morning/afternoon)
  - Average = 838/838 mm (33/33 in)

- Standard Deviation = 102/51 mm (4/2 in)
- Estimated ℓ value using actual slab thickness and concrete modulus of elasticity and assumed modulus of subgrade reaction
  - Value = 737 mm (29 in)
- Figure 48 show the variation of ℓ in relation to crack spacing (average of the closest five cracks)
- Average deflection under 40.03 kN (9,000 lb) load (0.000001 mm (0.001 in))
  - Basin test - 0.104 (4.1)
  - Mid-slab crack location (morning/afternoon) - 0.097/0.109 (3.8/4.3)
  - Edge crack location (morning/afternoon) - 0.175/0.135 (6.9/5.3)

Crack Width Measurements
- Number of cracks monitored within the 30.5-m (100-ft) subsection = 15
- Morning testing
  - Average temperature 102 mm (4 in) below surface = 19.4°C (67°F)
  - Average crack width = 0.46 mm (0.018 in)
- Afternoon testing
  - Average temperature 102 mm (4 in) below surface = 27.2°C (81°F)
  - Average crack width = 0.37 mm (0.015 in)
- Slab length change, mm/mm/°C (in/in/°F) = 5.8 millionth (3.2 millionth)

Reinforcing Steel Depths and Spacing
- Steel depth measurement offsets = 559, 1 219, 1 829 mm (22, 48, 72 in)
- Range of depth of cover = 81 to 132 mm (3.2 to 5.2 in)
- Average depth of cover = 107 mm (4.2 in)
- Standard deviation for depth of cover = 7.6 mm (0.3 in)
- Steel spacing was about 203 mm (8 in)

Corrosion Potential Testing (copper-copper sulfate half cell potentiometer)
- Range of measurements = -0.19 to -0.35 volts
- Average measurement = -0.29 volts
- Standard deviation for the measurement = -0.03 volts
- Potential for corrosion was marginal

Concrete Core Examination for Corrosion
- One core exhibited spot corrosion
- One core exhibited spread corrosion
- Two cores exhibited no corrosion

Profile Testing

- Average IRI over the 305-m (1,000-ft) section = 1 138 mm/km (72 in/mi)

Laboratory Testing

The following data were obtained from the testing of the concrete, base/subbase, and subgrade materials:

Concrete

Modulus of Elasticity	-	30 340 MPa (4,400,000 lbf/in^2)
Average Split Tensile Strength	-	3.31 MPa (480 lbf/in^2)
Split Tensile Strength Range	-	3.14 to 3.45 MPa (455 to 500 lbf/in^2)
Coefficient of Thermal Expansion	-	7.74 mm/mm/°C (4.30 in/in/°F)

Base

AASHTO Classification	-	CTB

Subbase

AASHTO Classification	-	N/A

Subgrade

Liquid Limit	-	15 percent
Plastic Limit	-	0 percent
Percent Passing #200 Sieve	-	7 percent
AASHTO Classification	-	A-2-6

Chloride content testing was conducted on pulverized concrete retrieved at various depths and locations. Total chloride content and water soluble chloride content values are listed below for IA-1.

Location	Depth, mm (in)	Total Chloride Content (% wt.)	Water Soluble Chlorides (% wt.)
No steel below sample	0-51 (0-2)	0.149	0.138
	51-102 (2-4)	0.077	0.068
Steel directly below sample	0-51 (0-2)	0.144	0.133
	51-102 (2-4)	0.080	0.066

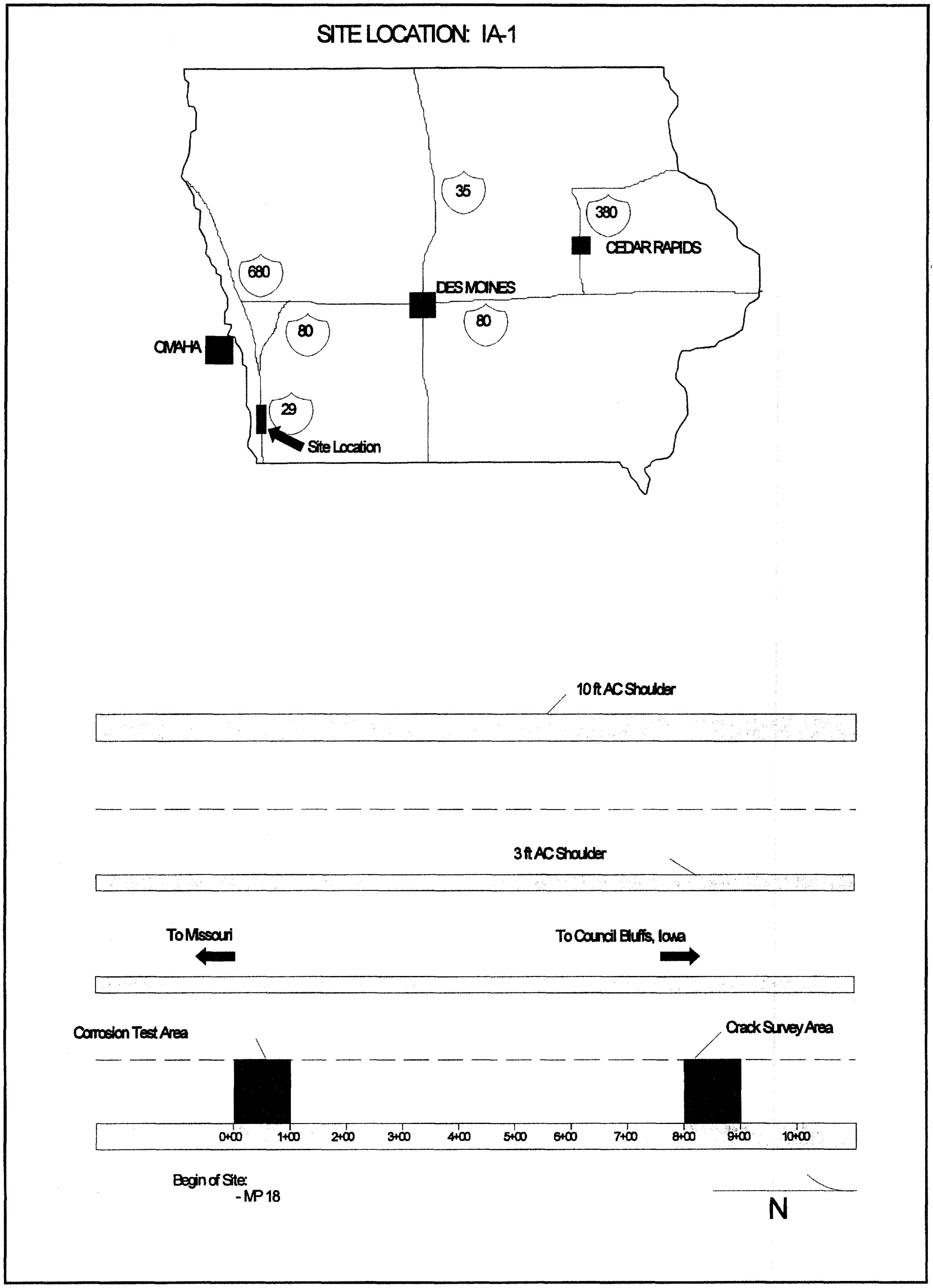


Figure 41. Site details for IA-1.

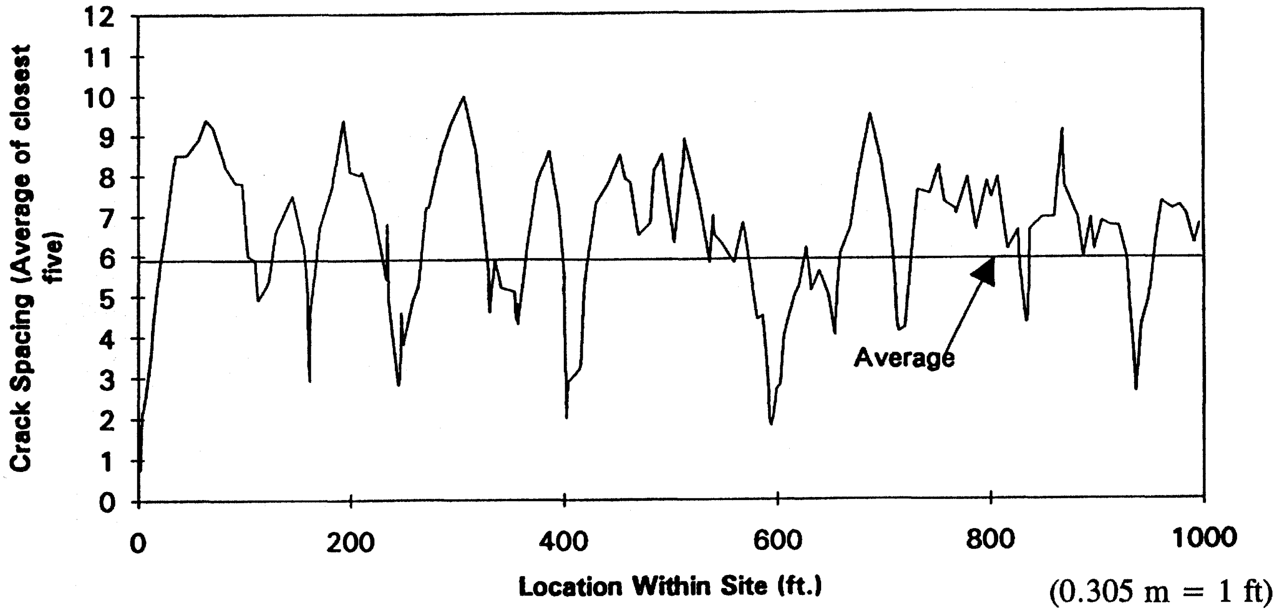


Figure 42. Crack spacing pattern at site IA-1.

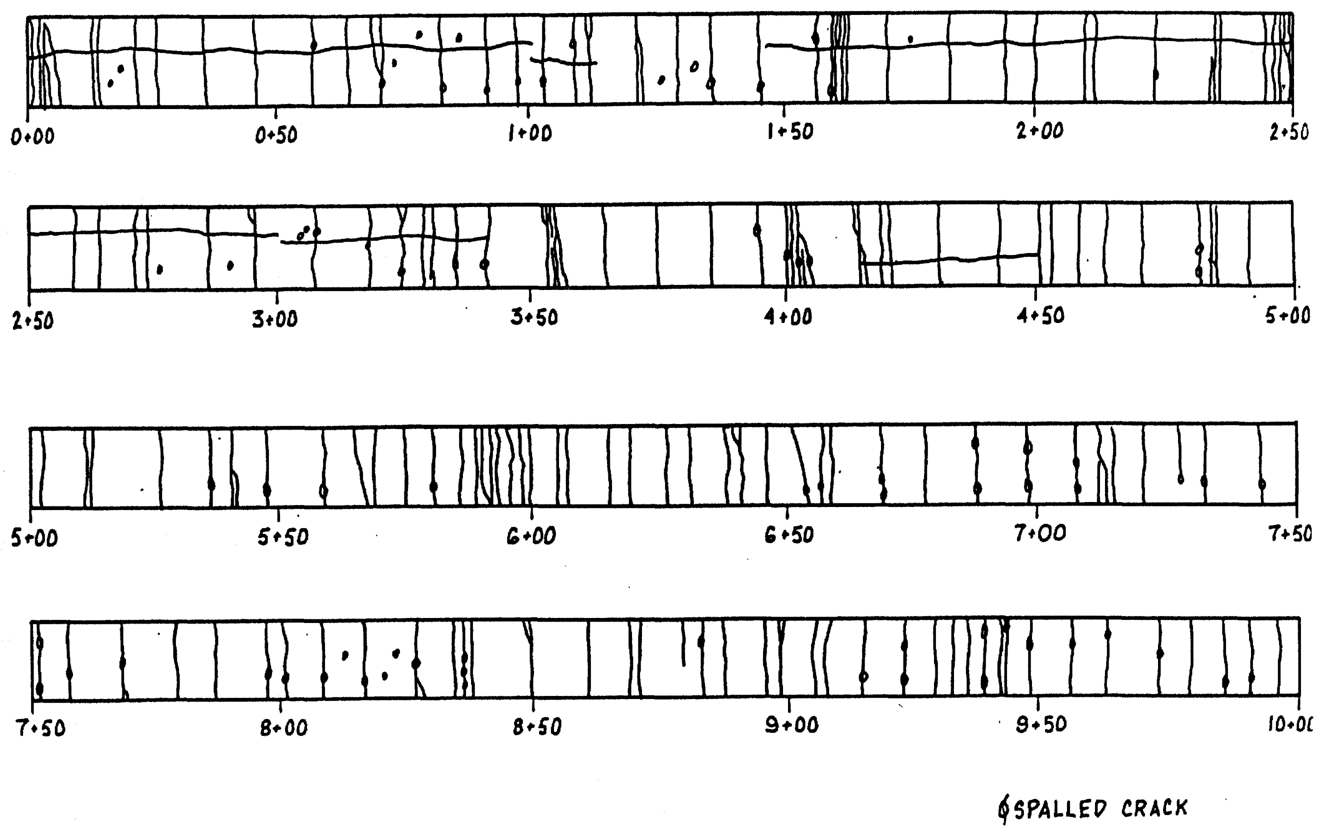
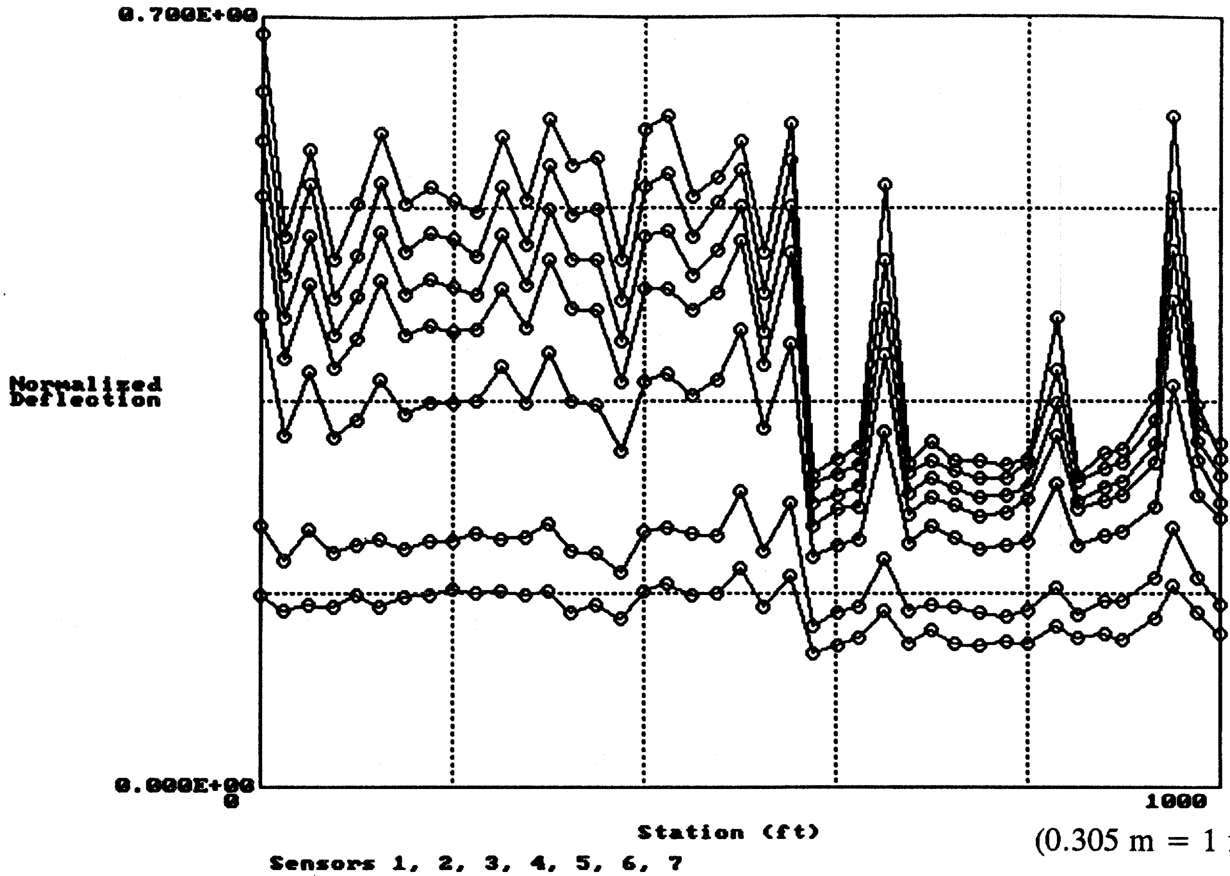
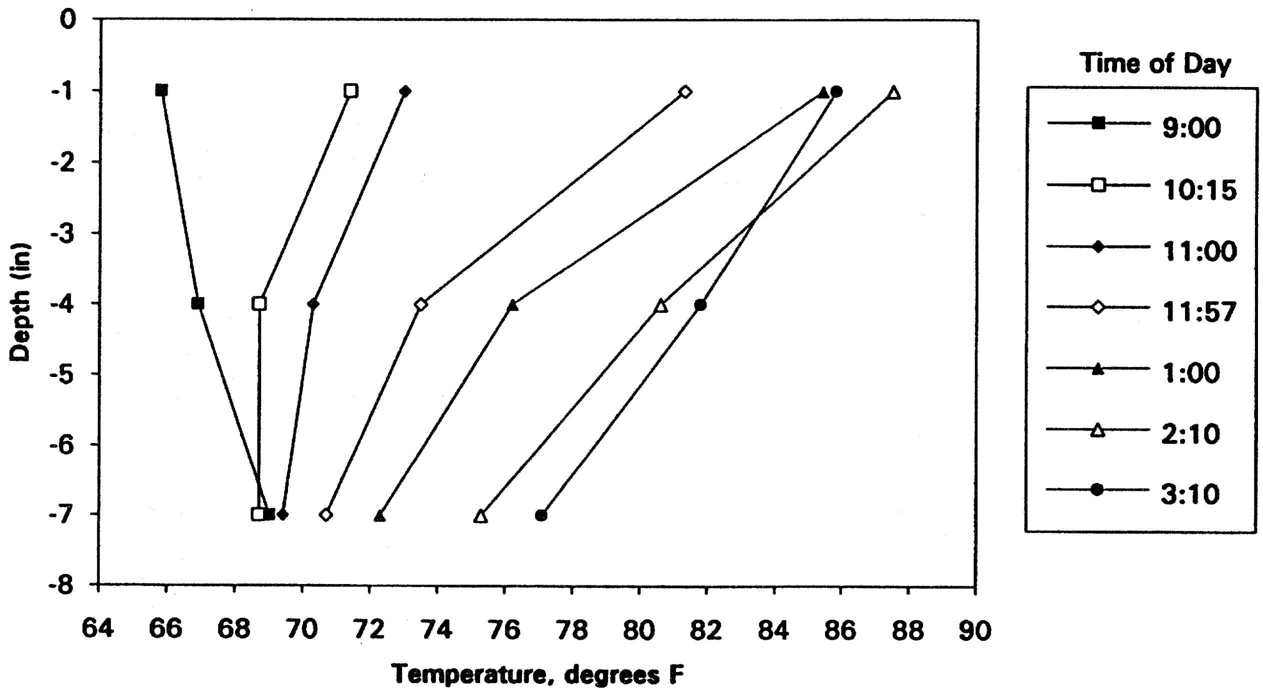


Figure 43. Crack pattern at site IA-1.



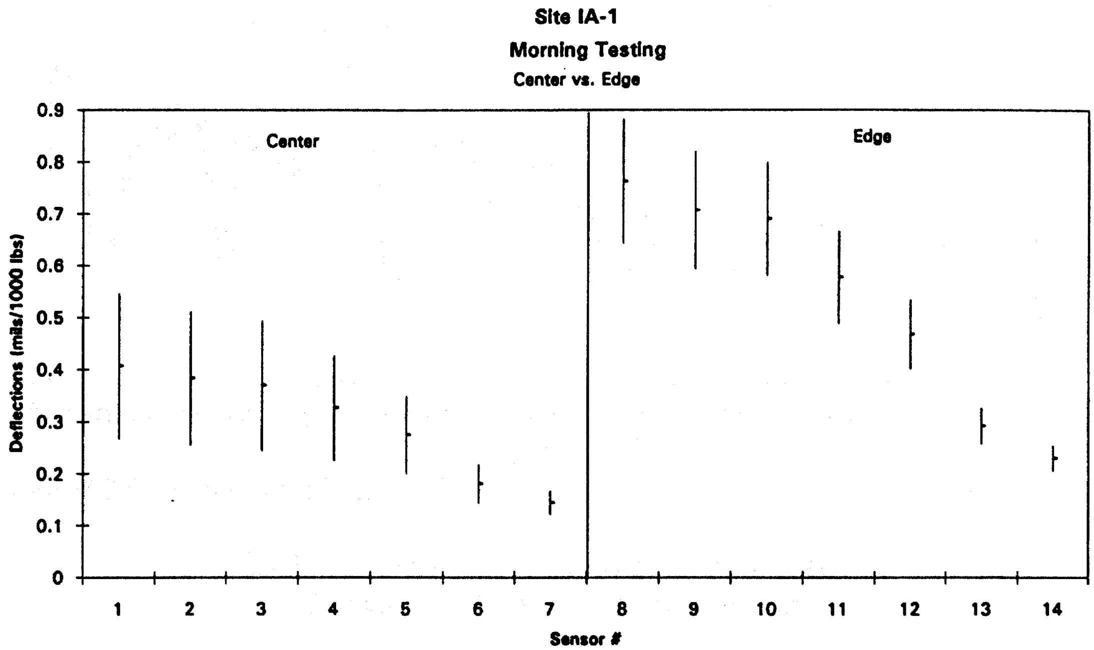
(0.305 m = 1 ft)

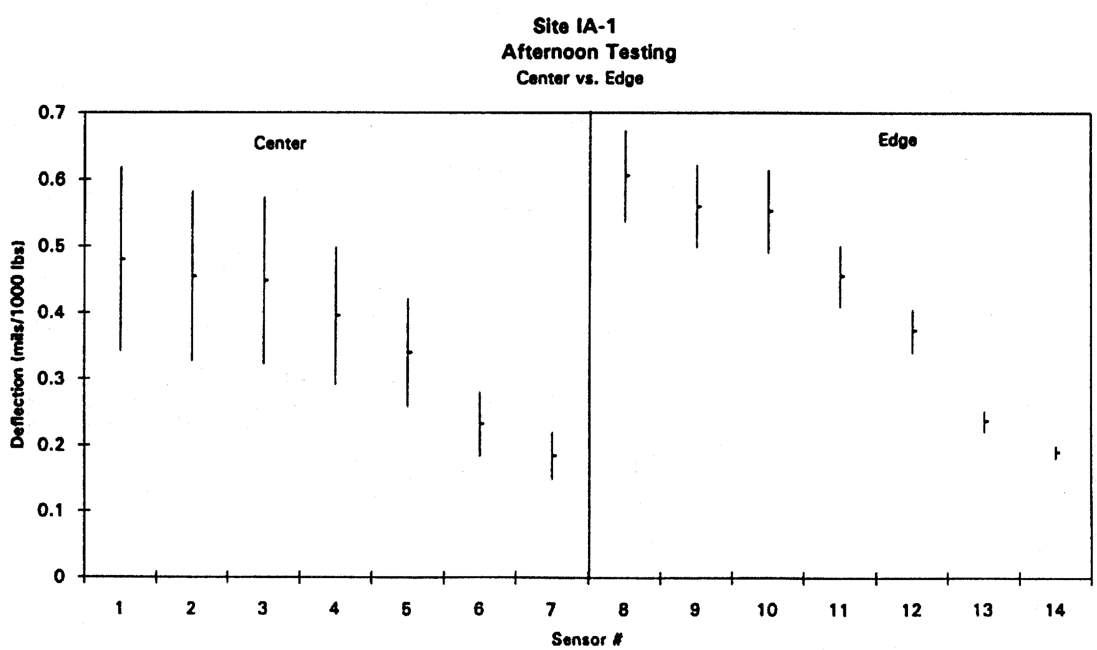
Figure 44. Deflections in inches along the section length at site IA-1.
[normalized per 4.45-kN [1,000-lb] load]



(25.4 mm = 1 in; 0.6°C = 1°F)

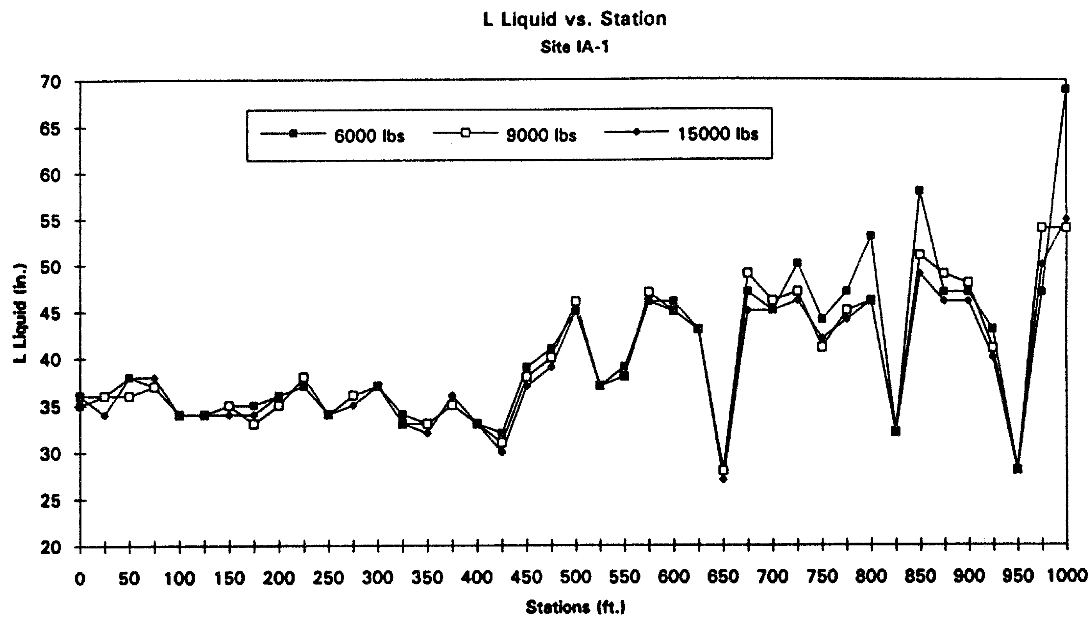
Figure 45. Temperature profile for site IA-1.





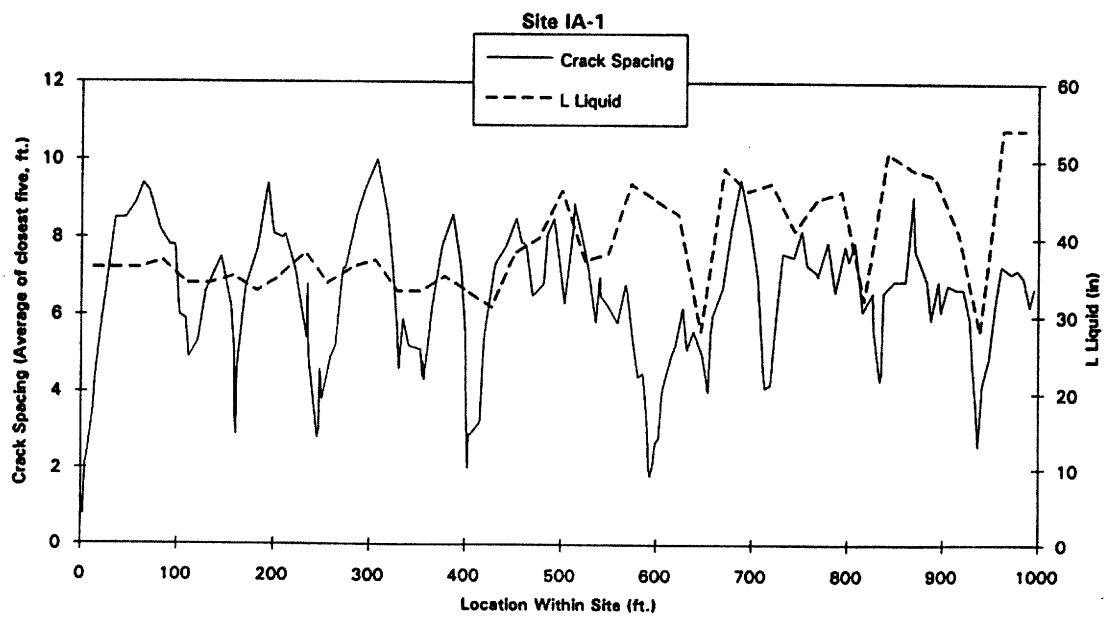
(0.0254 mm = 1 mil; 4.45 kN = 1,000 lb)

Figure 46. Center versus edge deflection for each of the seven sensors at site IA-1.
[40.03 kN (9,000 lb) load]



(25.4 mm = 1 in, 0.305 m = 1 ft)

Figure 47. Backcalculated values of radius of relative stiffness, ℓ (using program ILLI-BACK) for site IA-1.



(0.305 m = 1 ft)

Figure 48. Comparison of radius of relative stiffness values with average crack spacing (nearest five cracks) at site IA-1.

Section IA-2 - Interstate 80

Introduction

IA-2 or the second section in Iowa was constructed and opened to traffic in 1969. IA-2 is a section of I-80 that runs east to west from Council Bluffs to Des Moines onto Davenport, Iowa, as shown in figure 49. The length of the highway that was considered for testing runs from milepost 11.0 to 19.0. A 305-m (1,000-ft) east bound section was selected for detailed inspection between mileposts 15.00 and 16.00, as shown in figure 49. The section of I-80 that was chosen is in the western region of Iowa near the border with Nebraska.

The climate in this region is typically continental with relatively warm summers and cold, dry winters. Most of the precipitation, 75 percent, occurs during a 6-month period from April to September. The highest monthly average temperature is 25.4°C (77.7°F) during July, and the lowest monthly average temperature is -6.7°C (20.0°F) during January. I-80 lies within the wet-freeze environmental zone.

Traffic

Since its opening the pavement has experienced an estimated 8.85 million ESALs in its east bound lanes. The directional split for 1989 was 47.9 percent. The AADT for 1989 was 12,700 with a truck percentage of 34.8. From 1988 to 1989, there was a 5-percent increase in AADT. I-80 was designed for a traffic level of 14,500 vehicles per day.

1989 ESAL for Design Lane	=	1,300,000
1989 Two-Way AADT	=	12,700 vehicles

Structure

I-80 is a four-lane divided highway located in a rural section of Iowa. The pavement lane width is 3.7 m (12 ft) with an outside shoulder width of 3.0 m (10 ft) and an inside shoulder width of 0.9 m (3 ft). <u>The shoulders are constructed of asphaltic concrete.</u>

I-80 was designed as a three-layer system comprised of a 203-mm (8-in) thick CRCP and a <u>102-mm (4-in) thick ATB</u> over a subgrade of sandy, silty clay. The CRC pavement was designed with a steel percentage of 0.65. The reinforcing steel consists of #6 bars spaced at 216 mm (8.5 in) with no transverse steel. The bars were tube fed during the construction and placed as a single layer. The bars are not epoxy coated.

The concrete pavement thickness was determined to be 201 mm (7.9 in) through cores taken as part of this study. The concrete is considered to be well graded and is well consolidated based on core examination. The aggregate type used was a crushed stone with a maximum size of 44 mm (1¾ in).

The ATB was considered to be well graded and was measured to be about 99 mm (3.9 in) thick. The subgrade is a sandy, silty clay with the AASHTO classification of A-6. The subgrade has a liquid limit of 37 and a plastic limit of 16.

Design/Construction

The thickness design for I-80 was done in 1965 by the Iowa Department of Transportation using the PCA method. The design life chosen was 20 years with a design traffic level of 14,500 vehicles per day. A composite modulus of K equal to 27.1 MPa/m (100 pci) was assumed for subgrade reaction. Drainage was not initially designed but was retrofited after or during construction. The terminal joint design chosen was a lug anchor system. The bridge approach pavement type is concrete.

Performance

The performance of I-80 has been evaluated about every 2 to 3 years since its opening. The performance of I-80 has been recorded according to the PSI scale, 0.0 - 5.0, using a South Dakota profilometer. The first PSI value recorded in 1970 was 4.30 and since then has steadily decreased to a value of 3.40 in 1990. The PSI values for I-80 have been taken over the past 20 years and range from a value of 4.30 in 1970 to 3.40 in 1990.

Historical Data (M&R)

Very little maintenance and rehabilitation work has been recorded for I-80. The only maintenance that has been reported is a bituminous seal coat in 1981. Other maintenance known to be completed is a fog seal of the shoulders in selected areas and spot patching, but no dates were reported.

Field Investigation Data

Visual Condition Survey

- Total number of cracks = 455
  - 0 were at high severity
  - 420 were at medium severity
  - 35 were at low severity
- Average crack spacing = 0.9 m (2.98 ft)
- Standard deviation for crack spacing = 0.68 m (2.24 ft)
- Coefficient of variation of the crack spacing = 75 percent
- Figure 50 shows average crack spacing distribution based on the closest five cracks
- Figure 51 shows actual crack spacing
- Extent of Y cracks = 15 percent
- Cluster cracking was apparent at stations 1+00, 1+50, 4+00, and 8+00
- Other distress types included:

- One AC patch (low severity)
- Longitudinal cracking (1.2 m (4 ft) low, 14.9 m (49 ft) moderate, and 4.3 m (14 ft) high severity)
- Polished aggregates (wide spread)
- Popouts (few)

- Pavement shoulders were in fair condition.

Drainage Survey

- Ditches line both sides of the pavement.
- Ditches were lined with vegetation. There was no sign of ponded water or wet ground.

Windshield Survey

- 8.05 km (5 mi) of pavement surveyed was generally in good condition exhibiting about 21.3 m (70 ft) of longitudinal cracking condition, 7 PCC patches, and 11 AC patches.

Terminal Joint Survey

- Terminal joints used = Lug anchor (one surveyed)
- Joints exhibited the following distresses:
    - 1.11 m^2 (12 ft^2) of medium severity spalling
    - 0.74 m^2 (8 ft^2) of medium severity AC patch deterioration

Deflection Testing

- Figure 53 shows normalized/basin (per 4.45 kN (1,000 lb) load) deflections measured along the length of the section (for all seven sensors using the 40.03 kN (9,000 lb) load)
- Load transfer efficiency at cracks at mid-slab
    - 92 to 96 percent in the morning
    - 92 to 98 percent in the afternoon
- Load transfer efficiency at cracks along edge
    - 93 to 97 percent in the morning
    - 91 to 97 percent in the afternoon
- Figure 54 shows the distribution of slab temperatures with depth and time of day.
- Figure 55 shows the average deflections and ranges of deflections for each sensor for center and edge loading, for morning and afternoon testing (using the 40.03 kN (9,000 lb) load within the selected 30.5-m (100-ft) subsection).
- Figure 56 shows the backcalculated radius of relative stiffness, ℓ, values for basin testing for each of the three nominal load levels of 40.03, 53.38, and 71.17 kN (9,000, 12,000, and 16,000 lb).
- ℓ value based on 40.03 kN (9,000 lb) load basin testing
    - Average = 1 041 mm (41 in)
    - Standard Deviation = 127 mm (5 in)

- ℓ value at crack locations at mid-slab (morning/afternoon)
  - Average = 914/1 016 mm (36/40 in)
  - Standard Deviation = 102/51 mm (4/2 in)
- ℓ value at crack locations at edge (morning/afternoon)
  - Average = 1 067/1 016 mm (42/40 in)
  - Standard Deviation = 51/25 mm (2/1 in)
- Estimated ℓ value using actual slab thickness and concrete modulus of elasticity and assumed modulus of subgrade reaction
  - Value = 737 mm (29 in)
- Figure 56 shows the variation of ℓ in relation to crack spacing (average of the closest five cracks)
- Average deflection under 40.03 kN (9,000 lb) load (0.00001 mm (0.001 in.))
  - Basin test - 0.127 (5.0)
  - Mid-slab crack location (morning/afternoon) - 0.135/0.119 (5.3/4.7)
  - Edge crack location (morning/afternoon) - 0.340/0.244 (13.4/9.6)

Crack Width Measurements

- Number of cracks monitored within the 30.5-m (100-ft) subsection = 19
- Morning testing
  - Average temperature 25 mm (1 in) below surface = 13.9°C (57°F)
  - Average crack width = 0.20 mm (0.008 in)
- Afternoon testing
  - Average temperature 25 mm (1 in) below surface = 22.8°C (73°F)
  - Average crack width = 0.14 mm (0.006 in)
- Slab length change, mm/mm/°C (in/in/°F) = 4.1 millionth (2.3 millionth)

Reinforcing Steel Depths and Spacing

- Steel depth measurement offsets = 559, 914, 1 219 mm (22, 36, 48 in)
- Range of depth of cover = 56 to 99 mm (2.2 to 3.9 in)
- Average depth of cover = 79 mm (3.1 in)
- Standard deviation for depth of cover = 10 mm (0.4 in)
- Steel spacing was not measured

Corrosion Potential Testing (copper-copper sulfate half cell potentiometer)

- Range of measurements = -0.23 to -0.40 volts
- Average measurement = -0.31 volts
- Standard deviation for the measurement = -0.04 volts
- Potential for corrosion was marginal

Concrete Core Examination for Corrosion

- 2 cores exhibited spot corrosion
- 2 cores exhibited spread corrosion

Profile Testing

- Average IRI over the 305-m (1,000-ft) section = 1 280 mm/km (81 in/mi)

Laboratory Testing

The following data were obtained from the testing of the concrete, base/subbase, and subgrade materials:

Concrete

Modulus of Elasticity	-	27 580 MPa (4,000,000 lbf/in^2)
Average Split Tensile Strength	-	3.52 MPa (510 lbf/in^2)
Split Tensile Strength Range	-	3.10 to 3.90 MPa (450 to 565 lbf/in^2)
Coefficient of Thermal Expansion	-	8.55 mm/mm/°C (4.75 in/in/°F)

Base

AASHTO Classification	-	ATB

Subbase

AASHTO Classification	-	N/A

Subgrade

Liquid Limit	-	37 percent
Plastic Limit	-	16 percent
Percent Passing #200 Sieve	-	91 percent
AASHTO Classification	-	A-6

Chloride content testing was conducted on pulverized concrete retrieved at various depths and locations. Total chloride content and water soluble chloride content values are listed below for IA-2.

Location	Depth, mm (in)	Total Chloride Content (% wt.)	Water Soluble Chlorides (% wt.)
No steel below sample	0-51 (0-2)	0.141	0.172
	51-102 (2-4)	0.122	0.094
Steel directly below sample	0-51 (0-2)	0.195	0.174
	51-102 (2-4)	0.099	0.089

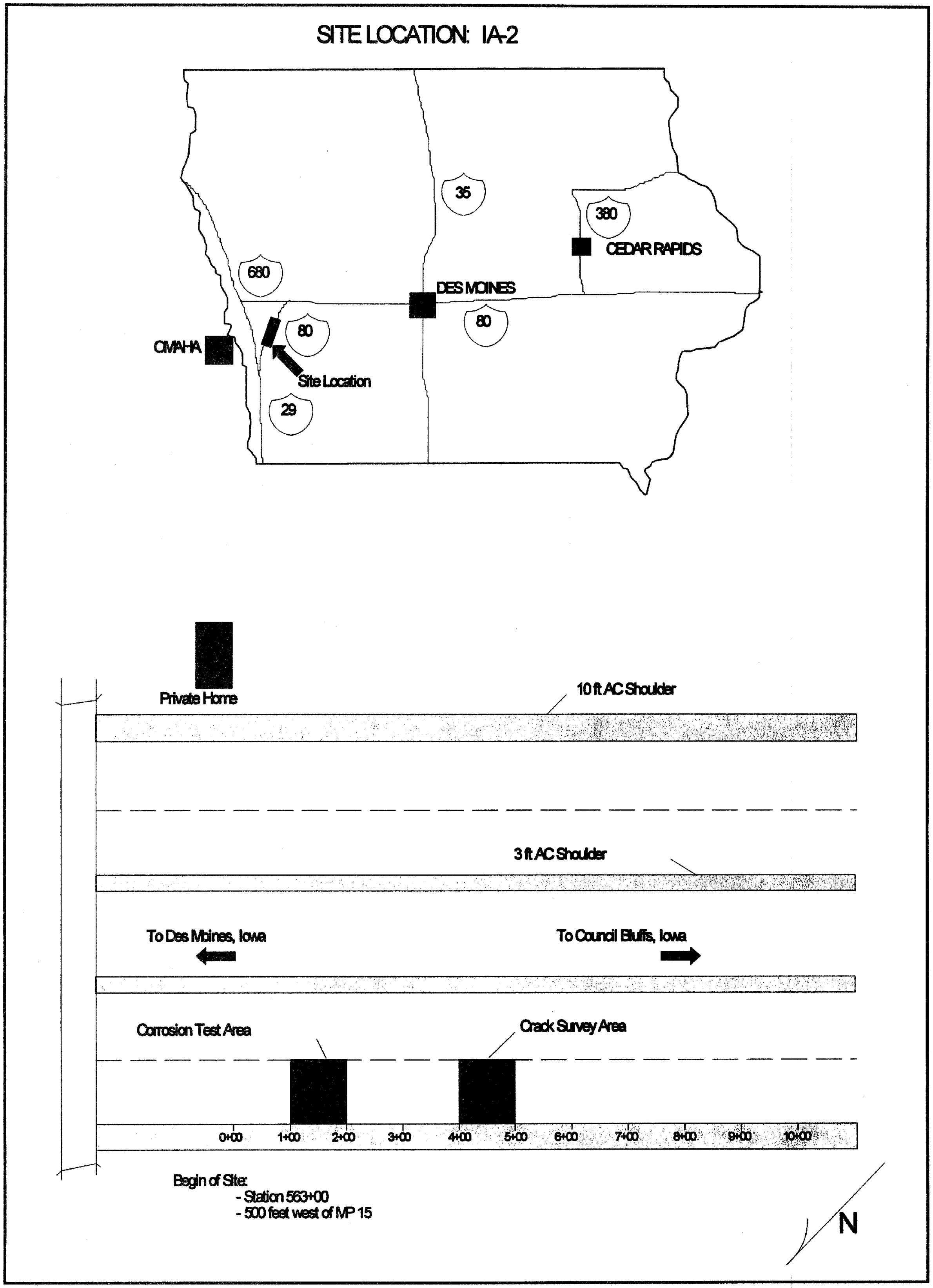


Figure 49. Site details for IA-2.

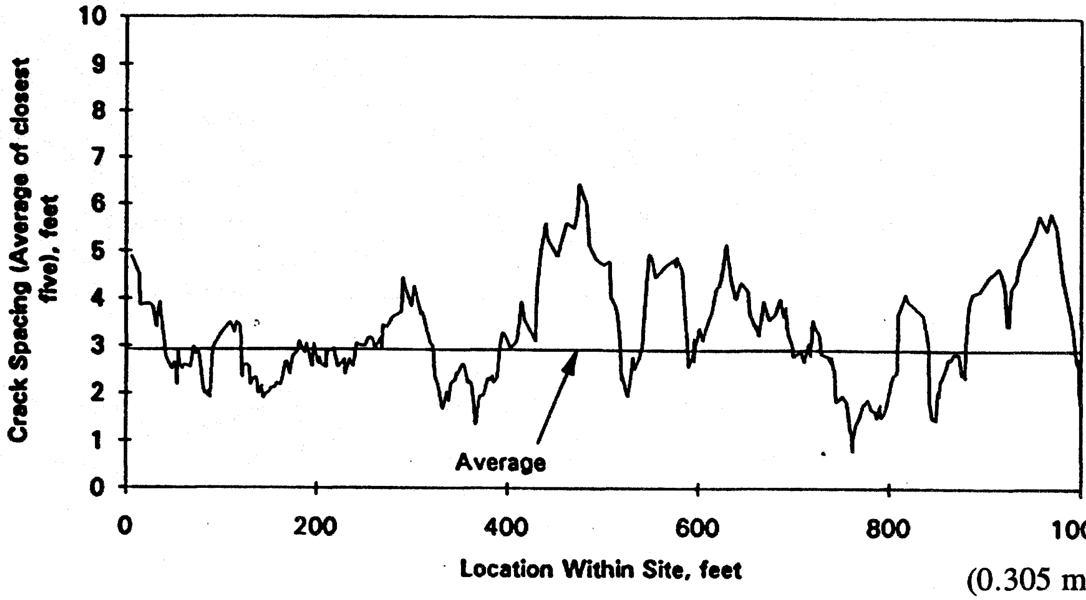


Figure 50. Crack spacing pattern at site IA-2.

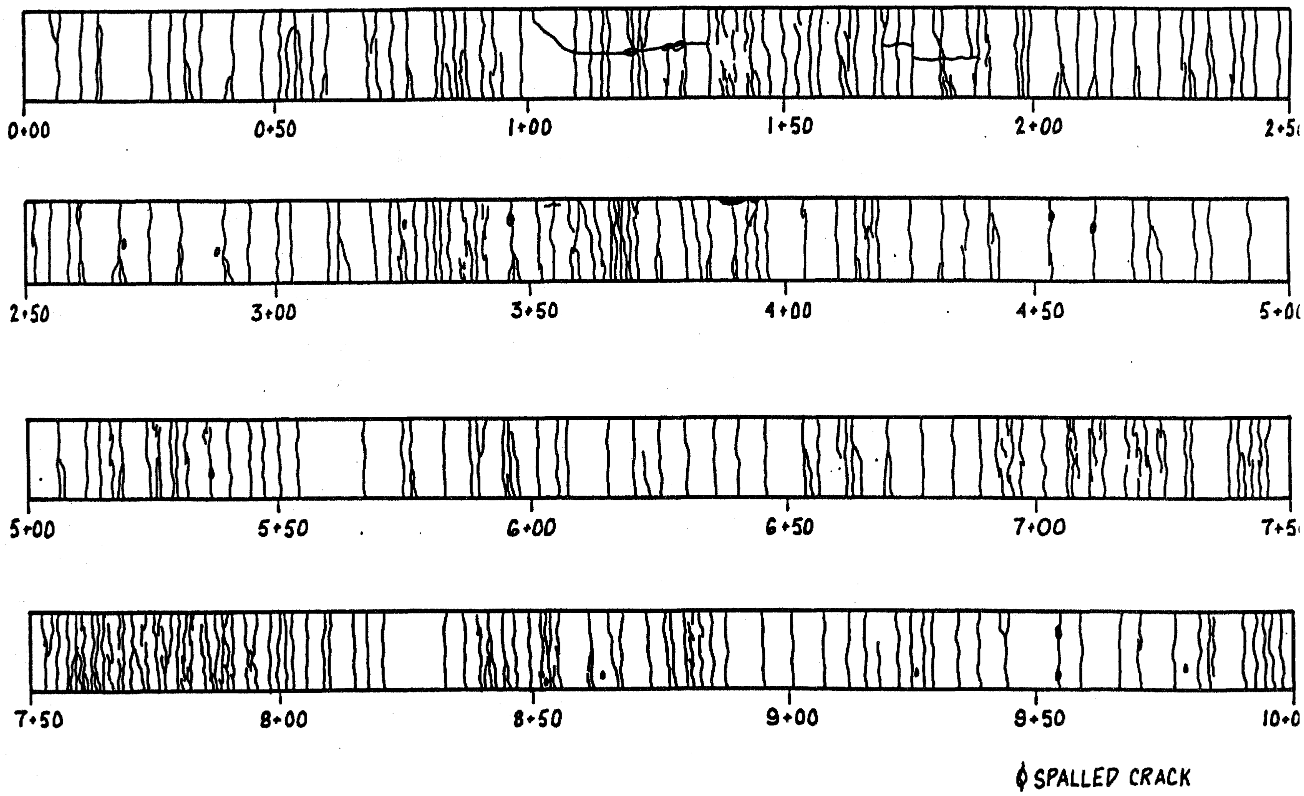
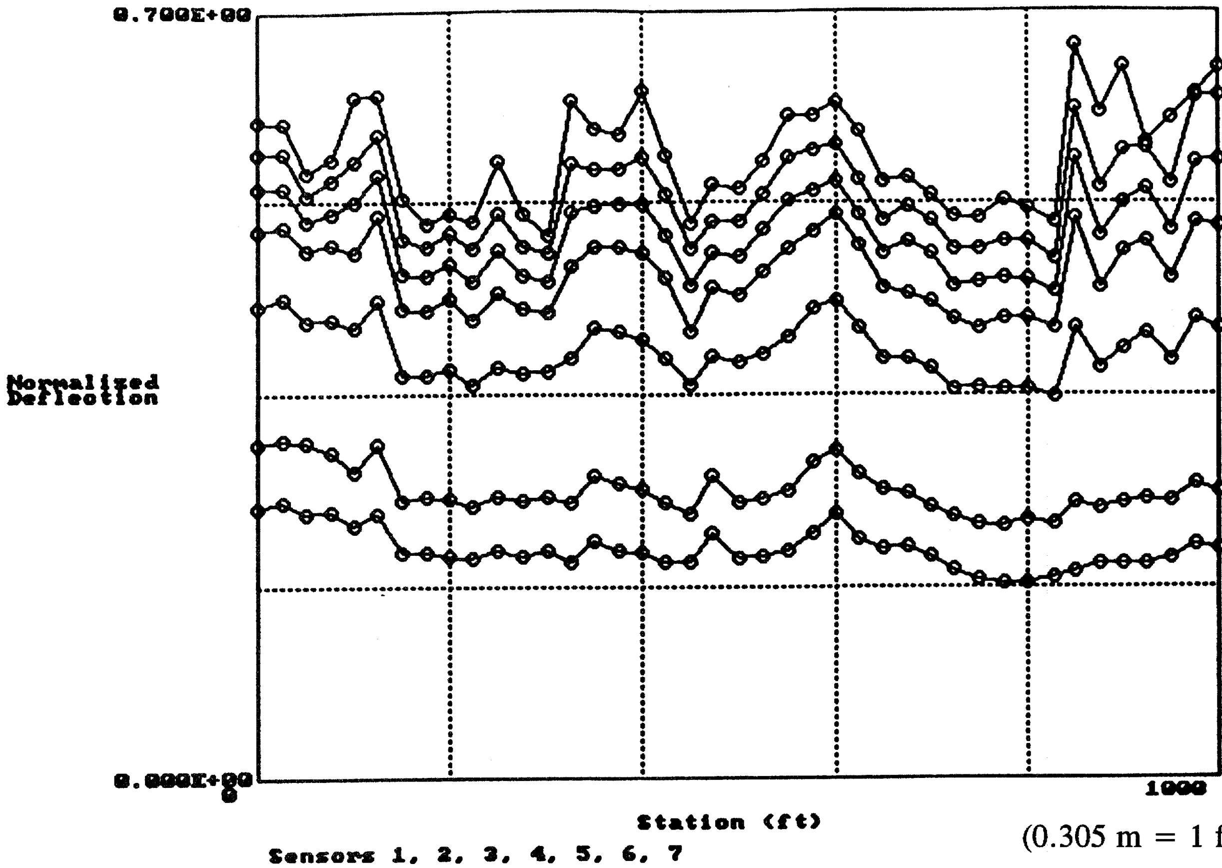
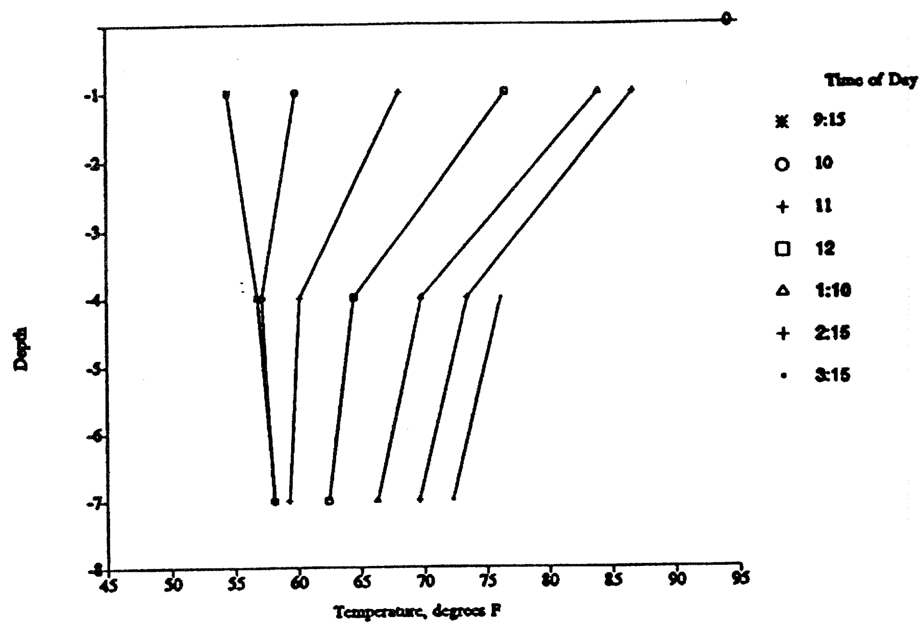


Figure 51. Crack pattern at site IA-2.



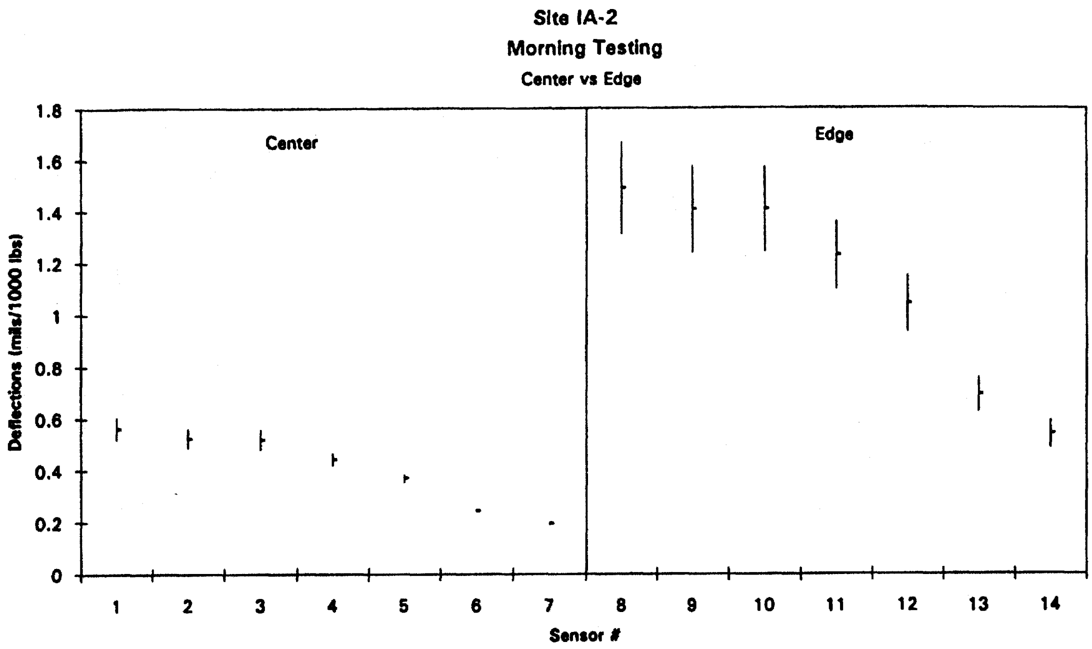
(0.305 m = 1 ft)

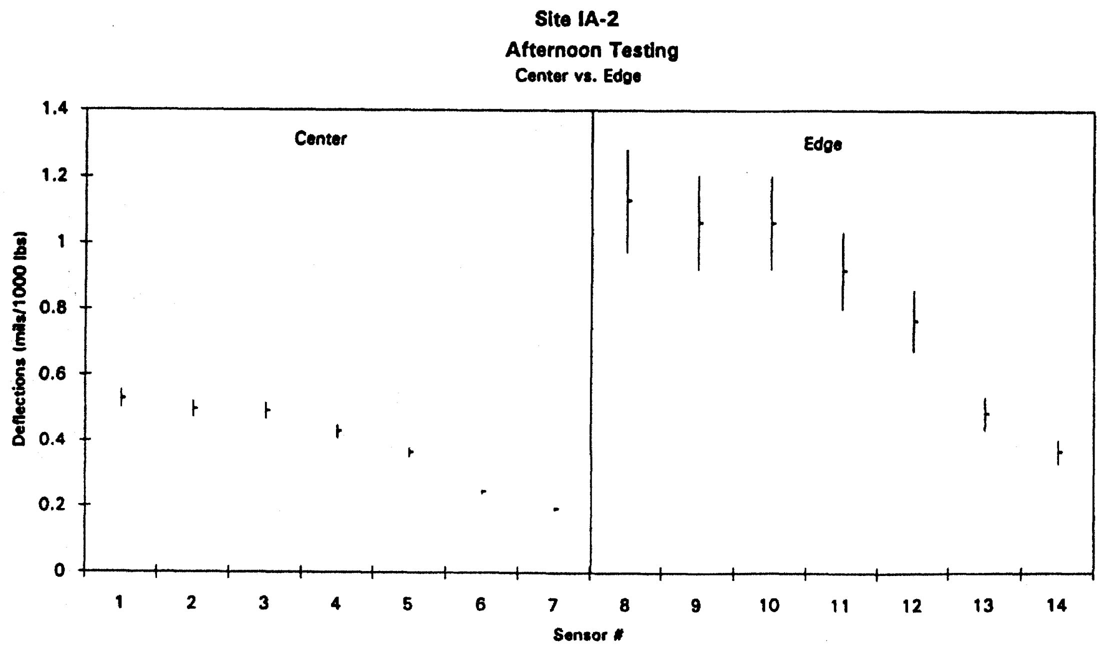
Figure 52. Deflections in inches along the section length at site IA-2.
[normalized per 4.45-kN [1,000-lb] load]



(25.4 mm = 1 in; 0.6°C = 1°F)

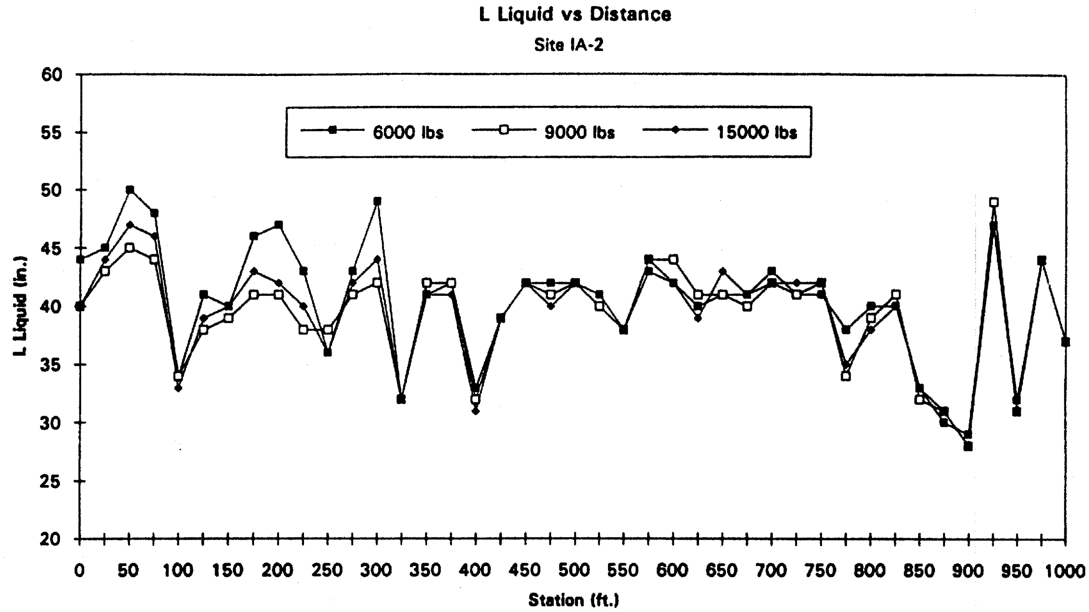
Figure 53. Temperature profile for site IA-2.





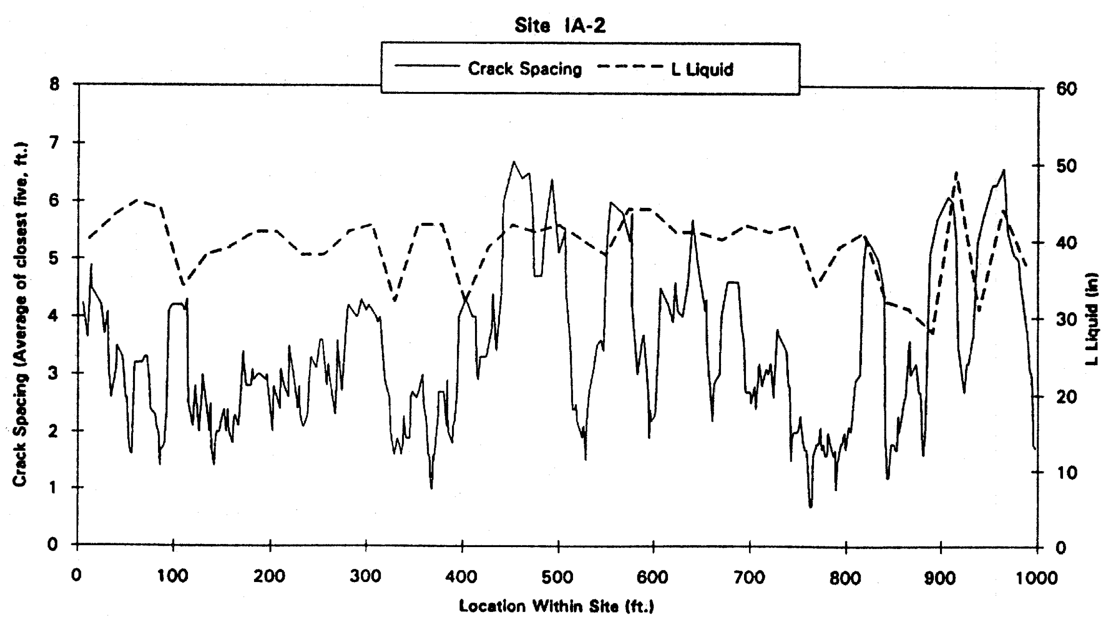
(0.0254 mm = 1 mil; 4.45 kN = 1,000 lb)

Figure 54. Center versus edge deflection for each of the seven sensors at site IA-2.
[40.03 kN (9,000 lb) load]



(25.4 mm = 1 in, 0.305 m = 1 ft)

Figure 55. Backcalculated values of radius of relative stiffness, ℓ (using program ILLI-BACK) for site IA-2.



(0.305 m = 1 ft)

Figure 56. Comparison of radius of relative stiffness values with average crack spacing (nearest five cracks) at site IA-2.

Section IA-3 - Interstate 380

Introduction

Section IA-3 was constructed and opened to traffic in 1976. IA-3 is a section of I-380 that runs northeast to southwest from Iowa City to Cedar Rapids, Iowa, as shown in figure 57. The length of the highway that was considered runs from milepost 14.0 to 17.0. A 305-m (1,000-ft) north bound section was selected for detailed inspection between mileposts 15.00 and 16.00 as shown in figure 57. The section of I-380 that was chosen is in the central eastern region of Iowa.

The climate in this region is a continental humid climate. Most of the precipitation, 72 percent, occurs during a 6-month period from April to September. The annual temperature range is large. The highest monthly average temperature is 22.6°C (72.6°F) during July, and the lowest monthly average is -10.1°C (13.9°F) during January. I-380 lies within the wet-freeze environmental zone.

Traffic

Since its opening, the pavement has experienced an estimated 5.3 million ESALs in its north bound lanes. The directional split for 1989 was 50.1 percent. The AADT for 1989 was 27,700 with a truck percentage of 14.0. From 1988 to 1989, there was a 6-percent increase in AADT. I-380 was designed for a traffic level of 15,000 vehicles per day.

1989 ESAL in Design Lane	=	1,020,000
1987 Two-Way AADT	=	27,700 vehicles

Structure

I-380 is a four-lane divided highway located in a rural section of Iowa. The pavement lane width is 3.7 m (12 ft) with an outside shoulder width of 3.0 m (10 ft) and an inside shoulder width of 0.9 m (3 ft). <u>The shoulders are constructed of PCC.</u>

I-380 was designed as a three-layer system comprised of a 203-mm (8-in) CRCP and a <u>102-mm (4-in) ATB</u> over a subgrade of a clayey soil. The CRC pavement was designed with a steel percentage of 0.65. The reinforcing steel is #6 bars spaced at 216 mm (8.5 in) with no transverse steel. The bars were tube fed during the construction and placed as a single layer. The bars are not epoxy coated. The portland cement shoulders were tied to the CRC mainline.

Average concrete pavement thickness along the test section was determined to be 206 mm (8.1 in) based on the cores taken as part of this study. The concrete is considered well graded and is well consolidated. The aggregate type used was a crushed stone with a maximum size of 38 mm (1½ in).

The ATB was considered to be well graded and had an average thickness of 114 mm (4.5 in). The subgrade is a clayey soil with the AASHTO classification of A-6. The Atterberg Limits of the subgrade were a liquid limit of 28 and a plastic limit of 15.

Design/Construction

Details on the development of the thickness design for I-380 were not available. It was reported that the design life chosen was 20 years with a design traffic level of 15,000 vehicles per day. A composite modulus of K equal to 27.1 MPa (100 pci) was assumed for subgrade reaction. No special drainage features were incorporated in the design. The terminal joint design chosen was a lug anchor system. The bridge approach pavement type is concrete.

Performance

The performance of I-380 has been evaluated about every 2 years since its opening. The performance of I-380 has been recorded according to the PSI scale, 0.0 - 5.0, using a South Dakota profilometer. The first PSI value recorded in 1976 was 4.15 and since then has steadily decreased to a value of 3.27 in 1990. The PSI values of I-380 have been taken over the past 14 years and range from 4.2 in 1976 to 3.3 in 1990.

Historical Data (M&R)

Very little maintenance and rehabilitation work has been reported for I-380. The only maintenance that has been reported was joint and crack sealing performed in 1987.

Field Investigation Data

Visual Condition Survey

- Total number of cracks = 439
  - 0 were at high severity
  - 423 were at medium severity
  - 16 were at low severity
- Average crack spacing = 0.9 m (2.98 ft)
- Standard deviation for crack spacing = 0.54 m (1.76 ft)
- Coefficient of variation of the crack spacing = 59 percent
- Figure 58 shows the average crack spacing distribution based on the closest five cracks
- Figure 59 shows actual crack spacing
- Extent of Y cracks = 15 percent
- Cluster cracking was apparent at stations 0+25, 2+00, 4+00, 4+60, and 8+00
- Other distress types included:
  - None
- Pavement shoulders were in fair condition.

Drainage Survey

- Ditches line both sides of the pavement.
- Ditches were lined with vegetation. There was no sign of ponded water or wet ground.

Windshield Survey

- 8.05 km (5 mi) of pavement surveyed was generally in good condition exhibiting no longitudinal cracking and had 13 PCC and 3 AC patches.

Terminal Joint Survey

- Terminal joints used = Lug anchor (two surveyed)
- Joints exhibited no distress

Deflection Testing

- Figure 60 shows normalized/basin (per 4.45 kN (1,000 lb) load) deflections measured along the length of the section (for all seven sensors using the 40.03 kN (9,000 lb) load)
- Load transfer efficiency at cracks at mid-slab
  - 91 to 94 percent in the morning
  - 91 to 93 percent in the afternoon
- Load transfer efficiency at cracks along edge
  - 92 to 94 percent in the morning
  - 92 to 94 percent in the afternoon
- Figure 61 shows the distribution of slab temperatures with depth and time of day.
- Figure 62 shows the average deflections and ranges of deflections for each sensor for center and edge loading, for morning and afternoon testing (using the 40.03 kN (9,000 lb) load within the selected 30.5-m (100-ft) subsection).
- Figure 63 shows the backcalculated radius of relative stiffness, ℓ, values for basin testing for each of the three nominal load levels of 40.03, 53.38, and 71.17 kN (9,000, 12,000, and 16,000 lb).
- ℓ value based on 40.03 kN (9,000 lb) load basin testing
  - Average = 940 mm (37 in)
  - Standard Deviation = 51 mm (2 in)
- ℓ value at crack locations at mid-slab (morning/afternoon)
  - Average = 889/889 mm (35/35 in)
  - Standard Deviation = 25/25 mm (1/1 in)
- ℓ value at crack locations at edge (morning/afternoon)
  - Average = 864/889 mm (34/35 in)
  - Standard Deviation = 25/51 mm (1/2 in)
- Estimated ℓ value using actual slab thickness and concrete modulus of elasticity and assumed modulus of subgrade reaction
  - Value = 787 mm (31 in)

- Figure 64 shows the variation of ℓ in relation to crack spacing (average of the closest five cracks)
- Average deflection under 40.03 kN (9,000 lb) load (0.000001 mm (0.001 in))
  - Basin test - 0.107 (4.2)
  - Mid-slab crack location (morning/afternoon) - 0.114/0.114 (4.5/4.5)
  - Edge crack location (morning/afternoon) - 0.196/0.231 (7.7/9.1)

Crack Width Measurements
- Number of cracks monitored within the 30.5-m (100-ft) subsection = 29
- Morning testing
  - Average temperature 25 mm (1 in) below surface = 18.3°C (65°F)
  - Average crack width = 0.47 mm (0.019 in)
- Afternoon testing
  - Average temperature 25 mm (1 in) below surface = 22.2°C (72°F)
  - Average crack width = 0.34 mm (0.013 in)
- Slab length change, mm/mm/°C (in/in/°F) = 31.9 millionth (17.7 millionth)

Reinforcing Steel Depths and Spacing
- Steel depth measurement offsets = 762, 1 524, 2 286 mm (30, 60, 90 in)
- Range of depth of cover = 58 to 102 mm (2.3 to 4.0 in)
- Average depth of cover = 81 mm (3.2 in)
- Standard deviation for depth of cover = 7.6 mm (0.3 in)
- Steel spacing was not measured

Corrosion Potential Testing (copper-copper sulfate half cell potentiometer)
- Range of measurements = -0.39 to -0.60 volts
- Average measurement = -0.49 volts
- Standard deviation for the measurement = -0.03 volts
- Potential for corrosion was indicated

Concrete Core Examination for Corrosion
- Three cores exhibited spot corrosion
- One core exhibited spread corrosion

Profile Testing
- Average IRI over the 305-m (1,000-ft) section = 1 864 mm/km (118 in/mi)

Laboratory Testing

The following data were obtained from the testing of the concrete, base/subbase, and subgrade materials:

Concrete

Modulus of Elasticity	-	35 850 MPa (5,200,000 lbf/in^2)
Average Split Tensile Strength	-	3.86 MPa (560 lbf/in^2)
Split Tensile Strength Range	-	3.38 to 4.14 MPa (490 to 600 lbf/in^2)
Coefficient of Thermal Expansion	-	9.40 mm/mm/°C (5.22 in/in/°F)

Base

AASHTO Classification	-	ATB

Subbase

AASHTO Classification	-	N/A

Subgrade

Liquid Limit	-	28 percent
Plastic Limit	-	15 percent
Percent Passing #200 Sieve	-	68.7 percent
AASHTO Classification	-	A-6

Chloride content testing was conducted on pulverized concrete retrieved at various depths and locations. Total chloride content and water soluble chloride content values are listed below for IA-3.

Location	**Depth, mm (in)**	**Total Chloride Content (% wt.)**	**Water Soluble Chlorides (% wt.)**
No steel below sample	0-51 (0-2)	0.167	0.154
	51-102 (2-4)	0.085	0.076
Steel directly below sample	0-51 (0-2)	0.184	0.151
	51-102 (2-4)	0.040	0.028

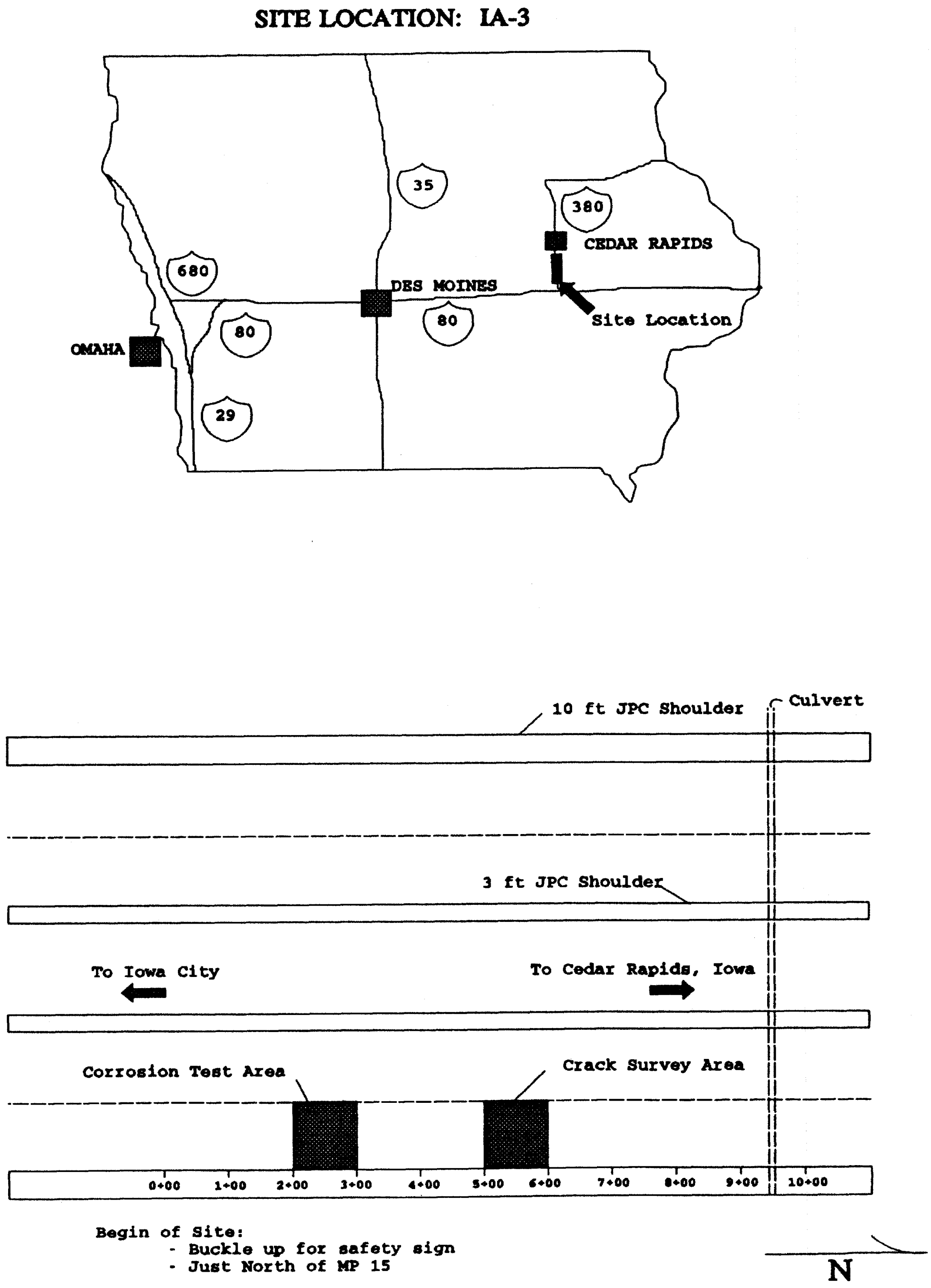
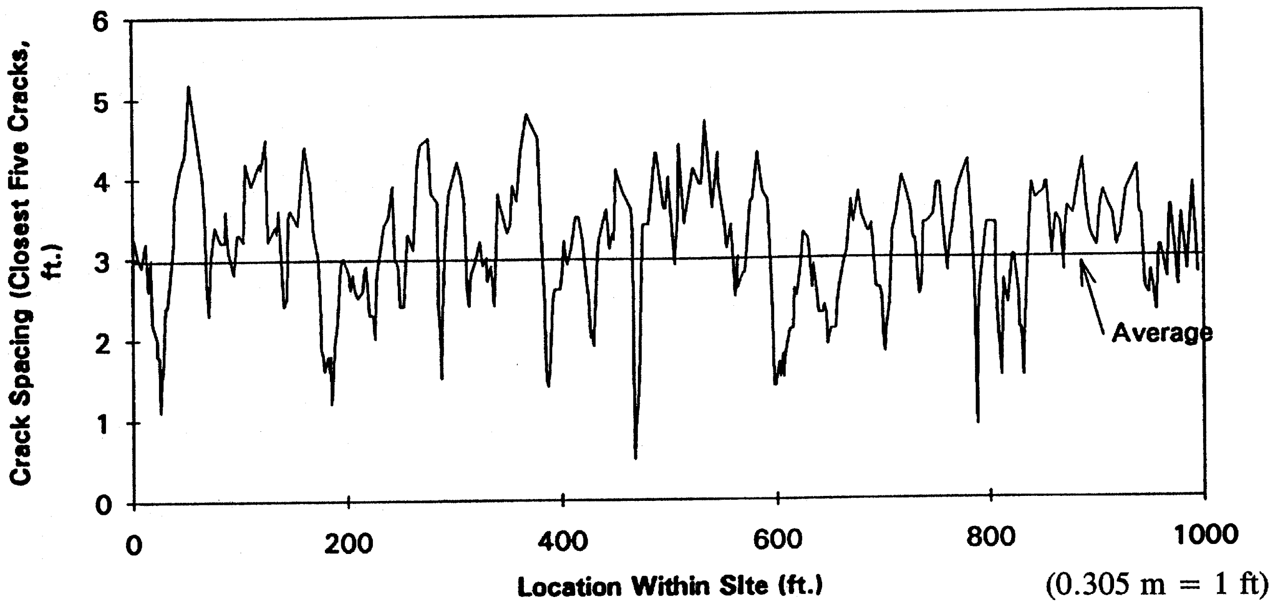


Figure 57. Site details for IA-3.



(0.305 m = 1 ft)

Figure 58. Crack spacing pattern at site IA-3.

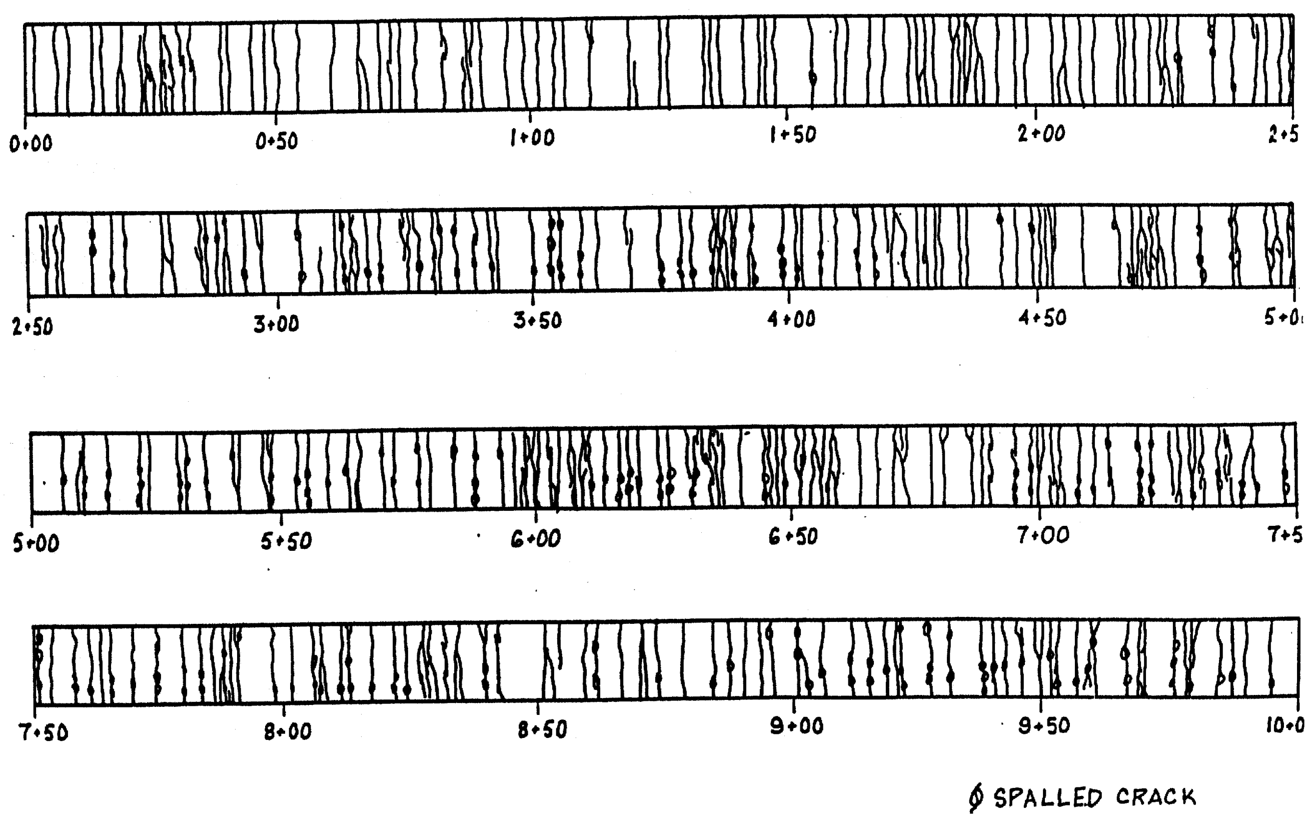
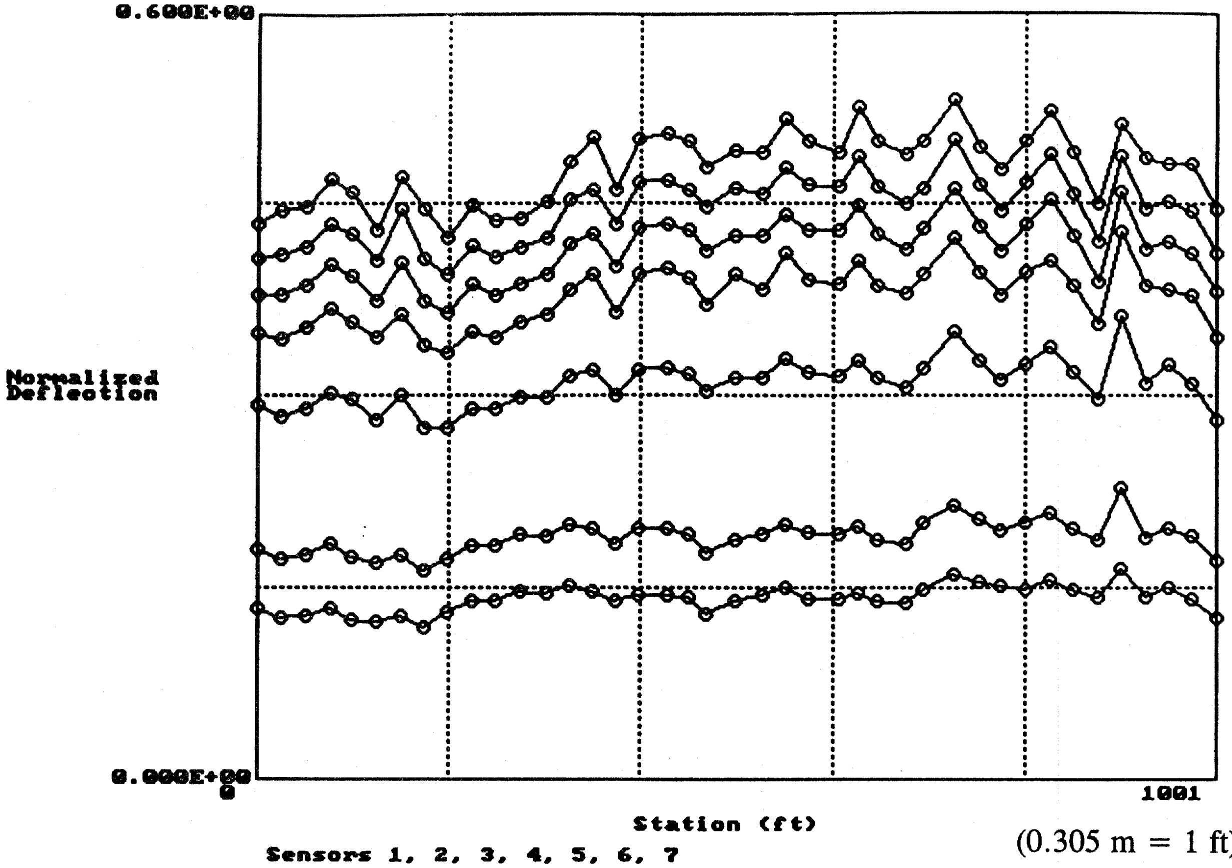
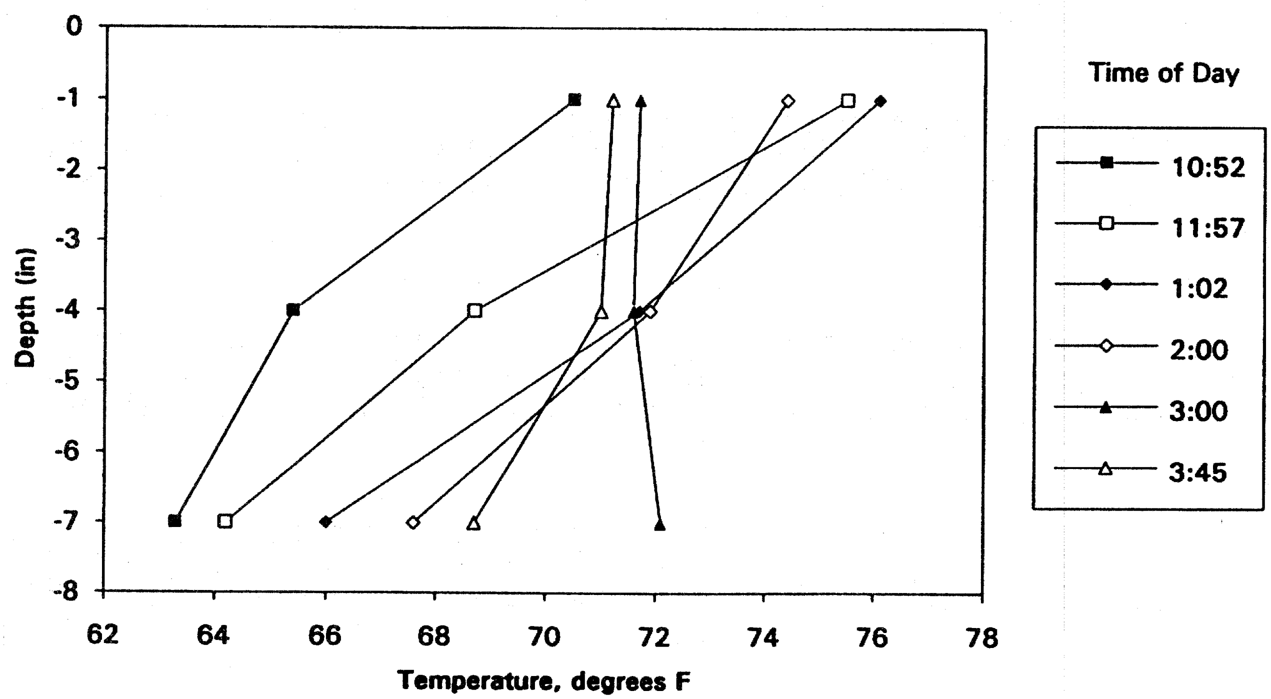


Figure 59. Crack pattern at site IA-3.



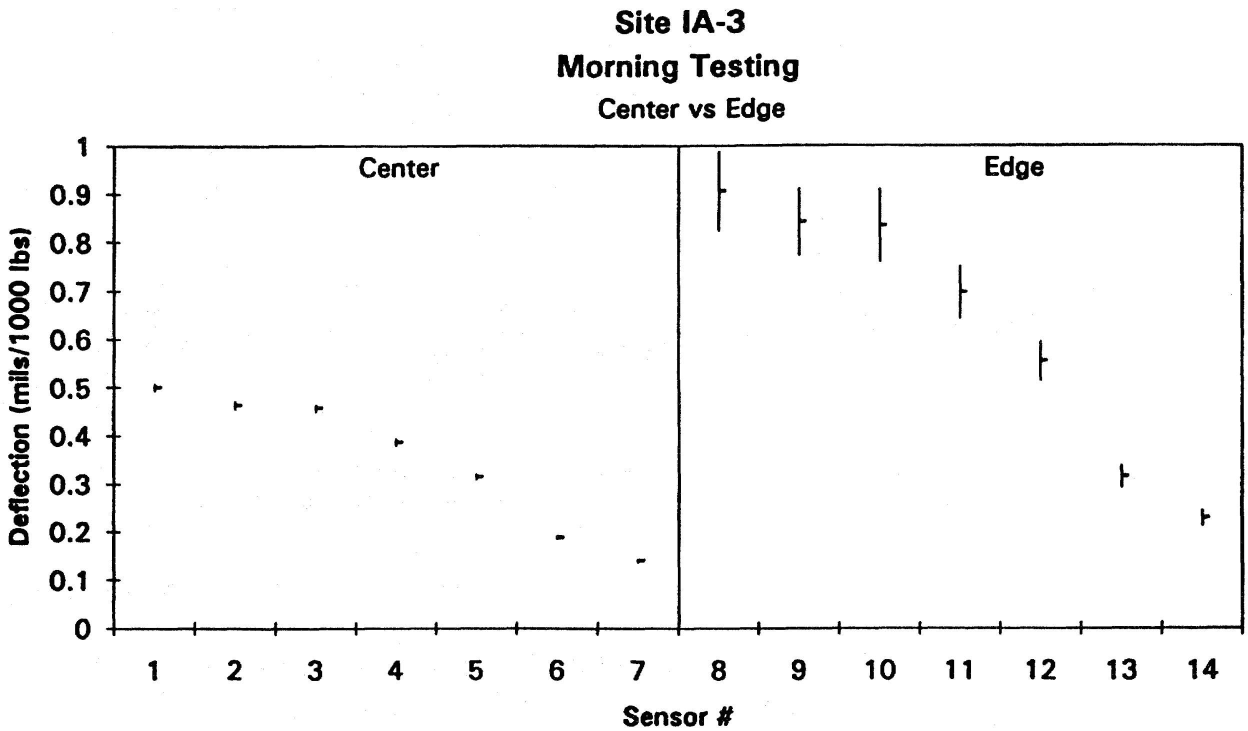
(0.305 m = 1 ft)

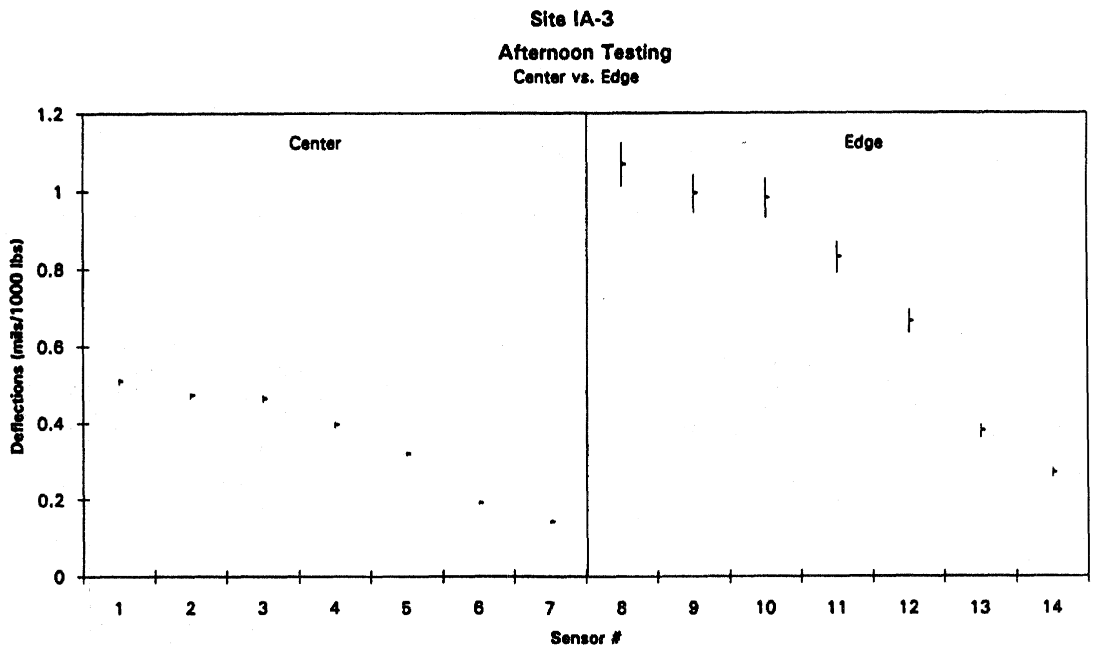
Figure 60. Deflections in inches along the section length at site IA-3.
[normalized per 4.45-kN [1,000-lb] load]



(25.4 mm = 1 in; 0.6°C = 1°F)

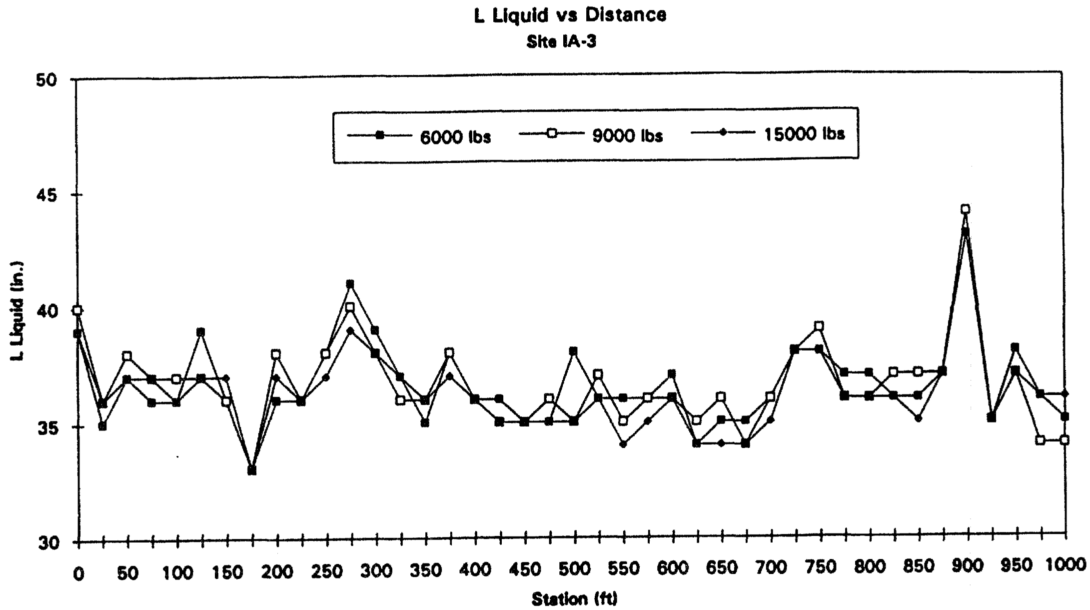
Figure 61. Temperature profile for site IA-3.





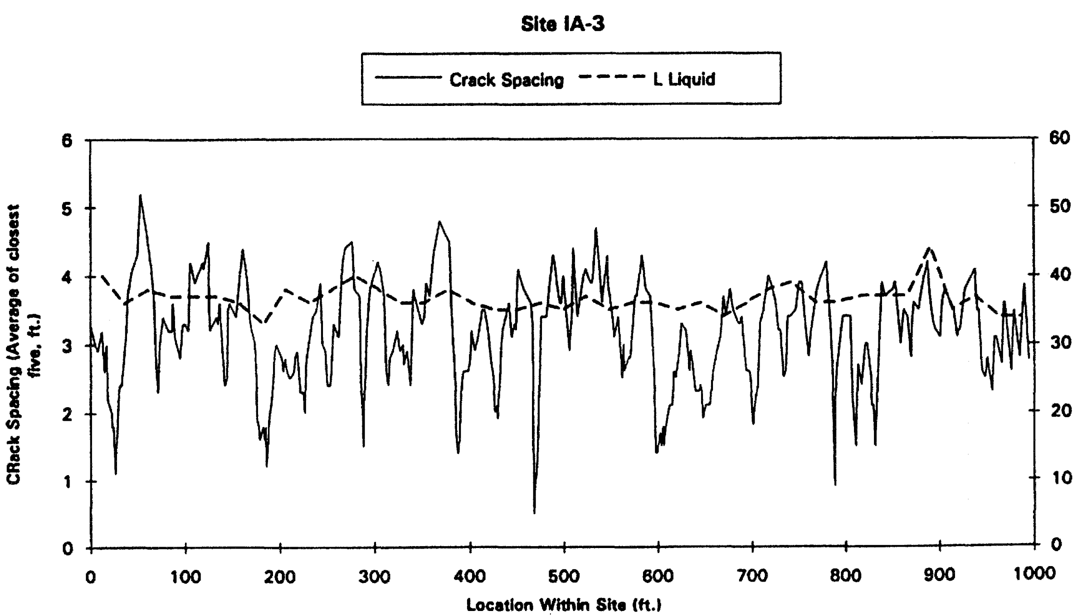
(0.0254 mm = 1 mil; 4.45 kN = 1,000 lb)

Figure 62. Center versus edge deflection for each of the seven sensors at site IA-3.
[40.03 kN (9,000 lb) load]



(25.4 mm = 1 in, 0.305 m = 1 ft)

Figure 63. Backcalculated values of radius of relative stiffness, ℓ (using program ILLI-BACK) for site IA-3.



(0.305 m = 1 ft)

Figure 64. Comparison of radius of relative stiffness values with average crack spacing (nearest five cracks) at site IA-3.

CHAPTER 5 - OKLAHOMA TEST SECTIONS

Section OK-1 - Interstate 40

Introduction

Section OK-1 was constructed in 1987 and was opened to traffic in 1989. OK-1 is a section of I-40 that runs east to west from Arkansas to Oklahoma City, Oklahoma, as shown in figure 65. The length of the highway that was considered runs from milepost 226 to 231. A 305-m (1,000-ft) west bound section was selected for detailed inspection between mileposts 230 and 231, shown in figure 65. The section of I-40 that was chosen is in the central region of Oklahoma.

The climate in this region is a continental climate. The continental climate produces considerable variation in seasonal and annual precipitation. The highest monthly average temperature is 27.8°C (82.1°F) during July, and the lowest monthly average is 2.2°C (35.9°F) during January. I-40 lies within the intermediate freeze-thaw environmental zone.

Traffic

Data on the number of ESALs in its west bound lanes has not been reported. The directional split was reported to be 55 percent for 1984 and 2004. The average daily traffic (ADT) for 1984 was estimated to be 10,500 with a truck percentage of 25.0. The ADT estimate for 2004 is 21,400 with a truck percentage of 25.0. For both years the directional split is 80 percent.

Structure

I-40 is a four-lane divided highway. The pavement lane width is 3.7 m (12 ft) with an outside shoulder width of 3.0 m (10 ft) and an inside shoulder width of 1.2 m (4 ft). The shoulders are constructed of PCC.

I-40 was designed as a three-layer system comprised of 229 mm (9 in) CRCP and a 102-mm (4-in) coarse aggregate bituminous base (CABB) over a subgrade of a clayey sand. The CRC pavement was designed with a longitudinal steel percentage of 0.5 and a transverse steel percentage of 0.08. The longitudinal reinforcing steel bars are #5 bars spaced at 222 mm (8.75 in) with the transverse steel bars being #5 spaced at 1 118 mm (44 in). The bars were placed on chairs as a single layer during the construction of the pavement. The bars are not epoxy coated. The PCC shoulders were 229 mm (9 in) thick and jointed. The shoulders were keyed and tied to the CRC mainline.

The concrete pavement thickness was determined to be 236 mm (9.3 in) based on cores taken as part of this study. The concrete is considered well graded and its consolidation is average. The aggregate type used was a crushed stone with a maximum size of 51 mm (2 in).

The coarse aggregate bituminous base was considered to be well graded and was 102 mm (4 in) thick. The subgrade is a clayey sand with the AASHTO classification of A-6. The Atterberg Limits of the subgrade were a liquid limit of 31 and a plastic limit of 16.

Design/Construction

The initial thickness design was done according to policy by the Oklahoma Department of Transportation. The design life chosen was 20 years with a design AADT level of 10,500 for the year 1984 with a growth rate of 3.6 percent. No specific drainage features were incorporated into the design. The terminal joint design chosen was a wide flange beam system. The bridge approach pavement type is concrete.

Construction began in October 1987 and ended in March 1989. The specified concrete properties were slump of 25 to 76 mm (1 to 3 in), air content of 4 to 6 percent, and a strength of 20.7 MPa (3,000 lbf/in^2) at 28 days.

Performance

The performance of I-40 has not been reported at this time.

Historical Data (M&R)

Very little maintenance and rehabilitation work has been reported for I-40. The only maintenance that has been reported is the full-depth repair of one punchout in 1990.

Field Investigation Data

Visual Condition Survey

- Total number of cracks = 124
  - 0 were at high severity
  - 124 were at medium severity
  - 0 were at low severity
- Average crack spacing = 2.59 m (8.51 ft)
- Standard deviation for crack spacing = 1.76 m (5.78 ft)
- Coefficient of variation of the crack spacing = 68 percent
- Figure 66 shows average crack spacing distribution based on the closest five cracks
- Figure 67 shows actual crack spacing
- Extent of Y cracks = 19 percent
- Cluster cracking was not apparent
- No other distress types were present
- Pavement shoulders were in good condition.

Drainage Survey

- Ditches line both sides of the pavement.

- Ditches were lined with vegetation. There were signs of ponded water and wet ground. It had rained before and during testing.

Windshield Survey

- 4.83 km (3 mi) of pavement surveyed was generally in good condition exhibiting only 1 PCC patch.

Terminal Joint Survey

- Terminal joints used = Wide flange (two surveyed)
- Joints exhibited no distress other than some sealant damage.

Deflection Testing

- Figure 68 shows normalized/basin (per 4.45 kN (1,000 lb) load) deflections measured along the length of the section (for all seven sensors using the 40.03 kN (9,000 lb) load)
- Load transfer efficiency at cracks at mid-slab
  - 80 to 97 percent in the morning
  - 80 to 96 percent in the afternoon
- Load transfer efficiency at cracks along edge
  - 71 to 94 percent in the morning
  - 56 to 92 percent in the afternoon
- Figure 69 shows the distribution of slab temperatures with depth and time of day.
- Figure 70 shows the average deflections and ranges of deflections for each sensor for center and edge loading, for morning and afternoon testing (using the 40.03 kN (9,000 lb) load within the selected 30.5-m (100-ft) subsection).
- Figure 71 shows the backcalculated radius of relative stiffness, ℓ, values for basin testing for each of the three nominal load levels of 40.03, 53.38, and 71.17 kN (9,000, 12,000, and 16,000 lb).
- ℓ value based on 40.03 kN (9,000 lb) load basin testing
  - Average = 889 mm (35 in)
  - Standard Deviation = 102 mm (4 in)
- ℓ value at crack locations at mid-slab (morning/afternoon)
  - Average = 610/584 mm (24/23 in)
  - Standard Deviation = 102/102 mm (4/4 in)
- ℓ value at crack locations at edge (morning/afternoon)
  - Average = 635/610 mm (25/24 in)
  - Standard Deviation = 76/102 mm (3/4 in)
- Estimated ℓ value using actual slab thickness and concrete modulus of elasticity and assumed modulus of subgrade reaction
  - Value = 889 mm (35 in)
- Figure 72 shows the variation of ℓ in relation to crack spacing (average of the closest five cracks)

- Average deflection under 40.03 kN (9,000 lb) load (0.000001 mm (0.001 in))
  - Basin test - 0.069 (2.7)
  - Mid-slab crack location (morning/afternoon) - 0.079/0.081 (3.1/3.2)
  - Edge crack location (morning/afternoon) - 0.127/0.135 (5.0/5.3)

Crack Width Measurements

- Number of cracks monitored within the 30.5-m (100-ft) subsection = 10
- Morning testing
  - Average temperature 102 mm (4 in) below surface = 18.3°C (65°F)
  - Average crack width = 0.63 mm (0.025 in)
- Afternoon testing
  - Average temperature 102 mm (4 in) below surface = 27.8°C (82°F)
  - Average crack width = 0.44 mm (0.017 in)
- Slab length change, mm/mm/°C (in/in/°F) = 6.7 millionth (3.7 millionth)

Reinforcing Steel Depths and Spacing

- Steel depth measurement offsets = 610, 1 829, 2 438 mm (24, 72, 96 in)
- Range of depth of cover = 71 to 137 mm (2.8 to 5.4 in)
- Average depth of cover = 99 mm (3.9 in)
- Standard deviation for depth of cover = 10 mm (0.4 in)
- Steel spacing was about 185 mm (7.3 in)

Corrosion Potential Testing (copper-copper sulfate half cell potentiometer)

- Range of measurements = -0.21 to -0.42 volts
- Average measurement = -0.28 volts
- Standard deviation for the measurement = -0.05 volts
- Potential for corrosion was marginal

Concrete Core Examination for Corrosion

- Three cores exhibited spot corrosion
- One core exhibited spread corrosion

Profile Testing

- Average IRI over the 305-m (1,000-ft) section = 837 mm/km (53 in/mi)

Laboratory Testing

The following data were obtained from the testing of the concrete, base/subbase, and subgrade materials:

Concrete

Modulus of Elasticity	-	39 990 MPa (5,800,000 lbf/in^2)
Average Split Tensile Strength	-	3.31 MPa (480 lbf/in^2)
Split Tensile Strength Range	-	2.90 to 3.59 MPa (420 to 520 lbf/in^2)
Coefficient of Thermal Expansion	-	8.89 mm/mm/°C (4.94 in/in/°F)

Base

AASHTO Classification	-	ATB

Subbase

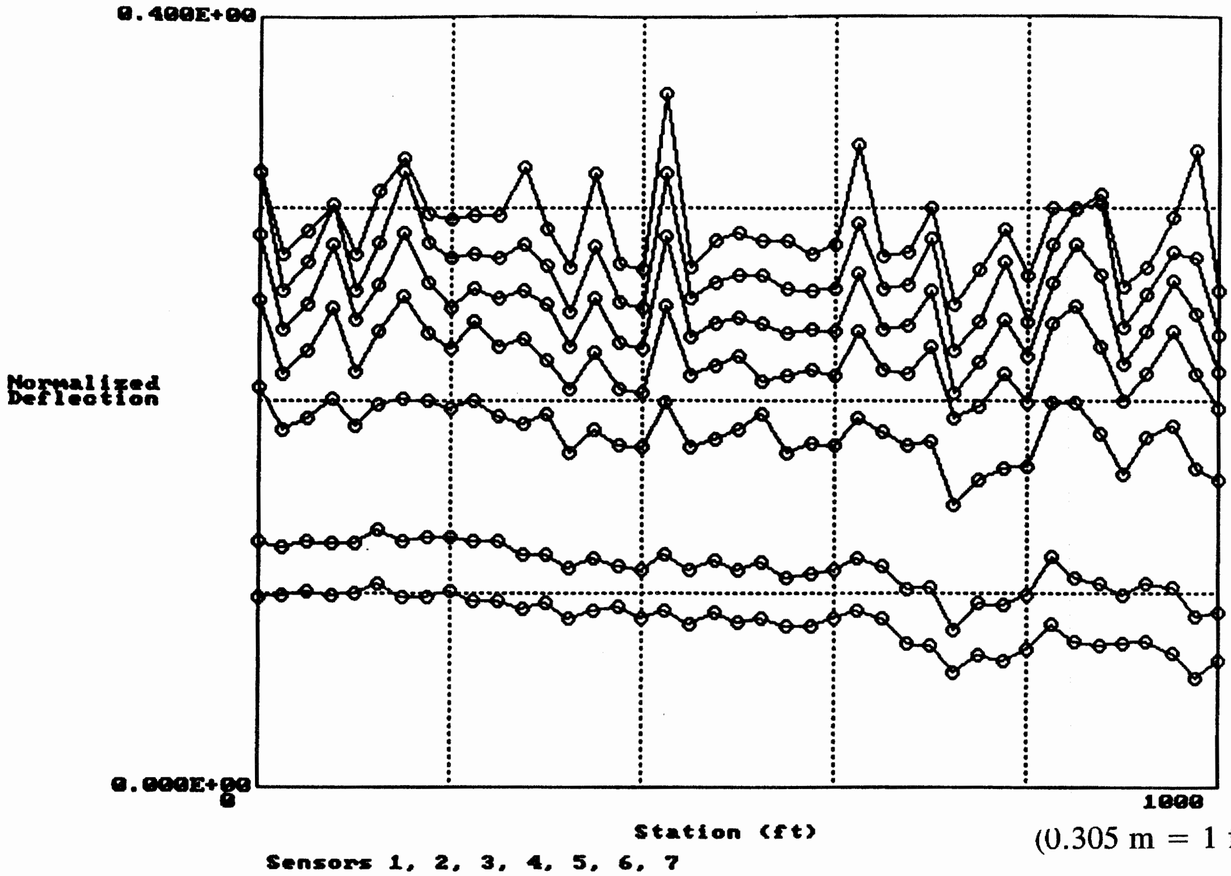
AASHTO Classification	-	N/A

Subgrade

Liquid Limit	-	31 percent
Plastic Limit	-	16 percent
Percent Passing #200 Sieve	-	61.3 percent
AASHTO Classification	-	A-6

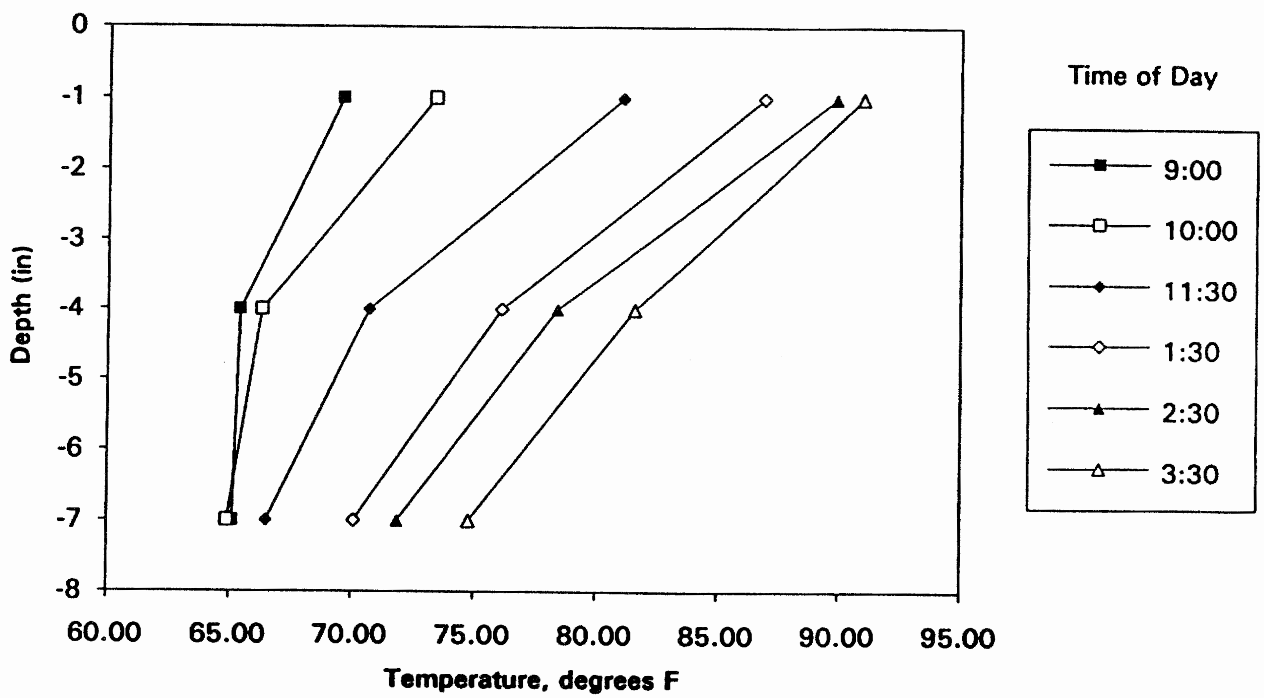
Chloride content testing was conducted on pulverized concrete retrieved at various depths and locations. Total chloride content and water soluble chloride content values are listed below for OK-1.

Location	Depth, mm (in)	Total Chloride Content (% wt.)	Water Soluble Chlorides (% wt.)
No steel below sample	0-51 (0-2)	0.054	0.036
	51-102 (2-4)	0.016	0.012
Steel directly below sample	0-51 (0-2)	0.045	0.036
	51-102 (2-4)	0.017	0.010



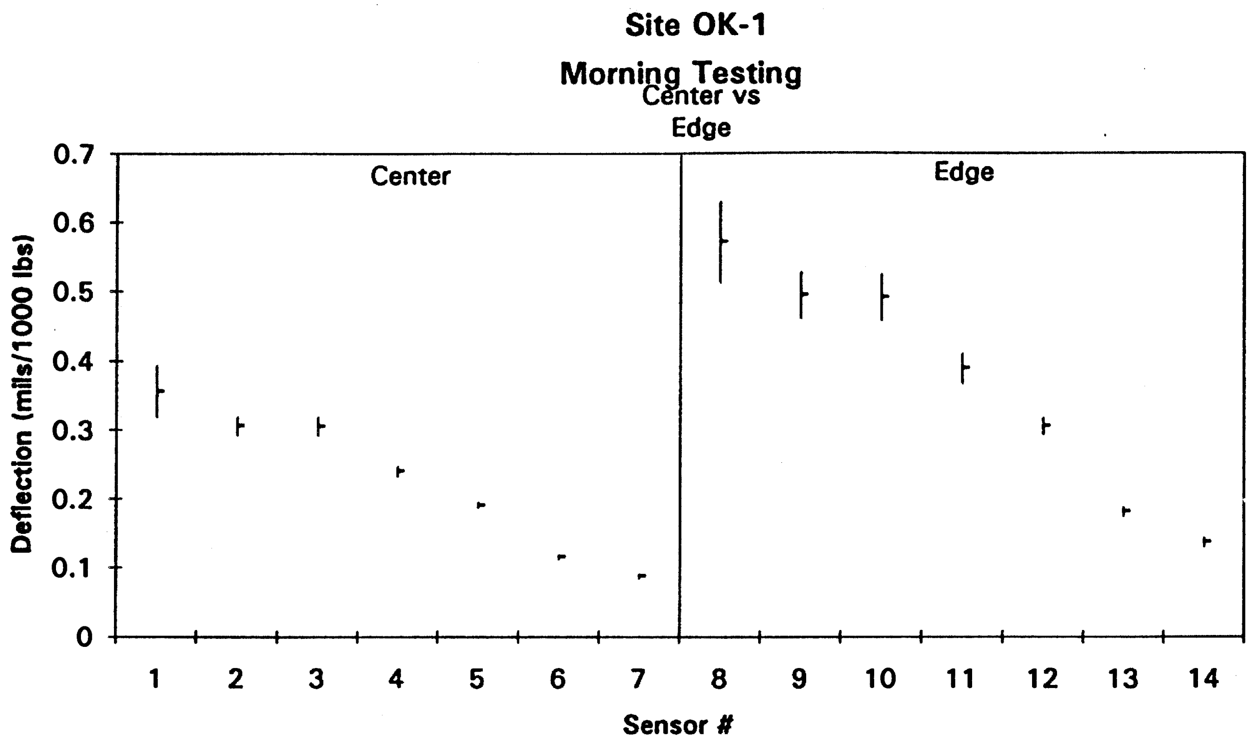
(0.305 m = 1 ft)

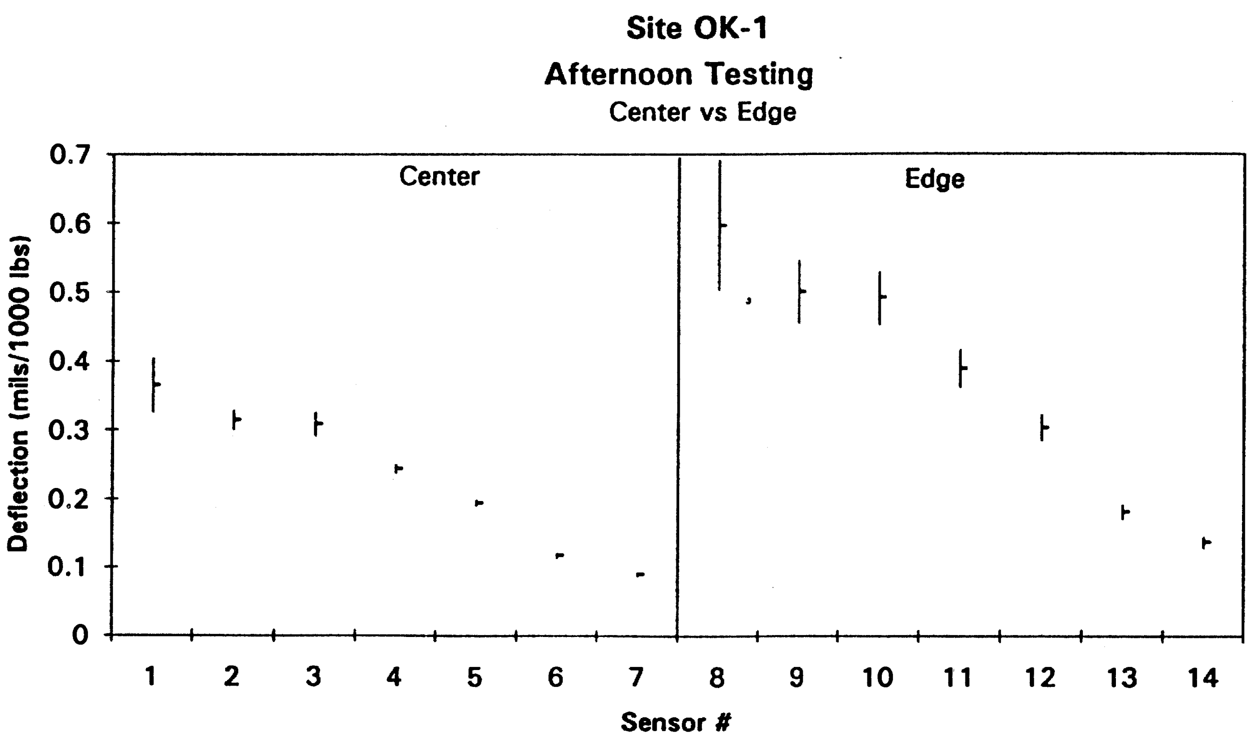
Figure 68. Deflections in inches along the section length at site OK-1.
[normalized per 4.45-kN [1,000-lb] load]



(25.4 mm = 1 in; 0.6°C = 1°F)

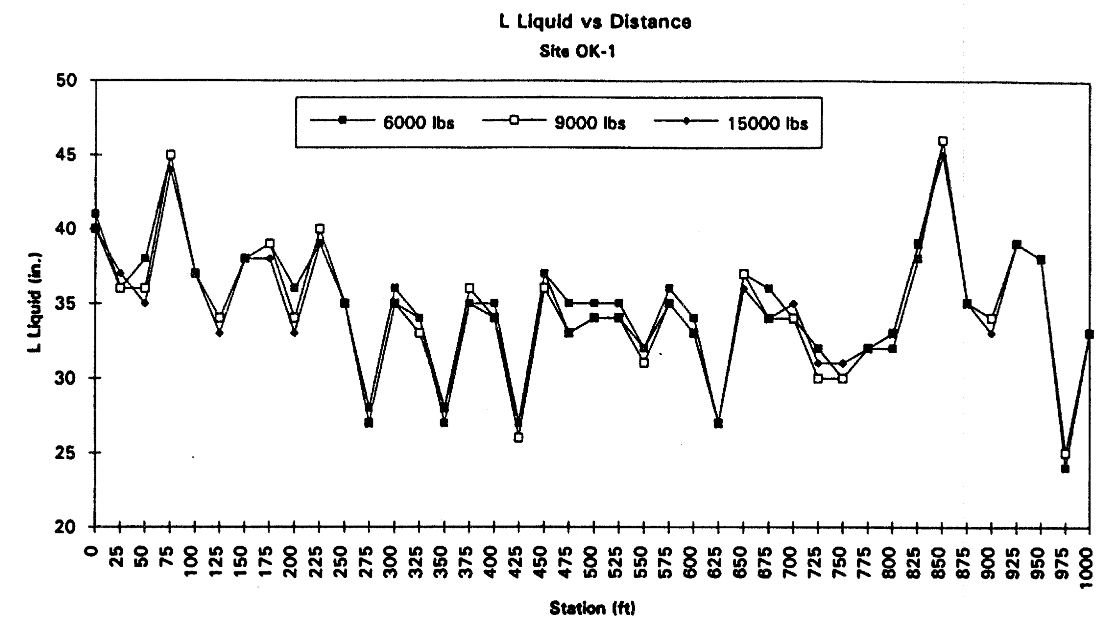
Figure 69. Temperature profile for site OK-1.





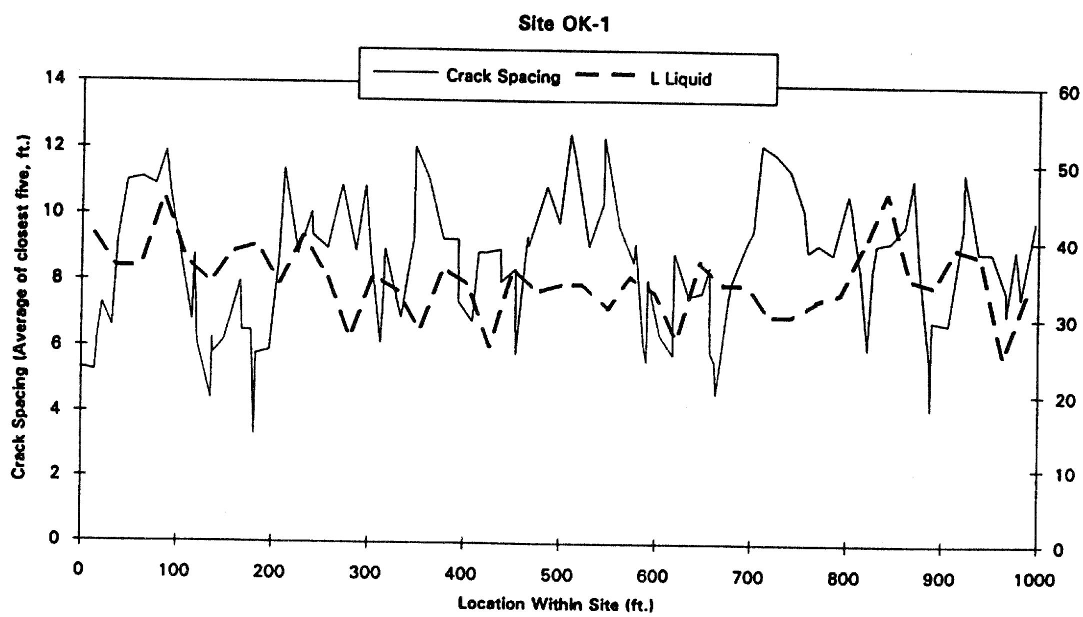
(0.0254 mm = 1 mil; 4.45 kN = 1,000 lb)

Figure 70. Center versus edge deflection for each of the seven sensors at site OK-1.
[40.03 kN (9,000 lb) load]



(25.4 mm = 1 in, 0.305 m = 1 ft)

Figure 71. Backcalculated values of radius of relative stiffness, ℓ (using program ILLI-BACK) for site OK-1.



(0.305 m = 1 ft)

Figure 72. Comparison of radius of relative stiffness values with average crack spacing (nearest five cracks) at site OK-1.

Section OK-2 - US Highway 69

Introduction

Section OK-2 was constructed in 1986 and 1988. It was opened to traffic in 1988. OK-2 is a section of US-69 that runs north to south from the Oklahoma-Texas border to McAlester, Oklahoma, as shown in figure 73. The length of the highway that was considered runs from 1.61 km (1 mi) north of Chokie, Oklahoma, to the county line of Atoka County. A 305-m (1,000-ft) north bound section was selected for detailed inspection 5.63 km (3.5 mi) north of the I-43 east intersection, as shown in figure 73. The section of US-69 that was chosen is in the south eastern region of Oklahoma.

The climate in this region is a continental climate. The continental climate produces considerable variation in seasonal and annual precipitation. The highest monthly average temperature is 27.8°C (82.1°F) during July, and the lowest monthly average is 2.2°C (35.9°F) during January. US-69 lies within the intermediate freeze-thaw environmental zone.

Traffic

Data on the number of ESALs in its north bound lanes were not available. The directional split for 1984 was reported to be 60 percent. The ADT for 1984 was estimated to be 6,900 with a truck percentage of 26.0. The ADT estimated for 2004 is 13,300. For 1984 the lane split was reported to be 80 percent. US-69 was designed for a 20-year design life.

Structure

US-69 is a four-lane divided highway. The pavement lane width is 3.7 m (12 ft) with an outside shoulder width of 3.0 m (10 ft) and an inside shoulder width of 1.2 m (4 ft). The shoulders are constructed of PCC.

US-69 was designed as a four-layer system comprised of a 229 mm (9 in) CRCP, a 76-mm (3-in) ATB and a 305-mm (12-in) aggregate subbase over a subgrade of clay. The CRC pavement was designed with a longitudinal steel percentage of 0.5 and a transverse steel percentage of 0.08. The longitudinal reinforcing steel bars are #5 bars spaced at 175 mm (6.875 in) with the transverse steel bars being #5 spaced at 1 118 mm (44 in). The bars were placed on chairs as a single layer during the construction of the pavement. The bars are not epoxy coated. The PCC shoulders were jointed, keyed, and tied to the CRC mainline.

The average concrete pavement thickness along OK-2 was determined to be 234 mm (9.2 in) based on cores taken as part of this study. The concrete is considered well graded and its consolidation was average. The aggregate type used was a crushed stone with a maximum size of 38 mm (1½ in).

The ATB was considered to be well graded and had an average thickness of 84 mm (3.3 in). The subbase layer was an unbound granular subbase that was 307 mm (12.1 in) thick and was well graded. The AASHTO classification for the subbase is A-1-a. The subgrade is a clay with the AASHTO classification of A-6. The Atterberg Limits of the subgrade were a liquid limit of 48 and a plastic limit of 27.

Design/Construction

The thickness design was done according to the Oklahoma Subgrade Index Method by the Oklahoma Department of Transportation. The design life chosen was 20 years. The allowable crack width was 1.02 mm (0.040 in), the allowable steel stress was 276 MPa (40 ksi), and the minimum crack spacing was 0.9 m (3 ft). Drainage was not considered in the design. The terminal joint design chosen was a wide flange beam system. The bridge approach pavement type was concrete.

Construction began in August 1986 and finished in September 1988. The specified concrete properties were slump of 25 to 76 mm (1 to 3 in), air content of 4 to 6 percent, and a strength of 20.7 MPa (3,000 lbf/in^2) at 28 days.

Performance

The performance of I-40 has not been reported at this time.

Historical Data (M&R)

No maintenance and rehabilitation work has been reported for US-69.

Field Investigation Data

Visual Condition Survey

- Total number of cracks = 229
  - 1 was at high severity
  - 201 were at medium severity
  - 27 were at low severity
- Average crack spacing = 1.39 m (4.57 ft)
- Standard deviation for crack spacing = 1.03 m (3.38 ft)
- Coefficient of variation of the crack spacing = 74 percent
- Figure 74 shows average crack spacing distribution based on the closest five cracks
- Figure 75 shows actual crack spacing
- Extent of Y cracks = 7 percent
- Cluster cracking was apparent at stations 1+00 and 4+50
- No other distress types were present.
- Pavement shoulders were in good condition.

Drainage Survey

- Ditches line both sides of the pavement.

- Ditches were lined with vegetation. There were signs of ponded water and wet ground. It had rained heavily before testing.

Windshield Survey
- No survey was conducted because of lack of time caused by the rainfall.

Terminal Joint Survey
- Terminal joints used = Wide flange (two surveyed)
- Joints exhibited the following distresses:
  - One joint exhibited 0.19 m^2 (2 ft^2) of low severity spalling
  - Other joint had 1.11 m^2 (12 ft^2) of medium severity spalling and 2.23 m^2 (24 ft^2) of high severity spalling.

Deflection Testing
- Figure 76 shows normalized/basin (per 4.45 kN (1,000 lb) load) deflections measured along the length of the section (for all seven sensors using the 40.03 kN (9,000 lb) load)
- Load transfer efficiency at cracks at mid-slab
  - 90 to 96 percent in the morning
  - 85 to 92 percent in the afternoon
- Load transfer efficiency at cracks along edge
  - 91 to 95 percent in the morning
  - 88 to 96 percent in the afternoon
- Figure 77 shows the distribution of slab temperature with depth and time of day.
- Figure 78 shows the average deflections and ranges of deflections for each sensor for center and edge loading, for morning and afternoon testing (using the 40.03 kN (9,000 lb) load within the selected 30.5-m (100-ft) subsection).
- Figure 79 shows the backcalculated radius of relative stiffness, ℓ, values for basin testing for each of the three nominal load levels of 40.03, 53.38, and 71.17 kN (9,000, 12,000, and 16,000 lb).
- ℓ value based on 40.03 kN (9,000 lb) load basin testing
  - Average = 1 016 mm (40 in)
  - Standard Deviation = 152 mm (6 in)
- ℓ value at crack locations at mid-slab (morning/afternoon)
  - Average = 787/787 mm (31/31 in)
  - Standard Deviation = 76/76 mm (3/3 in)
- ℓ value at crack locations at edge (morning/afternoon)
  - Average = 737/762 mm (29/30 in)
  - Standard Deviation = 102/102 mm (4/4 in)
- Estimated ℓ value using actual slab thickness and concrete modulus of elasticity and assumed modulus of subgrade reaction
  - Value = 914 mm (36 in)

- Figure 80 shows the variation of ℓ in relation to crack spacing (average of the closest five cracks)
- Average deflection under 40.03 kN (9,000 lb) load (0.000001 mm (0.001 in))
    - Basin test - 0.069 (2.7)
    - Mid-slab crack location (morning/afternoon) - 0.076/0.076 (3.0/3.0)
    - Edge crack location (morning/afternoon) - 0.104/0.094 (4.1/3.7)

Crack Width Measurements
- Number of cracks monitored within 30.5-m (100-ft) subsection = 28
- Morning testing
    - Average temperature 102 mm (4 in) below surface = 5.6°C (42°F)
    - Average crack width = 0.48 mm (0.019 in)
- Afternoon testing
    - Average temperature 102 mm (4 in) below surface = 12.2°C (54°F)
    - Average crack width = 0.38 mm (0.015 in)
- Slab length change, mm/mm/°C (in/in/°F) = 13.9 millionth (7.7 millionth)

Reinforcing Steel Depths and Spacing
- Steel depth measurement offsets = 610, 1 829, 2 438 mm (24, 72, 96 in)
- Range of depth of cover = 91 to 127 mm (3.6 to 5.0 in)
- Average depth of cover = 114 mm (4.5 in)
- Standard deviation for depth of cover = 7.6 mm (0.3 in).
- Steel spacing was about 203 mm (8 in)

Corrosion Potential Testing (copper-copper sulfate half cell potentiometer)
- Range of measurements = -0.03 to -0.23 volts
- Average measurement = -0.10 volts
- Standard deviation for the measurement = -0.04 volts
- Potential for corrosion was not indicated

Concrete Core Examination for Corrosion
- One core exhibited spot corrosion
- Three cores exhibited no corrosion

Profile Testing
- No profile testing was conducted because of equipment malfunction

Laboratory Testing

Concrete

Modulus of Elasticity	-	45 500 MPa (6,600,000 lbf/in^2)
Average Split Tensile Strength	-	3.93 MPa (570 lbf/in^2)
Split Tensile Strength Range	-	3.31 to 4.48 MPa (480 to 650 lbf/in^2)
Coefficient of Thermal Expansion	-	8.55 mm/mm/°C (4.75 in/in/°F)

Base

AASHTO Classification	-	ATB

Subbase

AASHTO Classification	-	Granular

Subgrade

Liquid Limit	-	48 percent
Plastic Limit	-	27 percent
Percent Passing #200 Sieve	-	77.5 percent
AASHTO Classification	-	A-6

Chloride content testing was conducted on pulverized concrete retrieved at various depths and locations. Total chloride content and water soluble chloride content values are listed below for OK-2.

Location	Depth, mm (in)	Total Chloride Content (% wt.)	Water Soluble Chlorides (% wt.)
No steel below sample	0-51 (0-2)	0.018	0.010
	51-102 (2-4)	0.015	0.000
Steel directly below sample	0-51 (0-2)	0.020	0.000
	51-102 (2-4)	0.012	0.000

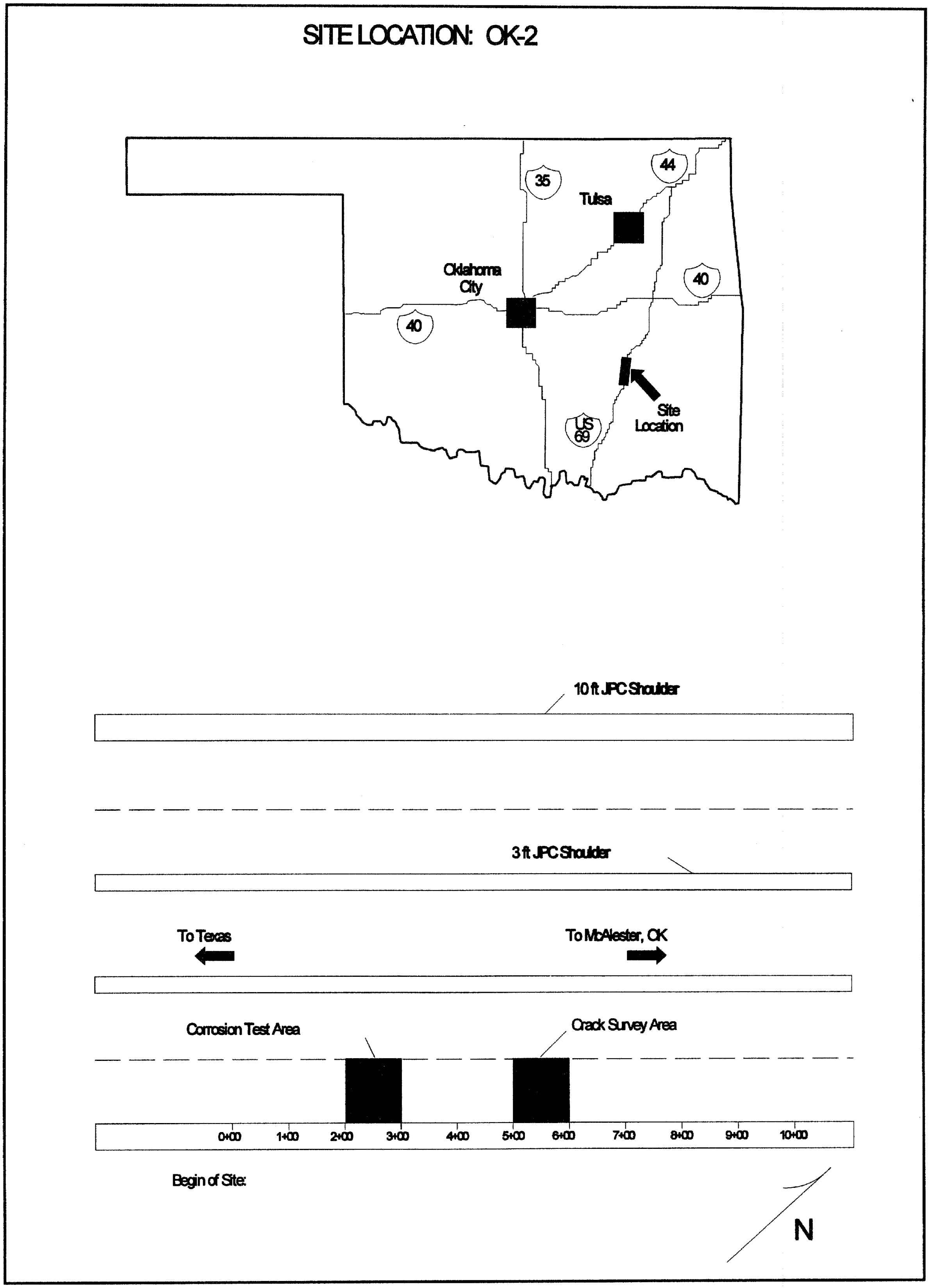
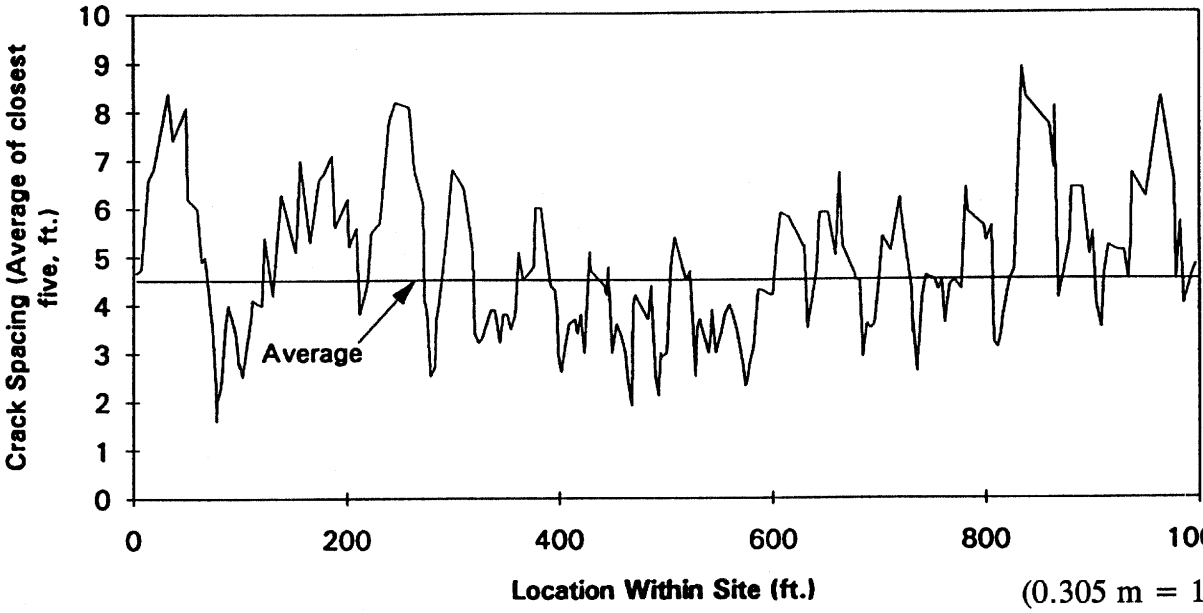


Figure 73. Site details for OK-2.



(0.305 m = 1 ft)

Figure 74. Crack spacing pattern at site OK-2.

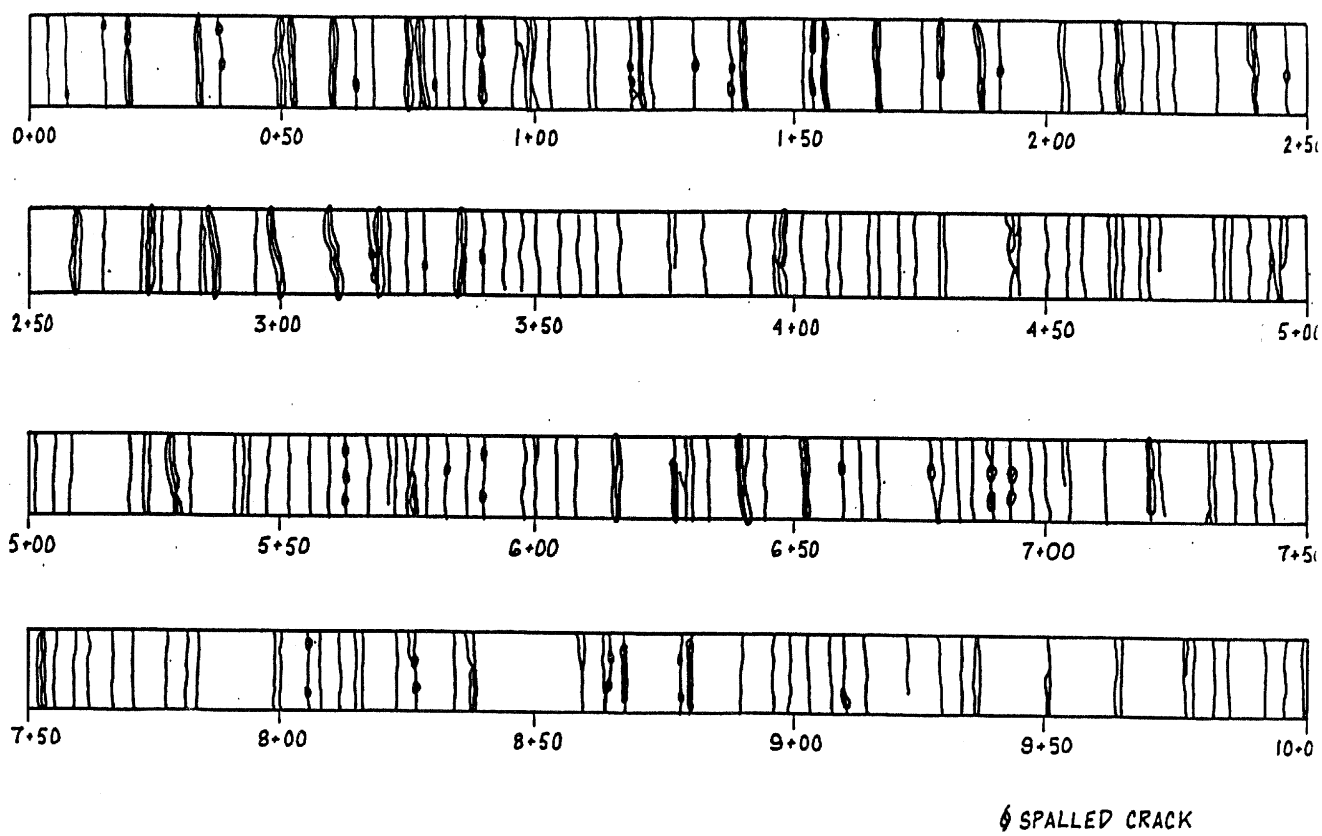
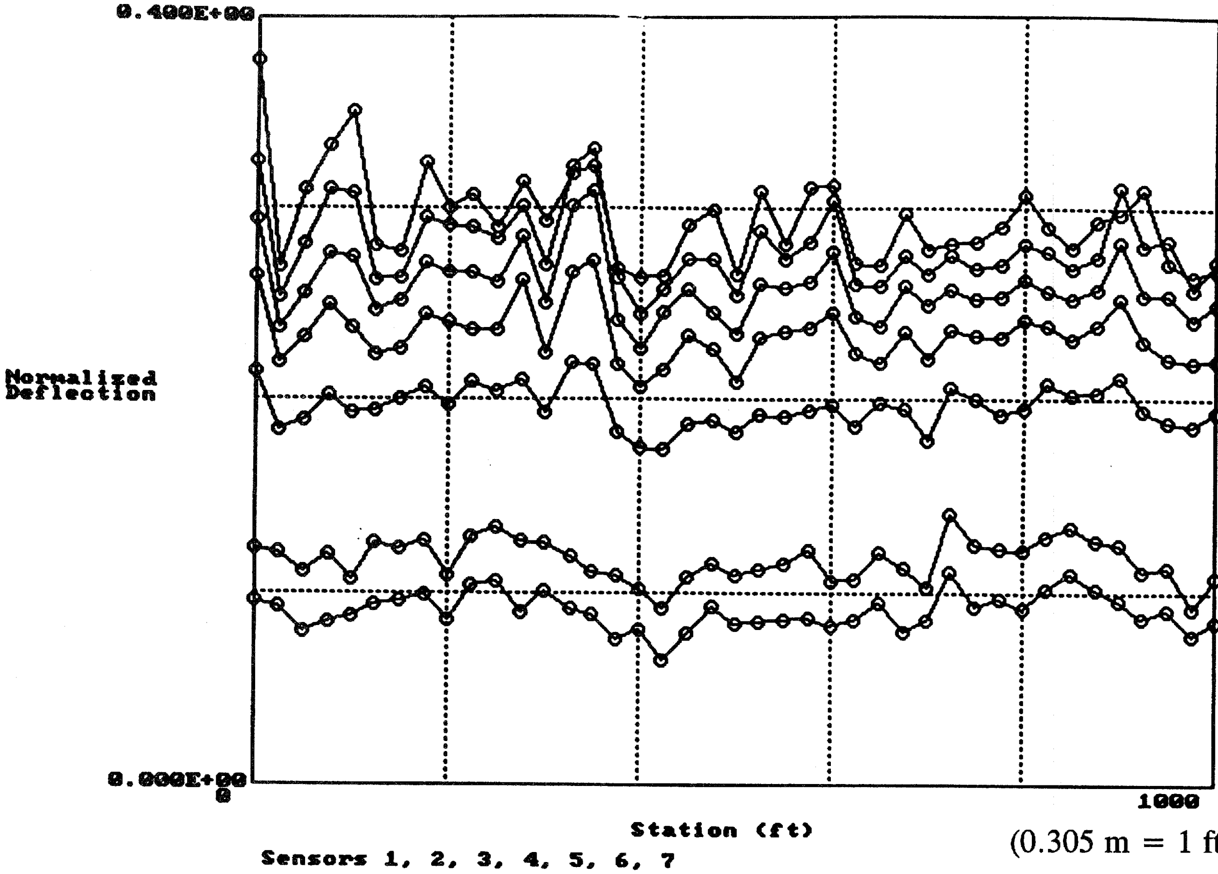
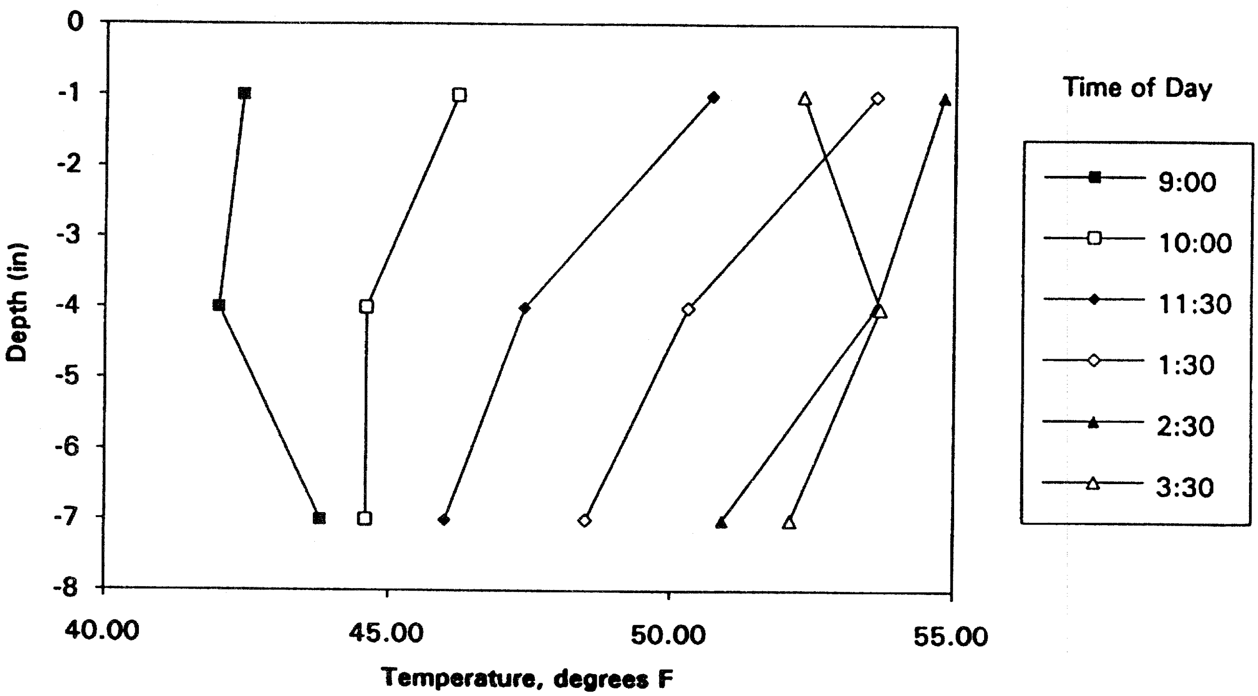


Figure 75. Crack pattern at site OK-2.



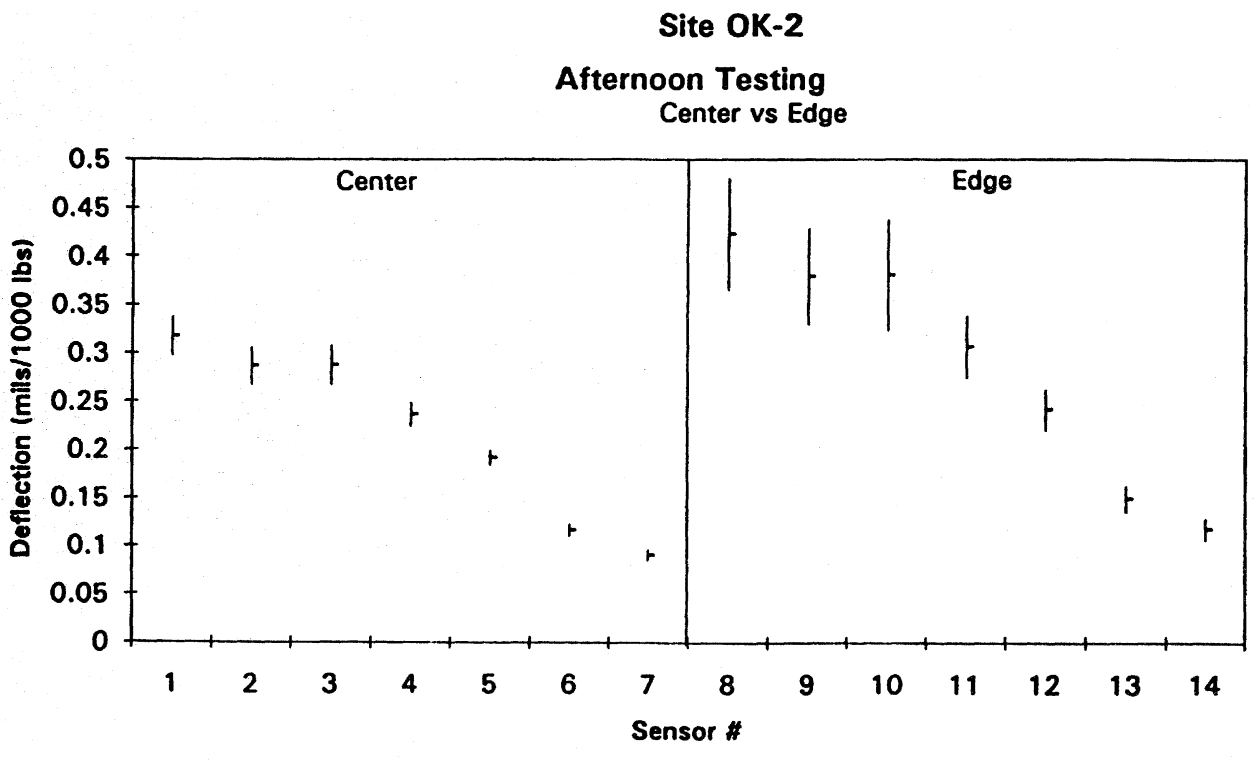
(0.305 m = 1 ft)

Figure 76. Deflections in inches along the section length at site OK-2.
[normalized per 4.45-kN [1,000-lb] load]



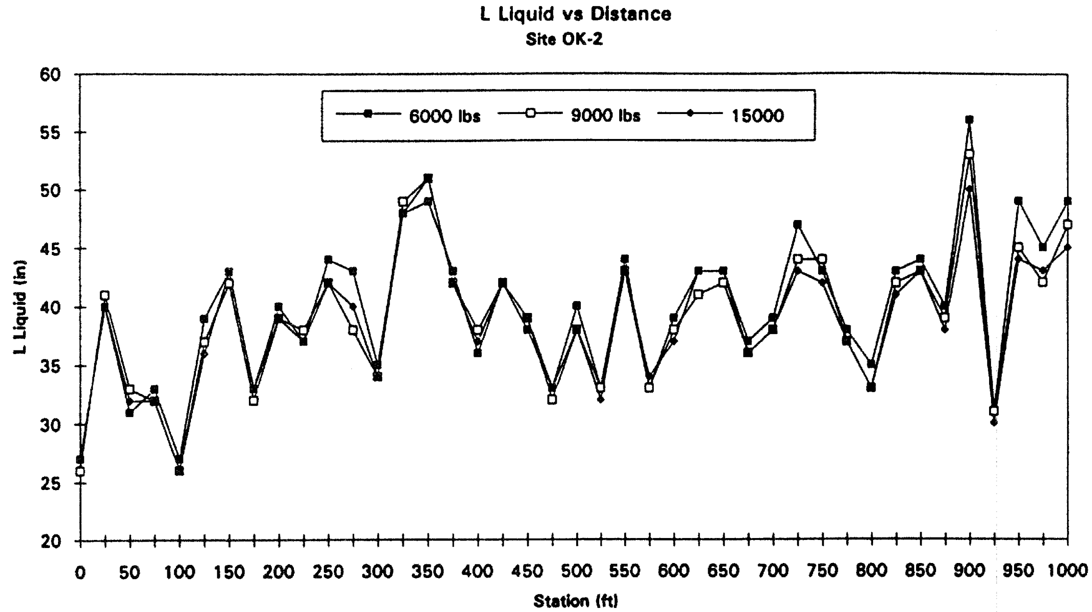
(25.4 mm = 1 in; 0.6°C = 1°F)

Figure 77. Temperature profile for site OK-2.



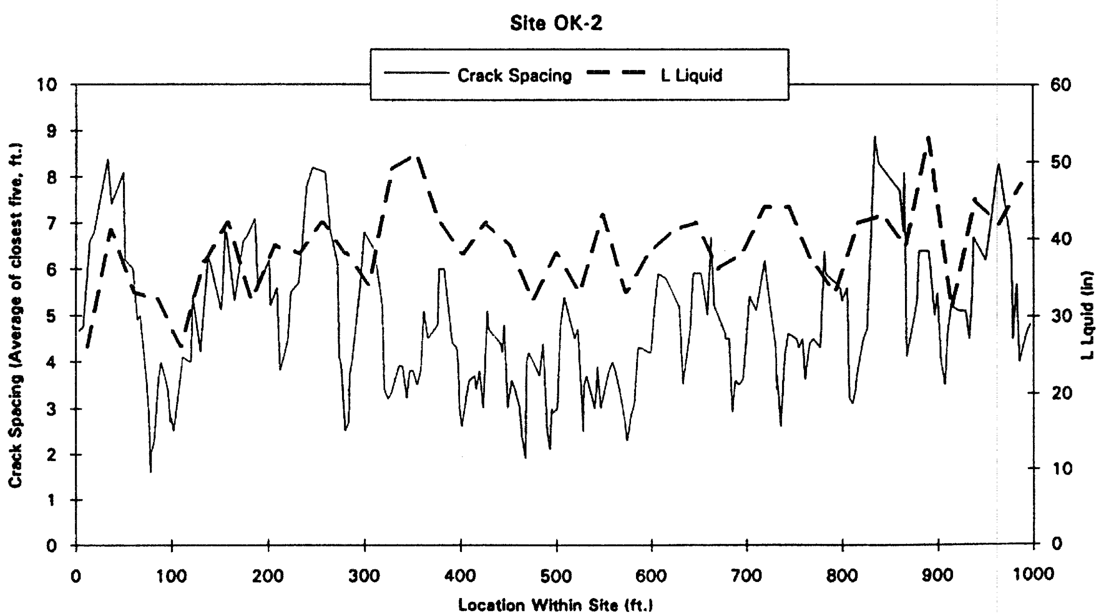
(0.0254 mm = 1 mil; 4.45 kN = 1,000 lb)

Figure 78. Center versus edge deflection for each of the seven sensors at site OK-2.
[40.03 kN (9,000 lb) load]



(25.4 mm = 1 in, 0.305 m = 1 ft)

Figure 79. Backcalculated values of radius of relative stiffness, ℓ (using program ILLI-BACK) for site OK-2.



(0.305 m = 1 ft)

Figure 80. Comparison of radius of relative stiffness values with average crack spacing (nearest five cracks) at site OK-2.

Section OK-3 - Interstate 35

Introduction

Section OK-3 was constructed during 1988 and 1989 and was opened to traffic in 1989. OK-3 is a section of I-35 that runs north to south from Oklahoma City to the Kansas-Oklahoma border as shown in figure 81. The length of the highway that was considered runs from mileposts 147 to 153. A 305-m (1,000-ft) north bound section was selected for detailed inspection between mileposts 148 and 149, as shown in figure 81. The section of I-35 that was chosen is in the central region of Oklahoma.

The climate in this region is a continental climate. The continental climate produces considerable variation in seasonal and annual precipitation. The highest monthly average temperature is 27.8°C (82.1°F) during July, and the lowest monthly average is 2.2°C (35.9°F) during January. I-35 lies within the intermediate freeze-thaw environmental zone.

Traffic

The number of ESALs in its north bound lanes has not been reported. The directional split for 1984 was reported to be 55 percent. The lane distribution for 1989 was reported to be 80 percent. The ADT for 1984 was estimated to be 22,000 with a truck percentage of 12.0. The ADT estimated for 2004 is 42,000. I-35 was designed for a 20-year design life.

Structure

I-35 is a four-lane divided highway. The pavement lane width is 3.7 m (12 ft) with an outside shoulder width of 3.0 m (10 ft) and an inside shoulder width of 1.2 m (4 ft). <u>The shoulders are constructed of PCC.</u>

I-35 was designed as a three-layer system comprised of a 254-mm (10-in) thick CRCP and a <u>76-mm (3-in) thick hot mix asphaltic concrete</u> over an already existing granular subbase. The CRC pavement was designed with a longitudinal steel percentage of 0.5 and a transverse steel percentage of 0.08. The longitudinal reinforcing steel bars are #5 bars spaced at 175 mm (6.875 in) with the transverse steel bars being #5 spaced at 1 118 mm (44 in). The bars were placed on chairs as a single layer during the construction of the pavement. <u>The bars are epoxy coated</u>. The portland cement shoulders were jointed, keyed, and tied to the CRC mainline pavement.

The concrete pavement thickness was determined to be 262 mm (10.3 in) based on cores taken as part of this study. The concrete is considered well graded and its consolidation was poor. The aggregate type used was a crushed stone with a maximum size of 38 mm (1½ in).

The asphaltic concrete base was considered to be well graded and had an average thickness of 76 mm (3 in). The subbase layer was an unbound granular subbase that was built in 1969 but could not be distinguished from the subgrade. The subgrade was classified as A-4. The Atterberg Limits of the subgrade were a liquid limit of 20 and a plastic limit of 0.

Design/Construction

The initial thickness design records were not available. The design was done by the Oklahoma Department of Transportation. The design life chosen was 20 years. The terminal joint design chosen was a wide flange beam system. The bridge approach pavement type is concrete.

Construction began in June 1988 and finished in May 1989. The specified concrete properties were slump of 25 to 76 mm (1 to 3 in), air content of 4 to 6 percent, and strength of 20.68 MPa (3,000 lbf/in^2) at 28 days.

Performance

The performance of I-35 has not been reported at this time.

Historical Data (M&R)

No maintenance and rehabilitation work has been reported for US-69.

Field Investigation Data

Visual Condition Survey

- Total number of cracks = 224
  - 0 were at high severity
  - 210 were at medium severity
  - 14 were at low severity
- Average crack spacing = 1.44 m (4.72 ft)
- Standard deviation for crack spacing = 0.70 m (2.29 ft)
- Coefficient of variation of the crack spacing = 63 percent
- Figure 82 shows average crack spacing distribution based on the closest five cracks
- Figure 83 shows actual crack spacing
- Extent of Y cracks = 12 percent
- Cluster cracking was apparent at stations 0+60 and 3+50
- No other distress types were present
- Pavement shoulders were in good condition.

Drainage Survey

- Ditches line the inside edge of the pavement. The outside edge side had a slope.
- Ditches were lined with vegetation. There was no sign of ponded water or wet ground.

Windshield Survey

- 6.44 km (4 mi) of pavement surveyed was generally in good condition exhibiting little distress.

Terminal Joint Survey

- Terminal joints used = Wide flange (two surveyed)
- Joints exhibited no distress other than minor seal damage.

Deflection Testing

- Figure 84 shows normalized/basin (per 4.45 kN (1,000 lb) load) deflections measured along the length of the section (for all seven sensors using the 40.03 kN (9,000 lb) load)
- Load transfer efficiency at cracks at mid-slab
  - 91 to 96 percent in the morning
  - 92 to 97 percent in the afternoon
- Load transfer efficiency at cracks along edge
  - 88 to 96 percent in the morning
  - 91 to 95 percent in the afternoon
- Figure 85 shows the distribution of slab temperature with depth and time of day.
- Figure 86 shows the average deflections and ranges of deflections for each sensor for center and edge loading, for morning and afternoon testing (using the 40.03 kN (9,000 lb) load within the selected 30.5-m (100-ft) subsection).
- Figure 87 shows the backcalculated radius of relative stiffness, ℓ, values for basin testing for each of the three nominal load levels of 40.03, 53.38, and 71.17 kN (9,000, 12,000, and 16,000 lb).
- ℓ value based on 40.03 kN (9,000 lb) load basin testing
  - Average = 1 041 mm (41 in)
  - Standard Deviation = 152 mm (6 in)
- ℓ value at crack locations at mid-slab (morning/afternoon)
  - Average = 940/940 mm (37/37 in)
  - Standard Deviation = 152/127 mm (6/5 in)
- ℓ value at crack locations at edge (morning/afternoon)
  - Average = 813/889 mm (32/35 in)
  - Standard Deviation = 127/127 mm (5/5 in)
- Estimated ℓ value using actual slab thickness and concrete modulus of elasticity and assumed modulus of subgrade reaction
  - Value = 914 mm (36 in)
- Figure 88 shows the variation of ℓ in relation to crack spacing (average of the closest five cracks)
- Average deflection under 40.03 kN (9,000 lb) load (0.000001 mm (0.001 in))
  - Basin test - 0.07 (2.9)

- Mid-slab crack location (morning/afternoon) - 0.08/0.08 (3.3/3.3)
- Edge crack location (morning/afternoon) - 0.132/0.117 (5.2/4.6)

Crack Width Measurements

- Number of cracks monitored within 30.5-m (100-ft) subsection = 19
- Morning testing
  - Average temperature 102 mm (4 in) below surface = 16.1°C (61°F)
  - Average crack width = 0.54 mm (0.021 in)
- Afternoon testing
  - Average temperature 102 mm (4 in) below surface = 25.6°C (78°F)
  - Average crack width = 0.44 mm (0.017 in)
- Slab length change, mm/mm/°C (in/in/°F) = 6.7 millionth (3.7 millionth)

Reinforcing Steel Depths and Spacing

- Steel depth measurement offsets = 610, 1 829, 2 438 mm (24, 72, 96 in)
- Range of depth of cover = 91 to 140 mm (3.6 to 5.5 in)
- Average depth of cover = 122 (4.8 in)
- Standard deviation for depth of cover = 7.6 mm (0.3 in)
- Steel spacing was about 221 mm (8.7 in)

Corrosion Potential Testing (copper-copper sulfate half cell potentiometer)

- No testing was performed as the steel was epoxy coated.

Concrete Core Examination for Corrosion

- No steel corrosion was detected

Profile Testing

- Average IRI over 305-m (1,000-ft) section = 1 169 mm/km (74 in/mi)

Laboratory Testing

The following data were obtained from the testing of the concrete, base/subbase, and subgrade materials:

Concrete

Modulus of Elasticity	-	35 850 MPa (5,200,000 lbf/in^2)
Average Split Tensile Strength	-	3.45 MPa (500 lbf/in^2)
Split Tensile Strength Range	-	3.17 to 3.86 MPa (460 to 560 lbf/in^2)
Coefficient of Thermal Expansion	-	6.37 mm/mm/°C (3.54 in/in/°F)

Base

AASHTO Classification - ATB

Subbase

AASHTO Classification - Granular

Subgrade

Liquid Limit - 20 percent
Plastic Limit - 0 percent
Percent Passing #200 Sieve - 35.8 percent
AASHTO Classification - A-4

Chloride content testing was conducted on pulverized concrete retrieved at various depths and locations. Total chloride content and water soluble chloride content values are listed below for OK-3.

Location	Depth, mm (in)	Total Chloride Content (% wt.)	Water Soluble Chlorides (% wt.)
No steel below sample	0-51 (0-2)	0.012	0.010
	51-102 (2-4)	0.000	0.000
Steel directly below sample	0-51 (0-2)	0.018	0.000
	51-102 (2-4)	0.010	0.000

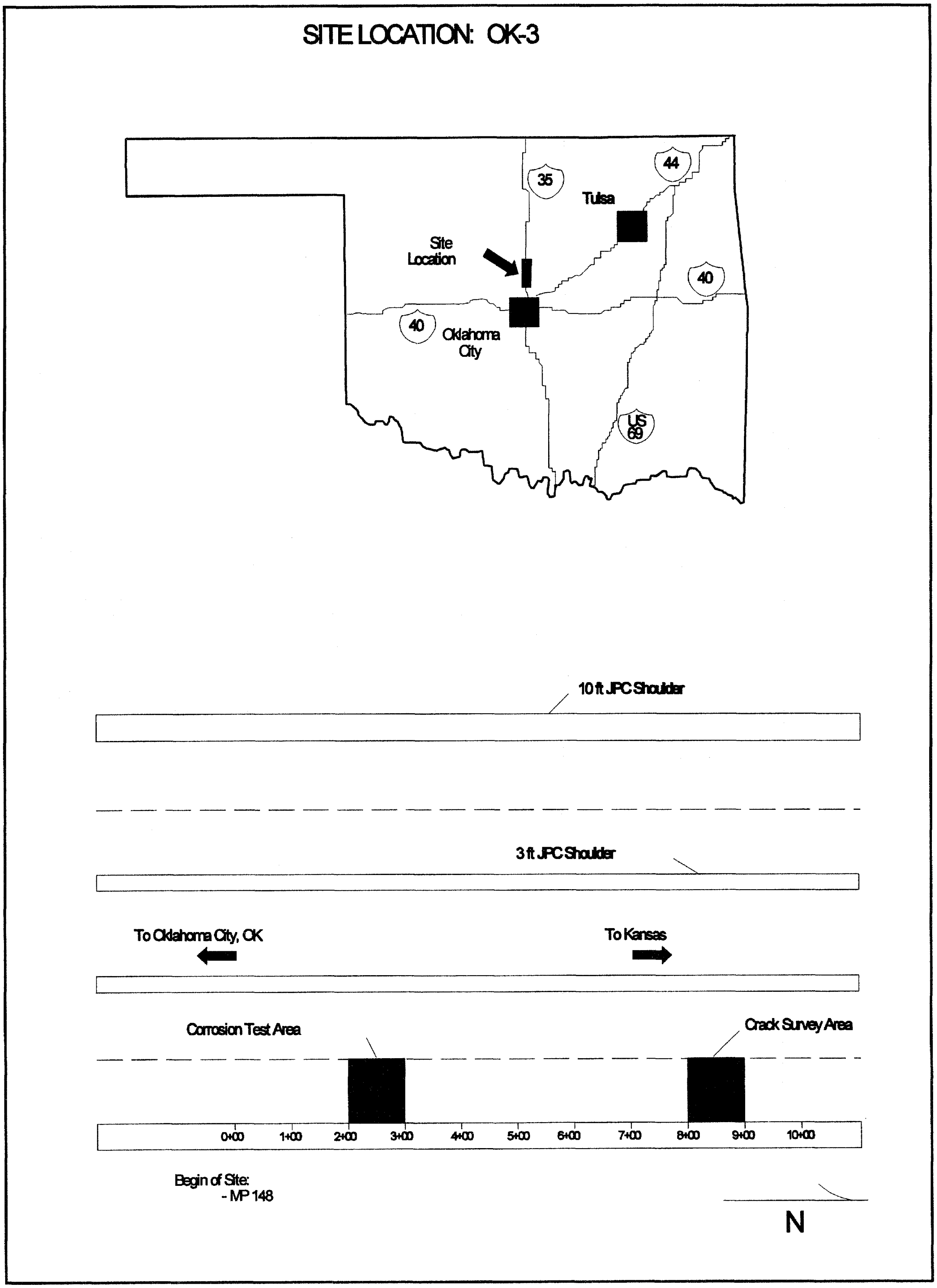
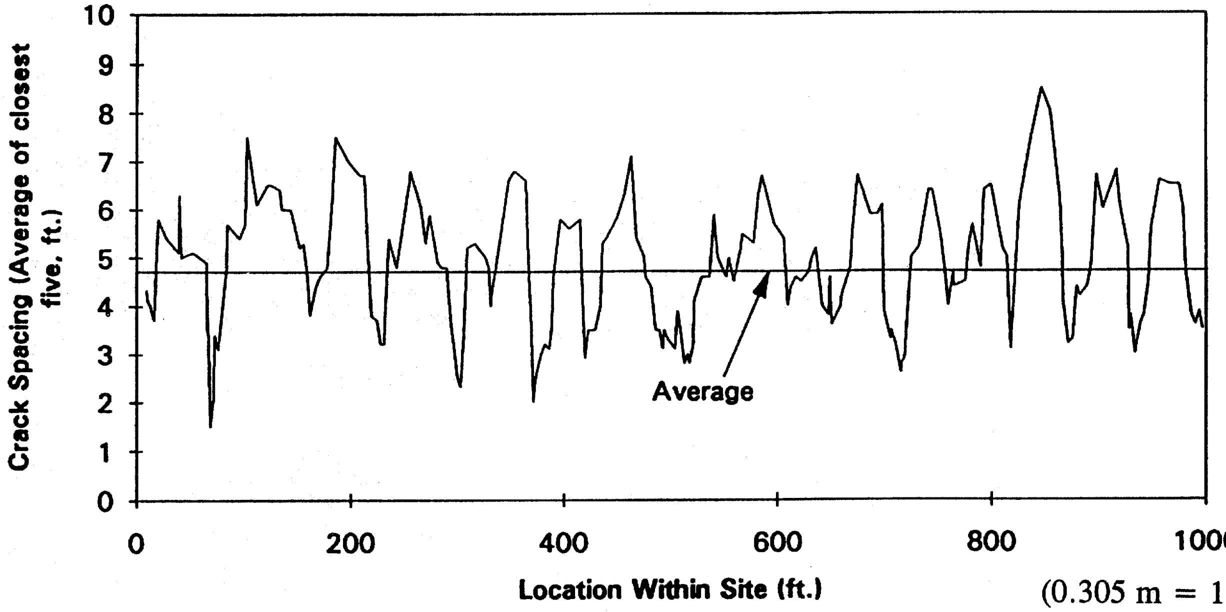


Figure 81. Site details for OK-3.



(0.305 m = 1 ft)

Figure 82. Crack spacing pattern at site OK-3.

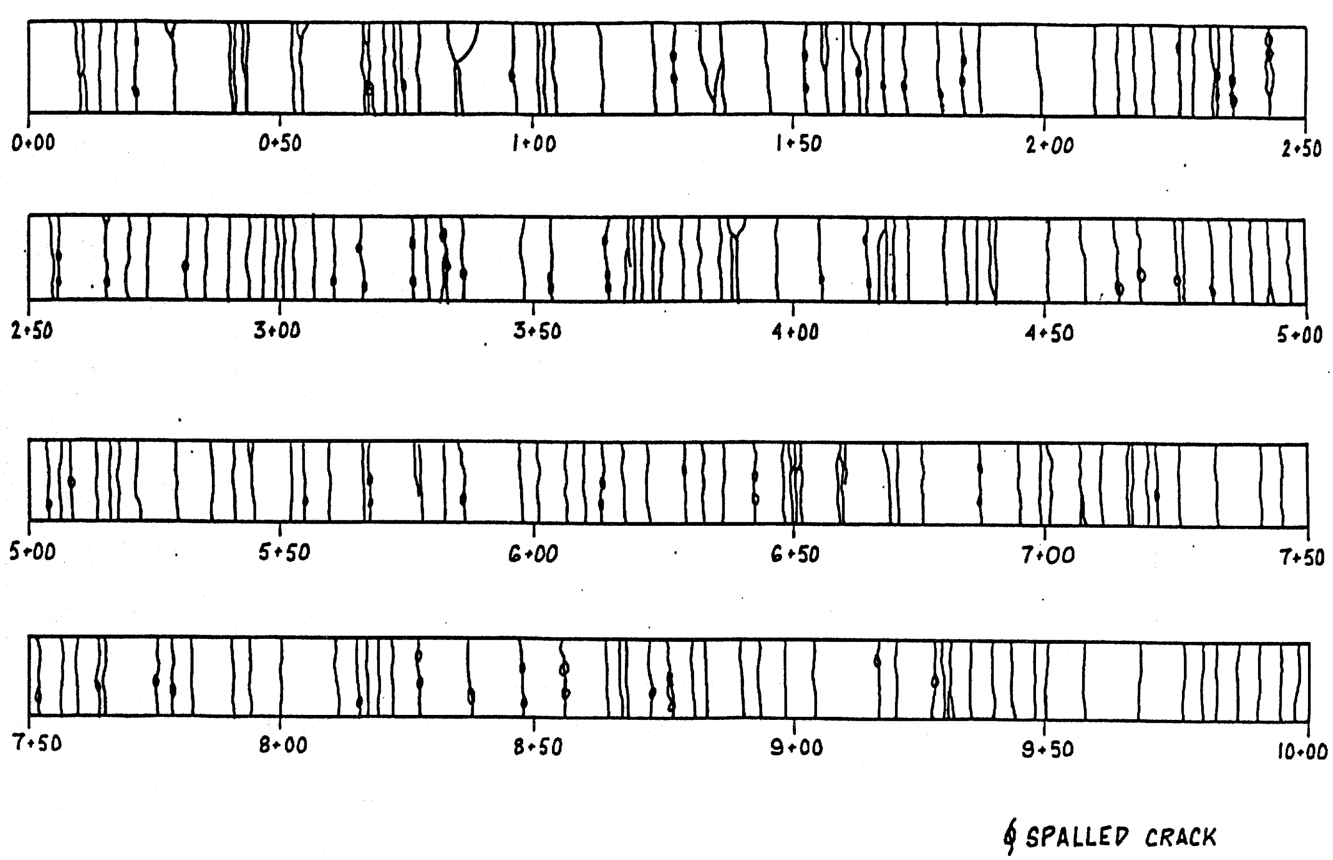
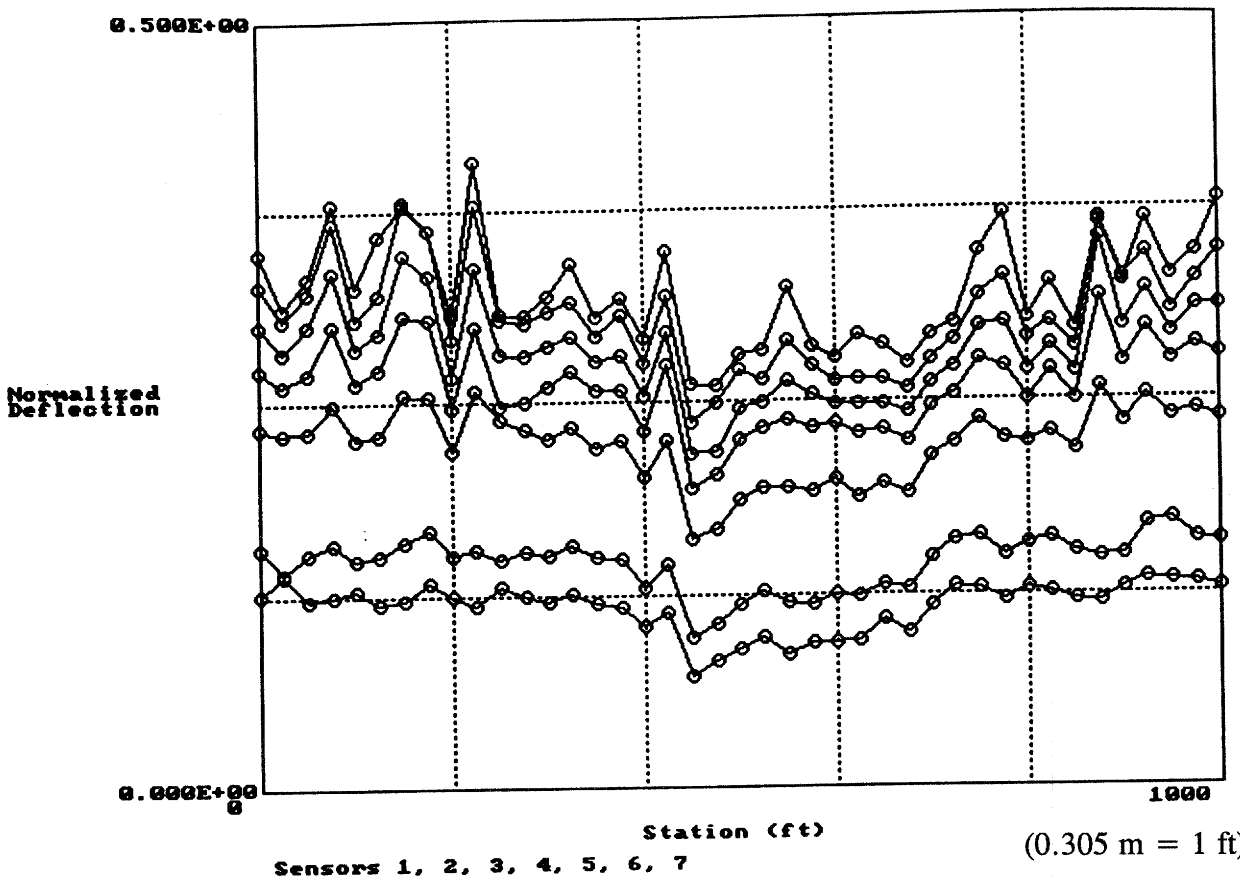
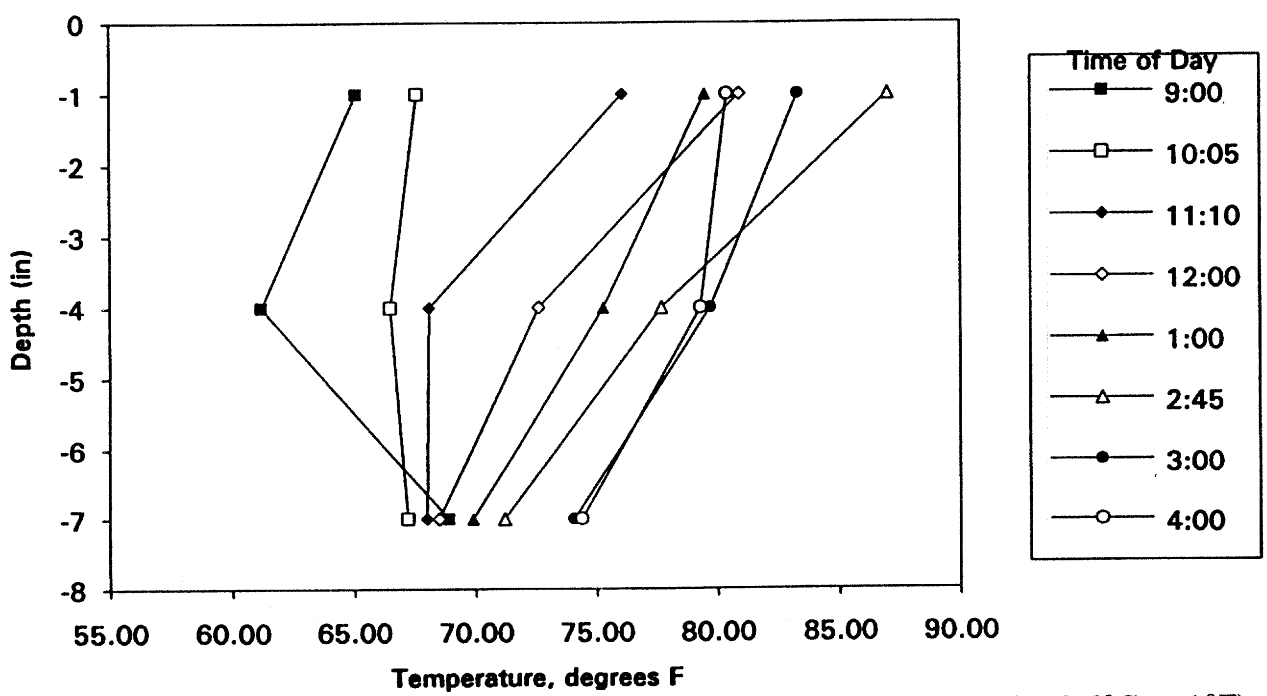


Figure 83. Crack pattern at site OK-3.



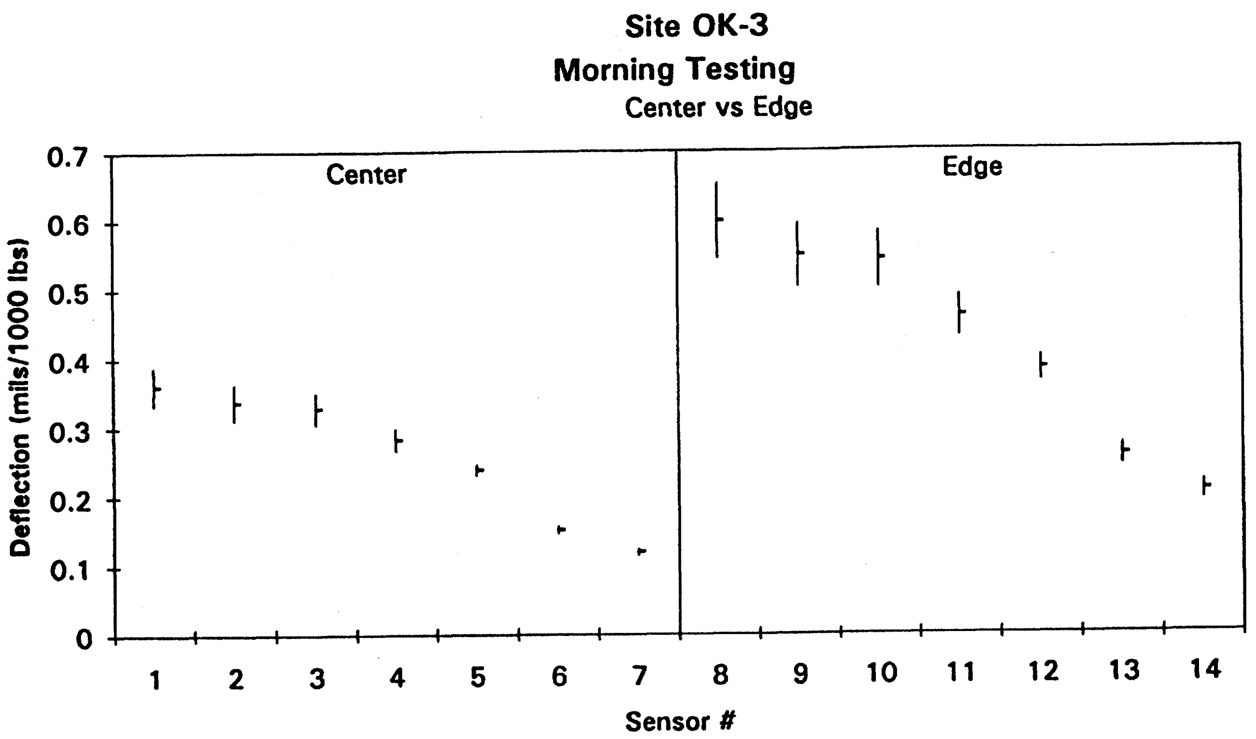
(0.305 m = 1 ft)

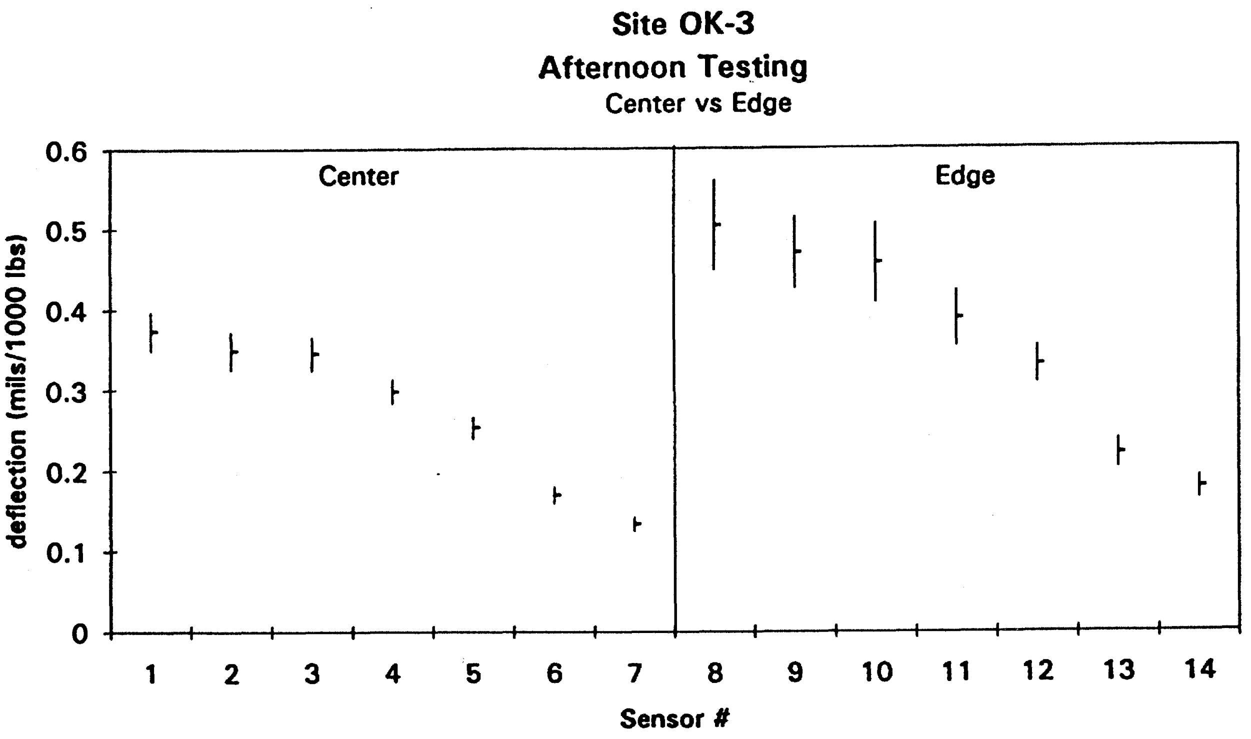
Figure 84. Deflections in inches along the section length at site OK-3.
[normalized per 4.45-kN [1,000-lb] load]



(25.4 mm = 1 in; 0.6°C = 1°F)

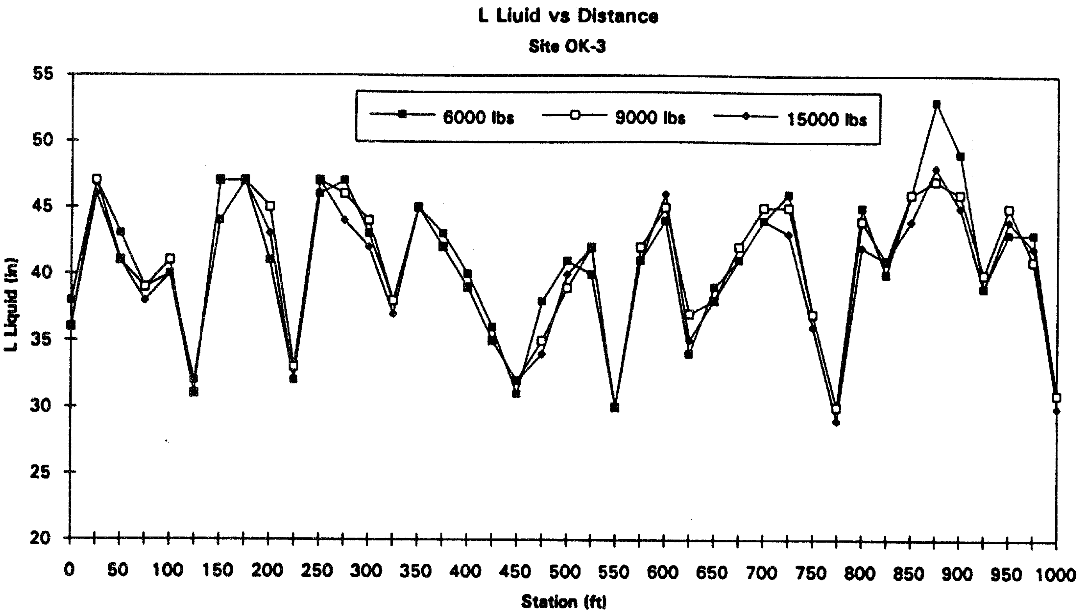
Figure 85. Temperature profile for site OK-3.





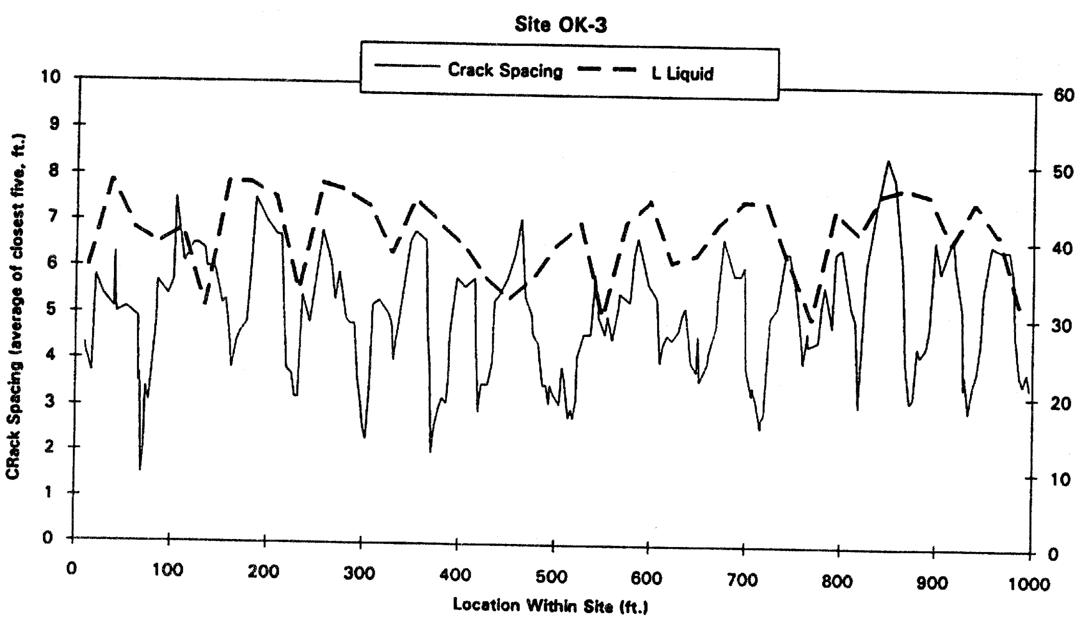
(0.0254 mm = 1 mil; 4.45 kN = 1,000 lb)

Figure 86. Center versus edge deflection for each of the seven sensors at site OK-3.
[40.03 kN (9,000 lb) load]



(25.4 mm = 1 in, 0.305 m = 1 ft)

Figure 87. Backcalculated values of radius of relative stiffness, ℓ (using program ILLI-BACK) for site OK-3.



(0.305 m = 1 ft)

Figure 88. Comparison of radius of relative stiffness values with average crack spacing (nearest five cracks) at site OK-3.

Section OK-4 - US Highway 69

Introduction

Section OK-4 was constructed during 1984 and 1985 and was opened to traffic in 1985. OK-4 is a section of US-69 that runs north to south from the Oklahoma-Texas border to McAlester, Oklahoma, as shown in figure 89. The length of the highway that was considered runs from station 1226+00, the Armstrong Interchange to station 1667+00, SH-22 Interchange. A 305-m (1,000-ft) south bound section was selected for detailed inspection 86 m (283 ft) south of SH-22 bridge, as shown in figure 89. The section of US-69 that was chosen is in the south eastern region of Oklahoma.

The climate in this region is a continental climate. The continental climate produces considerable variation in seasonal and annual precipitation. The highest monthly average temperature is 29.7°C (85.5°F) during July, and the lowest monthly average is 4.6°C (40.3°F) during January. US-69 lies within the intermediate freeze-thaw environmental zone.

Traffic

Data on the number of ESALs in its north bound lanes were not available. The directional split for 1983 was reported to be 60 percent. The lane distribution for 1983 was reported to be 80 percent. The ADT for 1983 was estimated to be 6,700 with a truck percentage of 35. The ADT estimated for 2003 is 14,000. US-69 was designed for a 20-year design life.

Structure

US-69 is a four-lane divided highway. The pavement lane width is 3.7 m (12 ft) with an outside shoulder width of 3.7 m (12 ft) and an inside shoulder width of 1.2 m (4 ft). The shoulders are constructed of PCC.

US-69 was designed as a four-layer system comprised of a 229-mm (9-in) CRCP, a 152-mm (6-in) soil asphalt base, and a 152-mm (6-in) select borrow subbase over a subgrade of clay. The CRC pavement was designed with a longitudinal steel percentage of 0.5 and a transverse steel percentage of 0.08. The longitudinal reinforcing steel bars are #5 bars spaced at 175 mm (6.875 in) with the transverse steel bars being #5 spaced at 1 118 mm (44 in). The bars were placed on chairs as a single layer during the construction of the pavement. The bars are not epoxy coated. The portland cement shoulders were jointed, keyed, and tied to the CRC mainline.

The concrete pavement thickness was determined to be 239 mm (9.4 in) based on cores taken as part of this study. The concrete is considered well graded, and its consolidation was average. The aggregate type used was a crushed stone with a maximum size of 38 mm (1½ in).

The soil asphalt base was uniformly graded and had an average thickness of 163 mm (6.4 in). The base had an AASHTO classification of A-2-4. The subbase layer was a sandy borrow material that was 135 mm (5.3 in) thick and uniformly graded. The AASHTO classification of the subbase is A-4. The subgrade is a clay with the AASHTO classification of A-6. The Atterberg Limits of the subgrade were a liquid limit of 24 and a plastic limit of 13.

Design/Construction

The initial thickness design was done according to the Oklahoma Subgrade Index Method by the Oklahoma Department of Transportation. The design life chosen was 20 years. The allowable crack width was 1.02 mm (0.040 in), the allowable steel stress was 6.89 MPa (40 ksi), and the minimum crack spacing was 0.9 m (3 ft). Drainage design was not considered. The terminal joint design chosen was a wide flange beam system. The bridge approach pavement type is concrete.

Construction began in May 1984 and finished in August 1985. The specified concrete properties were slump of 25 to 76 mm (1 to 3 in), air content of 4 to 6 percent, and the strength of 20.68 MPa (3,000 lbf/in^2) at 28 days.

Performance

The performance of I-40 has not been reported at this time.

Historical Data (M&R)

No maintenance and rehabilitation work has been reported for US-69.

Field Investigation Data

Visual Condition Survey

- Total number of cracks = 139
  - 7 were at high severity
  - 130 were at medium severity
  - 2 were at low severity
- Average crack spacing = 1.95 m (6.39 ft)
- Standard deviation for crack spacing = 0.98 m (3.20 ft)
- Coefficient of variation of the crack spacing = 50 percent
- Figure 90 shows average crack spacing distribution based on the closest five cracks
- Figure 91 shows actual crack spacing
- Extent of Y cracks = 3 percent
- Cluster cracking was apparent at station 6+00 only
- Other distress types included:
  - One punchout (low severity)
- Pavement shoulders were in fair condition.

Drainage Survey

- Ditches line both sides of the pavement.
- Ditches were lined with vegetation. There was no sign of ponded water or wet ground.

Windshield Survey

- No windshield survey was performed because of time constraints.

Terminal Joint Survey

- Terminal joints used = Wide flange (two surveyed)
- Joints exhibited no distress

Deflection Testing

- Figure 92 shows normalized/basin (per 4.45 kN (1,000 lb) load) deflections measured along the length of the section (for all seven sensors using the 40.03 kN (9,000 lb) load)
- Load transfer efficiency at cracks at mid-slab
  - 62 to 95 percent in the morning
  - 22 to 97 percent in the afternoon (very variable)
- Load transfer efficiency at cracks along edge
  - 66 to 98 percent in the morning
  - 21 to 95 percent in the afternoon (very variable)
- Figure 93 shows the distribution of slab temperature with depth and time of day.
- Figure 94 shows the average deflections and ranges of deflections for each sensor for center and edge loading, for morning and afternoon testing (using the 40.03 kN (9,000 lb) load within the selected 30.5-m (100-ft) subsection).
- Figure 95 shows the backcalculated radius of relative stiffness, ℓ, values for basin testing for each of the three nominal load levels of 40.03, 53.38, and 71.17 kN (9,000, 12,000, and 16,000 lb).
- ℓ value based on 40.03 kN (9,000 lb) load basin testing
  - Average = 838 mm (33 in)
  - Standard Deviation = 203 mm (8 in)
- ℓ value at crack locations at mid-slab (morning/afternoon)
  - Average = 610/737 (24/29 in)
  - Standard Deviation = 76/76 mm (3/3 in)
- ℓ value at crack locations at edge (morning/afternoon)
  - Average = 660/635 mm (26/25 in)
  - Standard Deviation = 127/76 mm (5/3 in)
- Estimated ℓ value using actual slab thickness and concrete modulus of elasticity and assumed modulus of subgrade reaction
  - Value = 940 mm (37 in)
- Figure 96 shows the variation of ℓ in relation to crack spacing (average of the closest five cracks)

- Average deflection under 40.03 kN (9,000 lb) load (0.000001 mm (0.001 in))
    - Basin test - 0.08 (3)
    - Mid-slab crack location (morning/afternoon) - 0.13/0.09 (5.0/3.7)
    - Edge crack location (morning/afternoon) - 0.24/0.20 (9.3/7.7)

Crack Width Measurements
- Number of cracks monitored within 30.5-m (100-ft) subsection = 17
- Morning testing
    - Average temperature 102 mm (4 in) below surface = 4.4°C (40°F)
    - Average crack width = 0.76 mm (0.030 in)
- Afternoon testing
    - Average temperature 102 mm (4 in) below surface = 12.2°C (54°F)
    - Average crack width = 0.70 mm (0.028 in)
- Slab length change, mm/mm/°C (in/in/°F) = 4.3 millionth (2.4 millionth)

Reinforcing Steel Depths and Spacing
- Steel depth measurement offsets = 610, 1 829, 2 438 mm (24, 72, 96 in)
- Range of depth of cover = 89 to 124 mm (3.5 to 4.9 in)
- Average depth of cover = 104 mm (4.1 in)
- Standard deviation for depth of cover = 7.6 mm (0.3 in)
- Steel spacing was about 170 mm (6.7 in)

Corrosion Potential Testing (copper-copper sulfate half cell potentiometer)
- Testing was not performed

Concrete Core Examination for Corrosion
- All four cores exhibited spot corrosion

Profile Testing
- No profile testing was conducted because of equipment malfunction

Laboratory Testing

The following data were obtained from the testing of the concrete, base/subbase, and subgrade materials:

Concrete

Modulus of Elasticity	-	44 130 MPa (6,400,000 lbf/in^2)
Average Split Tensile Strength	-	3.31 MPa (480 lbf/in^2)

Split Tensile Strength Range - 2.76 to 3.72 MPa (400 to 540 lbf/in^2)
Coefficient of Thermal Expansion - 8.75 mm/mm/°C (4.86 in/in/°F)

Base
AASHTO Classification - Soil-asphalt base

Subbase
AASHTO Classification - Borrow material (A-4)

Subgrade
Liquid Limit - 24 percent
Plastic Limit - 13 percent
Percent Passing #200 Sieve - 55.8 percent
AASHTO Classification - A-6

Chloride content testing was conducted on pulverized concrete retrieved at various depths and locations. Total chloride content and water soluble chloride content values are listed below for OK-4.

Location	**Depth, mm (in)**	**Total Chloride Content (% wt.)**	**Water Soluble Chlorides (% wt.)**
No steel below sample	0-51 (0-2)	0.018	0.010
	51-102 (2-4)	0.019	0.011
Steel directly below sample	0-51 (0-2)	0.021	0.010
	51-102 (2-4)	0.016	0.000

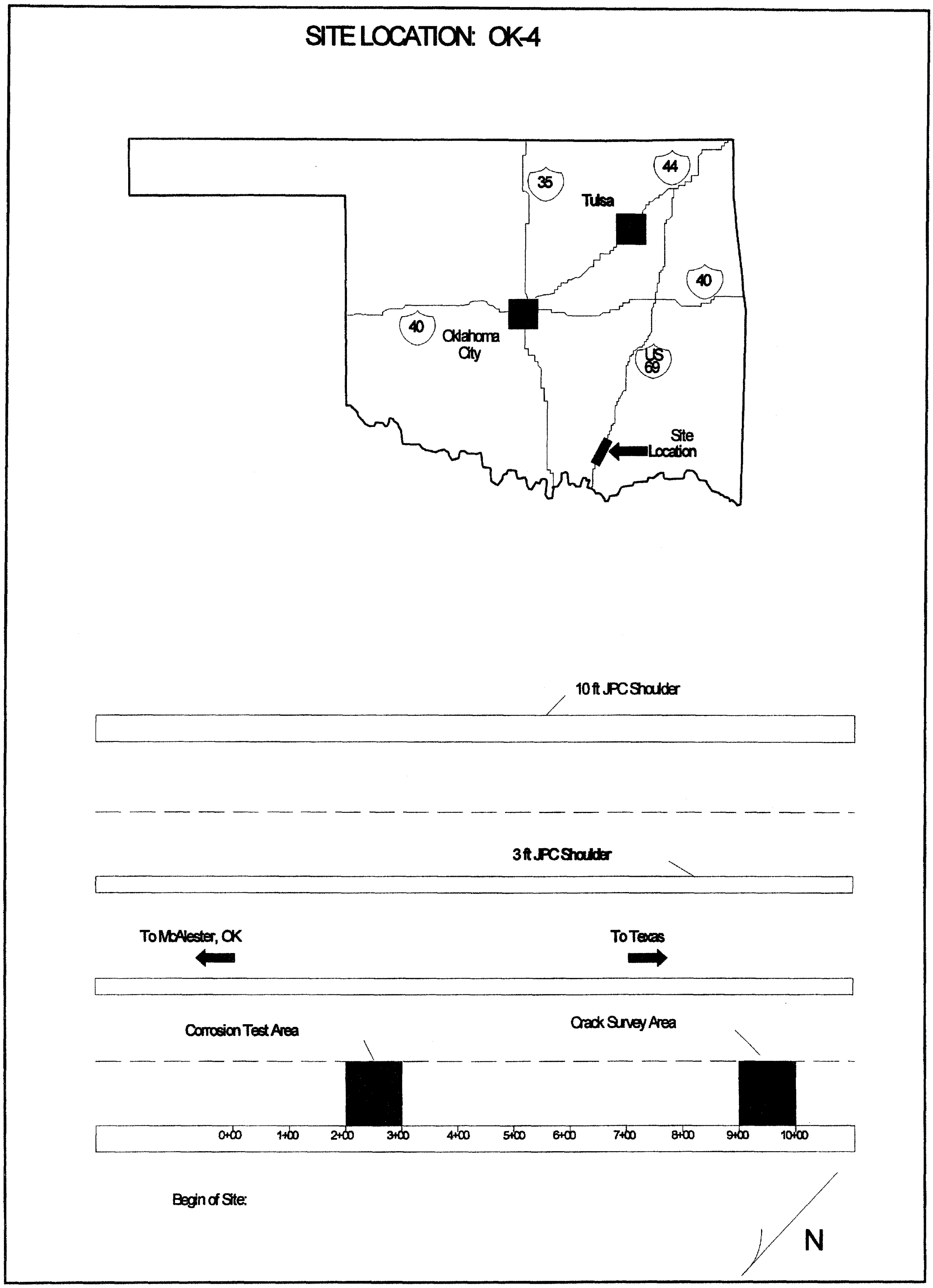


Figure 89. Site details for OK-4.

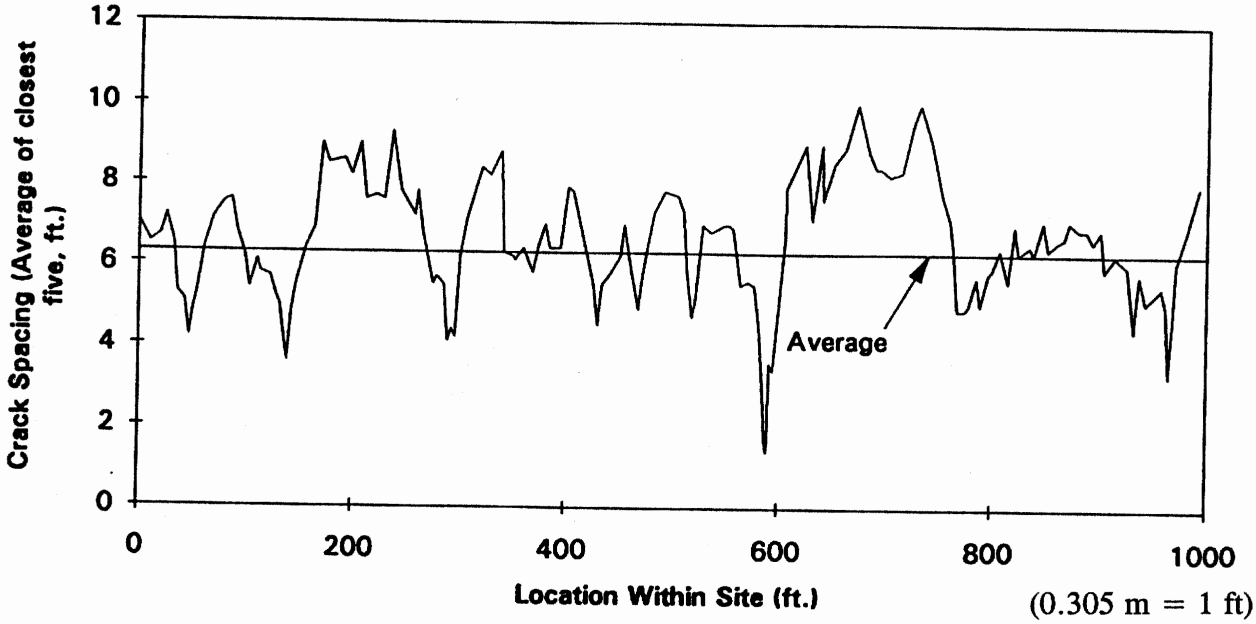


Figure 90. Crack spacing pattern at site OK-4.

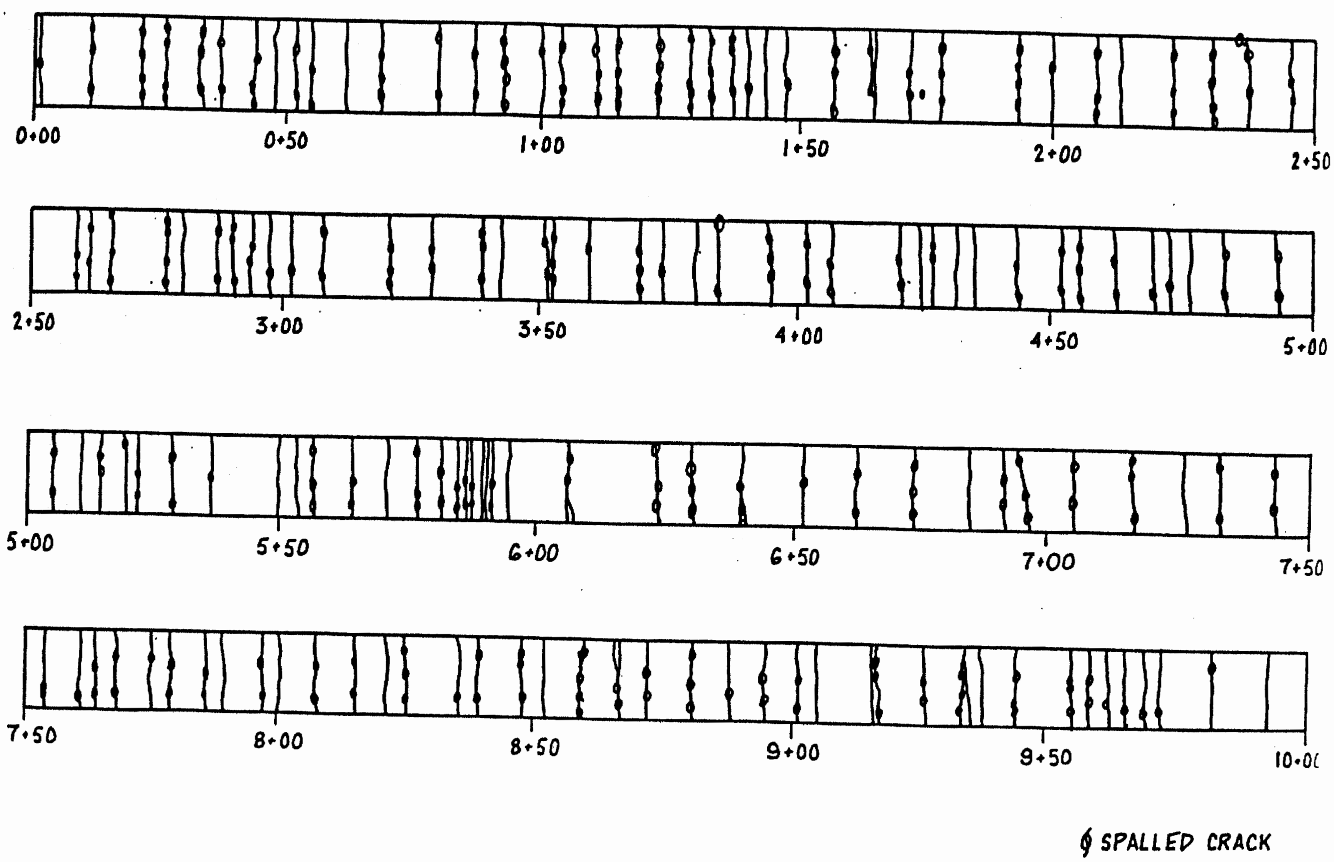
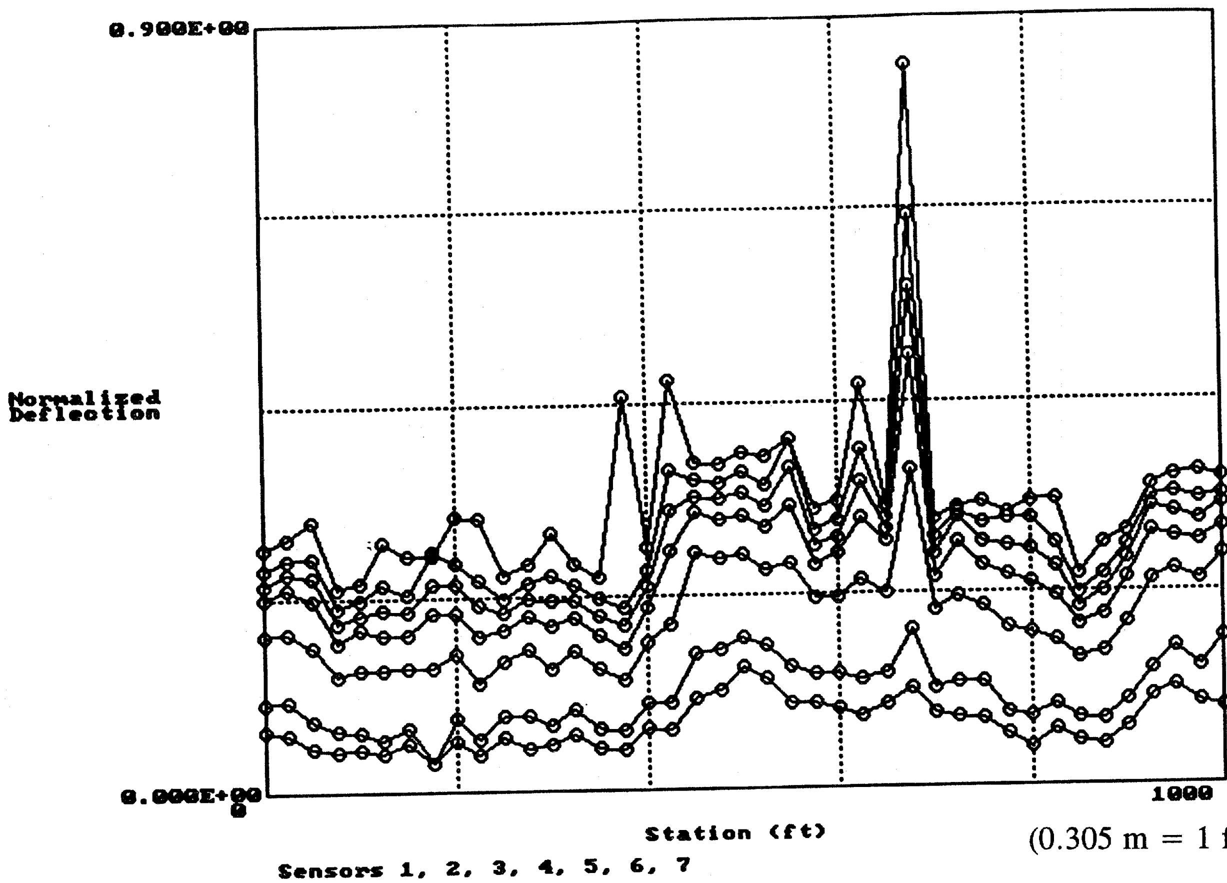
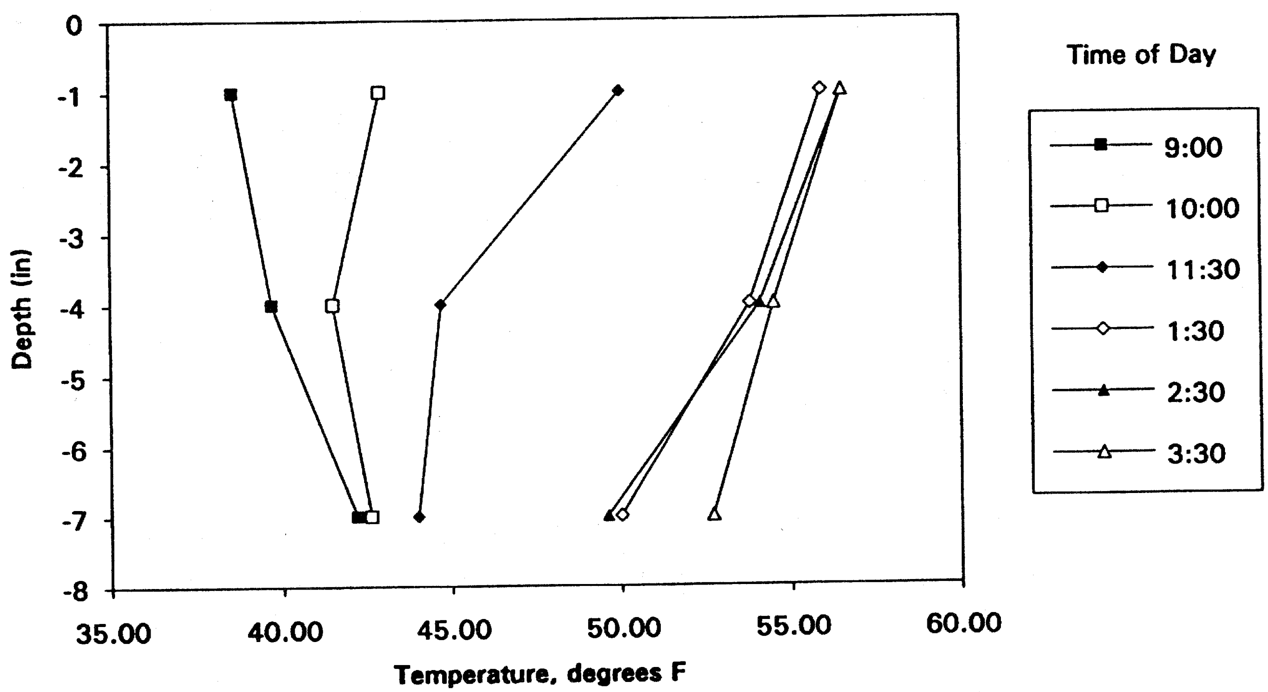


Figure 91. Crack pattern at site OK-4.



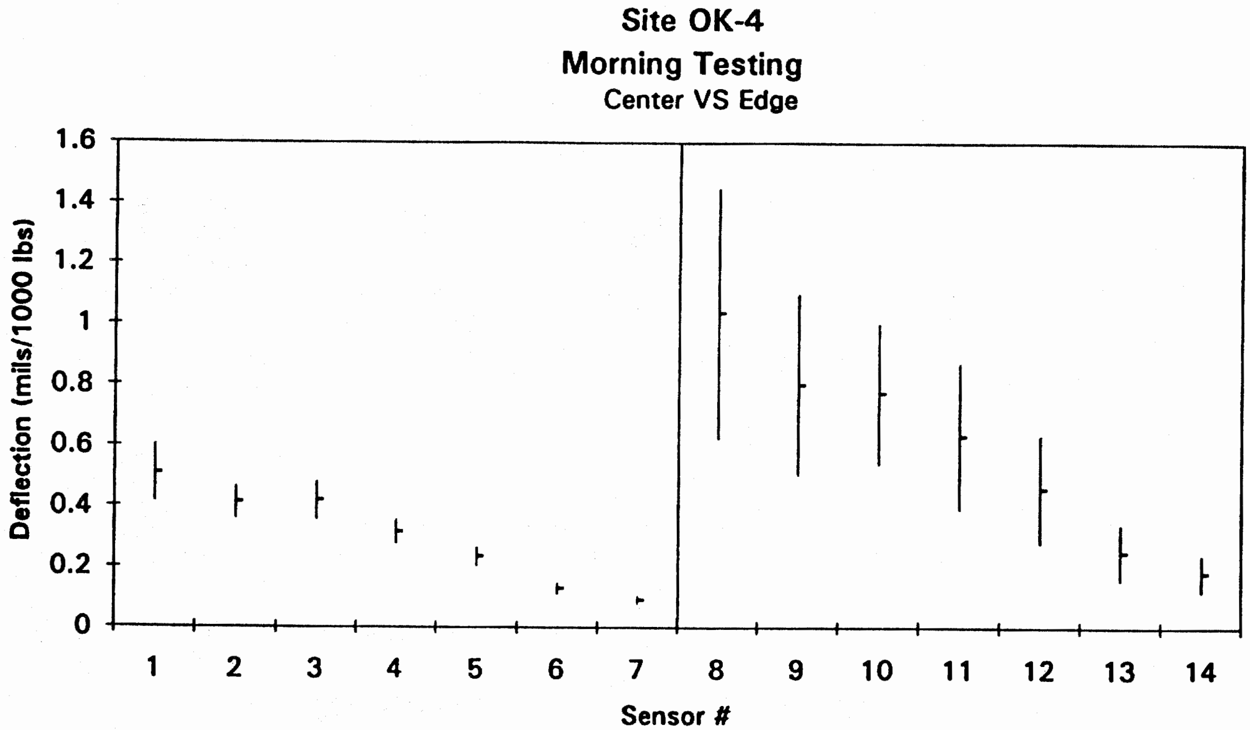
(0.305 m = 1 ft)

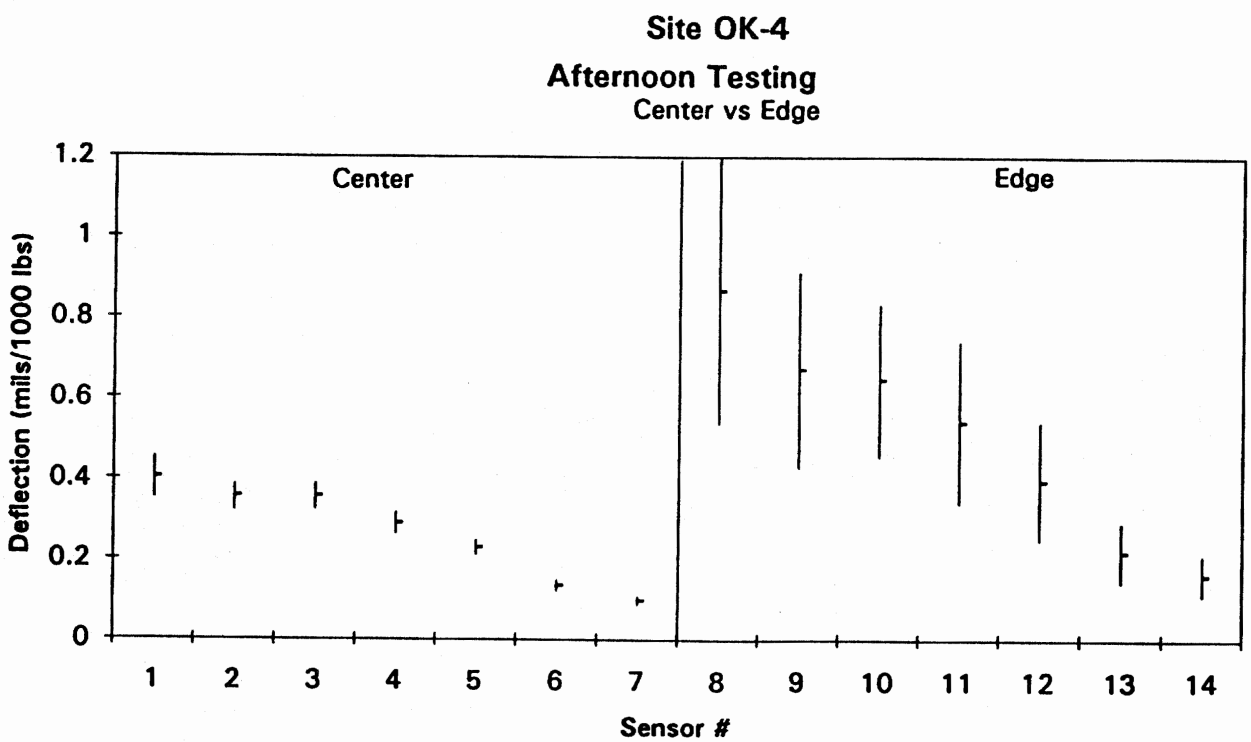
Figure 92. Deflections in inches along the section length at site OK-4.
[normalized per 4.45-kN [1,000-lb] load]



(25.4 mm = 1 in; 0.6°C = 1°F)

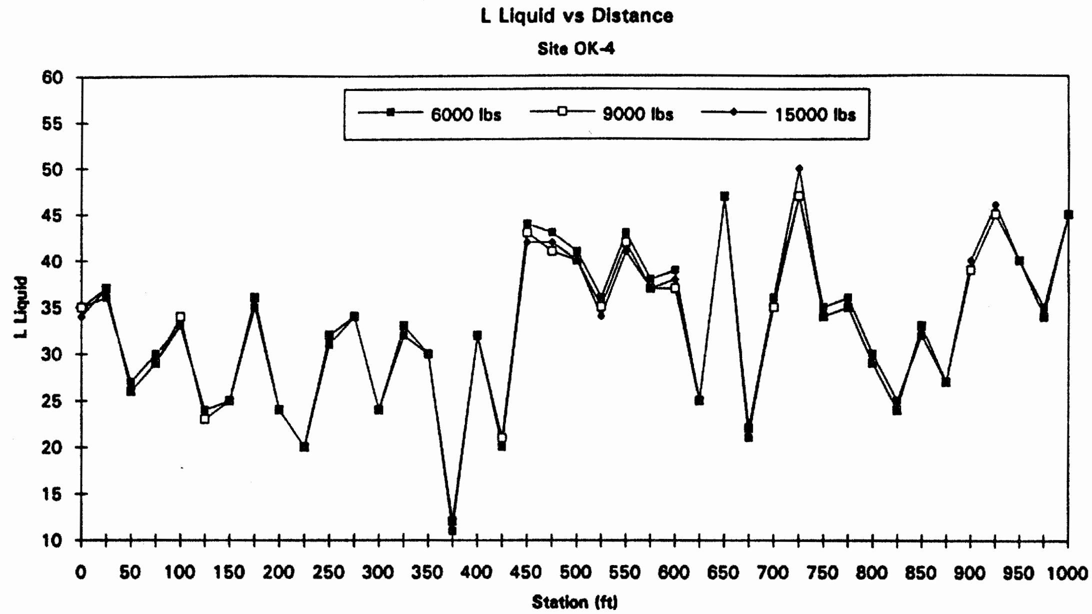
Figure 93. Temperature profile for site OK-4.





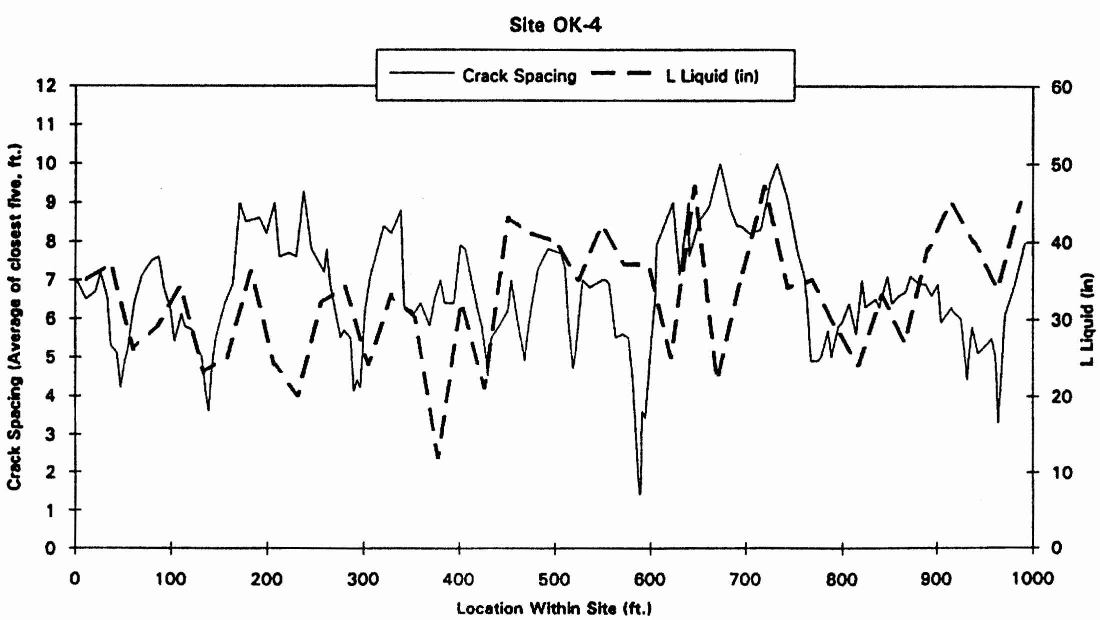
(0.0254 mm = 1 mil; 4.45 kN = 1,000 lb)

Figure 94. Center versus edge deflection for each of the seven sensors at site OK-4.
[40.03 kN (9,000 lb) load]



(25.4 mm = 1 in, 0.305 m = 1 ft)

Figure 95. Backcalculated values of radius of relative stiffness, ℓ (using program ILLI-BACK) for site OK-4.



(0.305 m = 1 ft)

Figure 96. Comparison of radius of relative stiffness values with average crack spacing (nearest five cracks) at site OK-4.

Section OK-5 - Interstate 40

Introduction

Section OK-5 was constructed during 1989 and 1990 and was opened to traffic in 1990. OK-5 is a section of I-40 that runs east to west from Arkansas to Oklahoma City, Oklahoma, as shown in figure 97. The length of the highway that was considered runs from milepost 298 to 303. A 305-m (1,000-ft) east bound section was selected for detailed inspection between mileposts 299 and 300, as shown in figure 97. The section of I-40 that was chosen is in the central eastern region of Oklahoma.

The climate in this region is a continental climate. The continental climate produces considerable variation in seasonal and annual precipitation. The highest monthly average temperature is 27.8°C (82.1°F) during July, and the lowest monthly average is 2.2°C (35.9°F) during January. I-40 lies within the intermediate freeze-thaw environmental zone.

Traffic

Data on the number of ESALs in its west bound lanes were not available. The directional split for 1989 was 55 percent and is estimated to be 55 percent in 2009. The ADT for 1989 was estimated to be 10,500 with a truck percentage of 31. The ADT estimate for 2009 is 22,000 with a truck percentage of 31. For both years, the lane distribution is 80 percent. I-40 was designed for a 20-year design life.

Structure

I-40 is a four-lane divided highway. The pavement lane width is 3.7 m (12 ft) with an outside shoulder width of 3.0 m (10 ft) and an inside shoulder width of 1.2 m (4 ft). <u>The shoulders are constructed of PCC.</u>

I-40 was designed as a three-layer system comprised of a 254-mm (10-in) thick CRCP and a <u>102-mm (4-in) thick permeable CTB</u> over a subgrade of select borrow material. The CRC pavement was designed with a longitudinal steel percentage of 0.61 and a transverse steel percentage of 0.08. The longitudinal reinforcing steel bars are #6 bars spaced at 184 mm (7.25 in) with the transverse steel bars being #5 spaced at 1 118 mm (44 in). The bars were placed on chairs as a single layer during the construction of the pavement. The bars are not epoxy coated. The shoulders were keyed and tied to the CRC mainline.

The average concrete pavement thickness was determined to be 257 mm (10.1 in) based on cores taken as part of this study. The concrete is considered well graded and its consolidation was average. The aggregate type used was a crushed stone with a maximum size of 38 mm (1½ in).

The permeable concrete base was considered to be uniformly graded and 83 mm (3.25 in) thick. The subgrade is a clayey sand with the AASHTO classification of A-2-6. The Atterberg Limits of the subgrade were a liquid limit of 29 and a plastic limit of 18.

Design/Construction

The initial thickness design was done according to the Oklahoma Subgrade Index Method by the Oklahoma Department of Transportation. The design life chosen was 20 years. Drainage was provided for by the permeable base and vertical edge drains. The terminal joint design chosen was an open joint on a sleeper slab. The bridge approach pavement type is concrete.

Construction began in May 1989 and ended in November 1990. The specified concrete properties were slump of 25 to 76 mm (1 to 3 in), air content of 4 to 6 percent, and strength of 20.68 MPa (3,000 lbf/in^2) at 28 days.

Performance

The performance of I-40 has not been reported at this time.

Historical Data (M&R)

No maintenance and rehabilitation work has been reported for I-40.

Field Investigation Data

Visual Condition Survey

- Total number of cracks = 156
  - 0 were at high severity
  - 48 were at medium severity
  - 108 were at low severity
- Average crack spacing = 1.88 m (6.16 ft)
- Standard deviation for crack spacing = 1.02 m (3.36 ft)
- Coefficient of variation of the crack spacing = 55 percent
- Figure 98 shows average crack spacing distribution based on the closest five cracks
- Figure 99 shows actual crack spacing
- Extent of Y cracks = 2 percent
- Cluster cracking was not apparent along this section
- No other distress types were present
- Pavement shoulders were in excellent condition.

Drainage Survey

- Ditches line both sides of the pavement.

- Ditches were lined with vegetation. There was no sign of ponded water but the ground was wet because of rainfall a few days before testing.

Windshield Survey

- 6.44 km (4 mi) of pavement surveyed was generally in excellent condition exhibiting two PCC patches.

Terminal Joint Survey

- Terminal joints used = Lug anchor (bridge end) and wide flange (construction joint)
- Joints exhibited no distress

Deflection Testing

- Figure 100 shows normalized/basin (per 4.45 kN (1,000 lb) load) deflections measured along the length of the section (for all seven sensors using the 40.03 kN (9,000 lb) load)
- Load transfer efficiency at cracks at mid-slab
  - 87 to 95 percent in the morning
  - 82 to 93 percent in the afternoon
- Load transfer efficiency at cracks along edge
  - 89 to 94 percent in the morning
  - 88 to 93 percent in the afternoon
- Figure 101 shows the distribution of slab temperature with depth and time of day.
- Figure 102 shows the average deflections and ranges of deflections for each sensor for center and edge loading, for morning and afternoon testing (using the 40.03 kN (9,000 lb) load within the selected 30.5-m (100-ft) subsection).
- Figure 103 shows the backcalculated radius of relative stiffness, ℓ, values for basin testing for each of the three nominal load levels of 40.03, 53.38, and 71.17 kN (9,000, 12,000, and 16,000 lb).
- ℓ value based on 40.03 kN (9,000 lb) load basin testing
  - Average = 864 mm (34 in)
  - Standard Deviation = 203 mm (8 in)
- ℓ value at crack locations at mid-slab (morning/afternoon)
  - Average = 610/660 mm (24/26 in)
  - Standard Deviation = 25/25 mm (1/1 in)
- ℓ value at crack locations at edge (morning/afternoon)
  - Average = 686/686 mm (27/27 in)
  - Standard Deviation = 51/51 mm (2/2 in)
- Estimated ℓ value using actual slab thickness and concrete modulus of elasticity and assumed modulus of subgrade reaction
  - Value = 838 mm (33 in)
- Figure 104 shows the variation of ℓ in relation to crack spacing (average of the closest five cracks)

- Average deflection under 40.03 kN (9,000 lb) load (0.000001 mm (0.001 in))
    - Basin test - 0.08 (3.0)
    - Mid-slab crack location (morning/afternoon) - 0.10/0.09 (4.0/3.5)
    - Edge crack location (morning/afternoon) - 0.13/0.11 (5.0/4.3)

Crack Width Measurements

- Number of cracks monitored within the 30.5-m (100-ft) subsection = 15
- Morning testing
    - Average temperature 102 mm (4 in) below surface = 5.6°C (42°F)
    - Average crack width = 0.45 mm (0.018 in)
- Afternoon testing
    - Average temperature 102 mm (4 in) below surface = 8.3°C (47°F)
    - Average crack width = 0.39 mm (0.015 in)
- Slab length change, mm/mm/°C (in/in/°F) = 10.6 millionth (5.9 millionth)

Reinforcing Steel Depths and Spacing

- Steel depth measurement offsets = 610, 1 829, 2 438 mm (24, 72, 96 in)
- Range of depth of cover = 86 to 163 mm (3.4 to 6.4 in)
- Average depth of cover = 117 mm (4.6 in)
- Standard deviation for depth of cover = 18 mm (0.7 in)
- Steel spacing was about 224 mm (8.8 in)

Corrosion Potential Testing (copper-copper sulfate half cell potentiometer)

- Range of measurements = -0.07 to -0.12 volts
- Average measurement = -0.09 volts
- Standard deviation for the measurement = -0.01 volts
- Potential for corrosion was not indicated

Concrete Core Examination for Corrosion

- No steel corrosion was detected in all four cores

Profile Testing

- Average IRI over the 305-m (1,000-ft) section = 790 mm/km (50 in/mi)

Laboratory Testing

The following data were obtained from the testing of the concrete, base/subbase, and subgrade materials:

Concrete

Modulus of Elasticity	-	22 750 MPa (3,300,000 lbf/in^2)
Average Split Tensile Strength	-	3.31 MPa (480 lbf/in^2)
Split Tensile Strength Range	-	3.24 to 3.59 MPa (470 to 520 lbf/in^2)
Coefficient of Thermal Expansion	-	13.34 mm/mm/°C (7.41 in/in/°F)

Base

AASHTO Classification	-	Permeable cement treated base

Subbase

AASHTO Classification	-	N/A

Subgrade

Liquid Limit	-	29 percent
Plastic Limit	-	18 percent
Percent Passing #200 Sieve	-	34.7 percent
AASHTO Classification	-	A-2-6

Chloride content testing was conducted on pulverized concrete retrieved at various depths and locations. Total chloride content and water soluble chloride content values are listed below for OK-5.

Location	Depth, mm (in)	Total Chloride Content (% wt.)	Water Soluble Chlorides (% wt.)
No steel below sample	0-51 (0-2)	0.010	0.000
	51-102 (2-4)	0.000	0.000
Steel directly below sample	0-51 (0-2)	0.010	0.000
	51-102 (2-4)	0.011	0.000

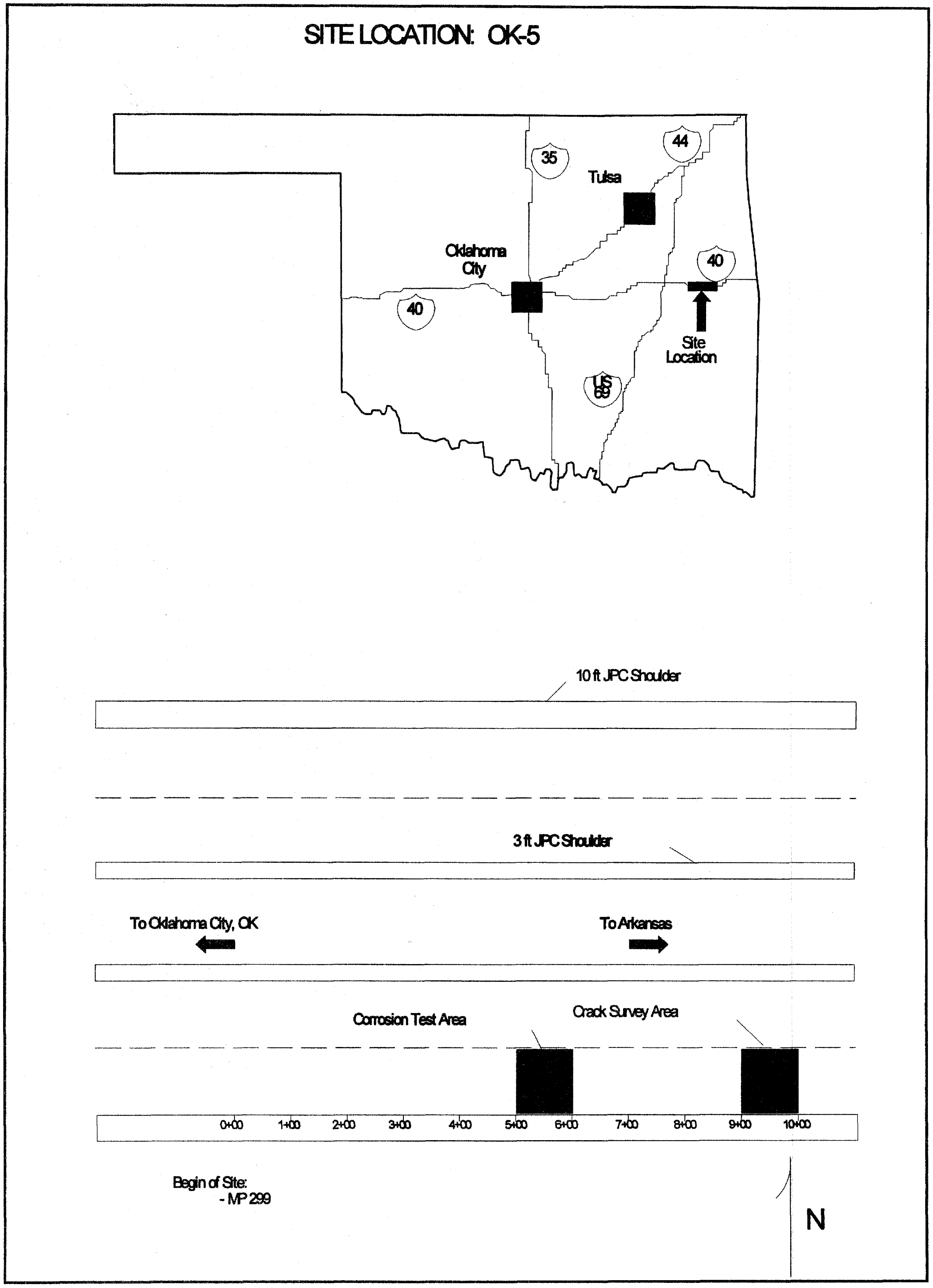
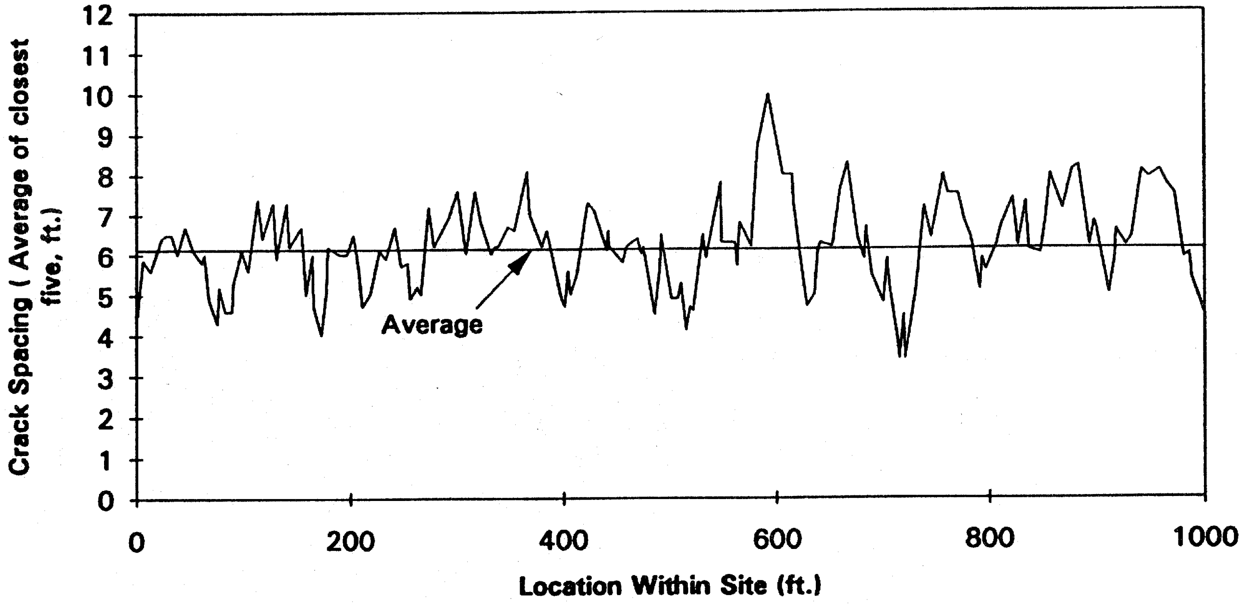


Figure 97. Site details for OK-5.



(0.305 m = 1 ft)

Figure 98. Crack spacing pattern at site OK-5.

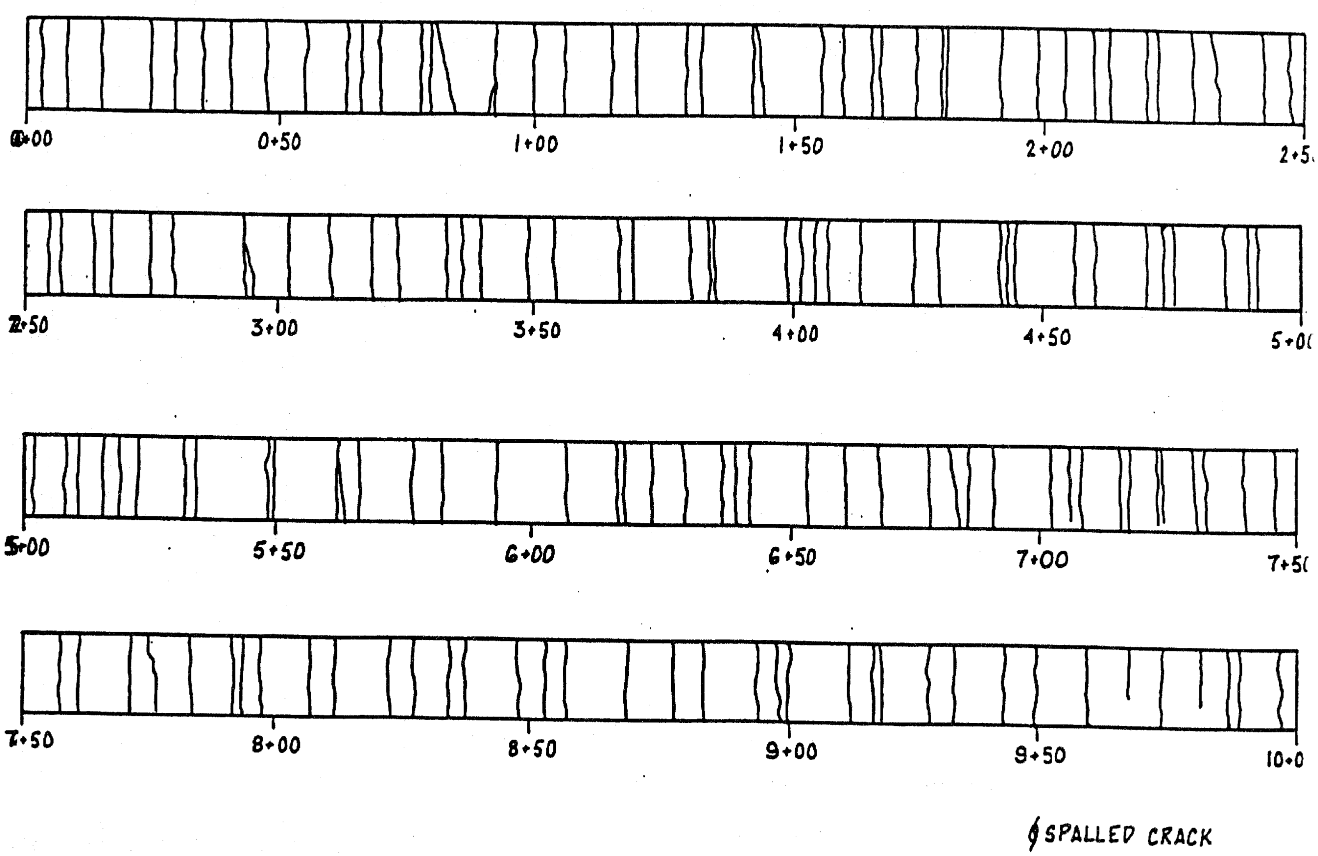
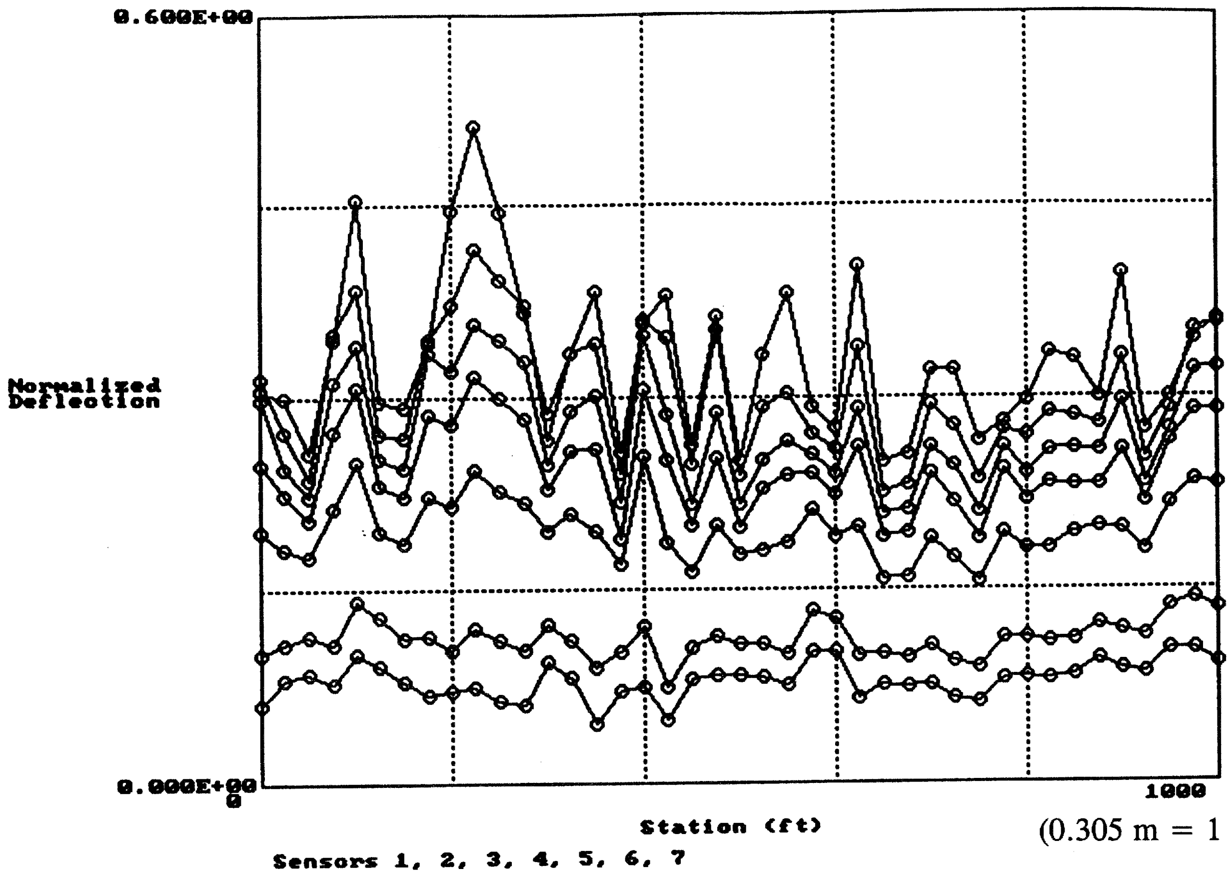
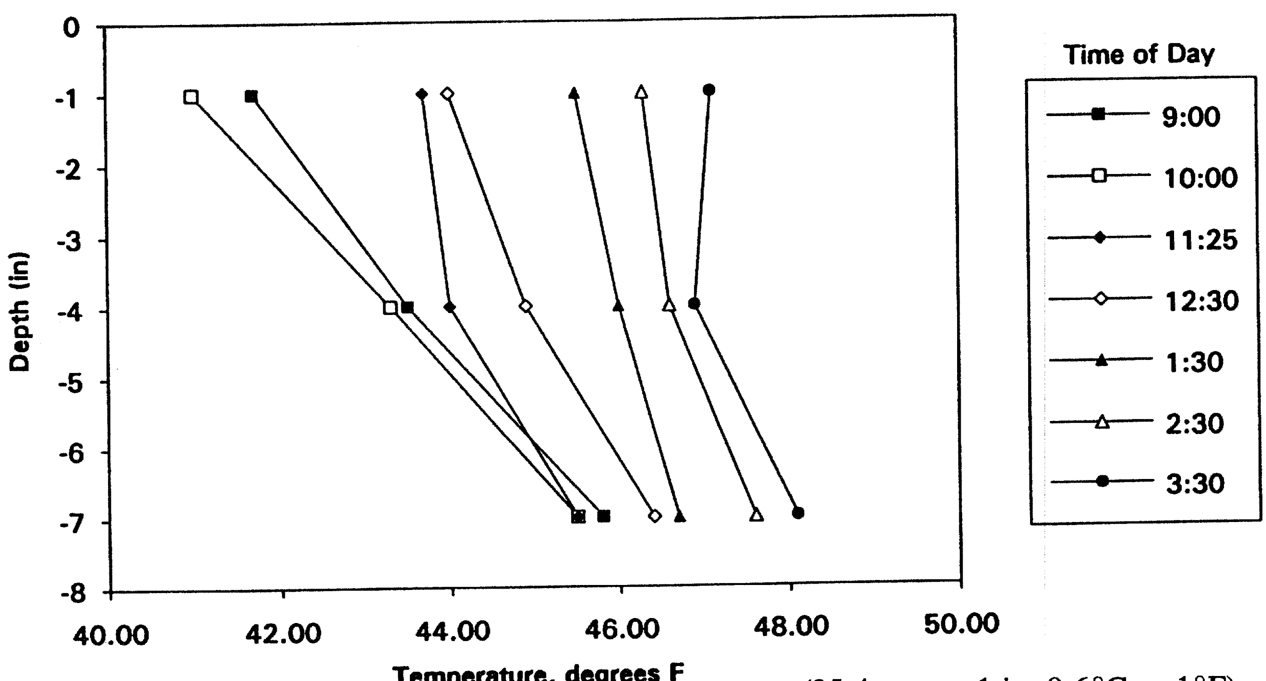


Figure 99. Crack pattern at site OK-5.



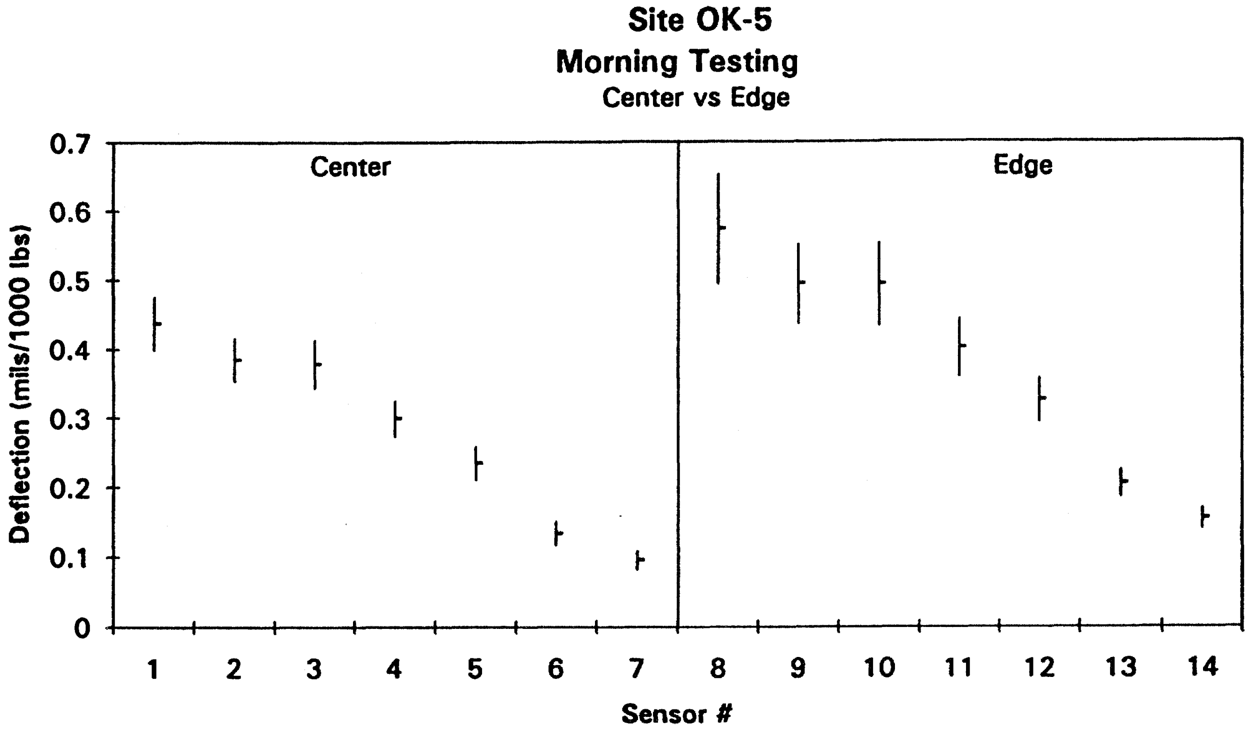
(0.305 m = 1 ft)

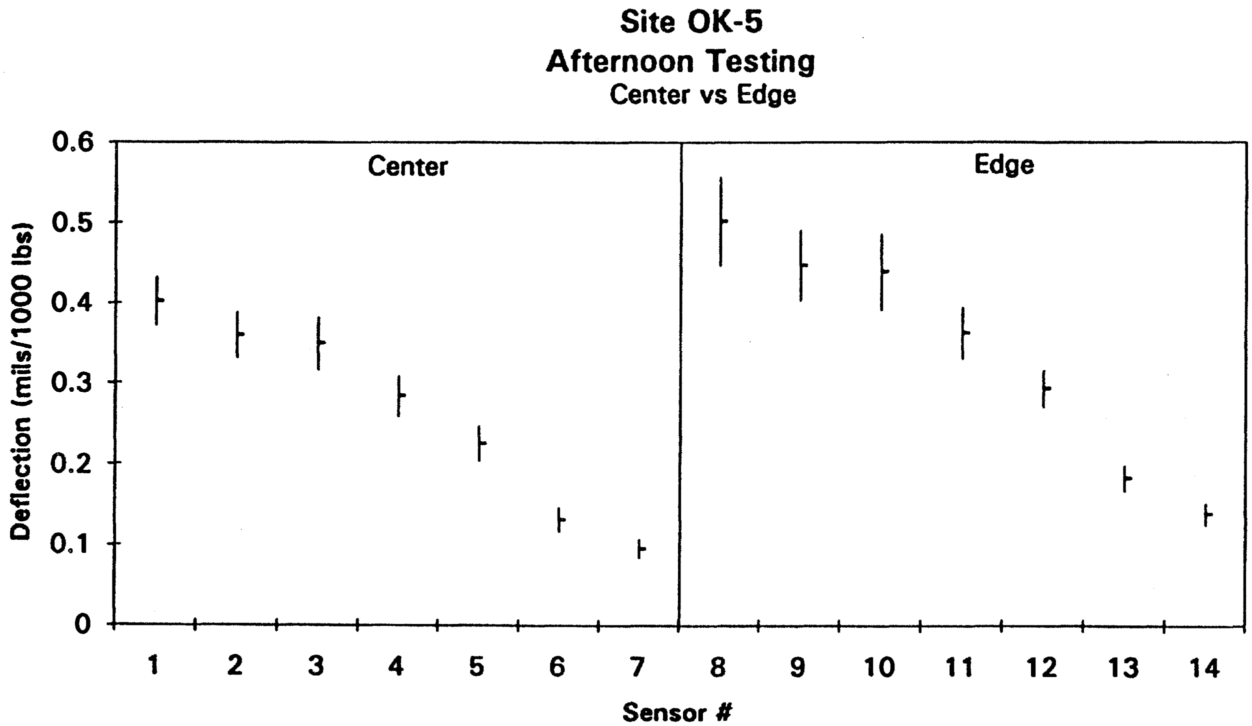
Figure 100. Deflections in inches along the section length at site OK-5.
[normalized per 4.45-kN [1,000-lb] load]



(25.4 mm = 1 in; 0.6°C = 1°F)

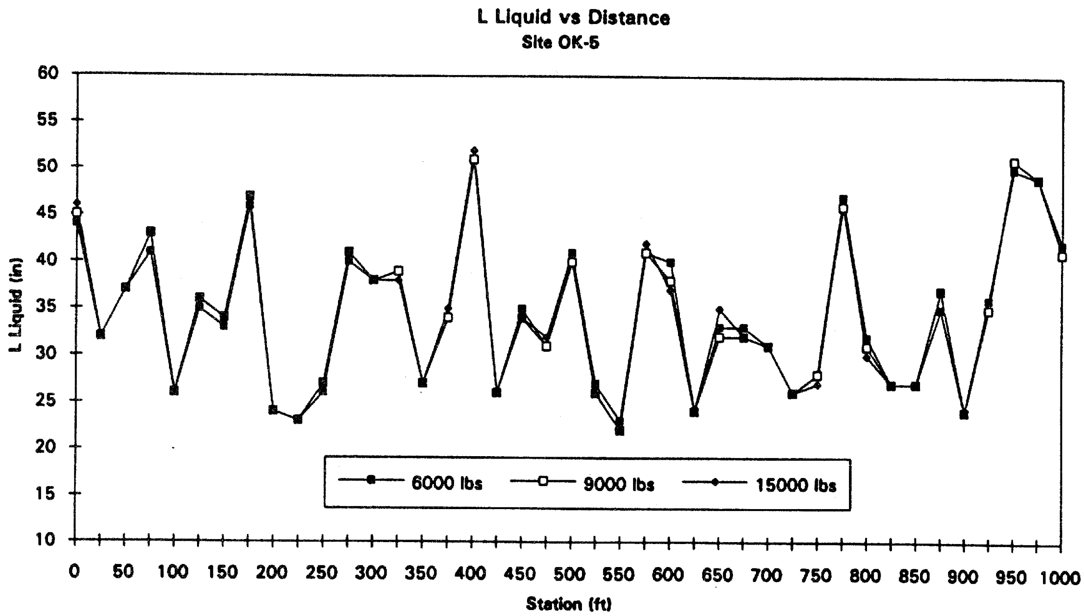
Figure 101. Temperature profile for site OK-5.





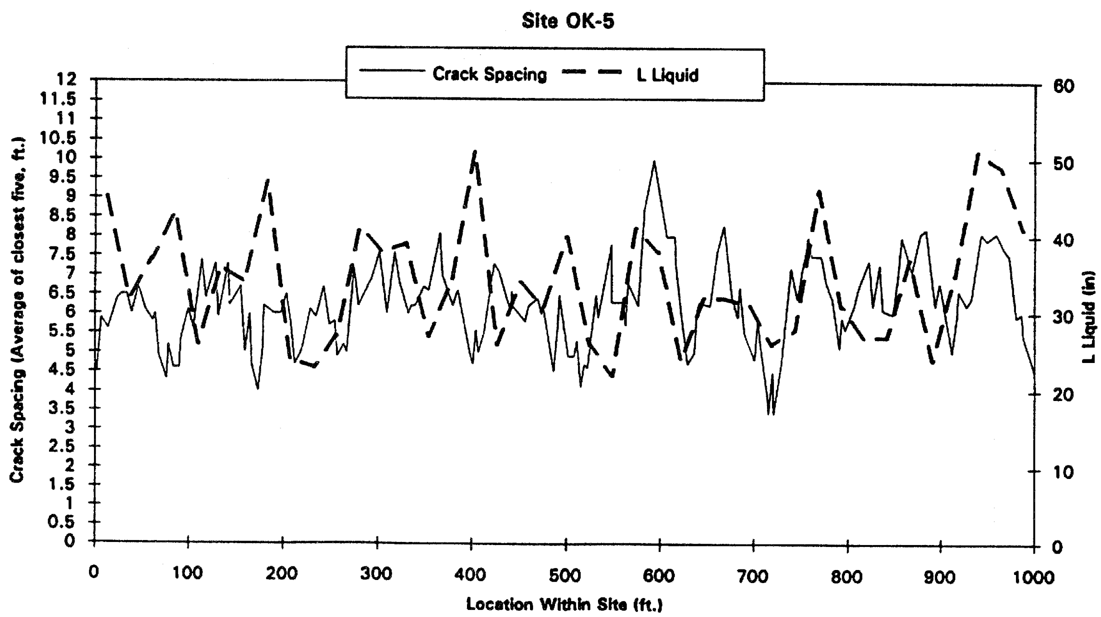
(0.0254 mm = 1 mil; 4.45 kN = 1,000 lb)

Figure 102. Center versus edge deflection for each of the seven sensors at site OK-5.
[40.03 kN (9,000 lb) load]



(25.4 mm = 1 in, 0.305 m = 1 ft)

Figure 103. Backcalculated values of radius of relative stiffness, ℓ (using program ILLI-BACK) for site OK-5.



(0.305 m = 1 ft)

Figure 104. Comparison of radius of relative stiffness values with average crack spacing (nearest five cracks) at site OK-5.

CHAPTER 6 - OREGON TEST SECTIONS

Section OR-1 - Interstate 5

Introduction

OR-1 was constructed between 1982 and 1985 and was opened to traffic in 1985. OR-1 is a section of I-5 that runs north to south from Eugene, Oregon, to the Oregon-California border as shown in figure 105. The length of the highway that was considered runs from milepost 147 to 188. A 305-m (1,000-ft) south bound section was selected for detailed inspection between mileposts 184 and 185, as shown in figure 105. The section of I-5 that was chosen is in the southern region of Oregon.

The climate in this region is affected by the Pacific Ocean. There is abundant moisture throughout the year, and the temperatures are moderate. The highest monthly average temperature is 19.3°C (66.8°F) during July, and the lowest monthly average is 4.4°C 40.0°F) during January. I-5 lies within the wet-no freeze environmental zone.

Traffic

Since its opening the pavement has experienced an estimated 11.3 million ESALs in its south bound lanes. The ADT for 1990 was 29,750 with a truck percentage of 26.2. From 1989 to 1990, there was a 2-percent decrease in ADT. I-5 was designed for a traffic level of 2,670 ESALs per day.

1990 ESAL in Design Lane = 2,900,000

Structure

OR-1 is a four-lane divided highway. <u>The pavement lane width is 4.0 m (13 ft)</u> with an outside shoulder width of 3.0 m (10 ft) and an inside shoulder width of 0.6 m (2 ft). <u>The shoulders are constructed of asphaltic concrete.</u>

The outside lane of I-80 was designed as a three-layer inlay system comprised of a 330-mm (13-in) CRCP and a 254-mm (10-in) granular base over native subgrade. The CRC pavement was designed with a steel percentage of 0.6. The reinforcing steel are #6 bars spaced at 121 mm (4.75 in). The transverse steel percentage is 0.08. The transverse steel are #4 spaced at 1 524 mm (60 in). The bars were placed on chairs as a single layer during the construction of the pavement. The bars are not epoxy coated.

The average concrete pavement thickness for OR-1 was determined to be 292 mm (11.5 in) based on cores taken as part of this study. The concrete is considered well graded, and its consolidation was average. The aggregate type used was not a rounded siliceous gravel and had a maximum size of 51 mm (2 in).

The granular base course was considered to be well graded and found to be 102 mm (4 in) thick. Its AASHTO classification is A-1-a. The subgrade classification was not determined as a sample and was not obtained. The subgrade was reported to be silty clay (AASHTO A-4).

Design/Construction

The thickness design for I-80 was done by the Oregon Department of Transportation using the PCA method. The design life chosen was 20 years with a design traffic level of 2,670 ESALs per day and a concrete modulus of rupture equal to 3.62 MPa (525 lbf/in^2). A composite modulus of K equal to 19.0 MPa/m (70 pci) and 80 were assumed for subgrade and subbase reaction, respectively. Edge drainage was designed before construction. Other design information that was used was an R-value of 10; a load safety factor of 1.3 and a concrete compressive strength of 3,950 were assumed. The terminal joint design chosen was a wide flange beam system. The bridge approach pavement type is concrete.

Construction began during May 1982 and was completed in August 1985. The concrete properties specified were slump of 25 to 76 mm (1 to 3 in), air content of 3 to 6 percent, and a concrete strength of 22.75 MPa (3,300 lbf/in^2) at 28 days.

Performance

The performance of I-5 has been evaluated every 2 years since its opening. The performance of I-5 has been recorded according to Visual Condition Survey scale, 0.0 - 5.0, 5.0 being very good and 1.0 being very poor. The first value recorded in 1985 was 4.7 and has decreased to a value of 3.5 in 1991.

Historical Data (M&R)

No major maintenance and rehabilitation work has been reported for I-5.

Field Investigation Data

Visual Condition Survey

- Total number of cracks = 302
  - 0 were at high severity
  - 301 were at medium severity
  - 1 were at low severity
- Average crack spacing = 1.2 m (4.0 ft)
- Standard deviation for crack spacing = 0.6 m (2.1 ft)
- Coefficient of variation of the crack spacing = 53 percent
- Figure 106 shows average crack spacing distribution based on the closest five cracks
- Figure 107 shows actual crack spacing
- Extent of Y cracks = 10 percent

- Cluster cracking was apparent at stations 1+00, 2+00, 5+00, and 5+50
- Other distress types included:
    - Popouts of low to moderate severity
- Pavement shoulders were in fair condition.

Drainage Survey
- No information was collected.

Windshield Survey
- Windshield survey was not conducted.

Terminal Joint Survey
- Terminal joints used = Wide flange (two surveyed)
- Joints exhibited the following distresses:
    - One joint exhibited 0.2 m^2 (2 ft^2) of low severity spalling.

Deflection Testing
- Figure 108 shows normalized/basin (per 4.45 kN (1,000 lb) load) deflections measured along the length of the section (for all seven sensors using the 40.03 kN (9,000 lb) load)
- Load transfer efficiency at cracks at mid-slab
    - 90 to 94 percent in the morning
    - No testing in the afternoon
- Load transfer efficiency at cracks along edge
    - 91 to 96 percent in the morning
    - No testing in the afternoon
- Figure 109 shows the distribution of slab temperature with depth and time of day.
- Figure 110 shows the average deflections and ranges of deflections for each sensor for center and edge loading, for morning and afternoon testing (using the 40.03 kN (9,000 lb) load within the selected 30.5-m (100-ft) subsection).
- Figure 111 shows the backcalculated radius of relative stiffness, ℓ, values for basin testing for each of the three nominal load levels of 40.03, 53.38, and 71.17 kN (9,000, 12,000, and 16,000 lb).
- ℓ value based on 40.03 kN (9,000 lb) load basin testing
    - Average = 965 mm (38 in)
    - Standard Deviation = 203 mm (8 in)
- ℓ value at crack locations at mid-slab (morning only)
    - Average = 838 mm (33 in)
    - Standard Deviation = 102 mm (4 in)
- ℓ value at crack locations at edge (morning only)
    - Average = 813 mm (32 in)
    - Standard Deviation = 51 mm (2 in)

- Estimated ℓ value using actual slab thickness and concrete modulus of elasticity and assumed modulus of subgrade reaction
  - Value = 1041 mm (41 in)
- Figure 112 shows the variation of ℓ in relation to crack spacing (average of the closest five cracks)
- Average deflection under 40.03 kN (9,000 lb) load (0.000001 mm (0.001 in))
  - Basin test - 0.07 (2.7)
  - Mid-slab crack location (morning) - 0.07 (2.8)
  - Edge crack location (morning) - 0.10 (4.1)

Crack Width Measurements
- Number of cracks monitored within the 30.5-m (100-ft) subsection = 21
- Morning testing
  - Average temperature 102 mm (4 in) below surface = 10.0°C (50°F)
  - Average crack width = 0.31 mm (0.012 in)
- Afternoon testing
  - Average temperature 102 mm (4 in) below surface = 11.7°C (53°F)
  - Average crack width = not measured
- Slab length change, mm/mm/°C (in/in/°F) = N/A

Reinforcing Steel Depths and Spacing
- Steel depth measurement offsets = 610, 1 219, 1 829 mm (24, 48, 72 in)
- Range of depth of cover = 107 to 325 mm (4.2 to 12.8 in)
- Average depth of cover = 191 mm (7.5 in)
- Standard deviation for depth of cover = 46 mm (1.8 in)
- Steel spacing was not measured

Corrosion Potential Testing (copper-copper sulfate half cell potentiometer)
- No testing was performed

Concrete Core Examination for Corrosion
- No steel corrosion was detected in three cores obtained over steel bars

Profile Testing
- Profile testing was not conducted

Laboratory Testing

The following data were obtained from the testing of the concrete, base/subbase, and subgrade materials:

Concrete

Modulus of Elasticity	-	24 820 MPa (3,600,000 lbf/in^2)
Average Split Tensile Strength	-	3.65 MPa (530 lbf/in^2)
Split Tensile Strength Range	-	3.03 to 4.07 MPa (440 to 590 lbf/in^2)
Coefficient of Thermal Expansion	-	8.84 mm/mm/°C (4.91 in/in/°F)

Base

AASHTO Classification	-	Granular base (A-1-a)

Subbase

AASHTO Classification	-	N/A

Subgrade

Liquid Limit	-	N/A
Plastic Limit	-	N/A
Percent Passing #200 Sieve	-	N/A
AASHTO Classification	-	Silty clay (A-4) (reported)

Chloride content testing was not conducted.

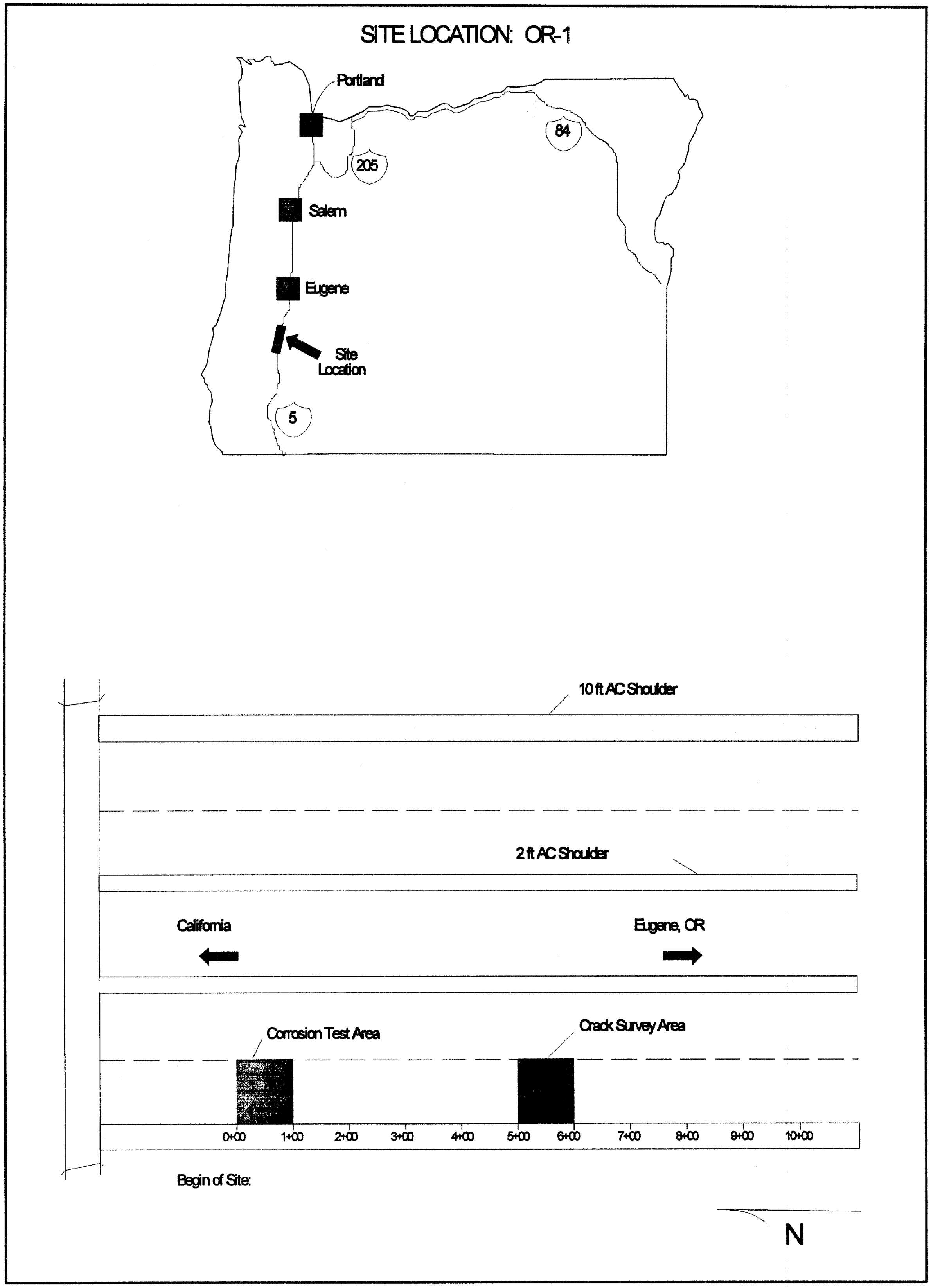


Figure 105. Site details for OR-1.

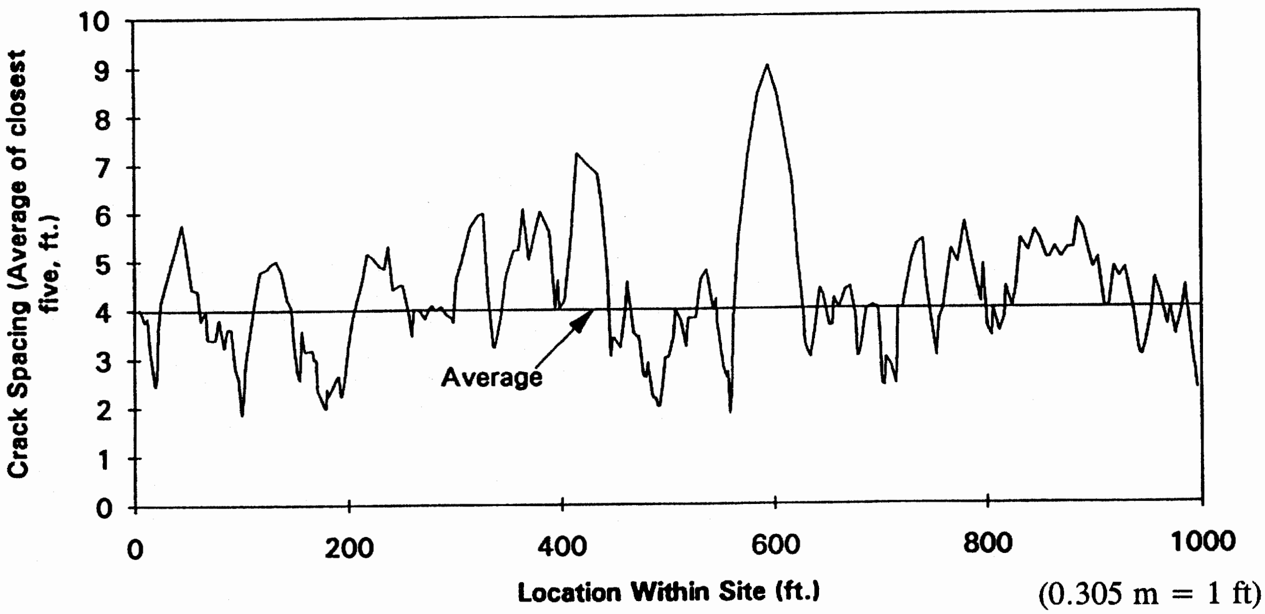


Figure 106. Crack spacing pattern at site OR-1.

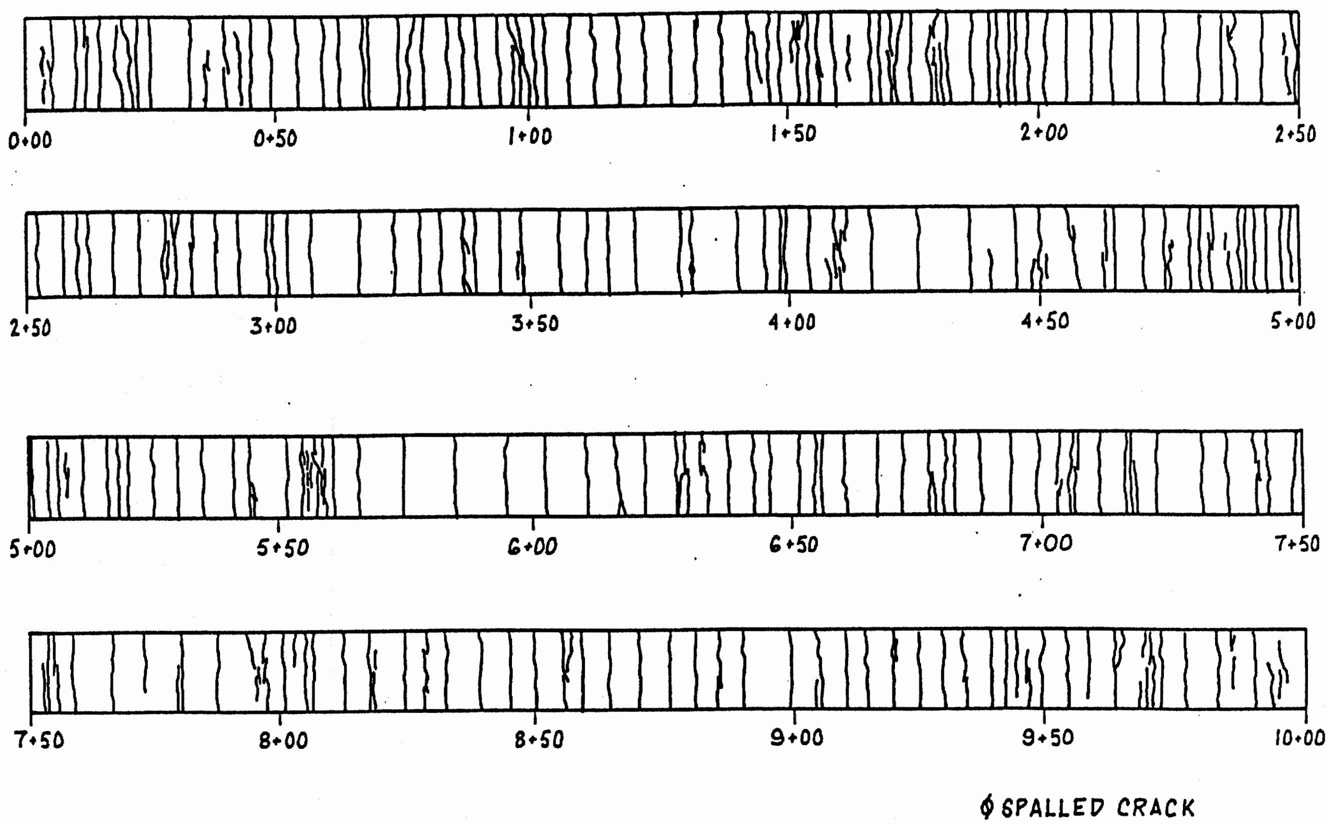
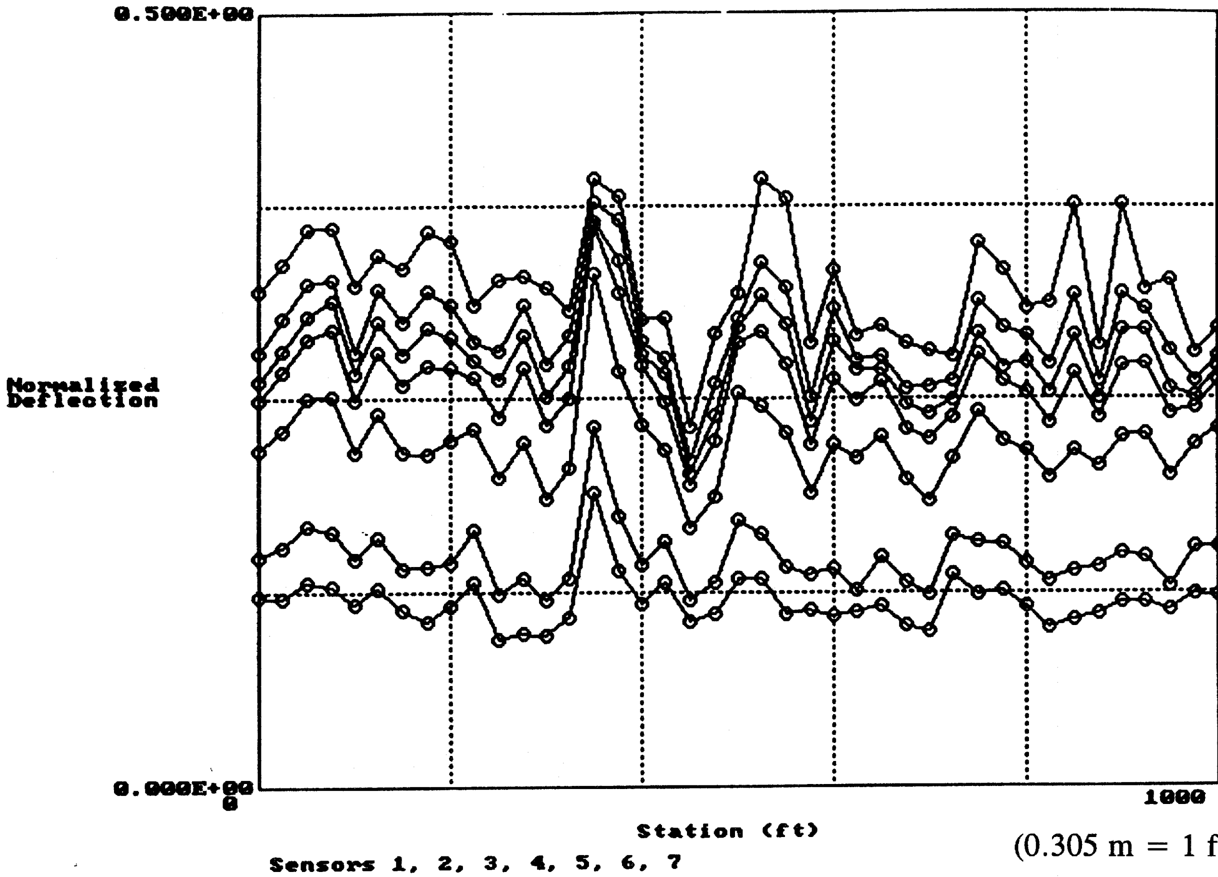
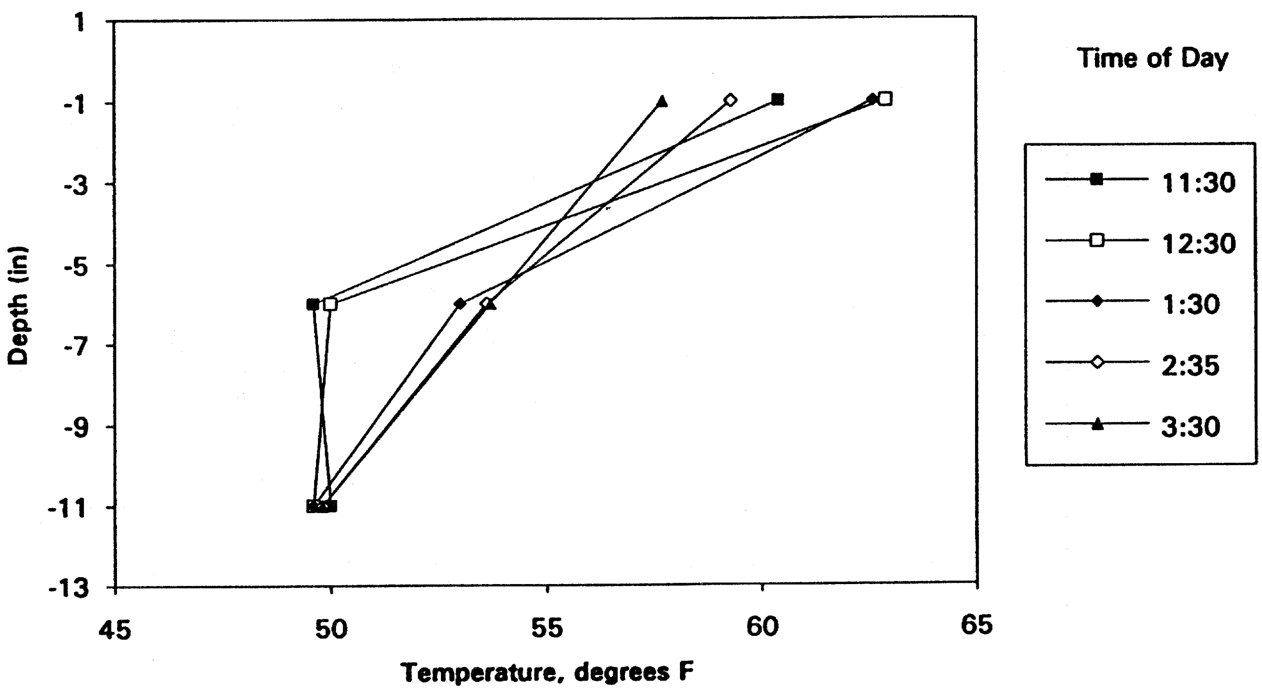


Figure 107. Crack pattern at site OR-1.



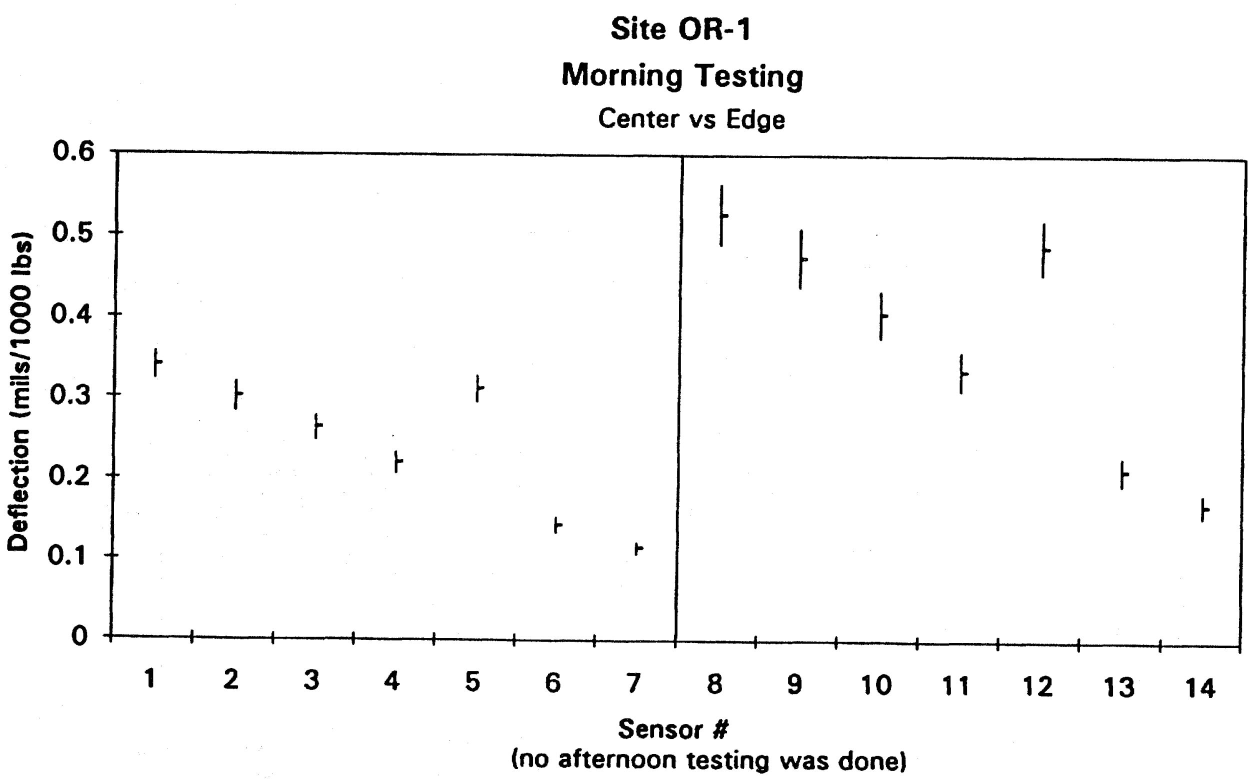
(0.305 m = 1 ft)

Figure 108. Deflections in inches along the section length at site OR-1.
[normalized per 4.45-kN [1,000-lb] load]



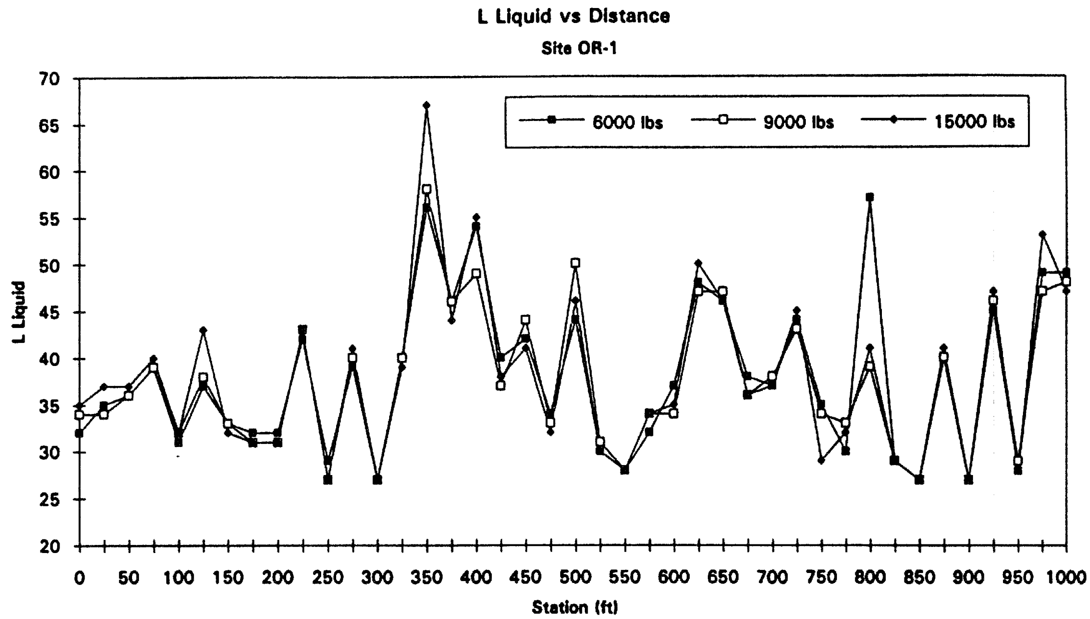
(25.4 mm = 1 in; 0.6°C = 1°F)

Figure 109. Temperature profile for site OR-1.



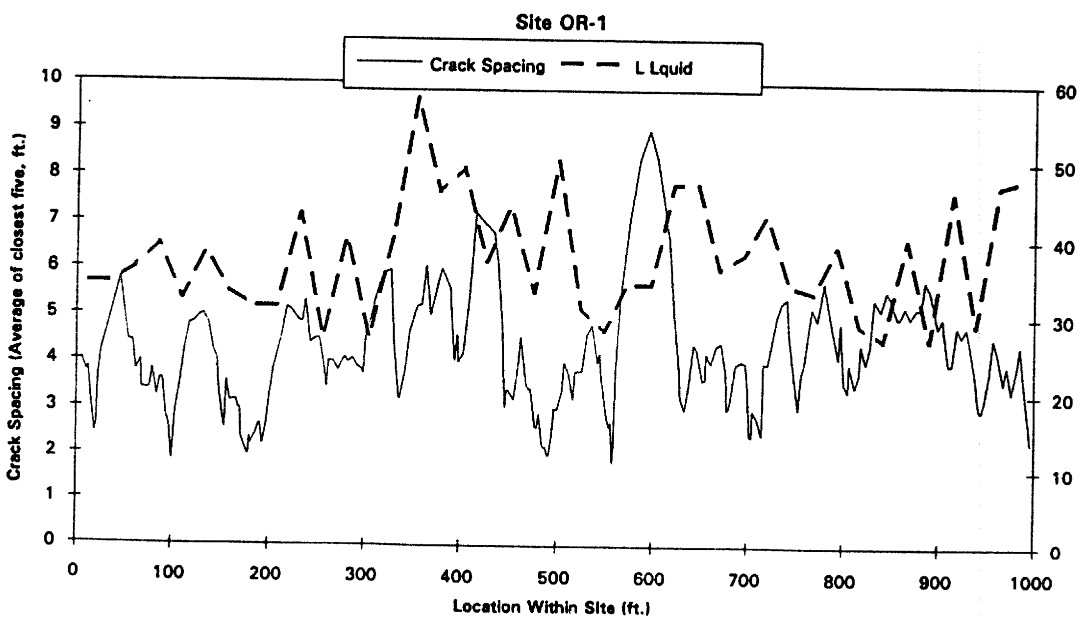
(0.0254 mm = 1 mil; 4.45 kN = 1,000 lb)

Figure 110. Center versus edge deflection for each of the seven sensors at site OR-1.
[40.03 kN (9,000 lb) load]



(25.4 mm = 1 in, 0.305 m = 1 ft)

Figure 111. Backcalculated values of radius of relative stiffness, ℓ (using program ILLI-BACK) for site OR-1.



(0.305 m = 1 ft)

Figure 112. Comparison of radius of relative stiffness values with average crack spacing (nearest five cracks) at site OR-1.

Section OR-2 - Interstate 5

Introduction

OR-2 was constructed between 1984 and 1987 and was opened to traffic in 1987. OR-2 is a section of I-5 that runs north to south from Eugene, Oregon, to the Oregon-California border as shown in figure 113. The length of the highway that was considered runs from milepost 174 to 188. A 305-m (1,000-ft) north bound section was selected for detailed inspection between mileposts 184 and 185, as shown in figure 113. The section of I-5 that was chosen is in the southern region of Oregon.

The climate in this region is affected by the Pacific Ocean. There is abundant moisture throughout the year, and the temperatures are moderate. The highest monthly average temperature is 19.3°C (66.8°F) during July, and the lowest monthly average is 4.4°C (40.0°F) during January. I-5 lies within the wet-no freeze environmental zone.

Traffic

The pavement has experienced an estimated 3 million ESALs in its north bound lanes in 1990. The ADT in the northbound lanes for 1990 was 30,300 with a truck percentage of 26.2. From 1989 to 1990, there was a minimal decrease in ADT. I-5 was designed for 33.88 million ESALs over 30 years.

Structure

OR-2 is a four-lane divided highway. The pavement lane width is 4.0 m (13 ft) with an outside shoulder width of 3.0 m (10 ft) and an inside shoulder width of 0.9 m (3 ft). The shoulders are constructed of asphaltic concrete.

This section of I-5 was designed as a three-layer system comprised of a 254-mm (10-in) continuously reinforced concrete pavement (CRCP) and a 229-mm (9-in) CTB over native subgrade. The CRC pavement was designed with a steel percentage of 0.6. The reinforcing steel are #6 bars spaced at 165 mm (6.5 in). The transverse steel percentage is 0.08. The transverse steel are #4 bars spaced at 1 524 mm (60 in). The bars were placed on chairs as a single layer during the construction of the pavement. The bars are not epoxy coated.

The average concrete pavement thickness for OR-2 was determined to be 264 mm (10.4 in) based on cores taken as part of this study. The concrete is considered well graded and is well consolidated. The aggregate type is a rounded siliceous gravel with a maximum size of 51 mm (2 in).

The CTB was found to be 236 mm (9.3 in) thick. The subgrade was a silty clay with an AASHTO classification of A-4. The liquid limit was 37, and its plastic limit was 33.

Design/Construction

The thickness deign for I-80 was done by the Oregon Department of Transportation using the PCA Design Method. The design life chosen was 30 years with a design traffic level of 33.8 million ESALs and a concrete modulus of rupture equal to 3.79 MPa (550 lbf/in^2). A composite modulus of subgrade reaction of 95.0 MPa/m (350 pci) was used for design. Edge drainage was incorporated into the design. Other design information that was used was an R-value of 10, and a load safety factor of 1.3 was assumed. The terminal joint design chosen was a wide flange beam system. The bridge approach pavement type is concrete.

Construction began during December 1984 and was completed in October 1987. The concrete properties specified were slump of 25 to 76 mm (1 to 3 in), air content of 3 to 6 percent, and a concrete strength of 22.75 MPa (3,300 lbf/in^2) at 28 days. Several short delays were encountered during construction because of rain. The pavement was then covered with plastic to protect the concrete. In some sections, excess water had to be removed from the pavement surface before tining.

Performance

The performance of I-5 has been evaluated every 2 years since its opening. The performance of I-5 has been recorded according to Visual Condition Survey scale, 0.0 - 5.0. The first value recorded in 1985 was 5.0 and has decreased to a value of 4.5 in 1991.

Historical Data (M&R)

No maintenance and rehabilitation work has been reported for I-5.

Field Investigation Data

Visual Condition Survey

- Total number of cracks = 277
  - 0 were at high severity
  - 274 were at medium severity
  - 3 were at low severity
- Average crack spacing = 1.68 m (5.51 ft)
- Standard deviation for crack spacing = 0.94 m (3.09 ft)
- Coefficient of variation of the crack spacing = 56 percent
- Figure 114 shows average crack spacing distribution based on the closest five cracks
- Figure 115 shows actual crack spacing
- Extent of Y cracks = 3 percent
- Cluster cracking was apparent at station 6+50 only.
- Other distress types included:
  - 5.5 m (18 ft) of longitudinal cracking (low severity)
- Pavement shoulders were in fair condition.

Drainage Survey

- No information was collected.

Windshield Survey

- Windshield survey was not performed.

Terminal Joint Survey

- Terminal joints used = Wide flange (two surveyed)
- Joints exhibited the following distresses:
    - One exhibited 0.22 m^2 (2.4 ft^2) of low severity spalling
    - Other exhibited 0.19 m^2 (2.0 ft^2) of low severity spalling

Deflection Testing

- Figure 116 shows normalized/basin (per 4.45 kN (1,000 lb) load) deflections measured along the length of the section (for all seven sensors using the 40.03 kN (9,000 lb) load)
- Load transfer efficiency at cracks at mid-slab
    - 87 to 93 percent in the morning
    - 90 to 96 percent in the afternoon
- Load transfer efficiency at cracks along edge
    - 90 to 94 percent in the morning
    - 91 to 95 percent in the afternoon
- Figure 117 shows the distribution of slab temperature with depth and time of day.
- Figure 118 shows the average deflections and ranges of deflections for each sensor for center and edge loading, for morning and afternoon testing (using the 40.03 kN (9,000 lb) load within the selected 30.5-m (100-ft) subsection).
- Figure 119 shows the backcalculated radius of relative stiffness, ℓ, values for basin testing for each of the three nominal load levels of 40.03, 53.38, and 71.17 kN (9,000, 12,000, and 16,000 lb).
- ℓ value based on 40.03 kN (9,000 lb) load basin testing
    - Average = 965 mm (38 in)
    - Standard Deviation = 203 mm (8 in)
- ℓ value at crack locations at mid-slab (morning/afternoon)
    - Average = 787/813 mm (31/32 in)
    - Standard Deviation = 127/152 mm (5/6 in)
- ℓ value at crack locations at edge (morning/afternoon)
    - Average = 762/787 mm (30/31 in)
    - Standard Deviation = 76/76 mm (3/3 in)
- Estimated ℓ value using actual slab thickness and concrete modulus of elasticity and assumed modulus of subgrade reaction
    - Value = 864 mm (34 in)
- Figure 120 shows the variation of ℓ in relation to crack spacing (average of the closest five cracks)

- Average deflection under 40.03 kN (9,000 lb) load (0.000001 mm (0.001 in))
    - Basin test - 0.048 (1.9)
    - Mid-slab crack location (morning/afternoon) - 0.053/0.053 (2.1/2.1)
    - Edge crack location (morning/afternoon) - 0.125/0.089 (4.9/3.5)

Crack Width Measurements

- Number of cracks monitored within the 30.5-m (100-ft) subsection = 20
- Morning testing
    - Average temperature 102 mm (4 in) below surface = 8.9°C (48°F)
    - Average crack width = 0.20 mm (0.008 in)
- Afternoon testing
    - Average temperature 102 mm (4 in) below surface = 11.7°C (53°F)
    - Average crack width = 0.20 mm (0.008 in)
- Slab length change, mm/mm/°C (in/in/°F) = 0 (outlier)

Reinforcing Steel Depths and Spacing

- Steel depth measurement offsets = 610, 1 219, 1 829 mm (24, 48, 72 in)
- Range of depth of cover = 33 to 155 (1.3 to 6.1 in)
- Average depth of cover = 102 mm (4.0 in)
- Standard deviation for depth of cover = 15 mm (0.6 in)
- Steel spacing was about 221 mm (8.7 in)

Corrosion Potential Testing (copper-copper sulfate half cell potentiometer)

- No testing was performed

Concrete Core Examination for Corrosion

- No steel corrosion was detected in the four cores obtained

Profile Testing

- Profile testing was not conducted

Laboratory Testing

The following data were obtained from the testing of the concrete, base/subbase, and subgrade materials:

Concrete

Modulus of Elasticity	-	29 650 MPa (4,300,000 lbf/in^2)
Average Split Tensile Strength	-	3.31 MPa (480 lbf/in^2)
Split Tensile Strength Range	-	3.10 to 3.59 MPa (450 to 520 lbf/in^2)

Coefficient of Thermal Expansion	-	8.69 mm/mm/°C (4.83 in/in/°F)

Base

AASHTO Classification	-	CTB

Subbase

AASHTO Classification	-	N/A

Subgrade

Liquid Limit	-	37 percent
Plastic Limit	-	33 percent
Percent Passing #200 Sieve	-	40.6 percent
AASHTO Classification	-	silty clay (A-4)

Chloride content testing was not conducted on this test section.

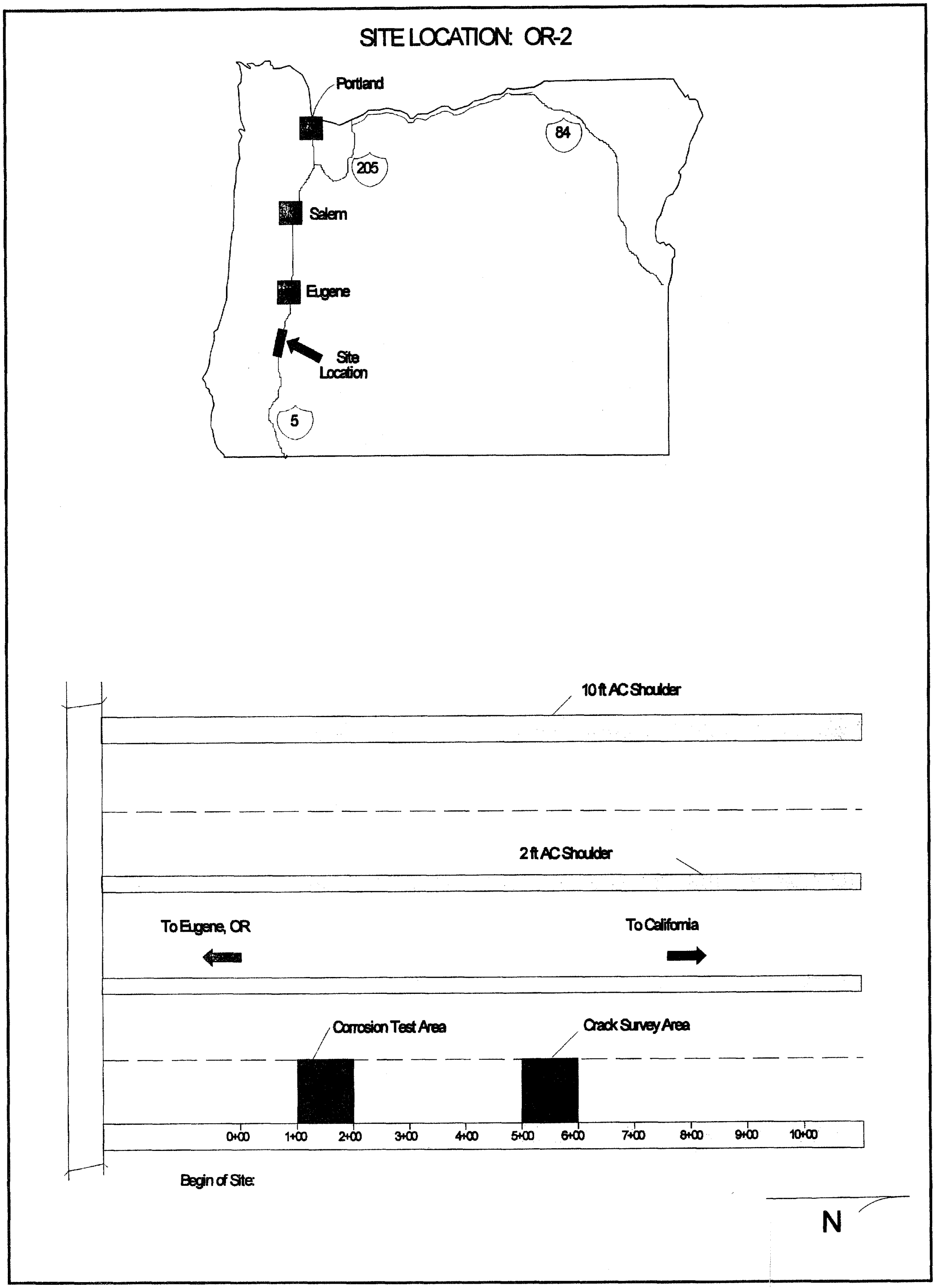
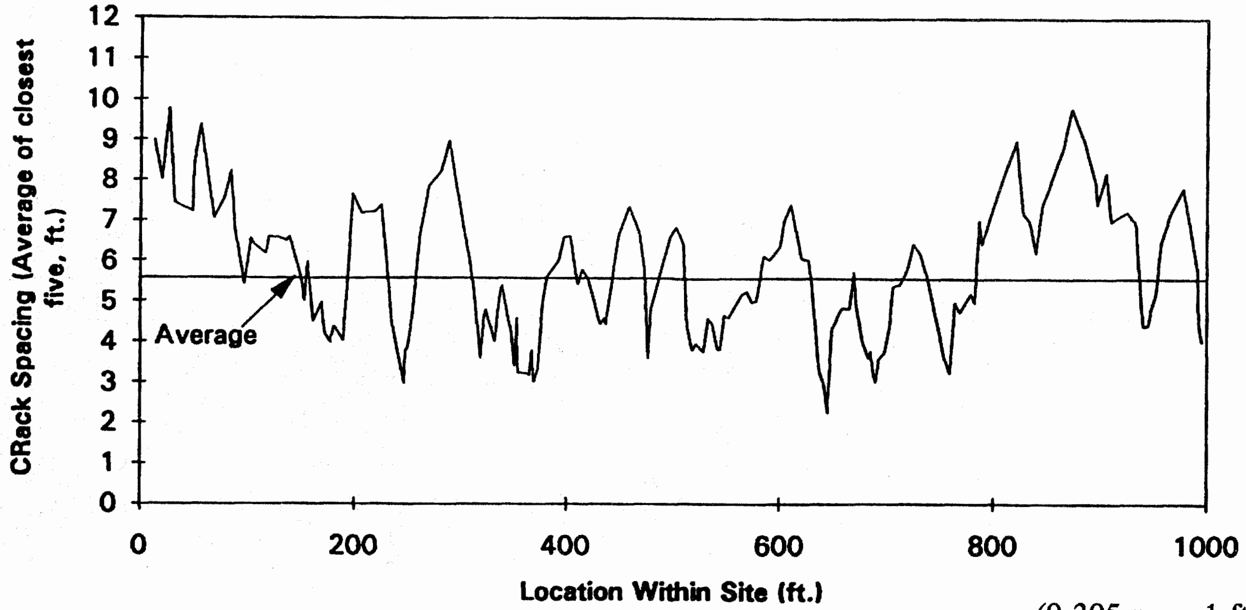


Figure 113. Site details for OR-2.



(0.305 m = 1 ft)

Figure 114. Crack spacing pattern at site OR-2.

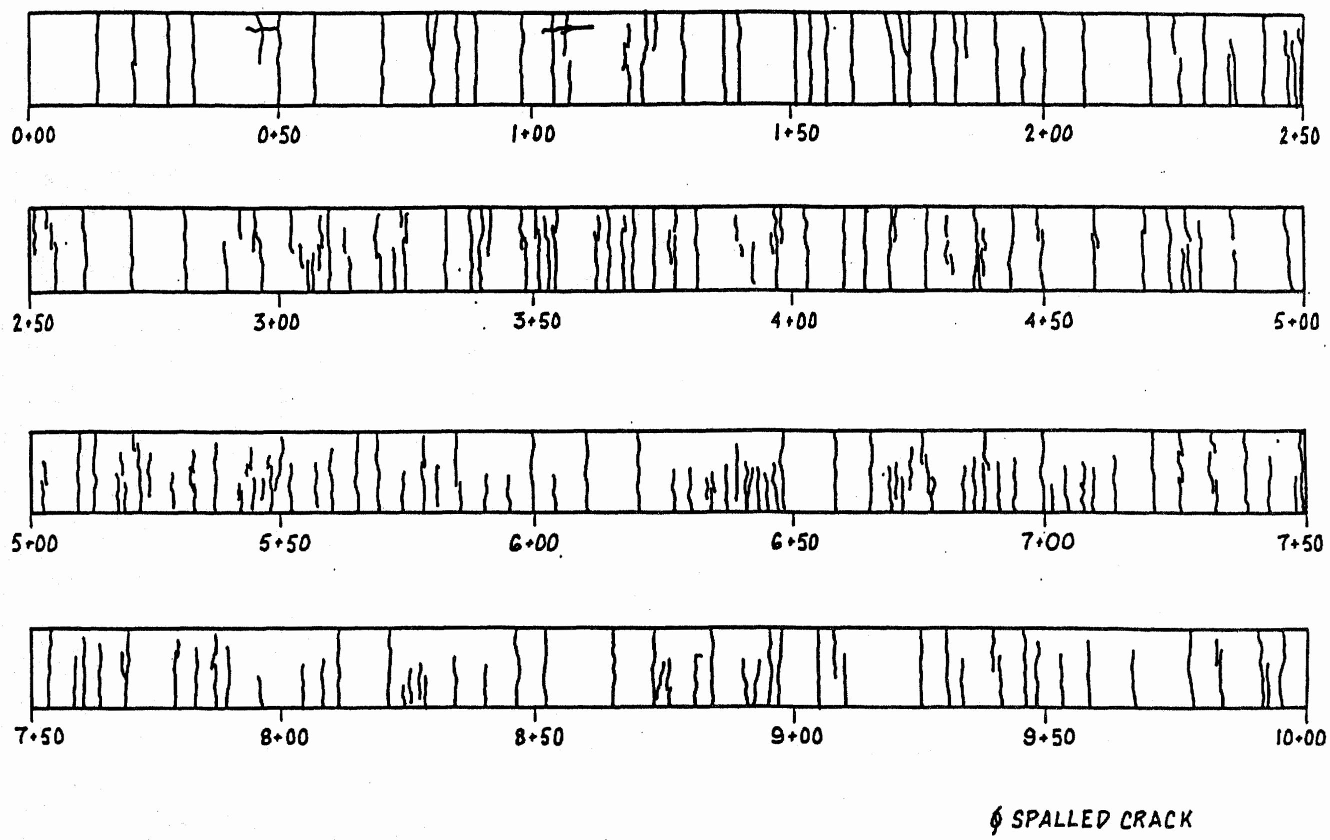
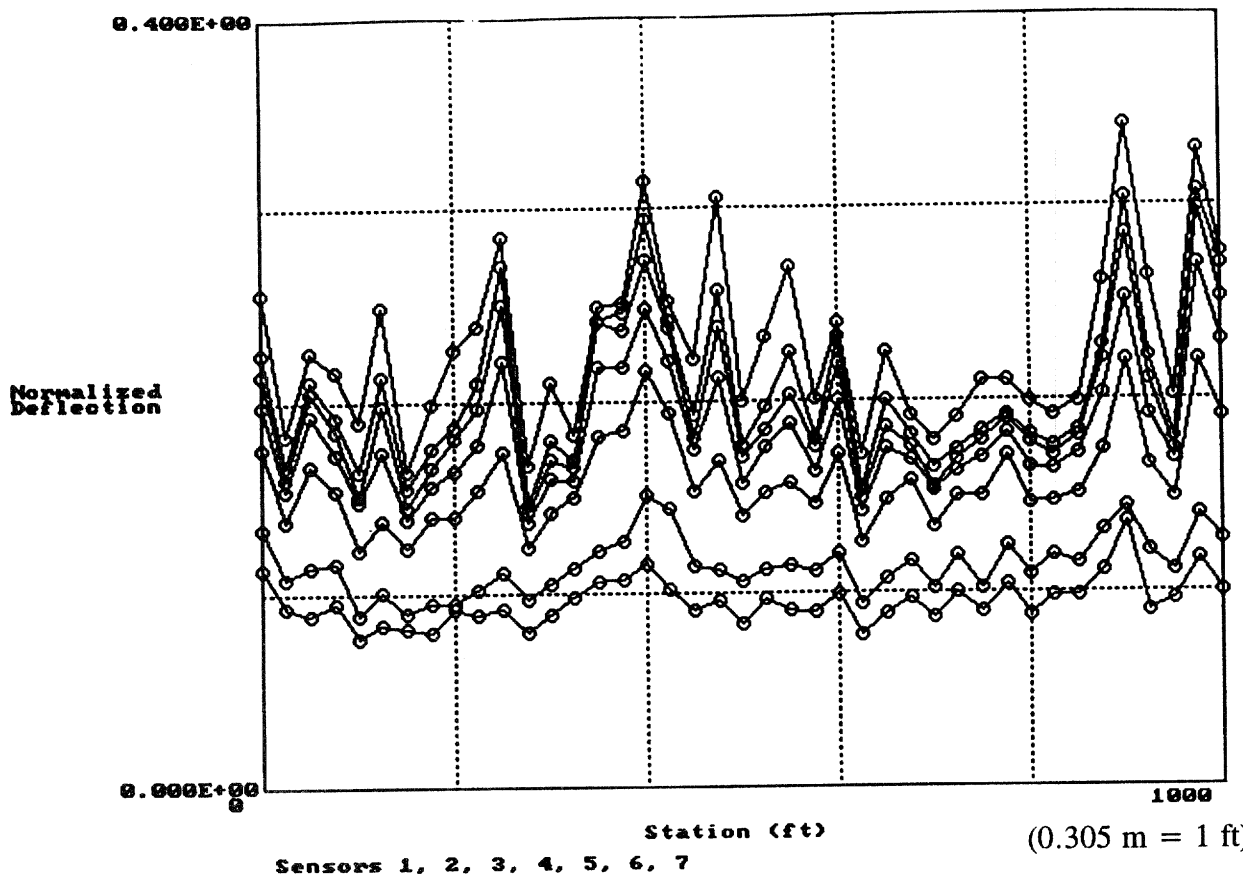
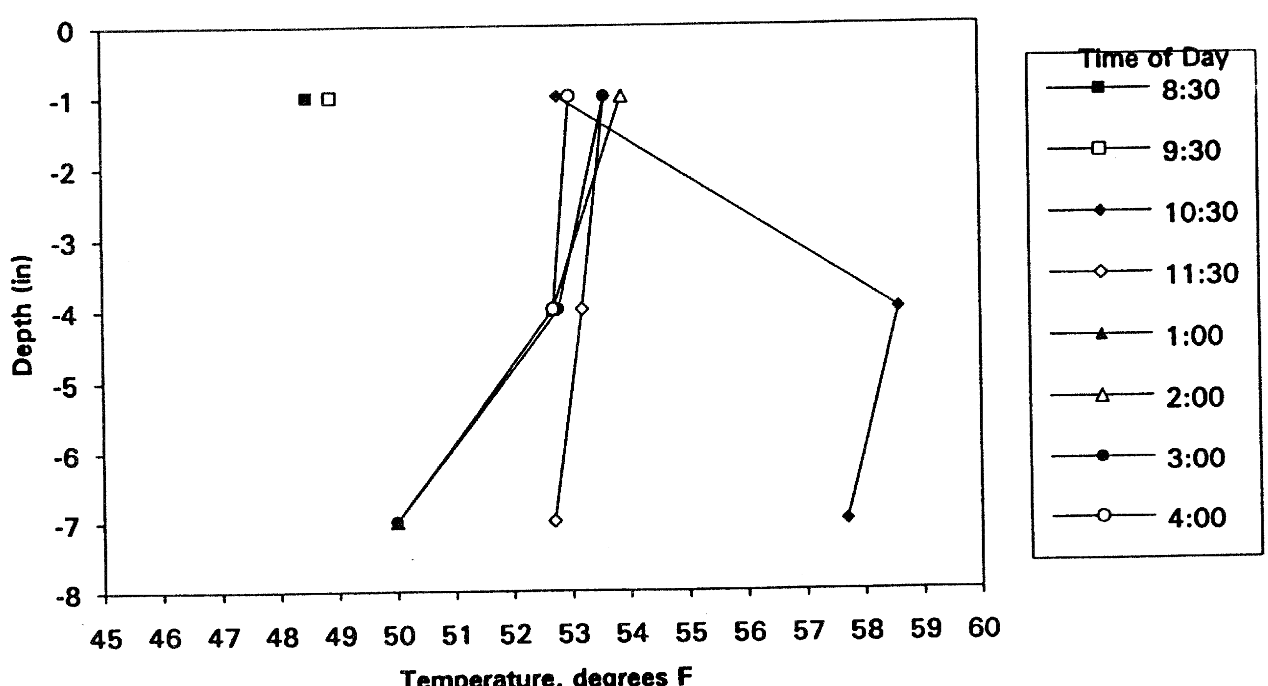


Figure 115. Crack pattern at site OR-2.



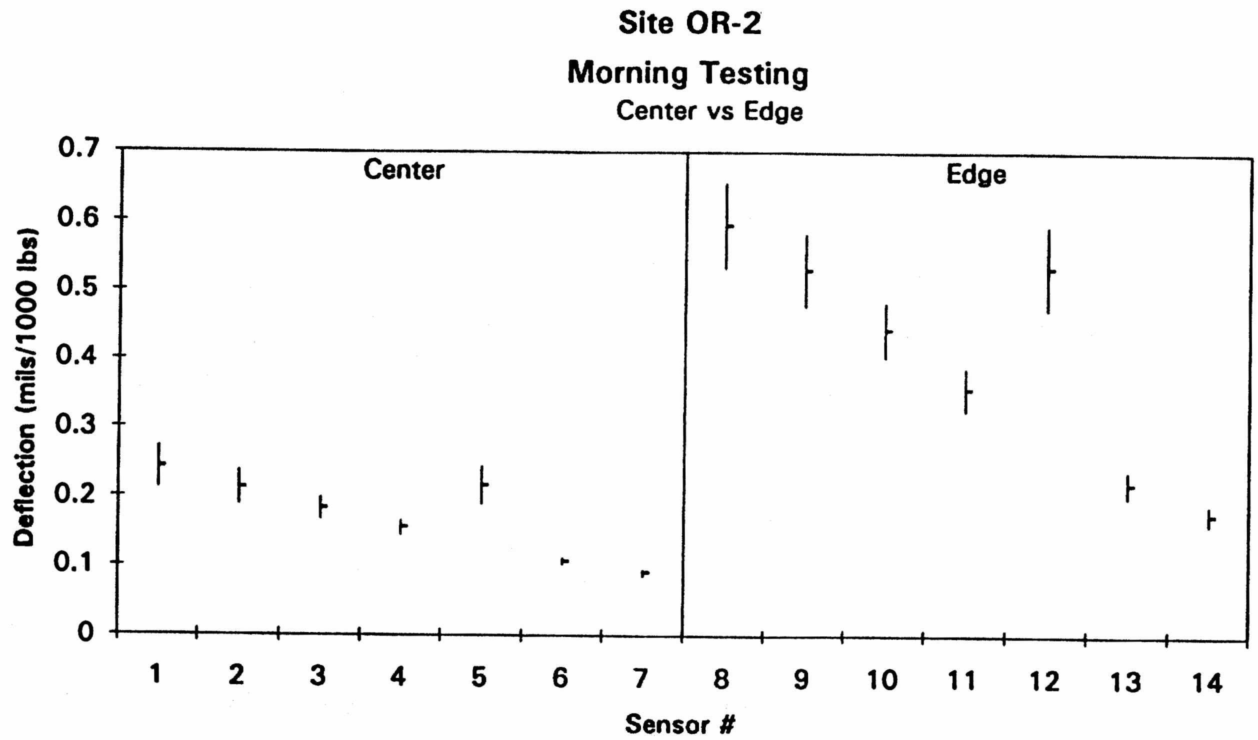
(0.305 m = 1 ft)

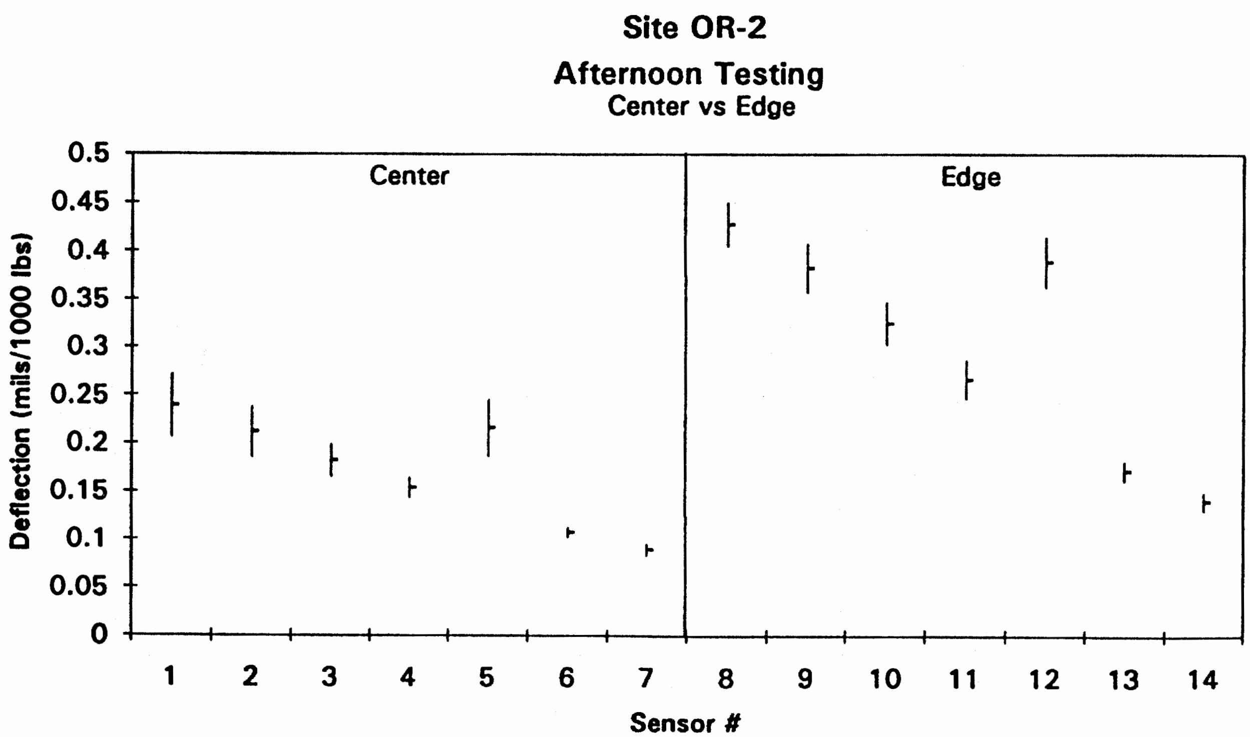
Figure 116. Deflections in inches along the section length at site OR-2.
[normalized per 4.45-kN [1,000-lb] load]



(25.4 mm = 1 in; 0.6°C = 1°F)

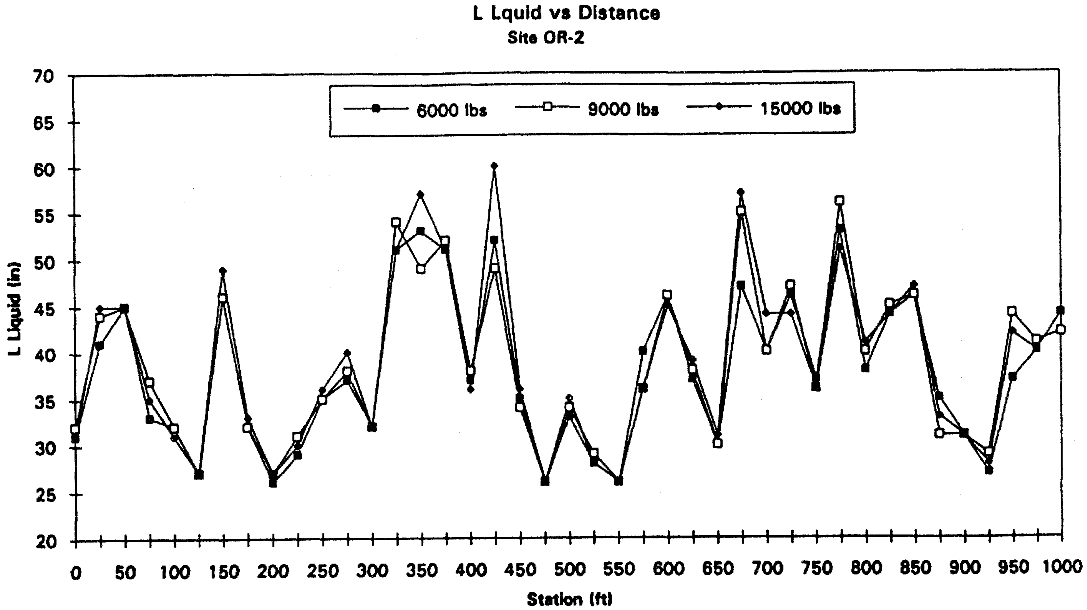
Figure 117. Temperature profile for site OR-2.





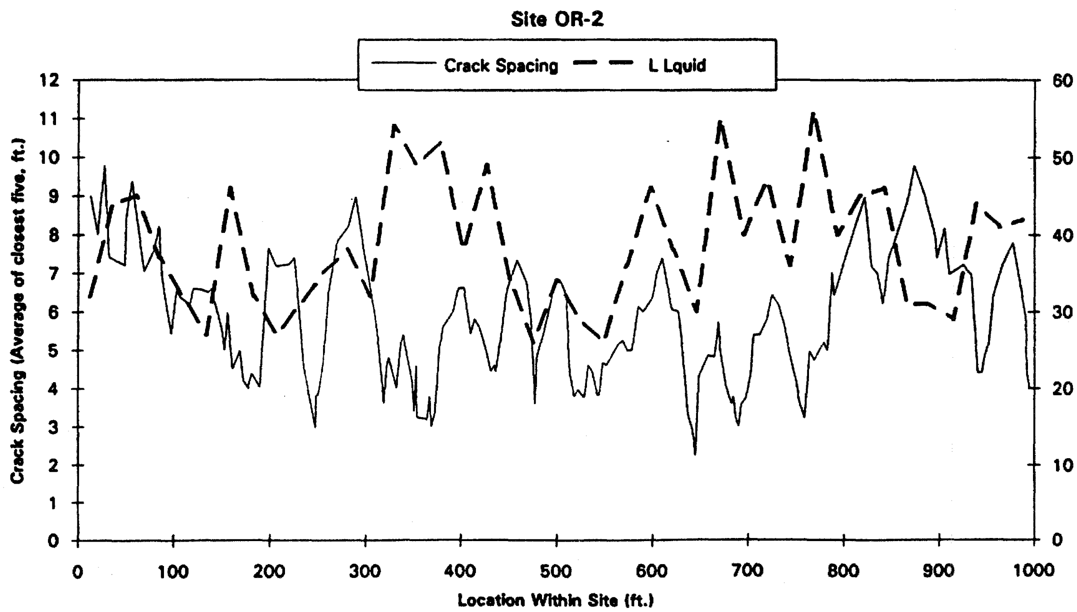
(0.0254 mm = 1 mil; 4.45 kN = 1,000 lb)

Figure 118. Center versus edge deflection for each of the seven sensors at site OR-2.
[40.03 kN (9,000 lb) load]



(25.4 mm = 1 in, 0.305 m = 1 ft)

Figure 119. Backcalculated values of radius of relative stiffness, ℓ (using program ILLI-BACK) for site OR-2.



(0.305 m = 1 ft)

Figure 120. Comparison of radius of relative stiffness values with average crack spacing (nearest five cracks) at site OR-2.

Section OR-3 - Interstate 205

Introduction

OR-3 was constructed between 1968 and 1971 and the pavement was opened to traffic in 1971. OR-3 is a section of I-205 that runs north to south from Portland, Oregon, to Oregon City as shown in figure 121. The length of the highway that was considered runs from milepost 0 to 3. A 305-m (1,000-ft) north bound section was selected for detailed inspection as shown in figure 121. The section of the I-205 that was chosen is in the northern region of Oregon.

The climate in this region is affected by the Pacific Ocean maritime climate. There is abundant moisture throughout the year, and the temperatures are moderate. Much of the precipitation, 70 percent, occurs over 5 months from November to March. The highest monthly average temperature is 19.1°C (66.3°F) during July, and the lowest monthly average is 4.1°C (39.3°F) during January. I-5 lies within the wet-no freeze environmental zone.

Traffic

The pavement has experienced an estimated 4.4 million ESALs in 1990. The ADT for 1990 was 59,100 with a truck percentage of 30.1. From 1989 to 1990, there was an increase of 5 percent in ADT. I-5 was designed for a 5,400,000 ESALs over 20 years.

1990 ESAL in Design Lane = 2,000,000

Structure

OR-3 is a four-lane divided highway. <u>The pavement lane width is 4.0 m (13 ft)</u> with an outside shoulder width of 3.0 m (10 ft) and an inside shoulder width of 0.6 m (2 ft). <u>The shoulders are constructed of asphaltic concrete.</u>

This section of I-5 was designed as a four-layer system comprised of an 203-mm (8-in) CRCP, a <u>229-mm (9-in) CTB</u>, and a 102-mm (4-in) plant mixed stone subbase over a subgrade of lime treated soil. The CRC pavement was designed with a steel percentage of 0.54. The reinforcing steel are #5 bars spaced at 165 mm (6.5 in). The transverse steel percentage is 0.12. The transverse steel are #4 bars spaced at 1 524 mm (60 in). The bars were placed on chairs as a single layer during the construction of the pavement. The bars are not epoxy coated.

The average concrete pavement thickness for OR-3 was determined to be 201 mm (7.9 in) based on cores taken as part of this study. The concrete is considered well graded and its consolidation is average. The aggregate type used was a rounded siliceous gravel with a maximum size of 25 mm (1 in).

The CTB was found to be 160 mm (6.3 in) thick. The subbase could not be distinguished from the subgrade. The subgrade was a lime modified soil with an AASHTO classification of A-6. The subgrade liquid limit was 37, and its plastic limit was 19.

Design/Construction

The thickness deign for I-80 was done by the Oregon Department of Transportation using the PCA Design Method. The design life chosen was 20 years with a design traffic level of 5,400,000 ESALs and a concrete modulus of rupture equal to 3.62 MPa (525 lbf/in^2). A composite modulus K for the three layers of CTB, subbase, and subgrade were assumed to be 47.5, 22.5, and 19.0 MPa/m (175, 83 and 70 pci), respectively. Edge drainage was designed before construction. Other design information used was an R-value of 10 , and a compressive strength of 27.23 MPa (3,950 lbf/in^2) was assumed for the concrete. The terminal joint design chosen was a lug anchor system. The bridge approach pavement type was concrete.

Construction began during July 1968 and was completed in November 1971. The concrete properties specified were slump of 25 to 76 mm (1 to 3 in), air content of 3 to 6 percent, and concrete strength of 22.75 MPa (3,300 lbf/in^2) at 7 days.

Performance

The performance of I-205 has been evaluated every 2 years since 1985. The performance of I-205 has been recorded according to the Visual Condition Survey scale, 0.0 to 5.0. The values recorded for 1985 and 1987 were 4.2 and for 1989 and 1990 were 4.0.

Historical Data (M&R)

No maintenance or rehabilitation work has been reported for I-205.

Field Investigation Data

Visual Condition Survey

- Total number of cracks = 226
  - 0 were at high severity
  - 226 were at medium severity
  - 0 were at low severity
- Average crack spacing = 1.35 m (4.42 ft)
- Standard deviation for crack spacing = 0.84 m (2.74 ft)
- Coefficient of variation of the crack spacing = 69 percent
- Figure 122 shows average crack spacing distribution based on the closest five cracks
- Figure 123 shows actual crack spacing
- Extent of Y cracks = 10 percent
- Cluster cracking was apparent at stations 1+00 and 9+00

- Other distress types included:
    - 12.19-m (40-ft) longitudinal cracking (moderate severity)
- Pavement shoulders were in fair condition.

Drainage Survey

- No information was collected

Windshield Survey

- Windshield survey was not conducted

Terminal Joint Survey

- Terminal joints used = Lug anchor (two surveyed)
- Joints exhibited the following distresses:
    - One exhibited 0.65 m^2 (7 ft^2) of low to medium severity PCC patch deterioration and some faulting
    - Others exhibited 0.37 m^2 (4 ft^2) of low to medium severity PCC patch deterioration

Deflection Testing

- Figure 124 shows normalized/basin (per 4.45 kN (1,000 lb) load) deflections measured along the length of the section (for all seven sensors using the 40.03 kN (9,000 lb) load)
- Load transfer efficiency at cracks at mid-slab
    - 90 to 96 percent in the morning
    - No testing in the afternoon
- Load transfer efficiency at cracks along edge
    - 91 to 95 percent in the morning
    - No testing in the afternoon
- Figure 125 shows the distribution of slab temperature with depth and time of day.
- Figure 126 shows the average deflections and ranges of deflections for each sensor for center and edge loading, for morning testing (using the 40.03 kN (9,000 lb) load within the selected 30.5-m (100-ft) subsection).
- Figure 127 shows the backcalculated radius of relative stiffness, ℓ, values for basin testing for each of the three nominal load levels of 40.03, 53.38, and 71.17 kN (9,000, 12,000, and 16,000 lb).
- ℓ value based on 40.03 kN (9,000 lb) load basin testing
    - Average = 889 mm (35 in)
    - Standard Deviation = 203 mm (8 in)
- ℓ value at crack locations at mid-slab (morning only)
    - Average = 737 mm (29 in)
    - Standard Deviation = 76 mm (3 in)
- ℓ value at crack locations at edge (morning only)
    - Average = 813 mm (32 in)
    - Standard Deviation = 152 mm (6 in)

- Estimated ℓ value using actual slab thickness and concrete modulus of elasticity and assumed modulus of subgrade reaction
  - Value = 787 mm (31 in)
- Figure 128 shows the variation of ℓ in relation to crack spacing (average of the closest five cracks)
- Average deflection under 40.03 kN (9,000 lb) load (0.000001 mm (0.001 in))
  - Basin test - 0.12 (4.7)
  - Mid-slab crack location (morning) - 0.11 (4.3)
  - Edge crack location (morning) - 0.18 (7.2)

Crack Width Measurements
- Number of cracks monitored within the 30.5-m (100-ft) subsection = 18
- Morning testing
  - Average temperature 102 mm (4 in) below surface = 9.4°C (49°F)
  - Average crack width = 0.84 mm (0.033 in)
- Afternoon testing
  - Average temperature 102 mm (4 in) below surface = 10.0°C (50°F)
  - Average crack width = not measured
- Slab length change, mm/mm/°C (in/in/°F) = N/A

Reinforcing Steel Depths and Spacing
- Steel depth measurement offsets = 610, 1 219, 1 829 mm (24, 48, 72 in)
- Range of depth of cover = 53 to 114 mm (2.1 to 4.5 in)
- Average depth of cover = 71 mm (2.8 in)
- Standard deviation for depth of cover = 10 mm (0.4 in)
- Steel spacing was about 165 mm (6.5 in)

Corrosion Potential Testing (copper-copper sulfate half cell potentiometer)
- Range of measurements = -0.06 to -0.13 volts
- Average measurement = -0.09 volts
- Standard deviation for the measurement = -0.01 volts
- Potential for corrosion was not indicated

Concrete Core Examination for Corrosion
- No steel corrosion was detected in four cores obtained over steel bars

Profile Testing
- Profile testing was not performed

Laboratory Testing

The following data were obtained from the testing of the concrete, base/subbase, and subgrade materials:

Concrete		
Modulus of Elasticity	-	32 400 MPa (4,700,000 lbf/in^2)
Average Split Tensile Strength	-	3.10 MPa (450 lbf/in^2)
Split Tensile Strength Range	-	2.83 to 3.31 MPa (410 to 480 lbf/in^2)
Coefficient of Thermal Expansion	-	7.60 mm/mm/°C (4.22 in/in/°F)
Base		
AASHTO Classification	-	CTB
Subbase		
AASHTO Classification	-	N/A
Subgrade		
Liquid Limit	-	37 percent
Plastic Limit	-	19 percent
Percent Passing #200 Sieve	-	43.9 percent
AASHTO Classification	-	A-6

Chloride content testing was not conducted on this test section.

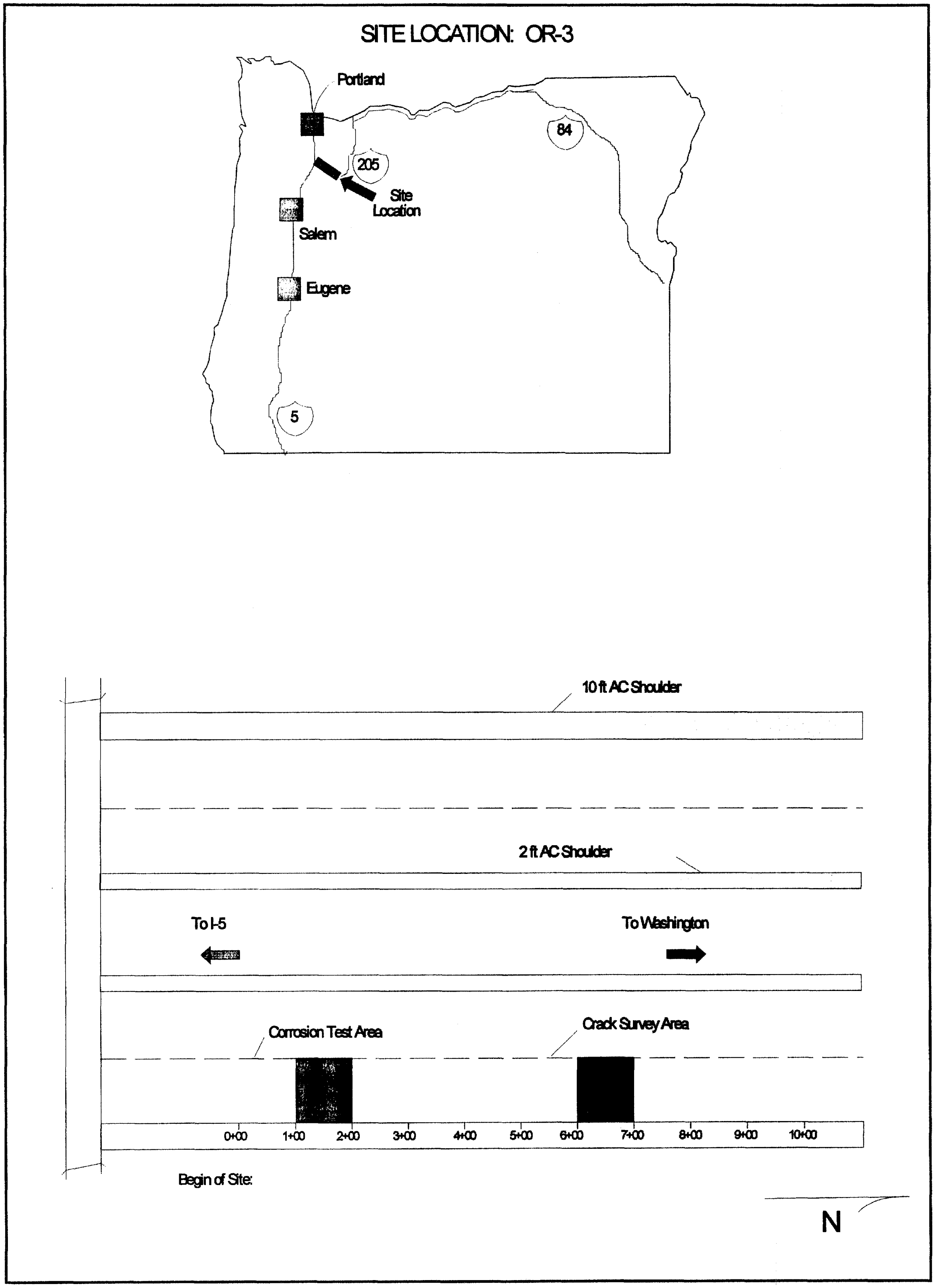
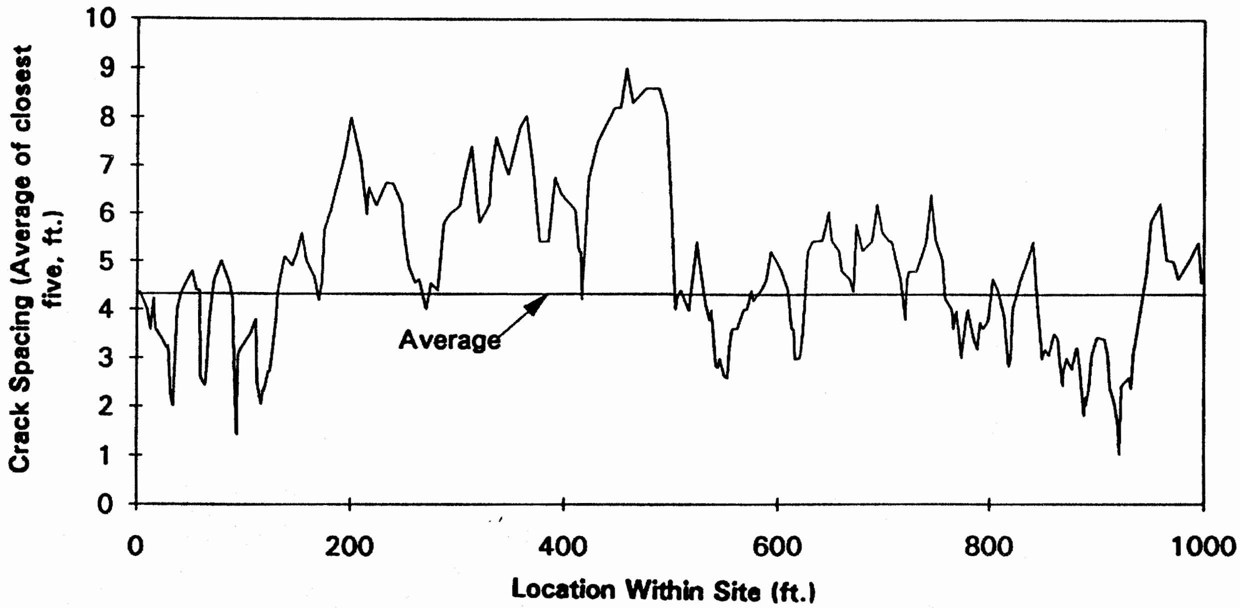


Figure 121. Site details for OR-3.



(0.305 m = 1 ft)

Figure 122. Crack spacing pattern at site OR-3.

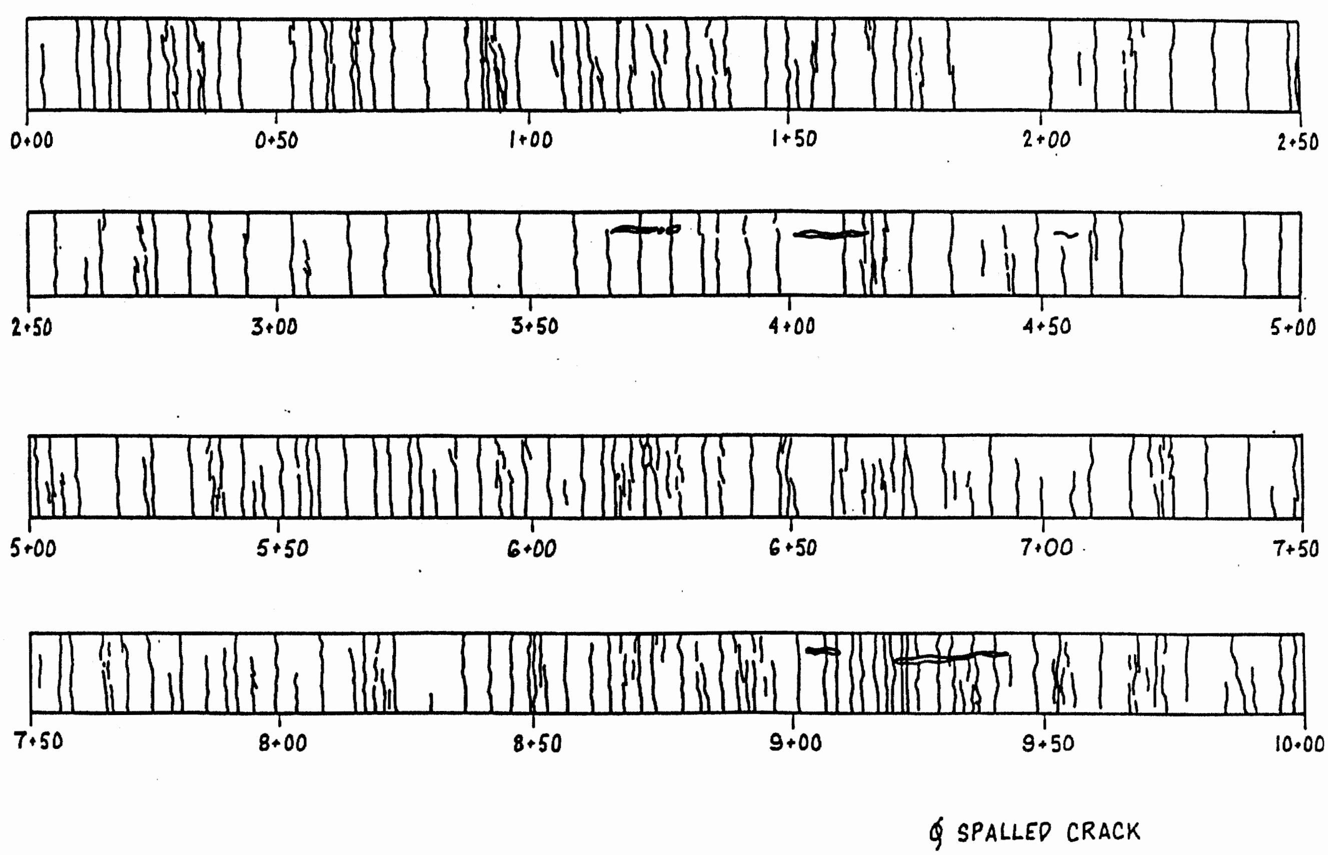
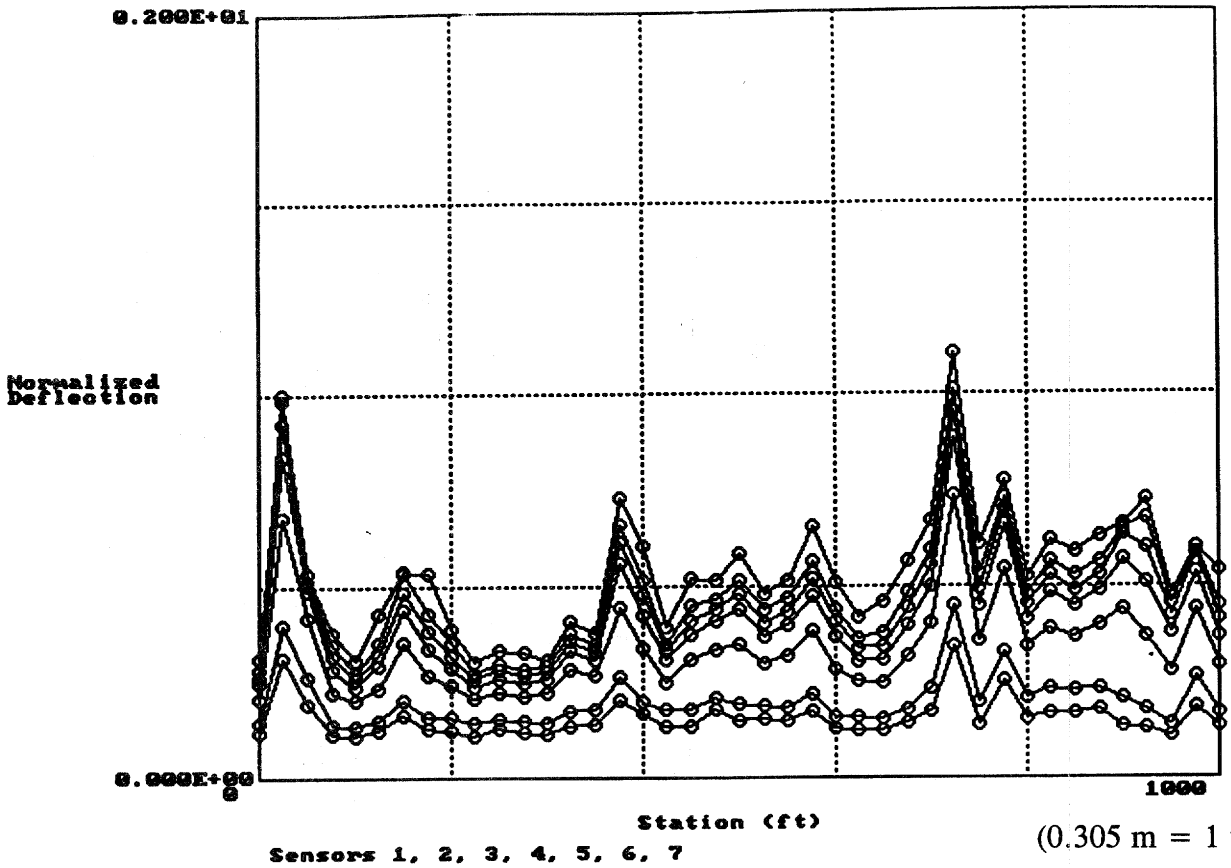
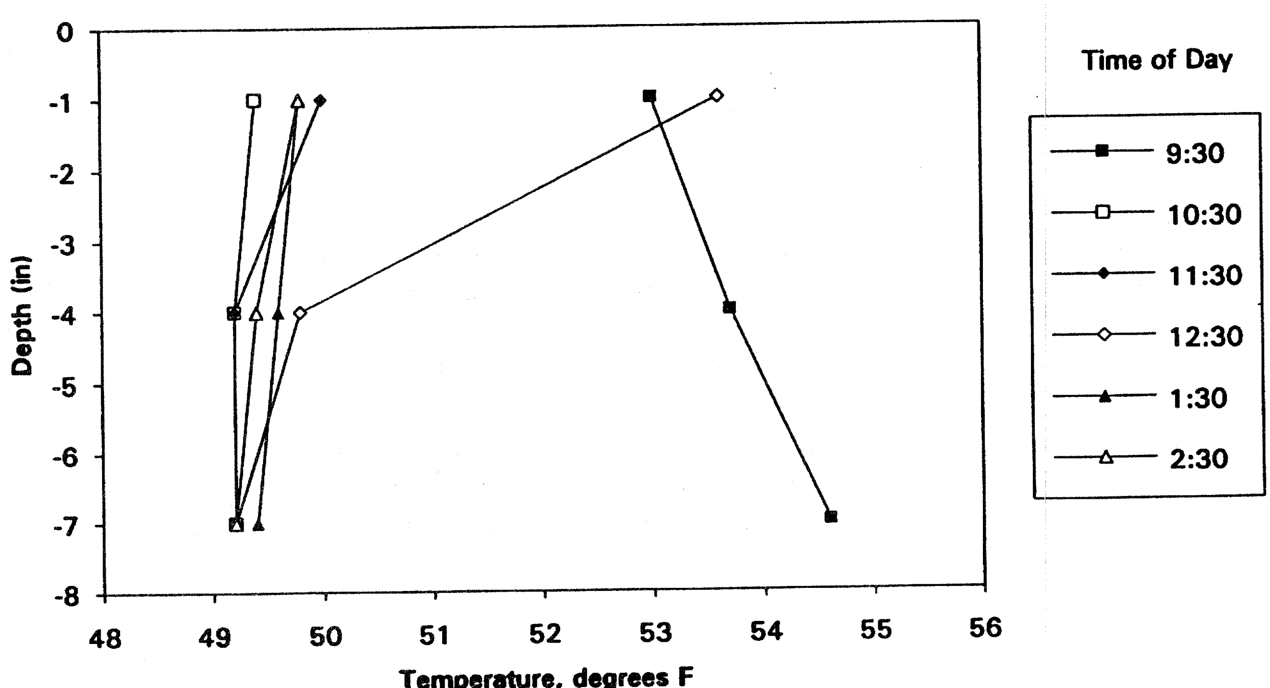


Figure 123. Crack pattern at site OR-3.



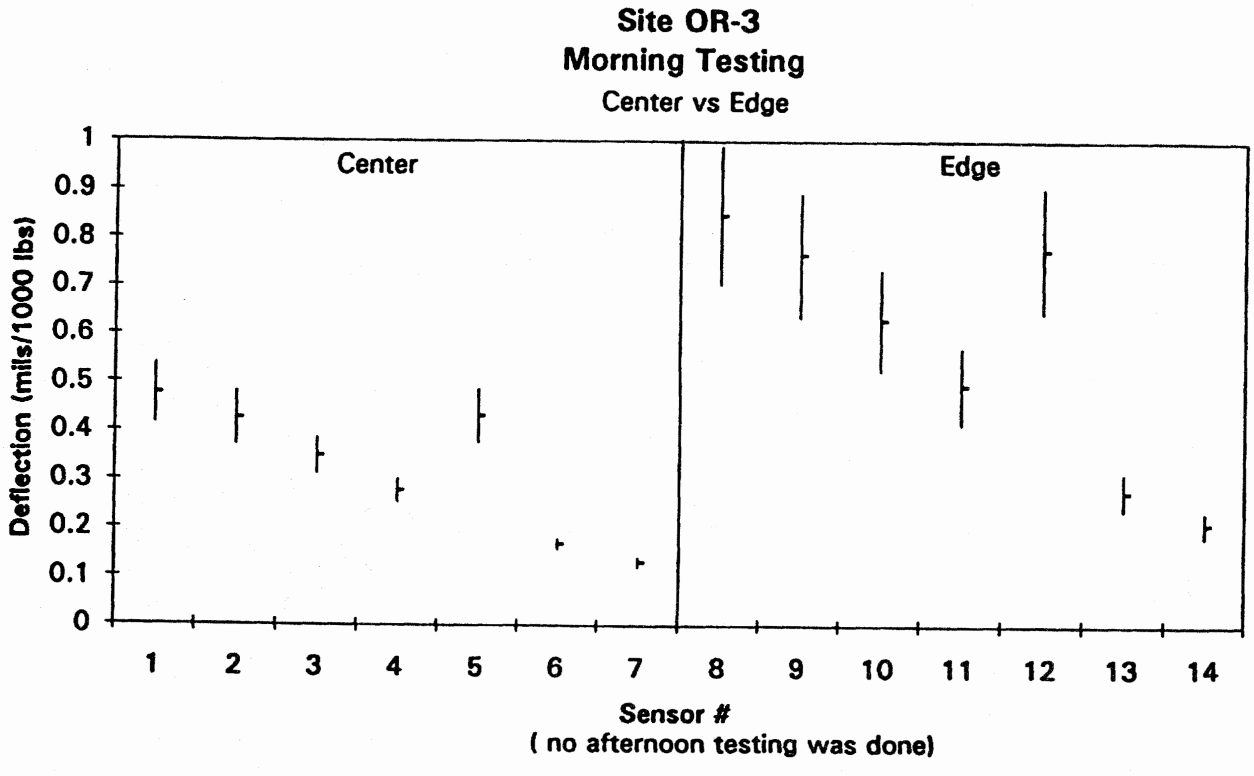
(0.305 m = 1 ft)

Figure 124. Deflections in inches along the section length at site OR-3.
[normalized per 4.45-kN [1,000-lb] load]



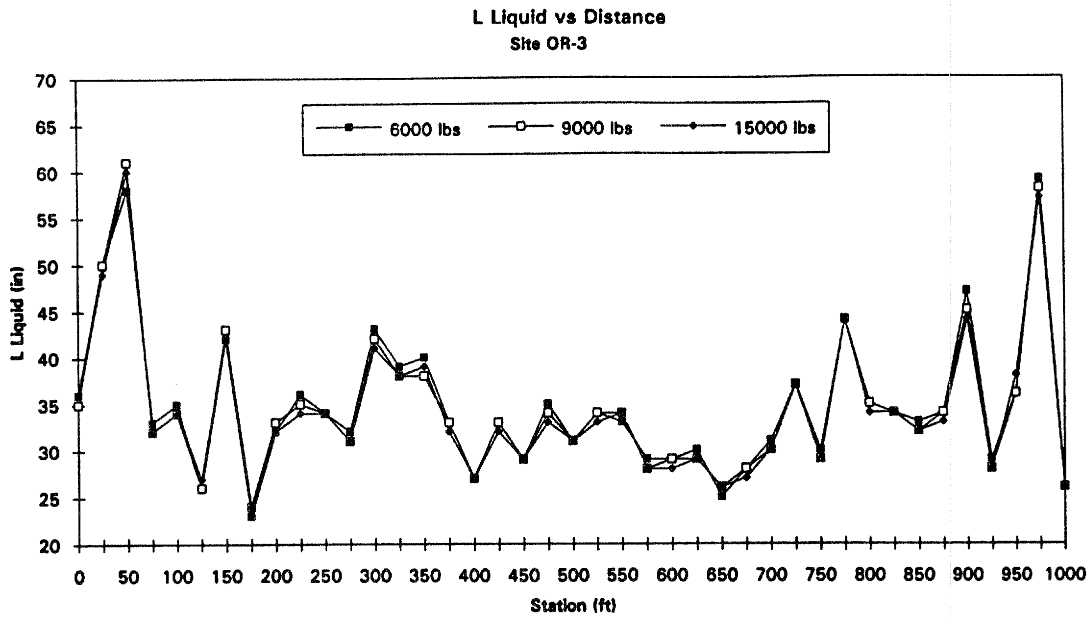
(25.4 mm = 1 in; 0.6°C = 1°F)

Figure 125. Temperature profile for site OR-3.



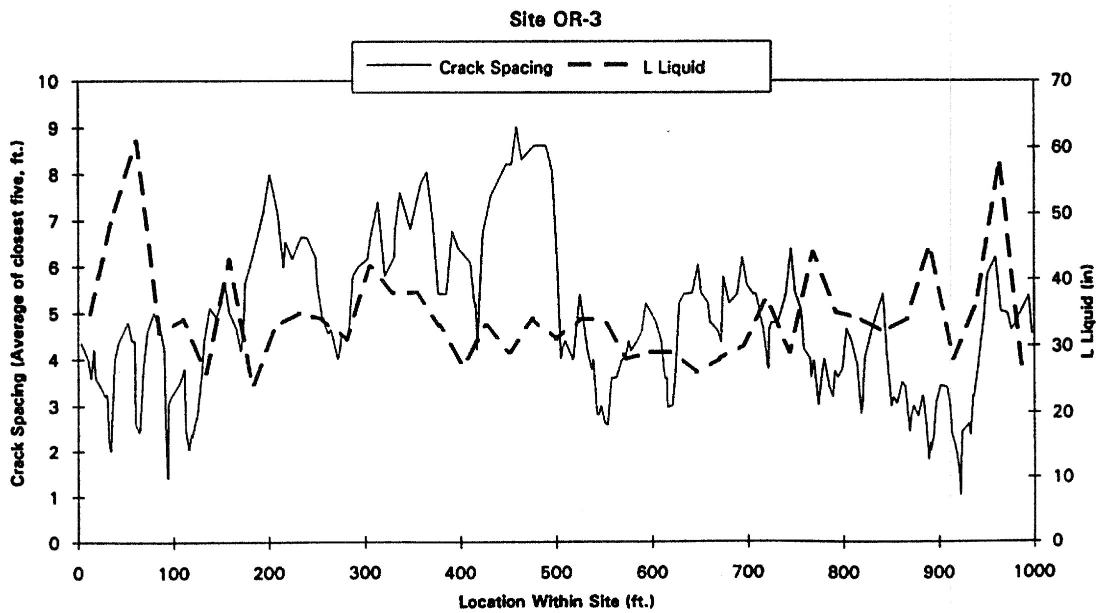
(0.0254 mm = 1 mil; 4.45 kN = 1,000 lb)

Figure 126. Center versus edge deflection for each of the seven sensors at site OR-3.
[40.03 kN (9,000 lb) load]



(25.4 mm = 1 in, 0.305 m = 1 ft)

Figure 127. Backcalculated values of radius of relative stiffness, ℓ (using program ILLI-BACK) for site OR-3.



(0.305 m = 1 ft)

Figure 128. Comparison of radius of relative stiffness values with average crack spacing (nearest five cracks) at site OR-3.

CHAPTER 7 - PENNSYLVANIA TEST SECTIONS

Section PA-1 - Interstate 180

Introduction

Section PA-1 was opened to traffic in 1976. PA-1 is a spur of I-180 that runs east to west from Williamsport to I-80 as shown in figure 129. The length of the highway that was considered runs from segment 370 to 380. A 305-m (1,000-ft) east bound section was selected for detailed inspection beginning at Segment 374, milepost 23, as shown in figure 129. The section of I-180 that was chosen is in the central region of Pennsylvania.

The climate in this region is a humid continental climate modified slightly by the Atlantic seaboard and the Great Lakes. The highest monthly average temperature is 22.5°C (72.5°F) during July, and the lowest monthly average is -3.2°C (26.2°F) during January. I-180 lies within the wet-freeze environmental zone.

Traffic

The 1985 ADT was 8,230 vpd, and the 1993 ADT was estimated to be 10,500 vpd with 11-percent trucks. The cumulative ESALs up to 1991 were estimated to be 5,500,000.

Structure

I-180 is a four-lane divided highway. The pavement lane width is 3.7 m (12 ft) with an outside shoulder width of 3.0 m (10 ft) and an inside shoulder width of 1.2 m (4 ft). The shoulders are constructed of asphaltic concrete.

I-380 was designed as a three-layer system comprised of a 229-mm (9-in) CRCP and a 305-mm (12-in) granular base over native subgrade. The CRC pavement was designed with a steel percentage of 0.45. From the site survey, it was found that the steel was tube feed and consisted of #5 bars spaced at 191 mm (7.5 in) as a single layer. The bars are not epoxy coated.

The average concrete pavement thickness for PA-1 was determined to be 231 mm (9.1 in) based on cores taken as part of this study. The concrete is considered well graded and well consolidated. The aggregate type used was a crushed gravel with a maximum size of 38 mm (1½ in).

The unbound base material was found to be well graded and was 356 mm (14 in) thick. It had an AASHTO classification of A-2-4. The subgrade also had an AASHTO classification of A-2-6. The Atterberg Limits of the subgrade was a liquid limit of 21 and a plastic limit of 14.

Design/Construction

No design information was available. The terminal joint design chosen was a wide flange beam system.

Performance

No performance data were available.

Historical Data (M&R)

No historical data on traffic were available.

Field Investigation Data

Visual Condition Survey

- Total number of cracks = 245
  - 7 were at high severity
  - 238 were at medium severity
  - 0 were at low severity
- Average crack spacing = 1.47 m (4.81 ft)
- Standard deviation for crack spacing = 0.81 m (2.67 ft)
- Coefficient of variation of the crack spacing = 55 percent
- Figure 130 shows average crack spacing distribution based on the closest five cracks
- Figure 131 shows actual crack spacing
- Extent of Y cracks = 7 percent
- Cluster cracking was apparent at stations 0+00, 6+00, and 7+00
- No other distress types were present
- Pavement shoulders were in poor condition.

Drainage Survey

- No information was collected

Windshield Survey

- 8.05 km (5 mi) of pavement surveyed was generally in good condition exhibiting about 244 m (800 ft) of longitudinal cracking condition.

Terminal Joint Survey

- Terminal joints used = Wide flange beam (two surveyed)
- Joints exhibited the following distresses:
  - Low severity AC patch deterioration
  - Faulting
  - Spalling

Deflection Testing

- Figure 132 shows normalized/basin (per 4.45 kN (1,000 lb) load) deflections measured along the length of the section (for all seven sensors using the 40.03 kN (9,000 lb) load)
- Load transfer efficiency at cracks at mid-slab
  - 82 to 90 percent in the morning
  - No testing in the afternoon
- Load transfer efficiency at cracks along edge
  - 82 to 89 percent in the morning
  - No testing in the afternoon
- Figure 133 shows the distribution of slab temperature with depth and time of day.
- Figure 134 shows the average deflections and ranges of deflections for each sensor for center and edge loading, for morning and afternoon testing (using the 40.03 kN (9,000 lb) load within the selected 30.5-m (100-ft) subsection).
- Figure 135 shows the backcalculated radius of relative stiffness, ℓ, values for basin testing for each of the three nominal load levels of 40.03, 53.38, and 71.17 kN (9,000, 12,000, and 16,000 lb).
- ℓ value based on 40.03 kN (9,000 lb) load basin testing
  - Average = 635 mm (25 in)
  - Standard Deviation = 76 mm (3 in)
- ℓ value at crack locations at mid-slab (morning/afternoon)
  - Average = 508/508 mm (20/20 in)
  - Standard Deviation = 51/25 mm (2/1 in)
- ℓ value at crack locations at edge (morning only)
  - Average = 508 mm (20 in)
  - Standard Deviation = 51 mm (2 in)
- Estimated ℓ value using actual slab thickness and concrete modulus of elasticity and assumed modulus of subgrade reaction
  - Value = 864 mm (34 in)
- Figure 136 shows the variation of ℓ in relation to crack spacing (average of the closest five cracks)
- Average deflection under 40.03 kN (9,000 lb) load (0.000001 mm (0.001 in))
  - Basin test - 0.056 (2.2)
  - Mid-slab crack location (morning) - 0.058 (2.3)
  - Edge crack location (morning) - 0.112 (4.4)

Crack Width Measurements

- Number of cracks monitored within the 30.5-m (100-ft) subsection = 19
- Morning testing
  - Average temperature 102 mm (4 in) below surface = 5.6°C (42°F)
  - Average crack width = 1.55 mm (0.061 in)

- Afternoon testing
    - Not conducted because of little temperature changes and traffic control concerns
- Slab length change, mm/mm/°C (in/in/°F) = N/A

Reinforcing Steel Depths and Spacing

- Steel depth measurement offsets = 610, 1219, 1829 mm (24, 48, 72 in)
- Range of depth of cover = 64 to 102 mm (2.5 to 4.0 in)
- Average depth of cover = 86 mm (3.4 in)
- Standard deviation for depth of cover = 5.1 mm (0.2 in)
- Steel spacing was about 203 mm (8.0 in)

Corrosion Potential Testing (copper-copper sulfate half cell potentiometer)

- Range of measurements = -0.20 to -0.35 volts
- Average measurement = -0.25 volts
- Standard deviation for the measurement = -0.02 volts
- Potential for corrosion was marginal

Concrete Core Examination for Corrosion

- No steel corrosion was detected in two cores
- Two cores exhibited spread corrosion

Profile Testing

- Average IRI over the 305-m (1,000-ft) section = 1 185 mm/km (75 in/mi)

Laboratory Testing

The following data were obtained from the testing of the concrete, base/subbase, and subgrade materials:

Concrete

Modulus of Elasticity	-	28 960 MPa (4,200,000 lbf/in^2)
Average Split Tensile Strength	-	3.31 MPa (480 lbf/in^2)
Split Tensile Strength Range	-	3.10 to 3.45 MPa (450 to 500 lbf/in^2)
Coefficient of Thermal Expansion	-	8.42 mm/mm/°C (4.68 in/in/°F)

Base

AASHTO Classification	-	Granular (A-2-4)

Subbase

AASHTO Classification	-	N/A

Subgrade

Liquid Limit	-	21 percent

Plastic Limit - 14 percent
Percent Passing #200 Sieve - 17.6 percent
AASHTO Classification - A-2-4

Chloride content testing was conducted on pulverized concrete retrieved at various depths and locations. Total chloride content and water soluble chloride content values are listed below for PA-1.

Location	Depth, mm (in)	Total Chloride Content (% wt.)	Water Soluble Chlorides (% wt.)
No steel below sample	0-51 (0-2)	0.197	0.193
	51-102 (2-4)	0.042	0.031
Steel directly below sample	0-51 (0-2)	0.226	0.181
	51-102 (2-4)	0.046	0.039

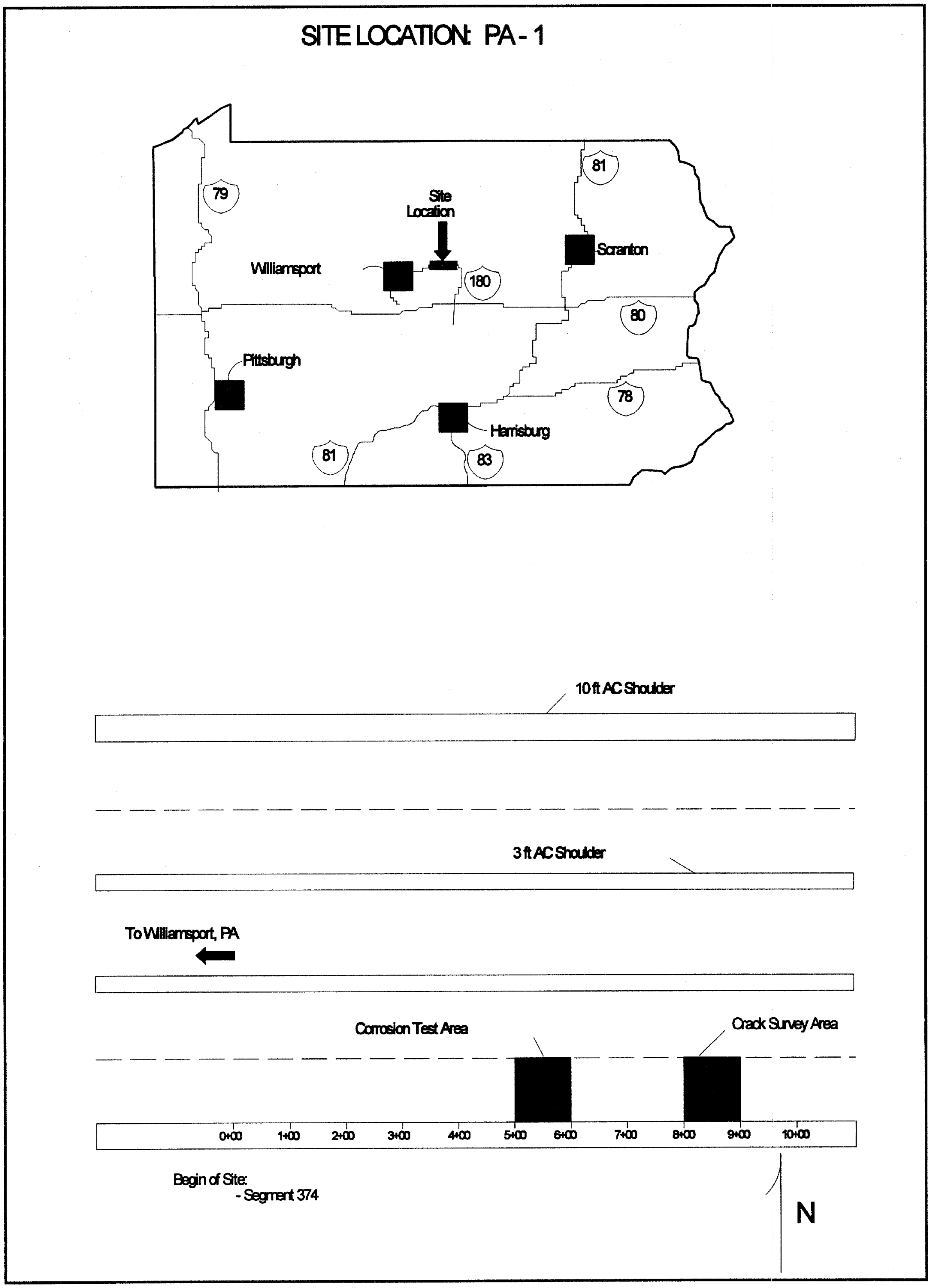
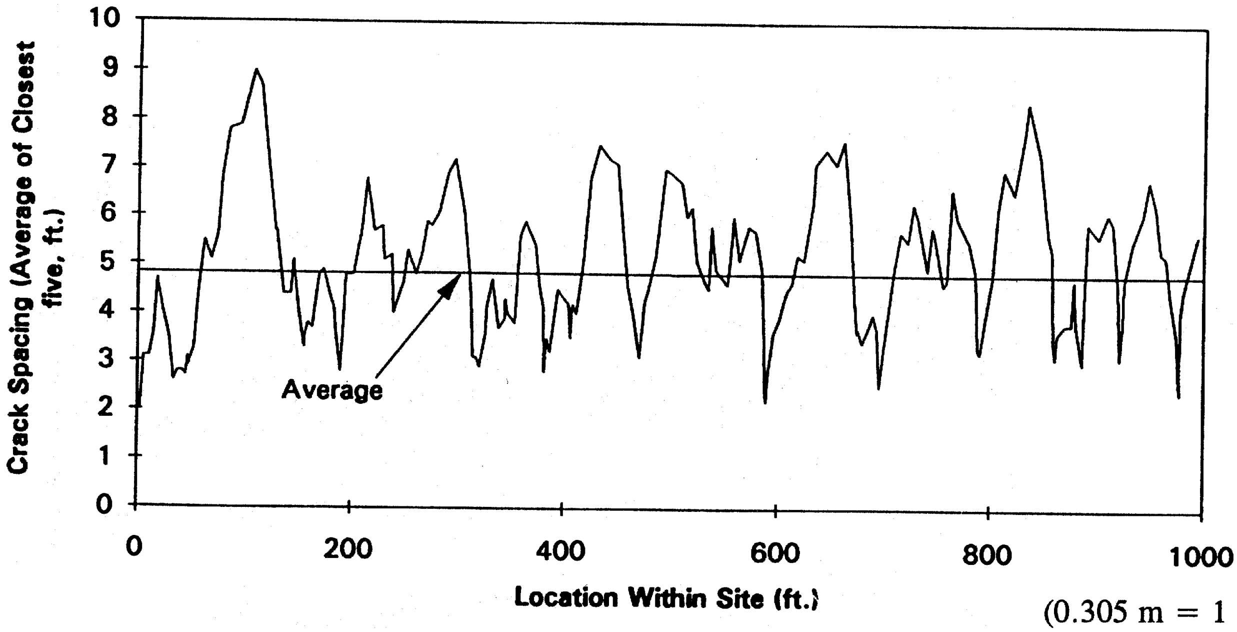


Figure 129. Site details for PA-1.



(0.305 m = 1 ft)

Figure 130. Crack spacing pattern at site PA-1.

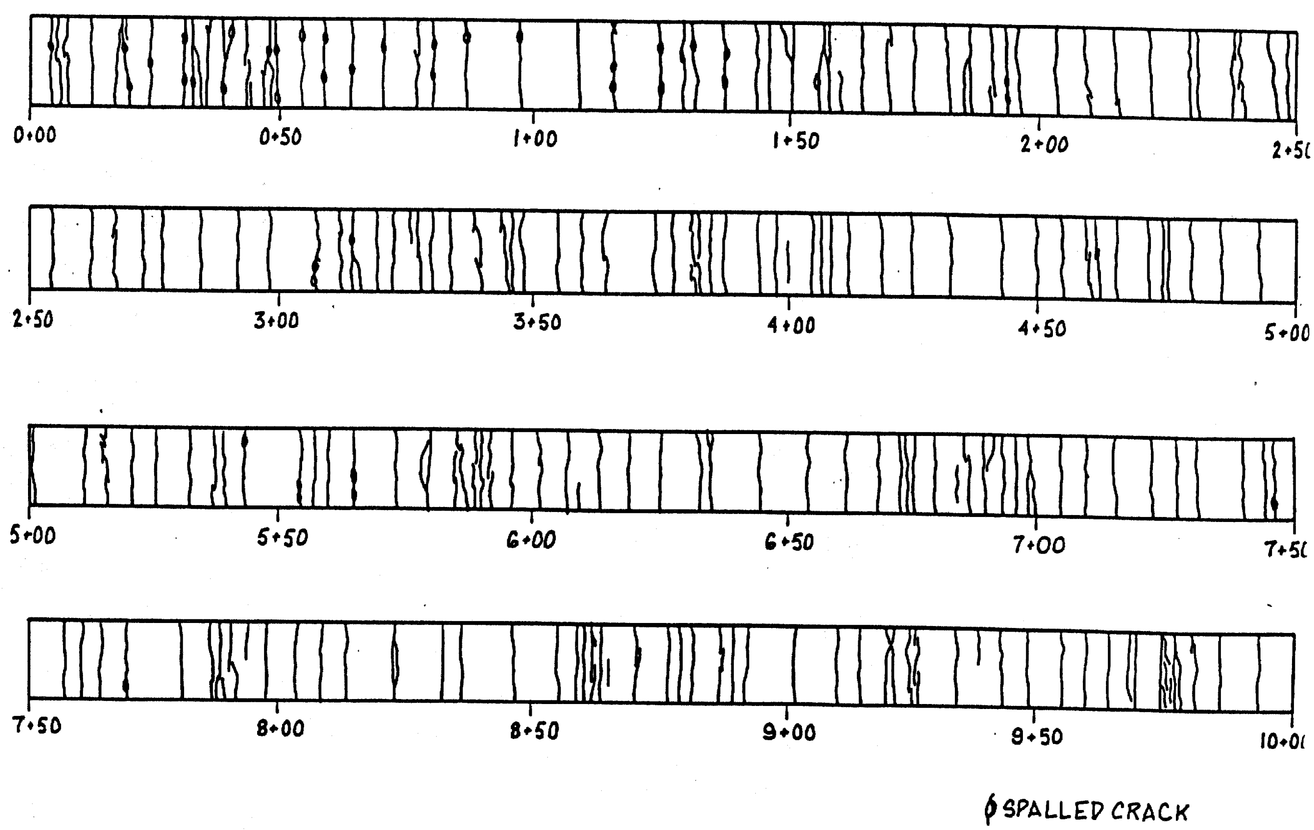
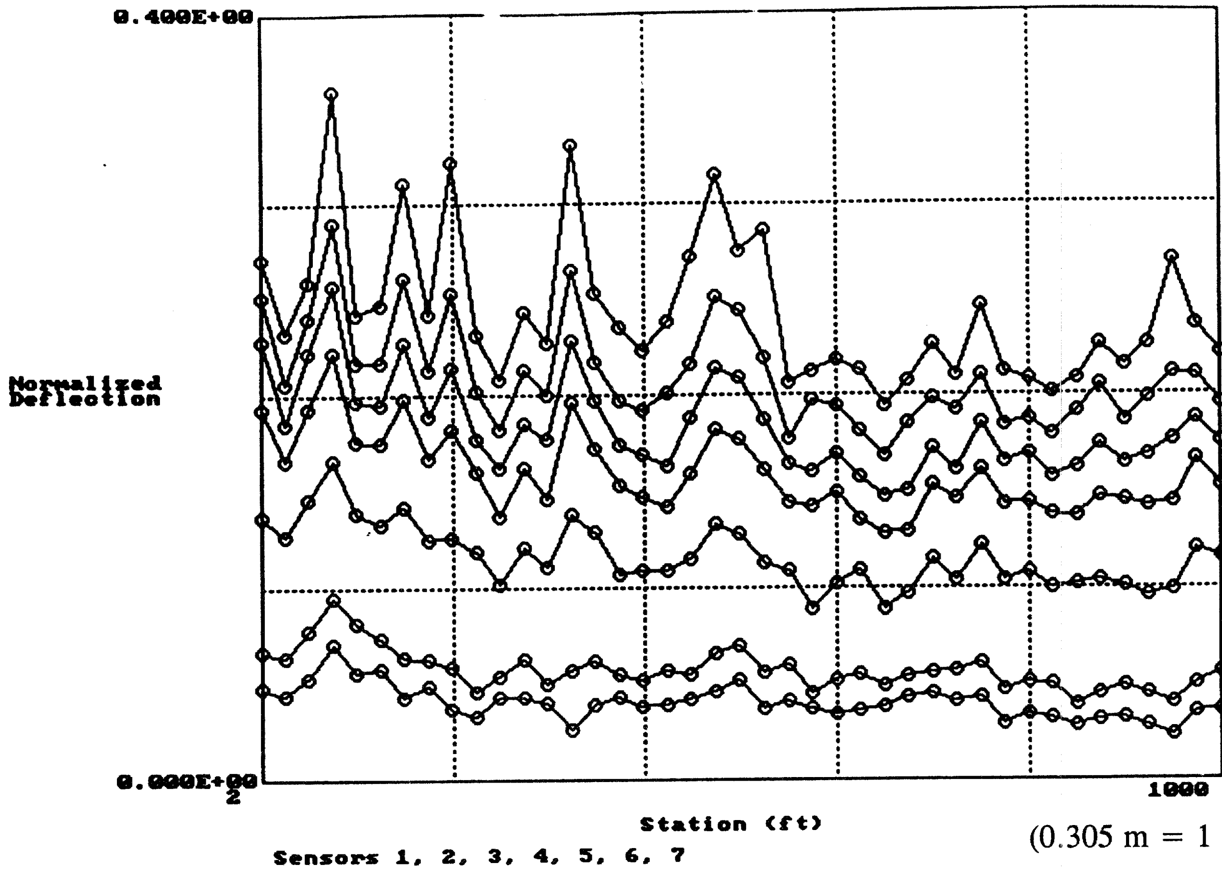
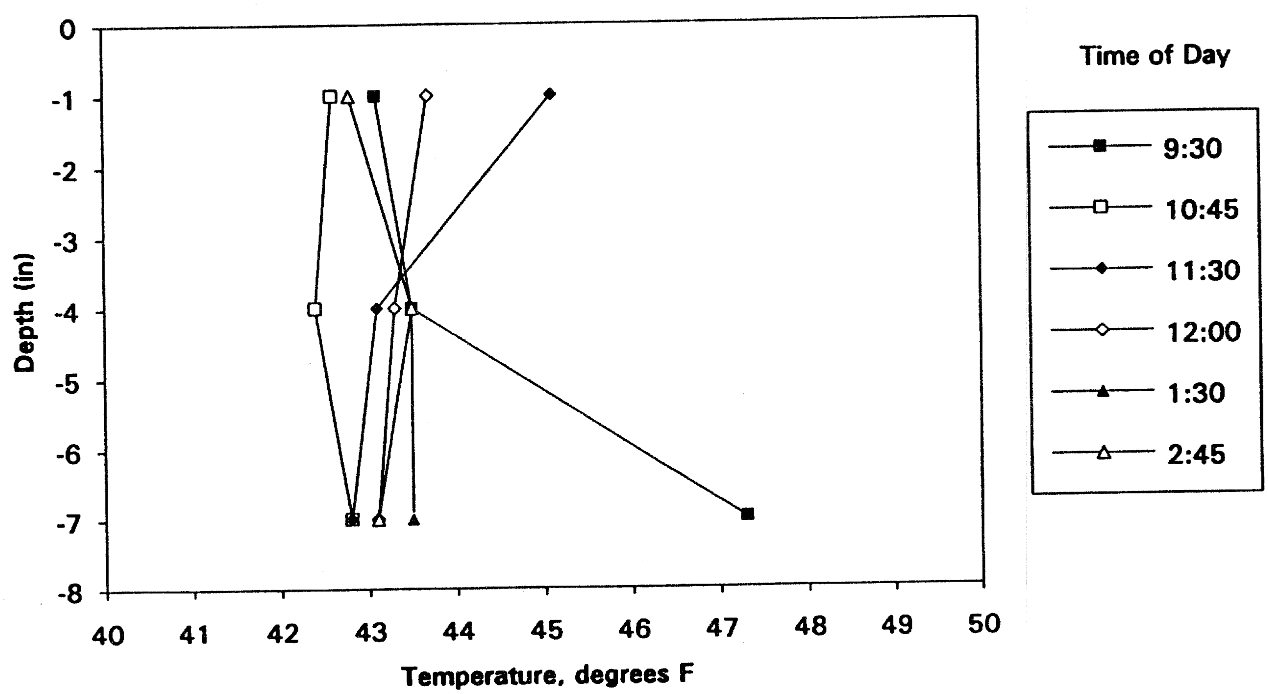


Figure 131. Crack pattern at site PA-1.



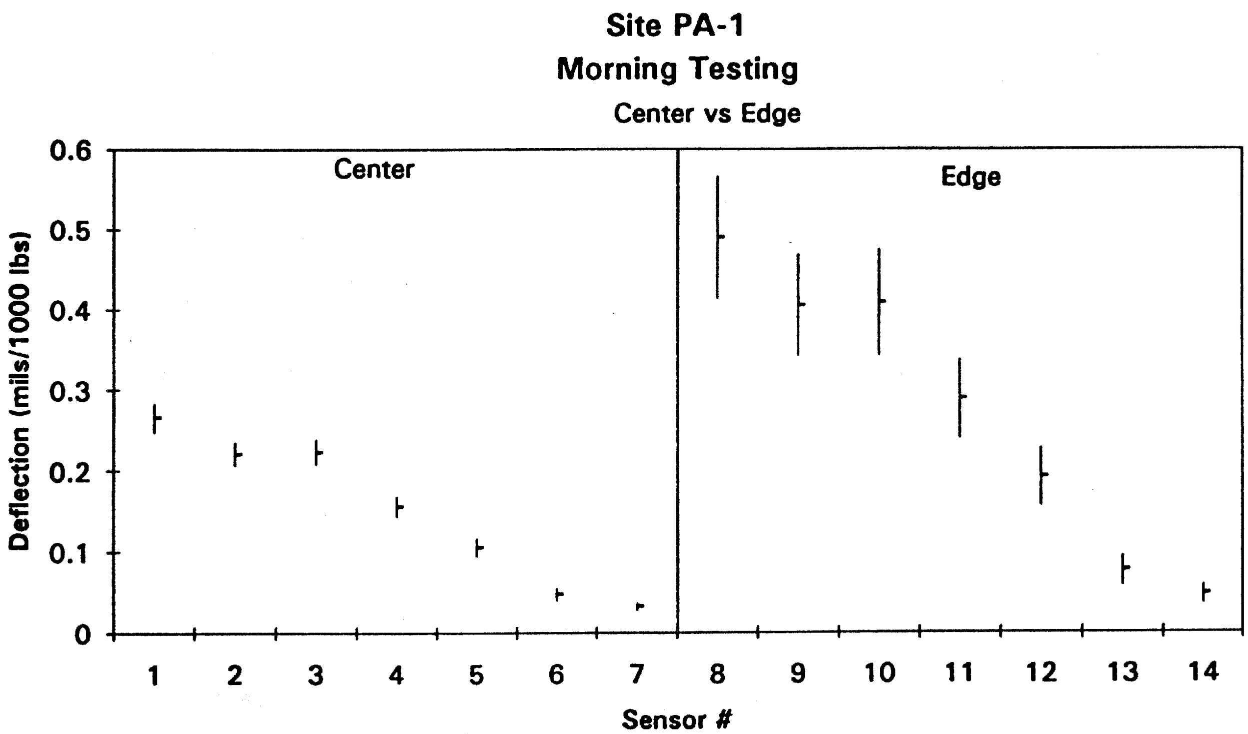
(0.305 m = 1 ft)

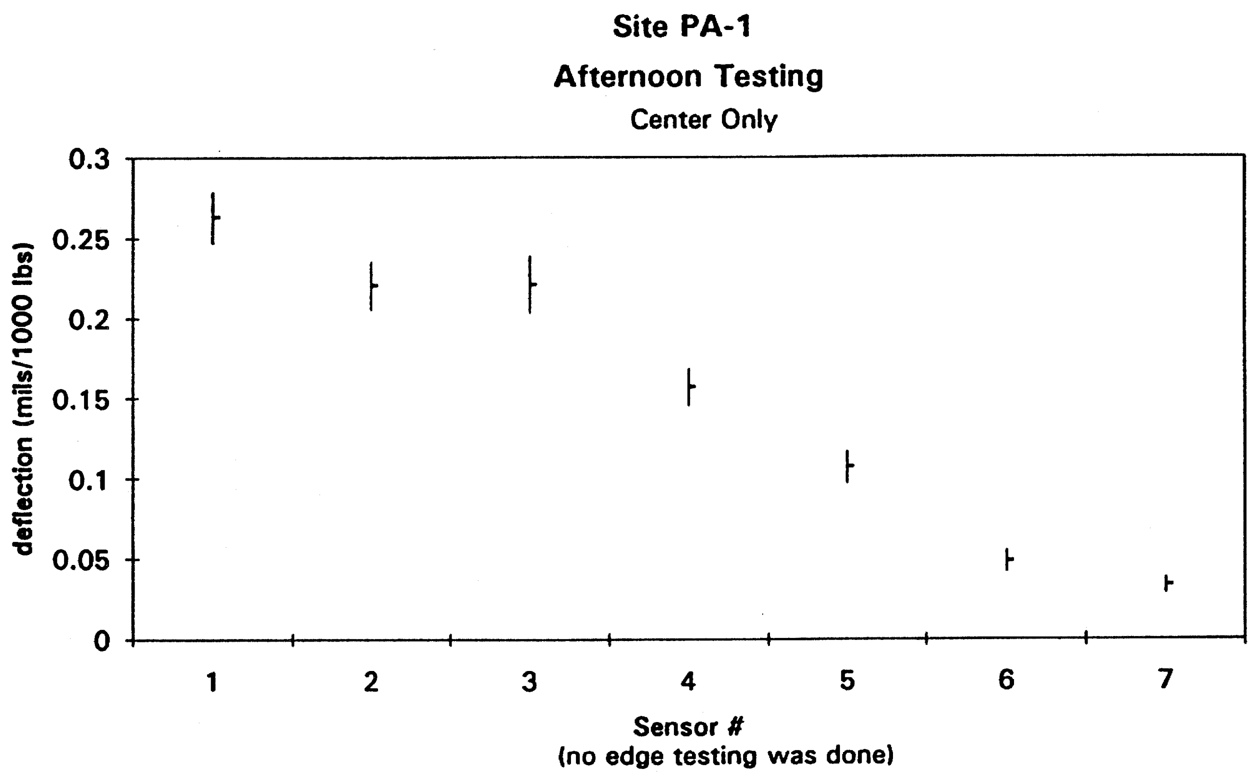
Figure 132. Deflections in inches along the section length at site PA-1.
[normalized per 4.45-kN [1,000-lb] load]



(25.4 mm = 1 in; 0.6°C = 1°F)

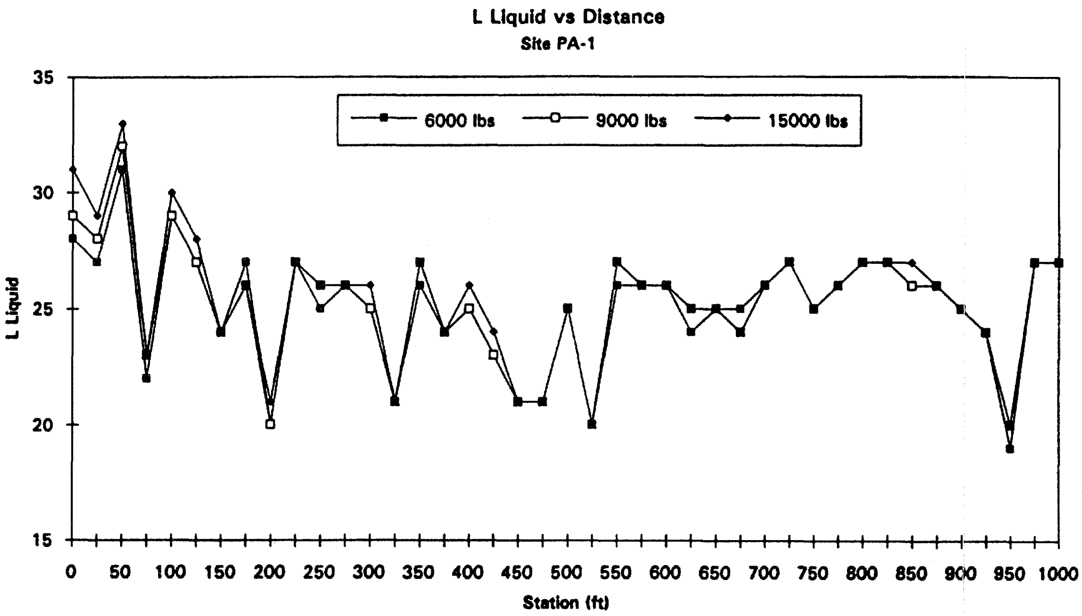
Figure 133. Temperature profile for site PA-1.





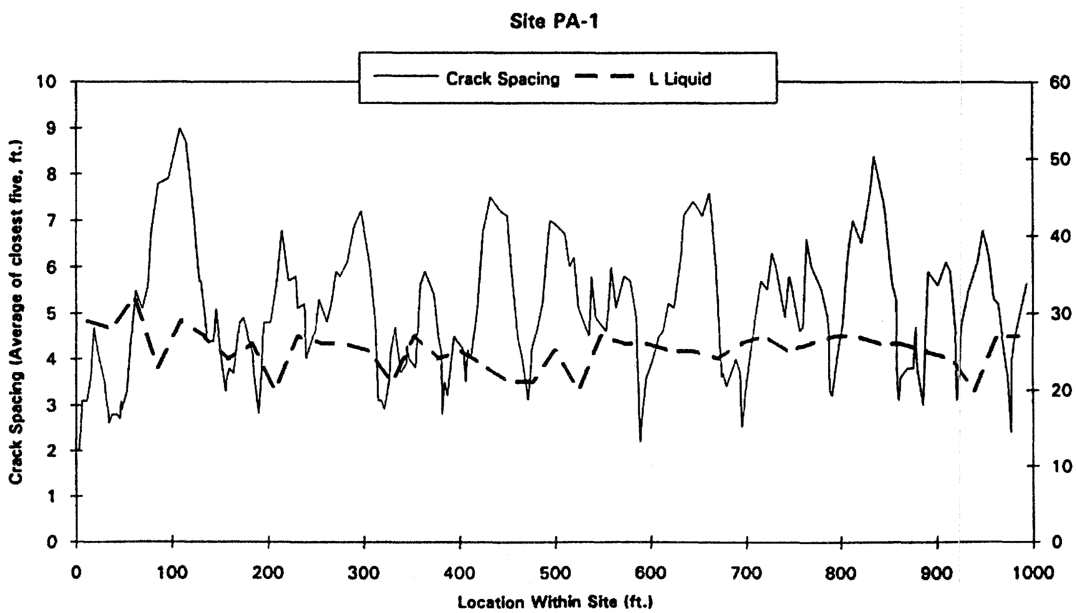
(0.0254 mm = 1 mil; 4.45 kN = 1,000 lb)

Figure 134. Center versus edge deflection for each of the seven sensors at site PA-1.
[40.03 kN (9,000 lb) load]



(25.4 mm = 1 in, 0.305 m = 1 ft)

Figure 135. Backcalculated values of radius of relative stiffness, ℓ (using program ILLI-BACK) for site PA-1.



(0.305 m = 1 ft)

Figure 136. Comparison of radius of relative stiffness values with average crack spacing (nearest five cracks) at site PA-1.

Section PA-2 - Interstate 81

Introduction

Section PA-2 was opened to traffic in 1969. PA-2 is a section of I-81 that runs north to south from Harrisburg to the Pennsylvania-Maryland border, as shown in figure 137. The length of the highway that was considered runs from segment 524 to 525. A 305-m (1,000-ft) east bound section was selected for detailed inspection beginning at Segment 524, milepost 52, as shown in figure 137. The section of I-81 that was chosen is in the southern region of Pennsylvania.

The climate in this region does not get the full benefits of the coastal climate. The highest monthly average temperature is 24.3°C (75.8°F) during July, and the lowest monthly average is -1.4°C (29.4°F) during January. I-81 lies within the wet-freeze environmental zone.

Traffic

The 1990 ADT was 13,660 vpd with about 30 percent trucks. The cumulative ESALs up to 1991 were estimated to be 32,000,000.

Structure

I-81 is a four-lane divided highway. The pavement lane width is 3.7 m (12 ft) with an outside shoulder width of 3.0 m (10 ft) and an inside shoulder width of 1.2 m (4 ft). <u>The shoulders are constructed of PCC.</u>

I-81 was designed as a three-layer system comprised of a 305-mm (12-in) CRCP and a <u>127 mm (5 in) LCB</u> over a subgrade of borrow material. The CRC pavement was designed with a steel percentage of 0.55. From the site survey though, it was found that the steel was placed on chairs. The transverse bars are #5 bars spaced at 140 mm (5.5 in). The longitudinal bars are #4 spaced at 1 245 mm (49 in). The bars are not epoxy coated.

The average concrete pavement thickness for PA-2 was determined to be 241 mm (9.5 in) based on cores taken as part of this study. The concrete is considered well graded and its consolidation is average. The aggregate type used was a crushed gravel with a maximum size of 38 mm (1½ in).

The LCB was 241 mm (9.5 in) thick. The subgrade had an AASHTO classification of A-2-4. The Atterberg Limits of the subgrade were a liquid limit of 33 and a plastic limit of 23.

Design/Construction

No design information was available for this section. The terminal joint design chosen was a lug anchor system.

Performance

No performance information was available for this section.

Historical Data (M&R)

No historical data were available for this section.

Field Investigation Data

Visual Condition Survey

- Total number of cracks = 256
  - 0 were at high severity
  - 256 were at medium severity
  - 0 were at low severity
- Average crack spacing = 1.32 m (4.33 ft)
- Standard deviation for crack spacing = 0.75 m (2.47 ft)
- Coefficient of variation of the crack spacing = 57 percent
- Figure 138 shows average crack spacing distribution based on the closest five cracks
- Figure 139 shows actual crack spacing
- Extent of Y cracks = 4 percent
- Cluster cracking was apparent at stations 1+00, 5+20, and 6+20
- Other distress types included:
  - 6.1 m (20 ft) longitudinal cracking (moderate severity)
  - Extensive map cracking, scaling, and polished aggregates
  - 3 punchouts (low severity)
  - 1 PCC patch (low severity)
- Pavement shoulders were in fair condition.

Drainage Survey

- No information was collected

Windshield Survey

- 8.05 km (5 mi) of pavement surveyed was generally in fair condition exhibiting about 26 m (85 ft) of longitudinal cracking condition, 11 PCC patches, 6 AC patches, and 36 punchouts.

Terminal Joint Survey

- Terminal joints used = Lug anchor (two surveyed)

- Joints exhibited the following distresses:
  - Both exhibited 3.3 m^2 (36 ft^2) of low to medium severity AC patch deterioration
  - One exhibited 2.2 m^2 (24 ft^2) of medium severity spalling

Deflection Testing

- Figure 140 shows normalized/basin (per 4.45 kN (1,000 lb) load) deflections measured along the length of the section (for all seven sensors using the 40.03 kN (9,000 lb) load)
- Load transfer efficiency at cracks at mid-slab
  - 90 to 96 percent in the morning
  - 92 to 96 percent in the afternoon
- Load transfer efficiency at cracks along edge
  - 89 to 93 percent in the morning
  - 87 to 94 percent in the afternoon
- Figure 141 shows the distribution of slab temperature with depth and time of day.
- Figure 142 shows the average deflections and ranges of deflections for each sensor for center and edge loading, for morning and afternoon testing (using the 40.03 kN (9,000 lb) load within the selected 30.5-m (100-ft) subsection).
- Figure 143 shows the backcalculated radius of relative stiffness, ℓ, values for basin testing for each of the three nominal load levels of 40.03, 53.38, and 71.17 kN (9,000, 12,000, and 16,000 lb).
- ℓ value based on 40.03 kN (9,000 lb) load basin testing
  - Average = 1 067 mm (42 in)
  - Standard Deviation = 305-mm (12 in)
- ℓ value at crack locations at mid-slab (morning/afternoon)
  - Average = 711/940 mm (28/37 in)
  - Standard Deviation = 127/254 mm (5/10 in)
- ℓ value at crack locations at edge (morning/afternoon)
  - Average = 584/660 mm (23/26 in)
  - Standard Deviation = 51/102 mm (2/4 in)
- Estimated ℓ value using actual slab thickness and concrete modulus of elasticity and assumed modulus of subgrade reaction
  - Value = 838 mm (33 in)
- Figure 144 shows the variation of ℓ in relation to crack spacing (average of the closest five cracks)
- Average deflection under 40.03 kN (9,000 lb) load (0.000001 mm (0.001 in))
  - Basin test - 0.13 (5.3)
  - Mid-slab crack location (morning/afternoon) - 0.11/0.14 (4.5/5.5)
  - Edge crack location (morning/afternoon) - 0.17/0.15 (6.5/5.8)

Crack Width Measurements

- Number of cracks monitored within the 30.5-m (100-ft) subsection = 24
- Morning testing
    - Average temperature 102 mm (4 in) below surface = 8.3°C (47°F)
    - Average crack width = 2.16 mm (0.085 in)
- Afternoon testing
    - Average temperature 102 mm (4 in) below surface = 14.4°C (58°F)
    - Average crack width = 2.11 mm (0.083 in)
- Slab length change, mm/mm/°C (in/in/°F) = 6.5 millionth (3.6 millionth)

Reinforcing Steel Depths and Spacing

- Steel depth measurement offsets = 610, 1 219, 1 829 mm (24, 48, 72 in)
- Range of depth of cover = 56 to 76 mm (2.2 to 3.0 in)
- Average depth of cover = 69 mm (2.7 in)
- Standard deviation for depth of cover = 2.5 mm (0.1 in)
- Steel spacing was about 145 mm (5.7 in)

Corrosion Potential Testing (copper-copper sulfate half cell potentiometer)

- Range of measurements = -0.29 to -0.60 volts
- Average measurement = -0.37 volts
- Standard deviation for the measurement = -0.05 volts
- Potential for corrosion was indicated

Concrete Core Examination for Corrosion

- All four cores exhibited spread corrosion

Profile Testing

- Average IRI over the 305-m (1,000-ft) section = 1 185 mm/km (75 in/mi)

Laboratory Testing

The following data were obtained from the testing of the concrete, base/subbase, and subgrade materials:

Concrete

Modulus of Elasticity	- 33 780 MPa (4,900,000 lbf/in^2)
Average Split Tensile Strength	- 3.79 MPa (550 lbf/in^2)
Split Tensile Strength Range	- 3.03 to 4.76 MPa (440 to 690 lbf/in^2)
Coefficient of Thermal Expansion	- 7.38 mm/mm/°C (4.10 in/in/°F)

Base

AASHTO Classification - LCB

Subbase

AASHTO Classification - N/A

Subgrade

Liquid Limit - 33 percent
Plastic Limit - 23 percent
Percent Passing #200 Sieve - 24.0 percent
AASHTO Classification - A-2-4

Chloride content testing was conducted on pulverized concrete retrieved at various depths and locations. Total chloride content and water soluble chloride content values are listed below for PA-2.

Location	Depth, mm (in)	Total Chloride Content (% wt.)	Water Soluble Chlorides (% wt.)
No steel below sample	0-51 (0-2)	0.160	0.146
	51-102 (2-4)	0.053	0.020
Steel directly below sample	0-51 (0-2)	0.081	0.069

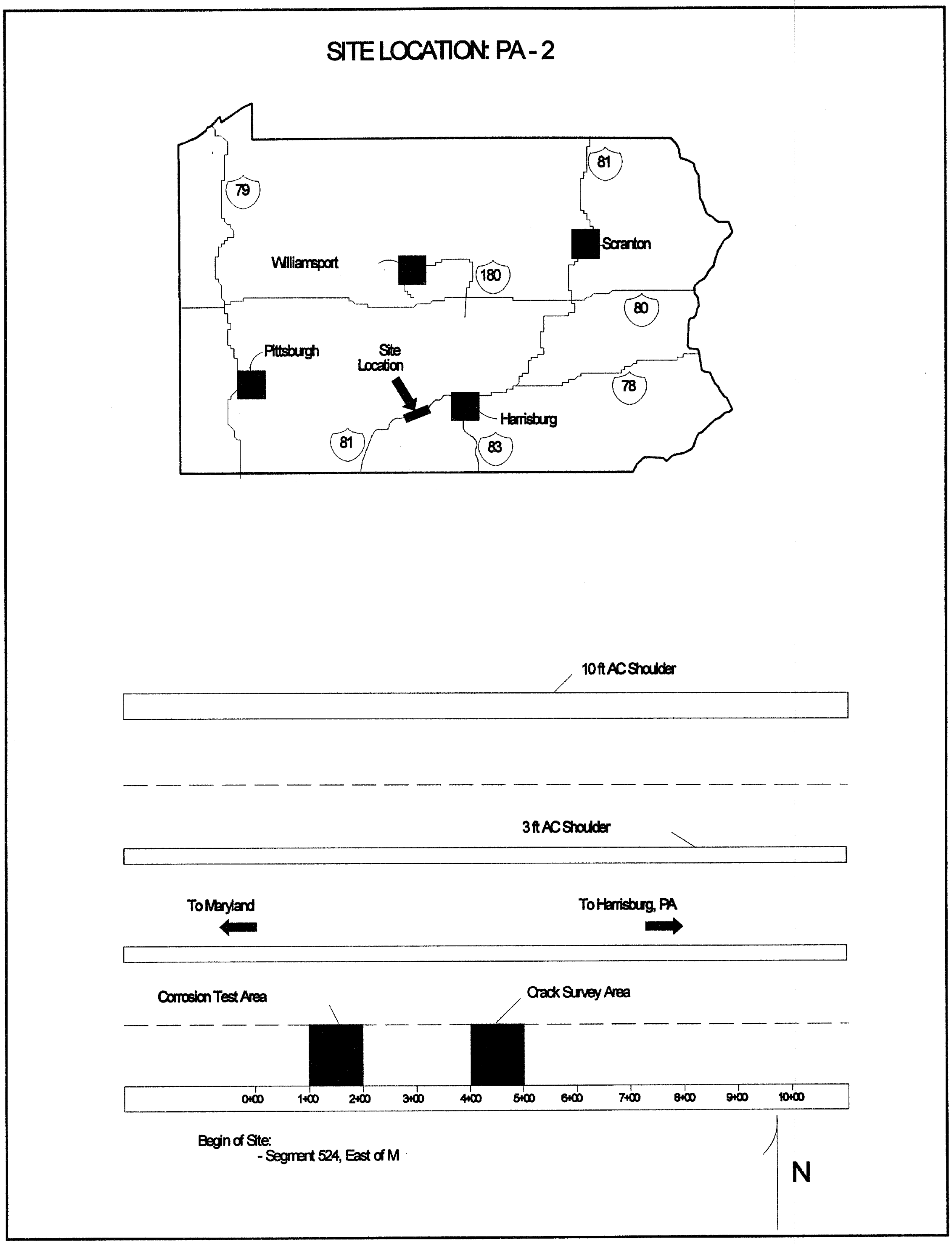
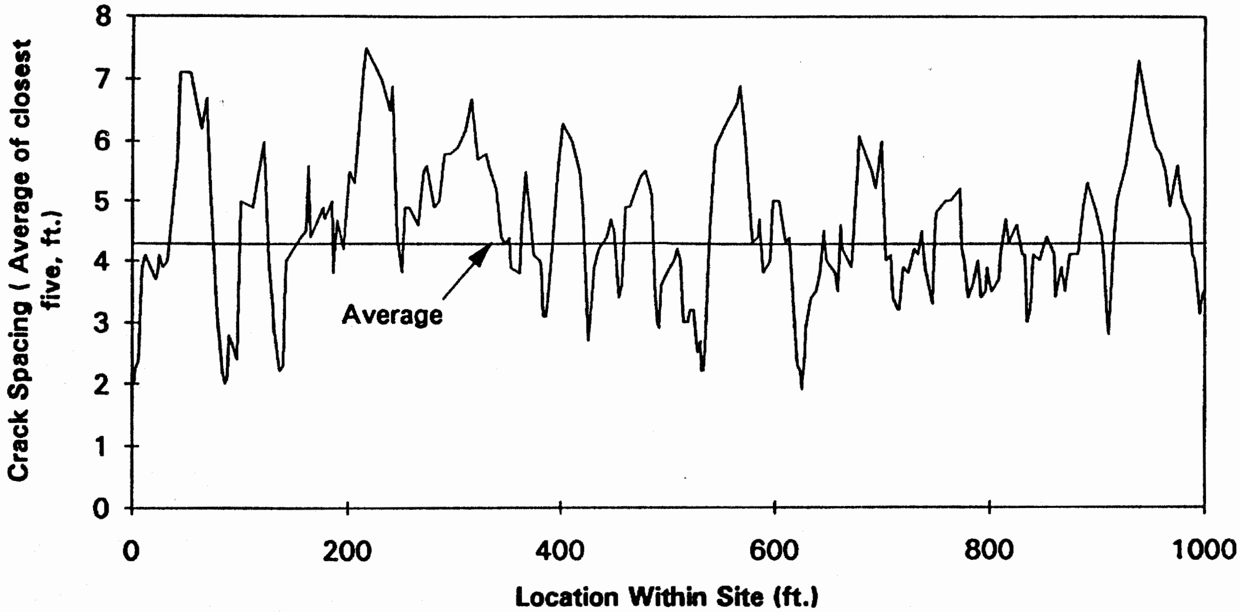


Figure 137. Site details for PA-2.



(0.305 m = 1 ft)

Figure 138. Crack spacing pattern at site PA-2.

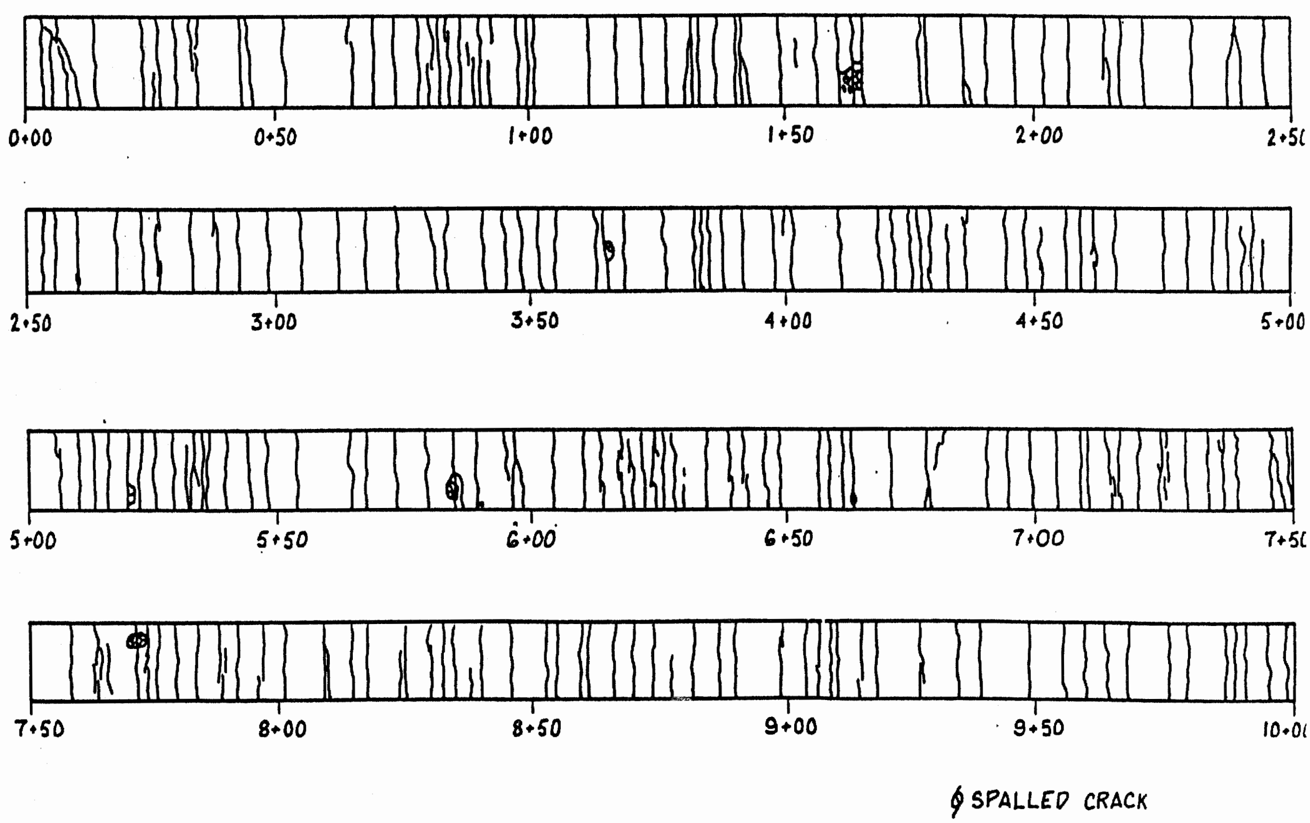
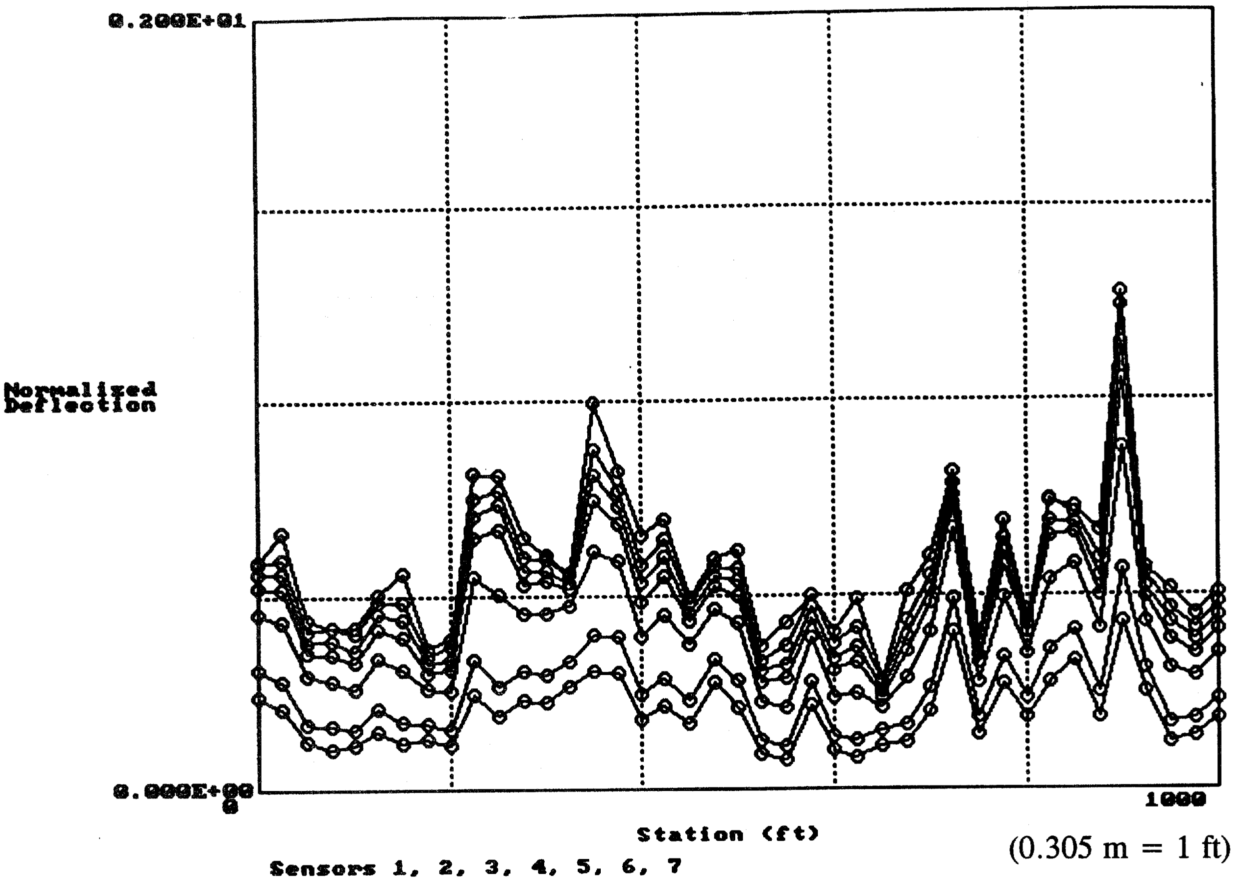
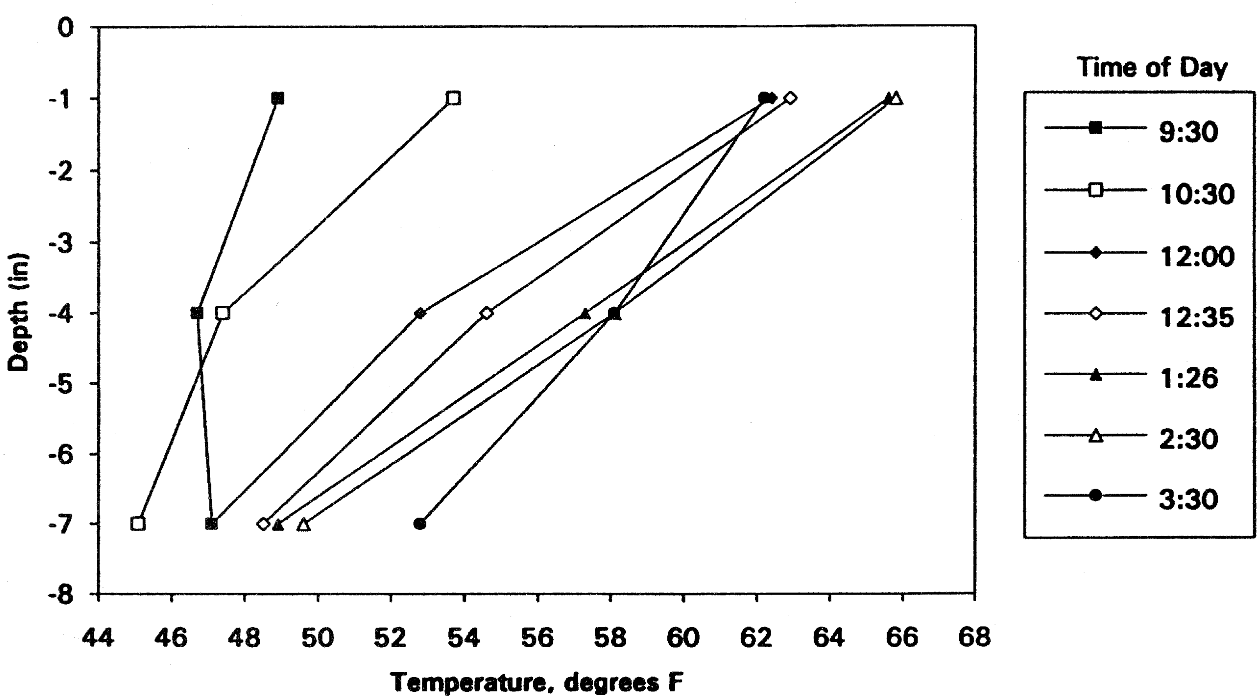


Figure 139. Crack pattern at site PA-2.



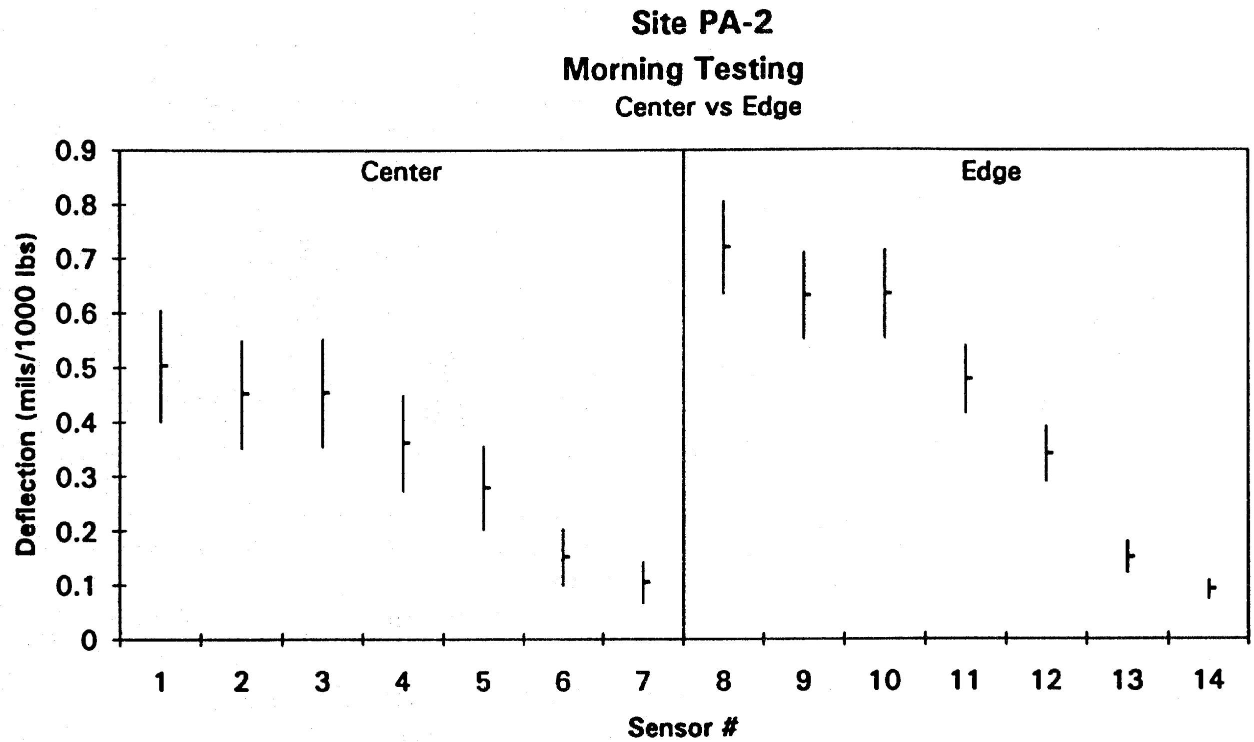
(0.305 m = 1 ft)

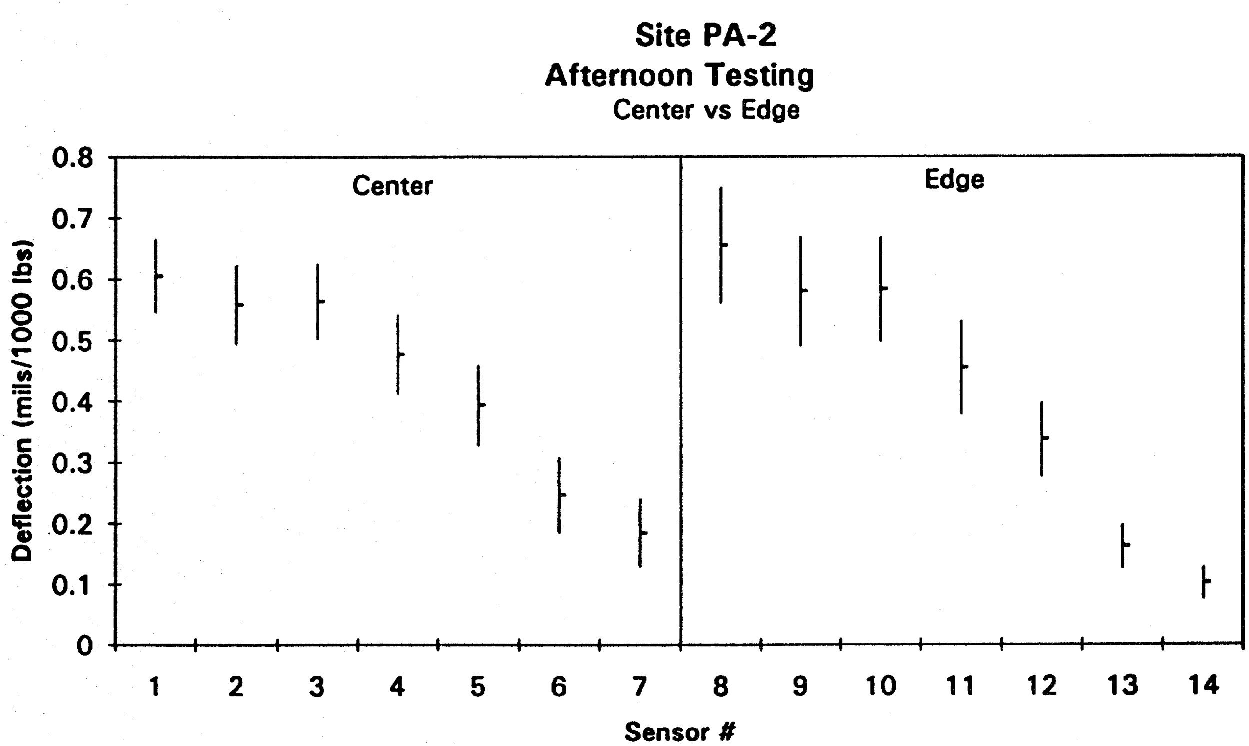
Figure 140. Deflections in inches along the section length at site PA-2.
[normalized per 4.45-kN [1,000-lb] load]



(25.4 mm = 1 in; 0.6°C = 1°F)

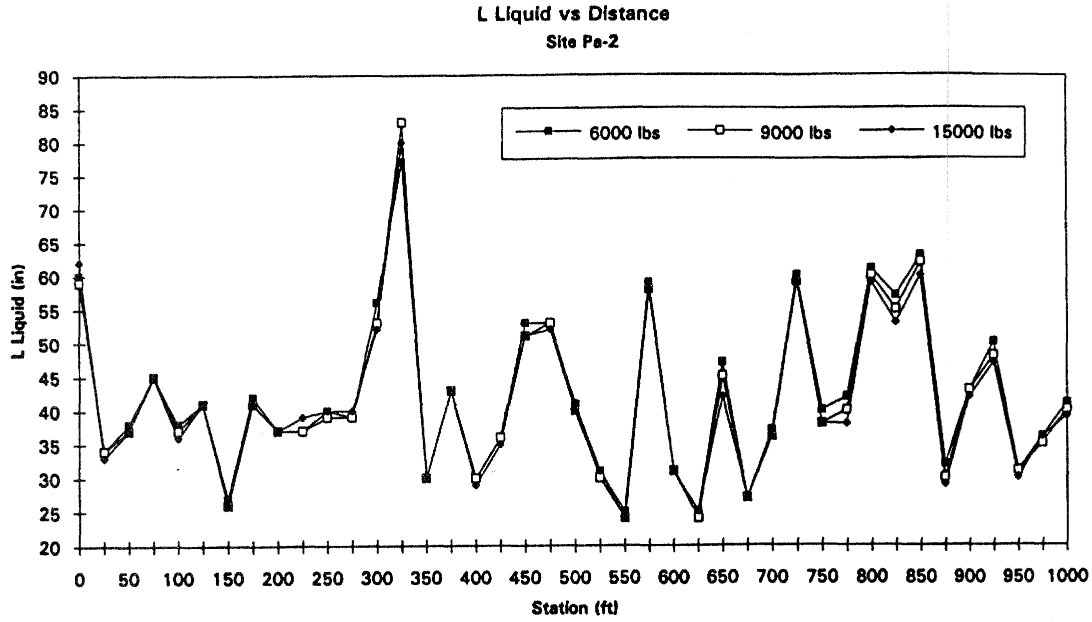
Figure 141. Temperature profile for site PA-2.





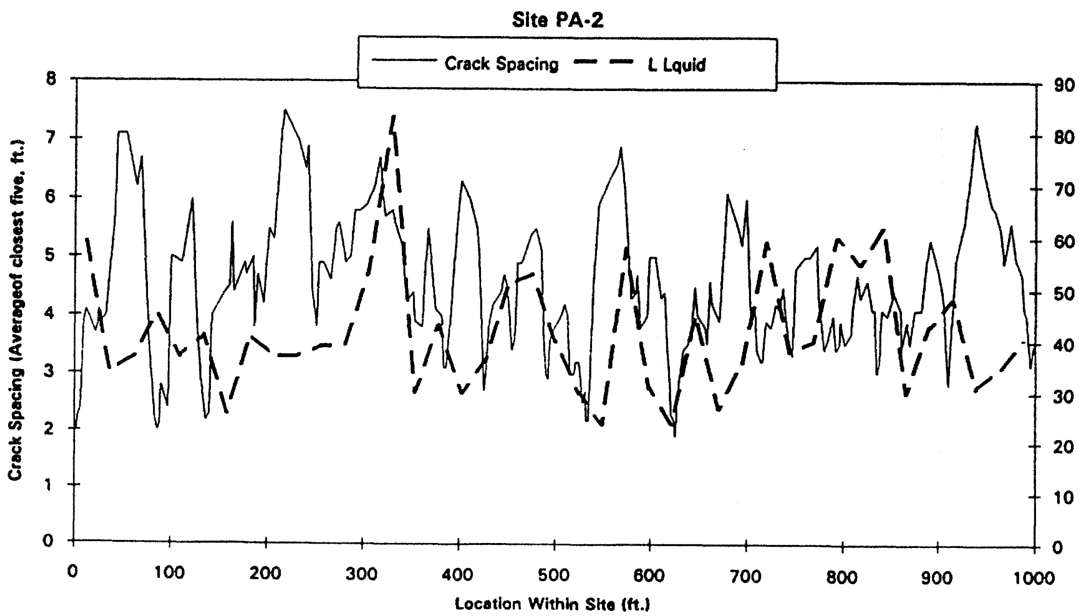
(0.0254 mm = 1 mil; 4.45 kN = 1,000 lb)

Figure 142. Center versus edge deflection for each of the seven sensors at site PA-2.
[40.03 kN (9,000 lb) load]



(25.4 mm = 1 in, 0.305 m = 1 ft)

Figure 143. Backcalculated values of radius of relative stiffness, ℓ (using program ILLI-BACK) for site PA-2.



(0.305 m = 1 ft)

Figure 144. Comparison of radius of relative stiffness values with average crack spacing (nearest five cracks) at site PA-2.

CHAPTER 8 - WISCONSIN TEST SECTIONS

Section WI-1 - Interstate 43

Introduction

Section WI-1 was opened to traffic in 1973. WI-1 is a section of I-43 that runs north to south from Milwaukee, Wisconsin, to the Wisconsin-Illinois border as shown in figure 145. The length of the highway that was considered runs from milepost 31 north to Milwaukee. A 305-m (1,000-ft) north bound section was selected for detailed inspection between milepost 31 and 32, as illustrated in figure 145. I-43 is in the southern region of Wisconsin.

The climate in this region is typically continental with a large annual temperature range and with frequent short-period temperature changes. Most of the precipitation, 60 percent, occurs during a 5-month period from May to September. The highest monthly average temperature is 21.4°C (70.6°F) during July, and the lowest monthly average is -9.1°C (15.6°F) during January. I-43 lies within the wet-freeze environmental zone.

Traffic

The 1991 ADT was 10,860 vpd, and the cumulative ESALs up to 1991 were estimated to be 2,287,000.

Structure

I-43 is a four-lane divided highway. The pavement lane width is 3.7 m (12 ft) with an outside shoulder width of 3.0 m (10 ft) and an inside shoulder width of 0.9 m (3 ft). The shoulders are constructed of asphaltic concrete.

I-43 was designed as a three-layer system comprised of a 203-mm (8-in) CRCP and a 229-mm (9-in) crushed gravel base course over a sandy subgrade. The CRC pavement was designed with a steel percentage of 0.65. The reinforcing steel are #4 bars spaced at 152 mm (6 in) with no transverse steel. The bars were tube fed during the construction and placed as a single layer. The bars are not epoxy coated.

The average concrete pavement thickness was determined to be 203 mm (8 in) based on cores taken as part of this study. The concrete is considered well graded and consolidated. The aggregate type used was a stone (quartz) with a maximum size of 51 mm (2 in).

The base course could not be distinguished from the subgrade material. The subgrade is a sandy soil with an AASHTO classification of A-2-4. It had a liquid limit of 12 and a plastic limit of 0.

Design/Construction

No design data were available for this section. The terminal end design chosen was a lug anchor system.

Performance

The performance of I-43 evaluated in 1990 according to the PSI scale, 0.0 to 5.0, was 2.5. A PDI survey was done in 1991, and the value was 77.

Historical Data (M&R)

No information about maintenance and rehabilitation was available for this section.

Field Investigation Data

Visual Condition Survey

- Total number of cracks = 259
  - 0 were at high severity
  - 259 were at medium severity
  - 0 were at low severity
- Average crack spacing = 0.87 m (2.87 ft)
- Standard deviation for crack spacing = 0.69 m (2.27 ft)
- Coefficient of variation of the crack spacing = 79 percent
- Figure 146 shows average crack spacing distribution based on the closest five cracks
- Figure 147 shows actual crack spacing
- Extent of Y cracks = 23 percent
- Cluster cracking was apparent at numerous locations
- Other distress types included:
  - 84 AC patches (low severity)
  - Longitudinal cracking (23 m (76 ft) low and 21 m (70 ft) moderate severity)
  - Popouts
- Pavement shoulders were in fair condition.

Drainage Survey

- Ditches line both sides of the pavement.
- Ditches were lined with vegetation. There was no sign of ponded water or wet ground.

Windshield Survey

- 8.05 km (5 mi) of pavement surveyed was generally in fair condition exhibiting about 2 700 m (9,000 ft) of longitudinal cracking condition, 9 PCC patches, 19 AC patches, and 3 punchouts.

Terminal Joint Survey

- Terminal joints used = Lug anchor (two surveyed)
- Joints exhibited the following distresses:
  - Both exhibited about 2.2 m^2 (24 ft^2) of medium severity AC patching
  - One joint exhibited some faulting

Deflection Testing

- Figure 148 shows normalized/basin (per 4.45 kN (1,000 lb) load) deflections measured along the length of the section (for all seven sensors using the 40.03 kN (9,000 lb) load)
- Load transfer efficiency at cracks at mid-slab
  - 77 to 89 percent in the morning
  - 82 to 91 percent in the afternoon
- Load transfer efficiency at cracks along edge
  - 81 to 91 percent in the morning
  - 83 to 91 percent in the afternoon
- Figure 149 shows the distribution of slab temperature with depth and time of day.
- Figure 150 shows the average deflections and ranges of deflections for each sensor for center and edge loading, for morning and afternoon testing (using the 40.03 kN (9,000 lb) load within the selected 30.5-m (100-ft) subsection).
- Figure 151 shows the backcalculated radius of relative stiffness, ℓ, values for basin testing for each of the three nominal load levels of 40.03, 53.38, and 71.17 kN (9,000, 12,000, and 16,000 lb).
- ℓ value based on 40.03 kN (9,000 lb) load basin testing
  - Average = 660 mm (26 in)
  - Standard Deviation = 127 mm (5 in)
- ℓ value at crack locations at mid-slab (morning/afternoon)
  - Average = 635/610 mm (25/24 in)
  - Standard Deviation = 51/51 mm (2/2 in)
- ℓ value at crack locations at edge (morning/afternoon)
  - Average = 610/660 mm (24/26 in)
  - Standard Deviation = 51/76 mm (2/3 in)
- Estimated ℓ value using actual slab thickness and concrete modulus of elasticity and assumed modulus of subgrade reaction
  - Value = 787 mm (31 in)
- Figure 152 shows the variation of ℓ in relation to crack spacing (average of the closest five cracks)
- Average deflection under 40.03 kN (9,000 lb) load (0.000001 mm (0.001 in))
  - Basin test - 0.071 (2.8)
  - Mid-slab crack location (morning/afternoon) - 0.086/0.081 (3.4/3.2)

- Edge crack location (morning/afternoon) - 0.155/0.112 (6.1/4.4)

Crack Width Measurements

- Number of cracks monitored within the 30.5-m (100-ft) subsection = 26
- Morning testing
  - Average temperature 102 mm (4 in) below surface = 12.8°C (55°F)
  - Average crack width = 0.91 mm (0.036 in)
- Afternoon testing
  - Average temperature 102 mm (4 in) below surface = 20.0°C (68°F)
  - Average crack width = 0.69 mm (0.027 in)
- Slab length change, mm/mm/°C (in/in/°F) = 25.9 millionth (14.4 millionth)

Reinforcing Steel Depths and Spacing

- Steel depth measurement offsets = 635, 1 346, 2 565 mm (25, 53, 101 in)
- Range of depth of cover = 46 to 99 mm (1.8 to 3.9 in)
- Average depth of cover = 71 mm (2.8 in)
- Standard deviation for depth of cover = 10 mm (0.4 in)
- Steel spacing was not measured

Corrosion Potential Testing (copper-copper sulfate half cell potentiometer)

- Range of measurements = -0.04 to -1.00 volts
- Average measurement = -0.53 volts
- Standard deviation for the measurement = -0.15 volts
- Potential for corrosion was indicated

Concrete Core Examination for Corrosion

- One core exhibited spot corrosion
- Three cores exhibited spread corrosion

Profile Testing

- Average IRI over the 305-m (1,000-ft) section = 1 770 mm/km (112 in/mi)

Laboratory Testing

The following data were obtained from the testing of the concrete, base/subbase, and subgrade materials:

Concrete

Modulus of Elasticity - 38 610 MPa (5,600,000 lbf/in^2)

Average Split Tensile Strength - 4.55 MPa (660 lbf/in^2)
Split Tensile Strength Range - 3.86 to 5.10 MPa (560 to 740 lbf/in^2)
Coefficient of Thermal Expansion - 10.06 mm/mm/°C (5.59 in/in/°F)

Base
AASHTO Classification - Granular base

Subbase
AASHTO Classification - N/A

Subgrade
Liquid Limit - 12 percent
Plastic Limit - 0 percent
Percent Passing #200 Sieve - 8.4 percent
AASHTO Classification - A-2-4

Chloride content testing was conducted on pulverized concrete retrieved at various depths and locations. Total chloride content and water soluble chloride content values are listed below for WI-1.

Location	Depth, mm (in)	Total Chloride Content (% wt.)	Water Soluble Chlorides (% wt.)
No steel below sample	0-51 (0-2)	0.204	0.173
	51-102 (2-4)	0.069	0.062
Steel directly below sample	0-51 (0-2)	0.270	0.207
	51-102 (2-4)	0.086	0.074

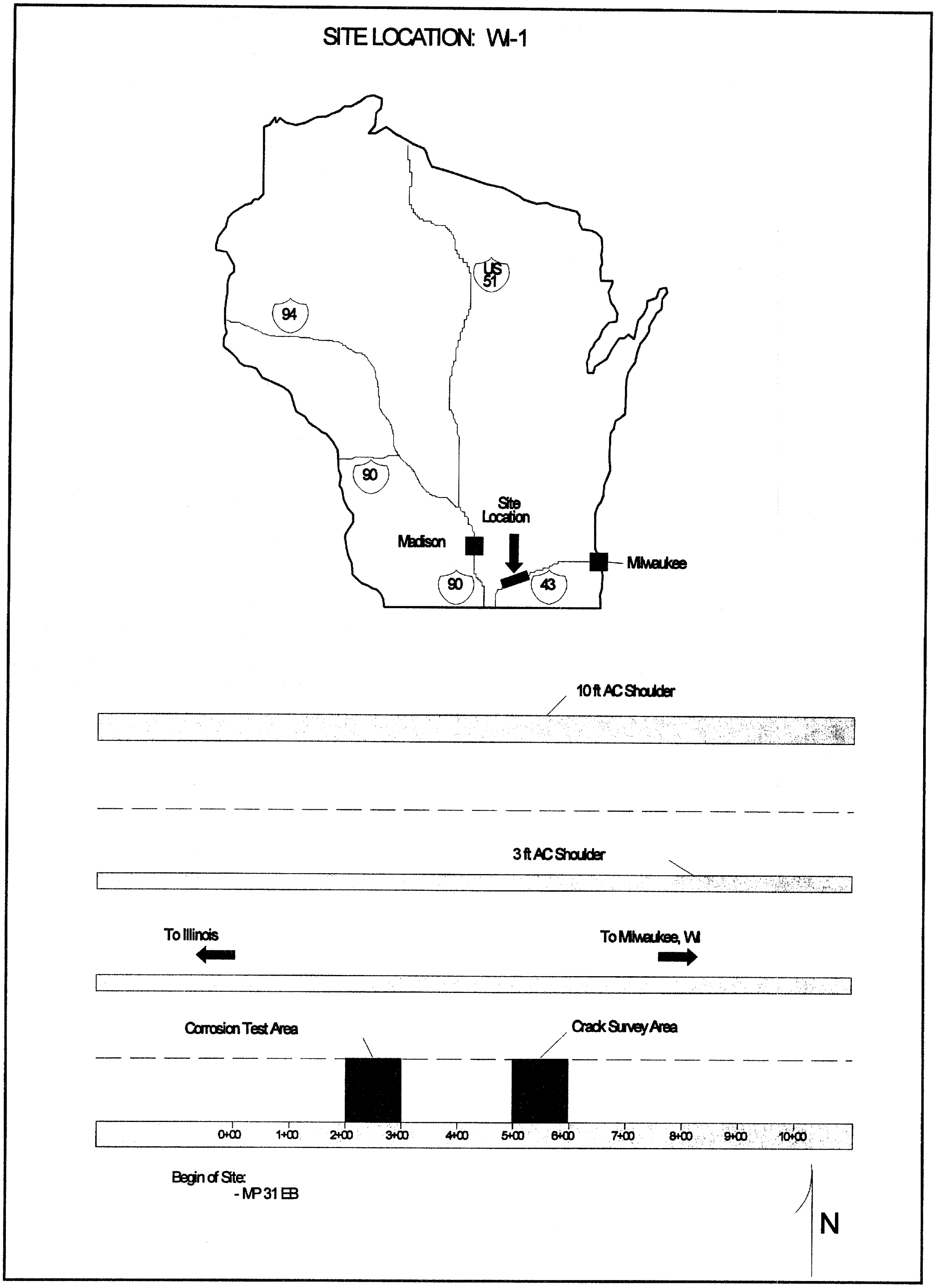


Figure 145. Site details for WI-1.

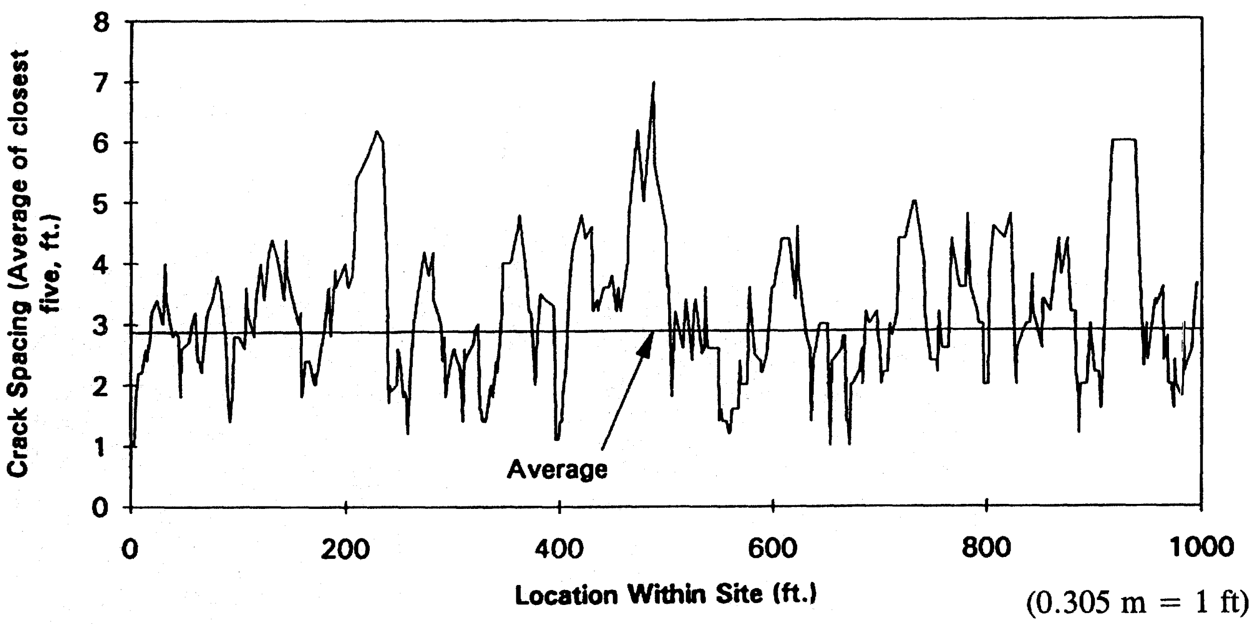


Figure 146. Crack spacing pattern at site WI-1.

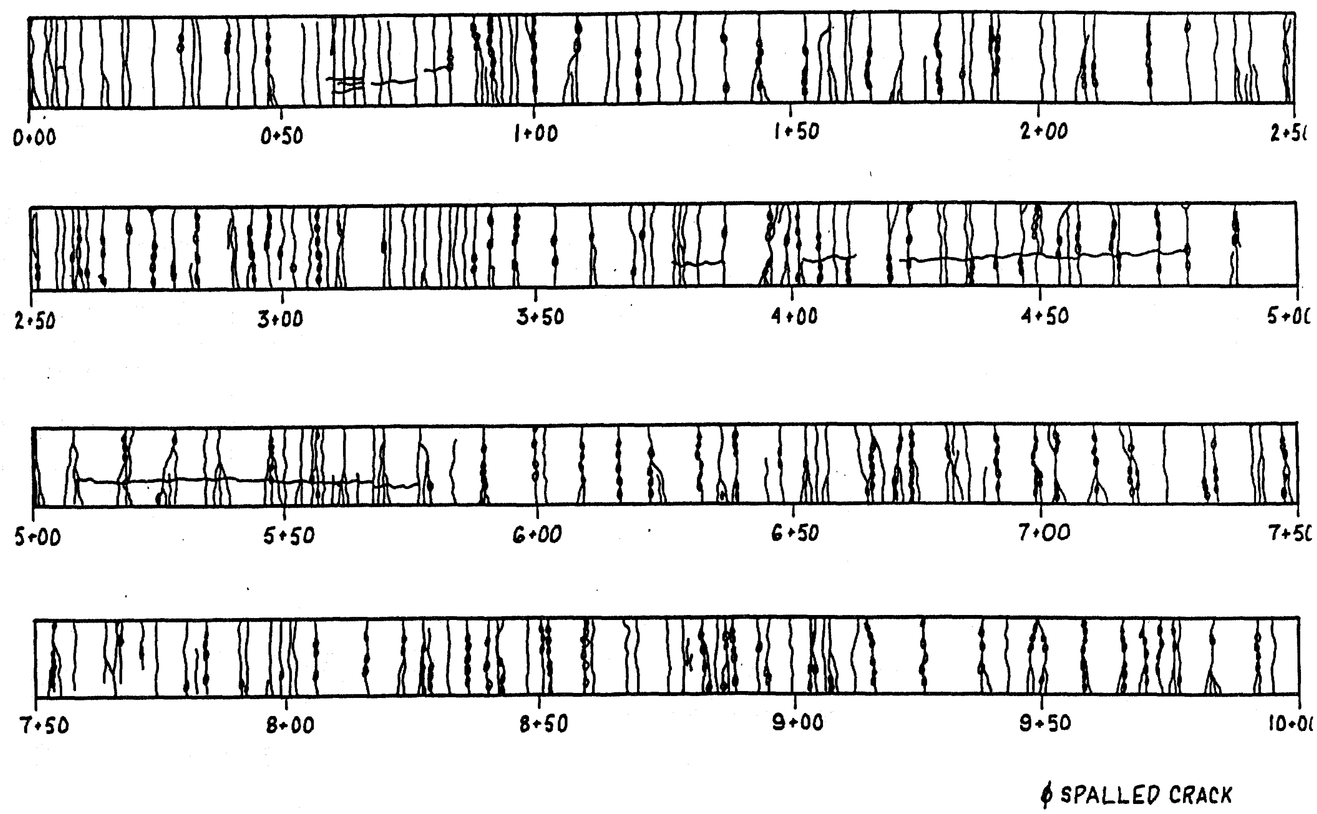
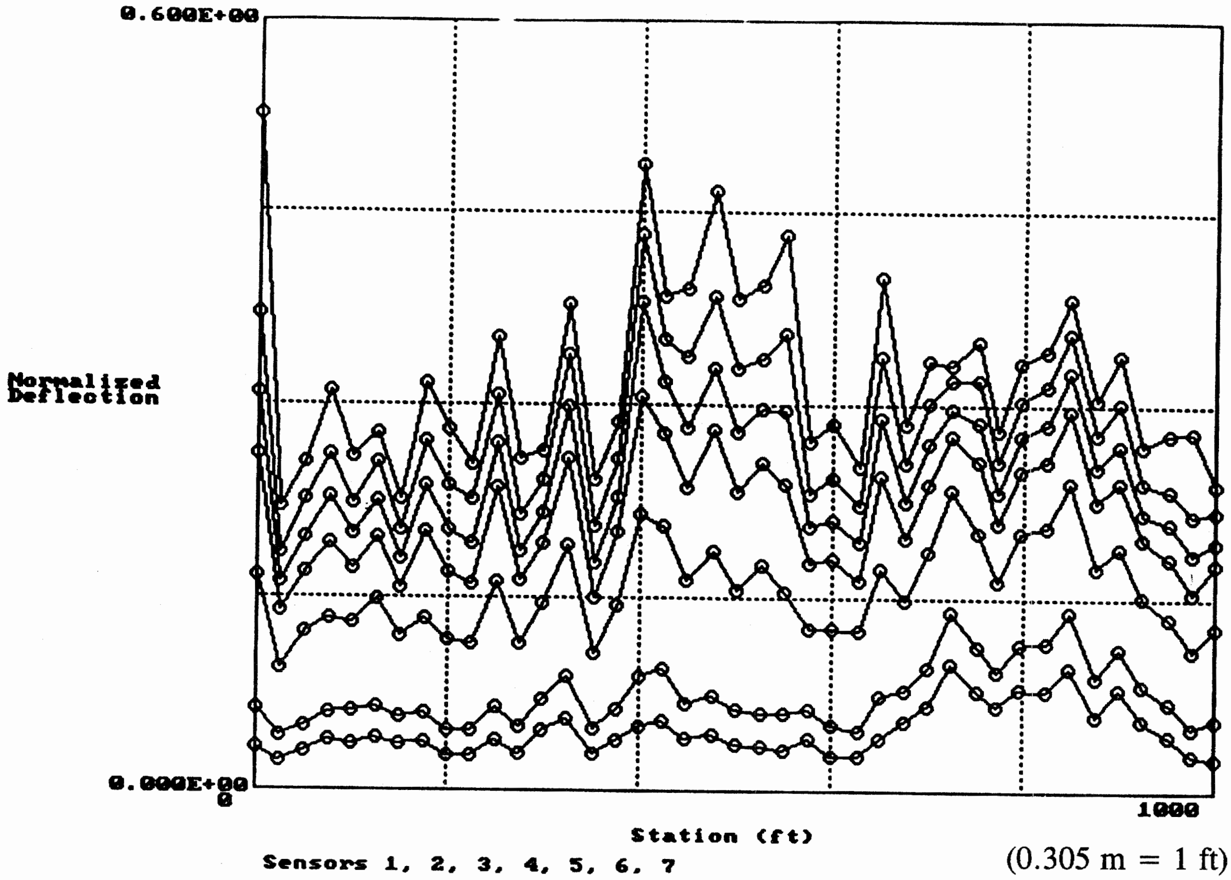
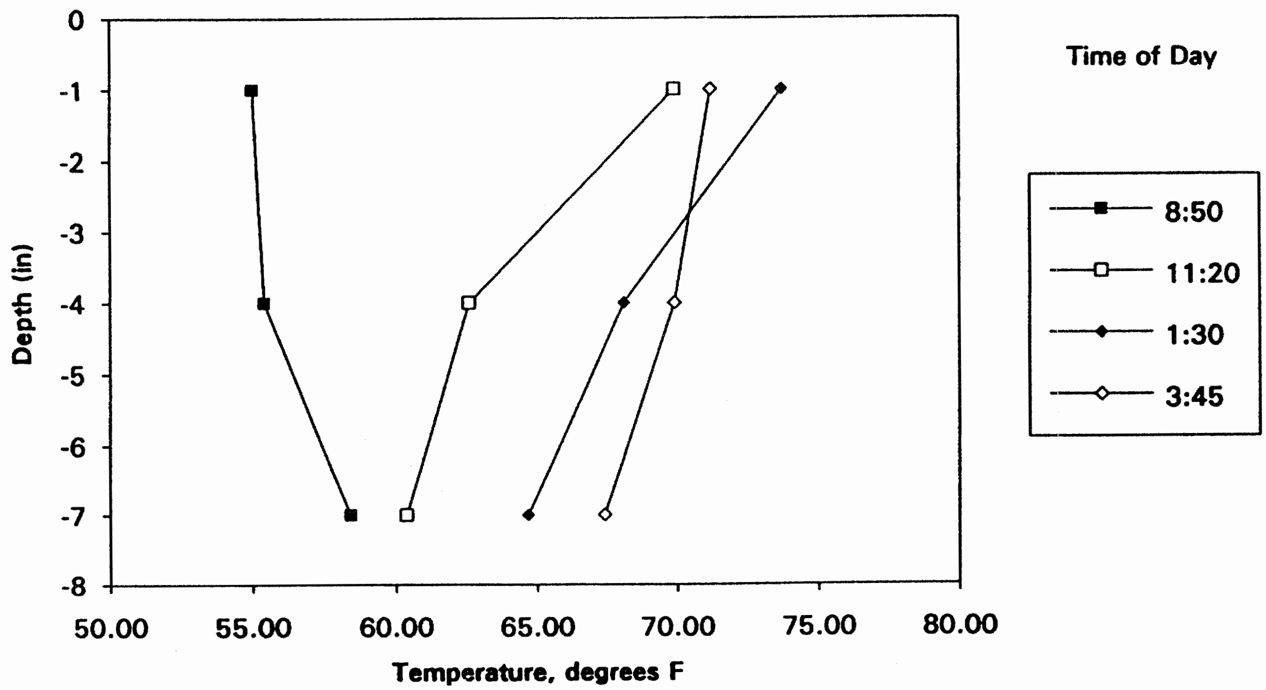


Figure 147. Crack pattern at site WI-1.



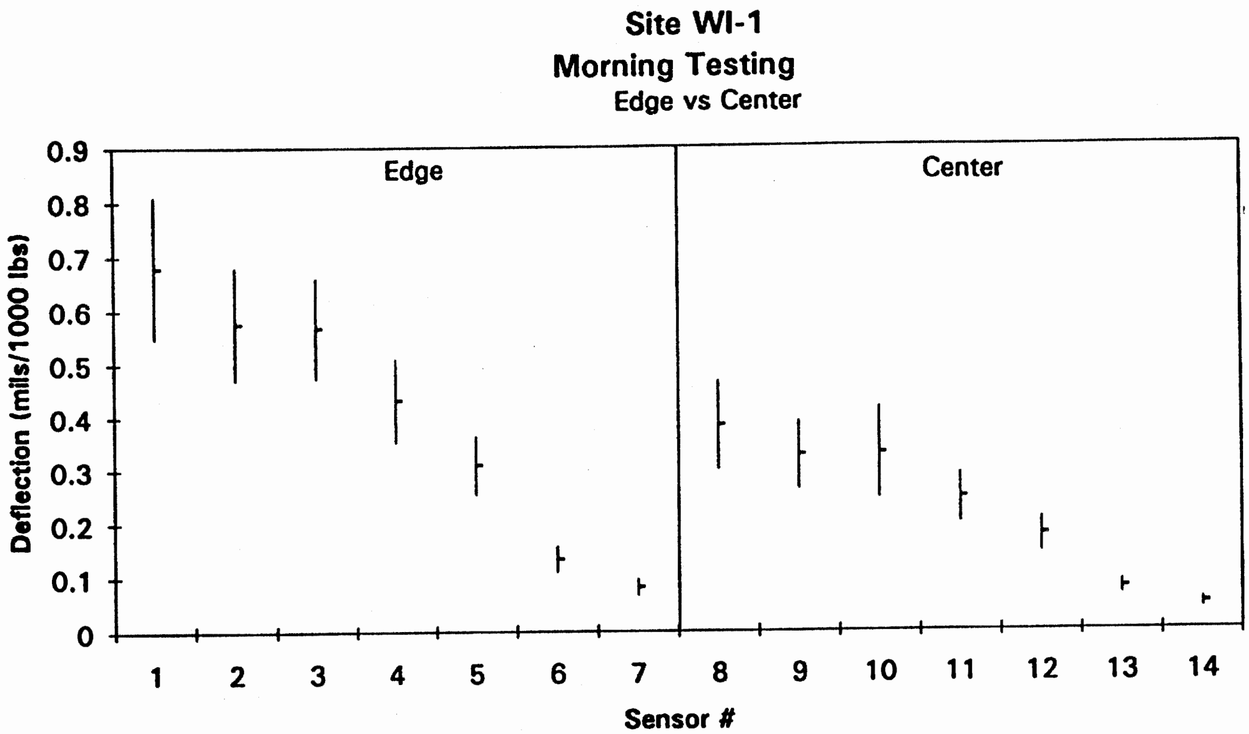
(0.305 m = 1 ft)

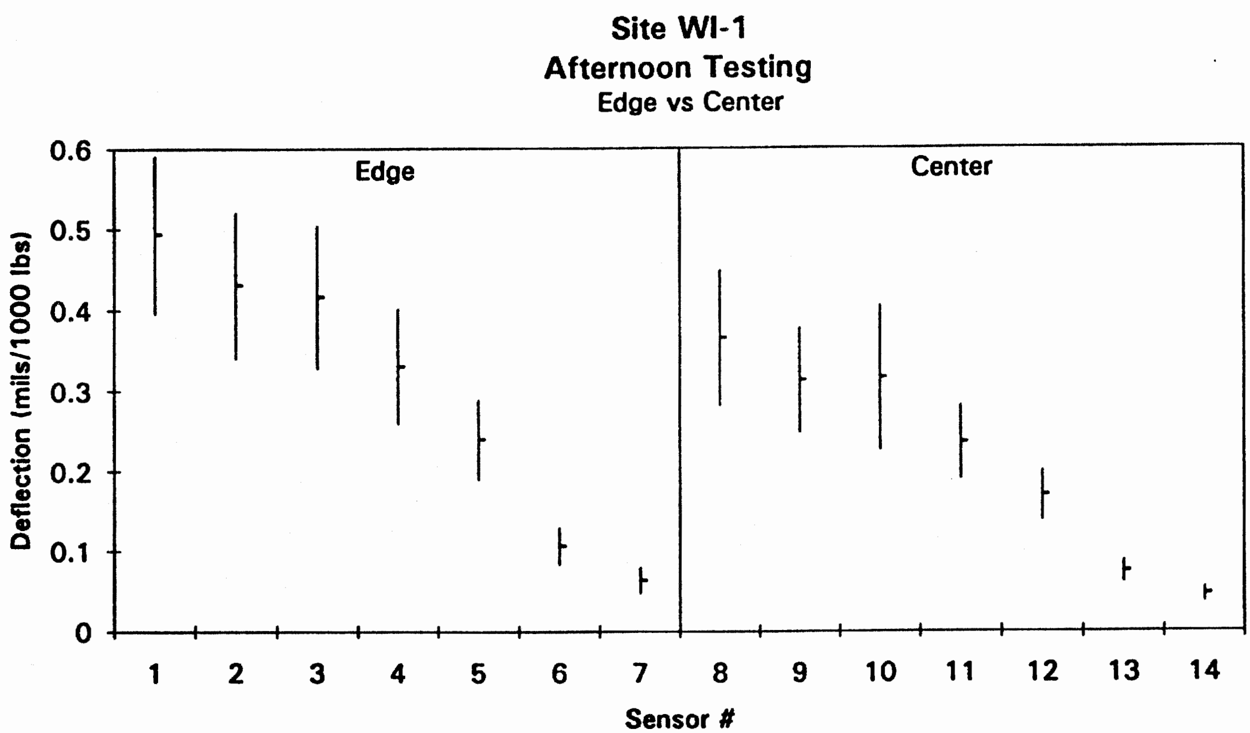
Figure 148. Deflections in inches along the section length at site WI-1.
[normalized per 4.45-kN [1,000-lb] load]



(25.4 mm = 1 in; 0.6°C = 1°F)

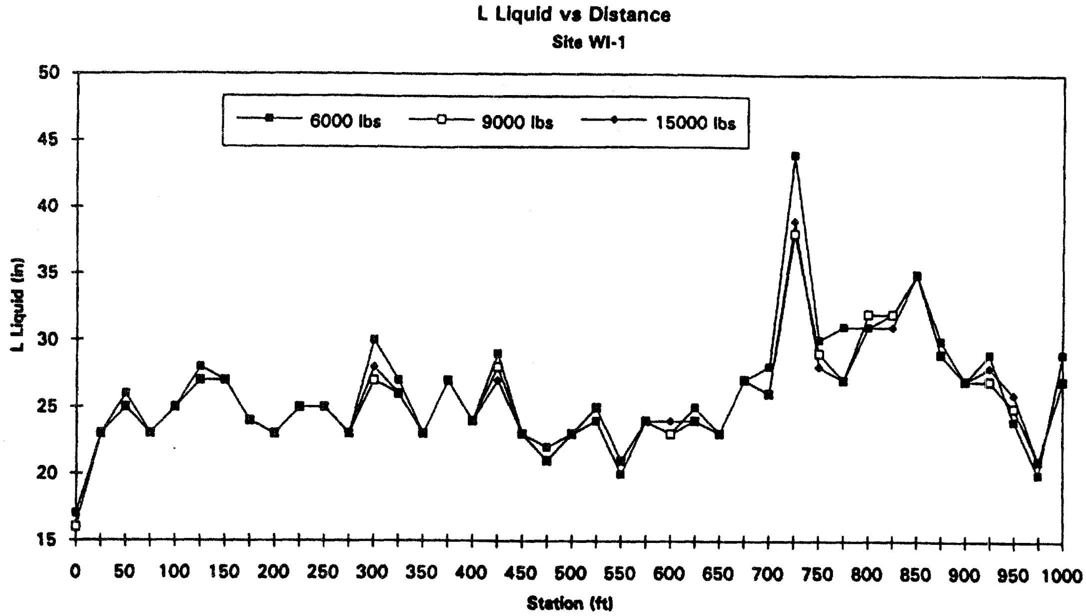
Figure 149. Temperature profile for site WI-1.





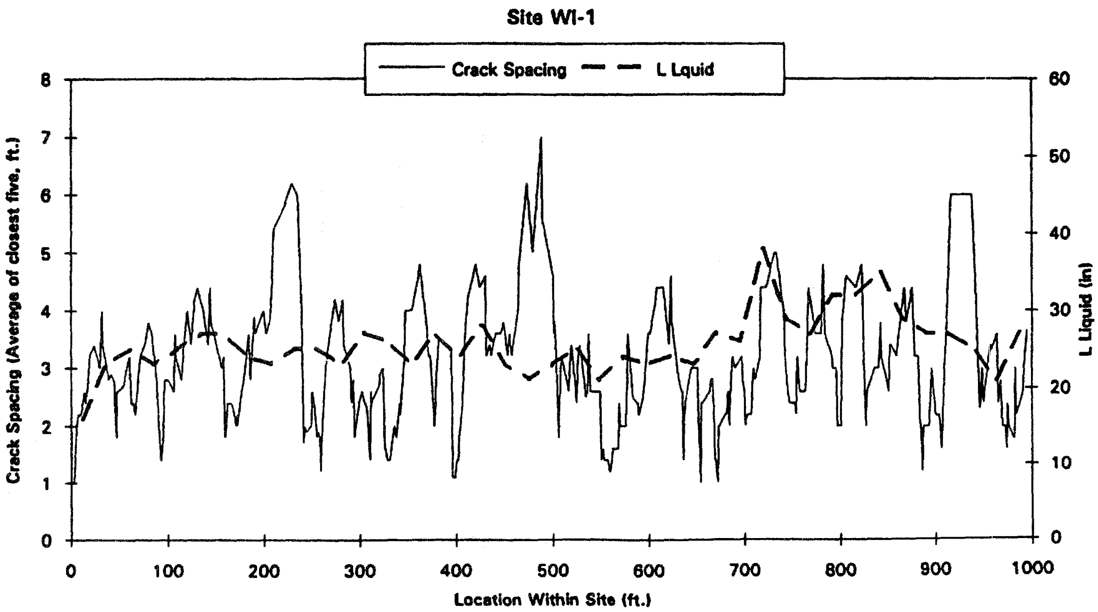
(0.0254 mm = 1 mil; 4.45 kN = 1,000 lb)

Figure 150. Center versus edge deflection for each of the seven sensors at site WI-1.
[40.03 kN (9,000 lb) load]



(25.4 mm = 1 in, 0.305 m = 1 ft)

Figure 151. Backcalculated values of radius of relative stiffness, ℓ (using program ILLI-BACK) for site WI-1.



(0.305 m = 1 ft)

Figure 152. Comparison of radius of relative stiffness values with average crack spacing (nearest five cracks) at site WI-1.

Section WI-2 - Interstate 90

Introduction

Section WI-2 was opened to traffic in 1985. WI-2 is a section of I-90/I-94 that runs northwest to southeast from Madison, Wisconsin to the Wisconsin-Illinois border as shown in figure 153. The length of the highway that was considered runs from milepost 180 east to Illinois. A 305-m (1,000-ft) north bound section was selected for detailed inspection between mileposts 180 and 181, as illustrated in figure 153. This section is in the southern region of Wisconsin.

The climate in this region is typically continental with a large annual temperature range and frequent short-period temperature changes. Most of the precipitation, 60 percent, occurs during a 5-month period from May to September. The highest monthly average temperature is 21.4°C (70.6°F) during July, and the lowest monthly average is -9.1°C (15.6°F) during January. I-43 lies within the wet-freeze environmental zone.

Traffic

The 1991 ADT was 31,390 vpd, and the cumulative ESALs up to 1991 were estimated to be 2,528,000.

Structure

I-90/I-94 is a four-lane divided highway. The pavement lane width is 3.7 m (12 ft) with an outside shoulder width of 3.0 m (10 ft) and an inside shoulder width of 0.9 m (3 ft). The shoulders are constructed of PCC.

I-90/I-94 was designed as a four-layer system comprised of a 254-mm (10-in) CRCP, a 152-mm (6-in) dense graded crushed gravel base course, and a 229-mm (9-in) granular pit-run subbase over a subgrade of in-situ material. The CRC pavement was designed with a steel percentage of 0.67. The reinforcing steel are #6 bars spaced at 165 mm (6.5 in). The transverse steel are #4 bars spaced at 1 219 mm (48 in). The bars were placed on chairs as a single layer during the construction of the pavement. The bars are epoxy coated. The concrete shoulders were tied to the mainline CRC pavement.

The average concrete pavement thickness for WI-2 was determined to be 254 mm (10 in) based on cores taken as part of this study. The concrete is considered well graded, and its consolidation is average. The aggregate type used was a stone (quartz) with a maximum size of 38 mm (1½ in).

The granular base course was 152 mm (6 in) and well graded. Its AASHTO classification was A-1-a. The subbase material could not be distinguished from the subgrade. The subgrade is a sandy soil with an AASHTO classification of A-4. It had a liquid limit of 14 and a plastic limit of 0.

Design/Construction

No design information was available for this section. The terminal end design system used was a wide flange beam system.

Performance

The performance of I-90 taken in 1990 according to the PSI scale, 0.0 to 5.0, was 4.8.

Historical Data (M&R)

No information about maintenance and rehabilitation work was available for this section.

Field Investigation Data

Visual Condition Survey

- Total number of cracks = 350
  - 0 were at high severity
  - 314 were at medium severity
  - 36 were at low severity
- Average crack spacing = 0.88 m (2.90 ft)
- Standard deviation for crack spacing = 0.43 m (1.41 ft)
- Coefficient of variation of the crack spacing = 49 percent
- Figure 154 shows average crack spacing distribution based on the closest five cracks
- Figure 155 shows actual crack spacing
- Extent of Y cracks = 10 percent
- Cluster cracking was apparent at numerous locations
- No other distress was present.
- Pavement shoulders were in fair condition.

Drainage Survey

- Ditches line both sides of the pavement.
- Ditches were lined with vegetation. There was no sign of ponded water or wet ground.

Windshield Survey

- 8.05 km (5 mi) of pavement surveyed was generally in good condition exhibiting very little distress.

Terminal Joint Survey

- Terminal joints used = Wide flange beam (two surveyed)
- Joints exhibited no distress.

Deflection Testing

- Figure 156 shows normalized/basin (per 4.45 kN (1,000 lb) load) deflections measured along the length of the section (for all seven sensors using the 40.03 kN (9,000 lb) load)
- Load transfer efficiency at cracks at mid-slab
    - 91 to 97 percent in the morning
    - 90 to 96 percent in the afternoon
- Load transfer efficiency at cracks along edge
    - 90 to 95 percent in the morning
    - 90 to 96 percent in the afternoon
- Figure 157 shows the distribution of slab temperature with depth and time of day.
- Figure 158 shows the average deflections and ranges of deflections for each sensor for center and edge loading, for morning and afternoon testing (using the 40.03 kN (9,000 lb) load within the selected 30.5-m (100-ft) subsection).
- Figure 159 shows the backcalculated radius of relative stiffness, ℓ, values for basin testing for each of the three nominal load levels of 40.03, 53.38, and 71.17 kN (9,000, 12,000, and 16,000 lb).
- ℓ value based on 40.03 kN (9,000 lb) load basin testing
    - Average = 864 mm (34 in)
    - Standard Deviation = 127 mm (5 in)
- ℓ value at crack locations at mid-slab (morning/afternoon)
    - Average = 838/864 mm (33/34 in)
    - Standard Deviation = 76/102 mm (3/4 in)
- ℓ value at crack locations at edge (morning/afternoon)
    - Average = 838/762 (33/30 in)
    - Standard Deviation = 203/102 mm (8/4 in)
- Estimated ℓ value using actual slab thickness and concrete modulus of elasticity and assumed modulus of subgrade reaction
    - Value = 864 mm (34 in)
- Figure 160 shows the variation of ℓ in relation to crack spacing (average of the closest five cracks)
- Average deflection under 40.03 kN (9,000 lb) load (0.000001 mm (0.001 in))
    - Basin test - 0.071 (2.8)
    - Mid-slab crack location (morning/afternoon) - 0.071/0.071 (2.8/2.8)
    - Edge crack location (morning/afternoon) - 0.178/0.137 (7.0/5.4)

Crack Width Measurements

- Number of cracks monitored within the 30.5-m (100-ft) subsection = 35
- Morning testing

- Average temperature 102 mm (4 in) below surface = 13.9°C (57°F)
- Average crack width = 0.58 mm (0.023 in)

- Afternoon testing
  - Average temperature 102 mm (4 in) below surface = 15.6°C (60°F)
  - Average crack width = 0.27 mm (0.011 in)
- Slab length change, mm/mm/°C (in/in/°F) = outlier value

Reinforcing Steel Depths and Spacing

- Steel depth measurement offsets = 305, 914, 1 524 mm (12, 36, 60 in)
- Range of depth of cover = 91 to 130 mm (3.6 to 5.1 in)
- Average depth of cover = 109 mm (4.3 in)
- Standard deviation for depth of cover = 7.6 mm (0.3 in)
- Steel spacing was not measured

Corrosion Potential Testing (copper-copper sulfate half cell potentiometer)

- Testing was not performed as the steel was epoxy coated

Concrete Core Examination for Corrosion

- No cores over steel bars were obtained

Profile Testing

- Average IRI over the 305-m (1,000-ft) section = 1 533 mm/km (97 in/mi)

Laboratory Testing

The following data were obtained from the testing of the concrete, base/subbase, and subgrade materials:

Concrete

Modulus of Elasticity	-	31 030 MPa (4,500,000 lbf/in^2)
Average Split Tensile Strength	-	3.38 MPa (490 lbf/in^2)
Split Tensile Strength Range	-	2.96 to 3.72 MPa (430 to 540 lbf/in^2)
Coefficient of Thermal Expansion	-	10.21 mm/mm/°C (5.67 in/in/°F)

Base

AASHTO Classification	-	Granular base (A-1-a)

Subbase

AASHTO Classification	-	N/A

Subgrade

Liquid Limit	-	14 percent

Plastic Limit	-	0 percent
Percent Passing #200 Sieve	-	3.6 percent
AASHTO Classification	-	A-4

Chloride content testing was not conducted at this test section. This test section was built with epoxy coated steel bars.

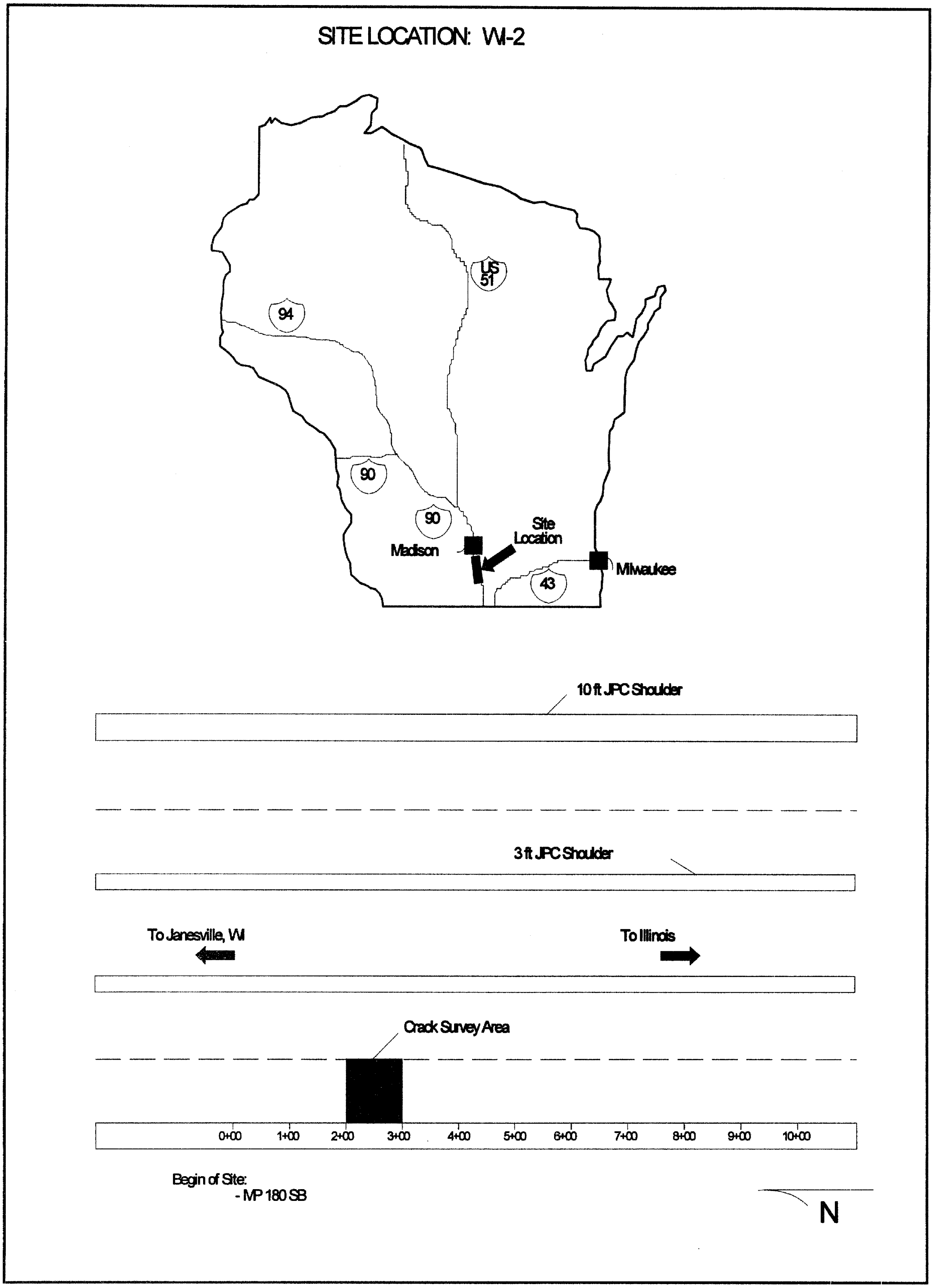


Figure 153. Site details for WI-2.

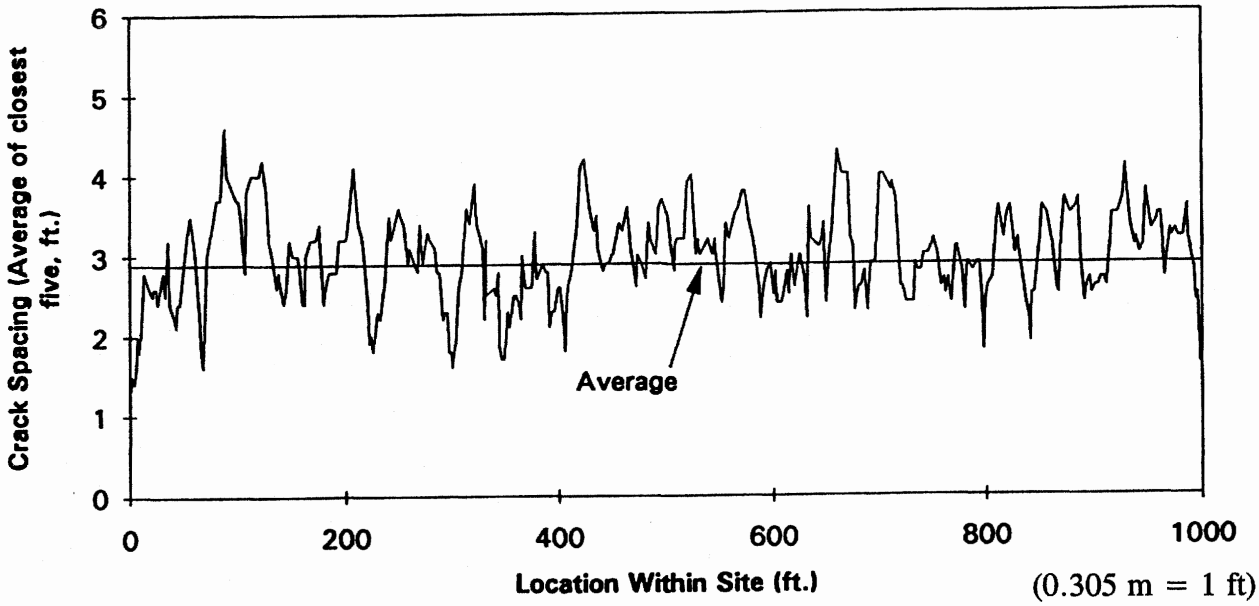


Figure 154. Crack spacing pattern at site WI-2.

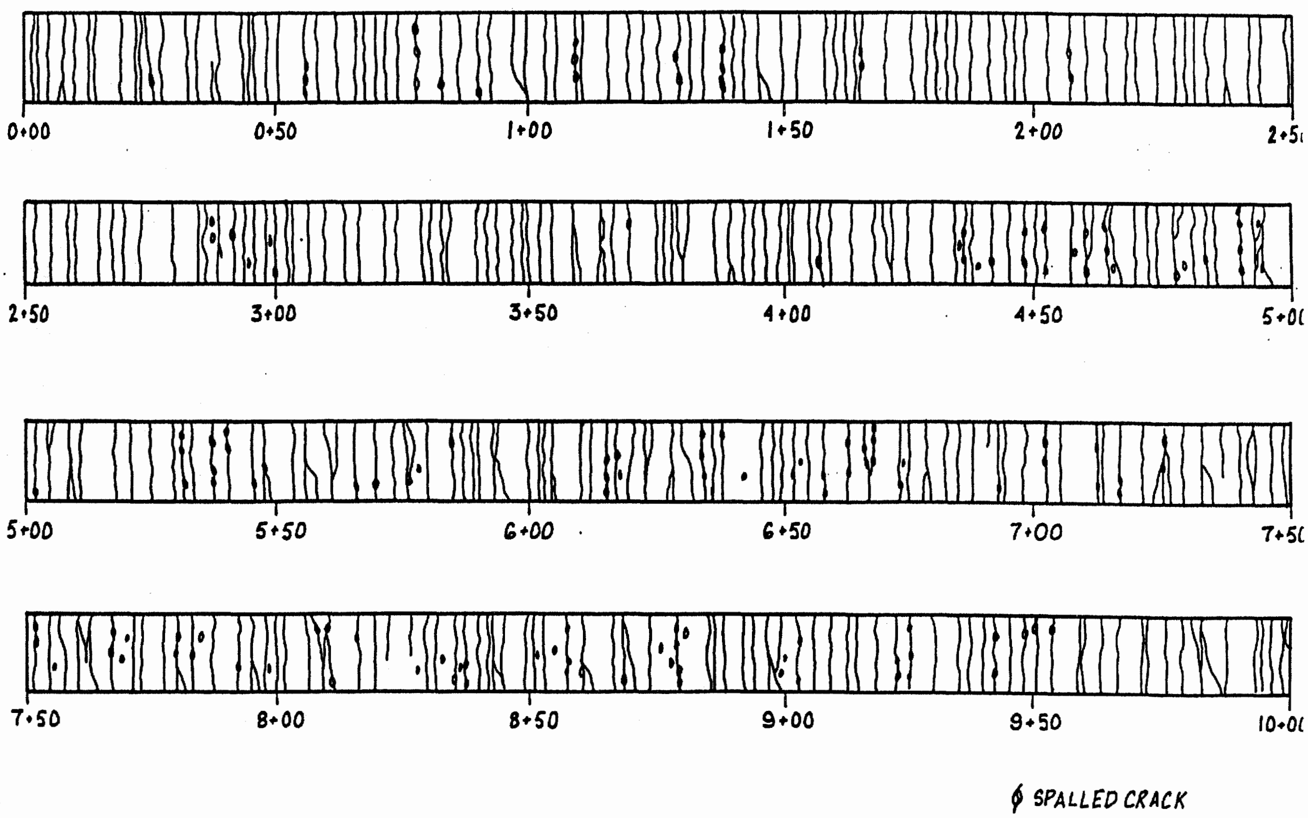
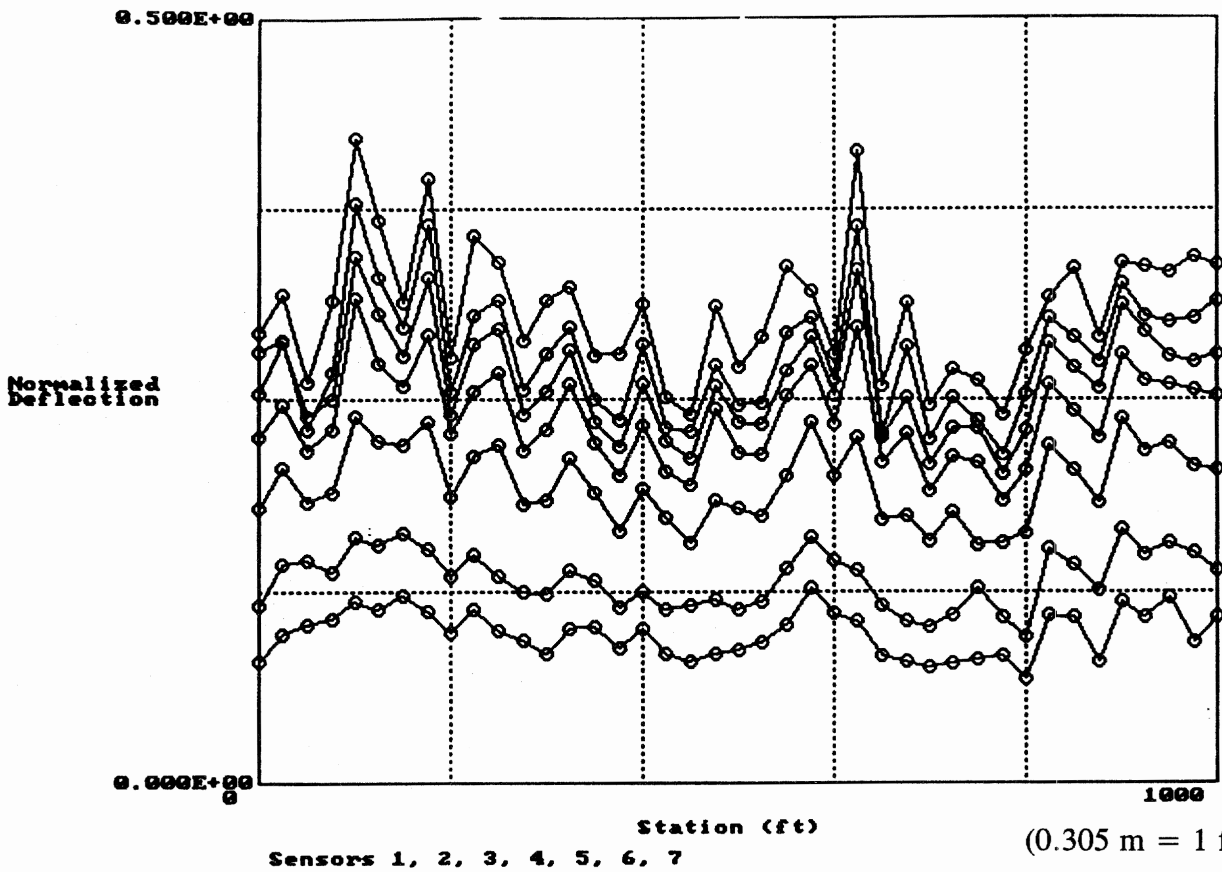
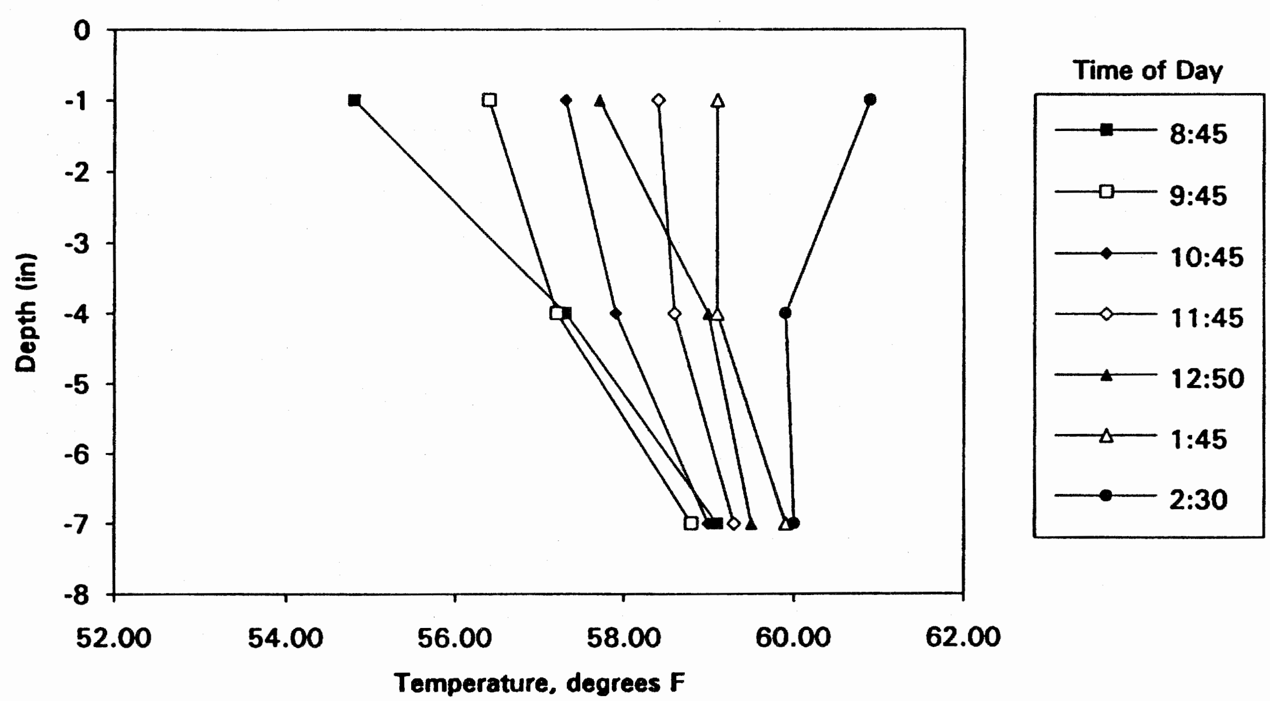


Figure 155. Crack pattern at site WI-2.



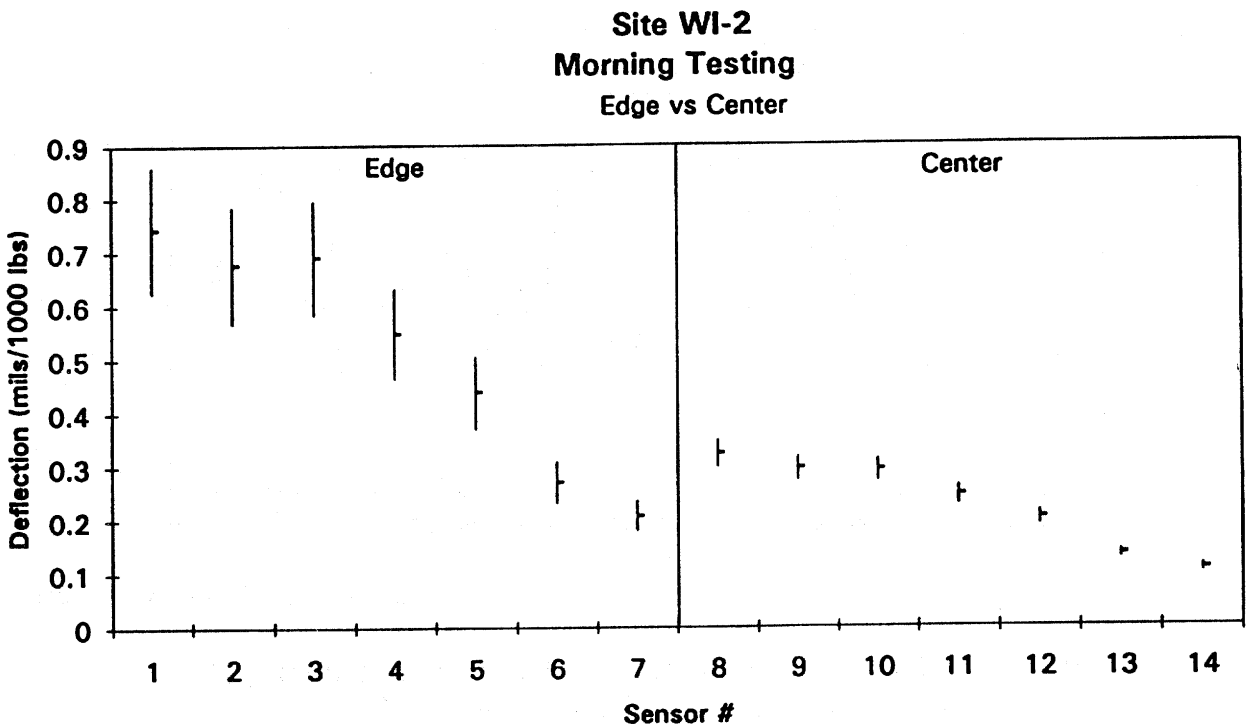
(0.305 m = 1 ft)

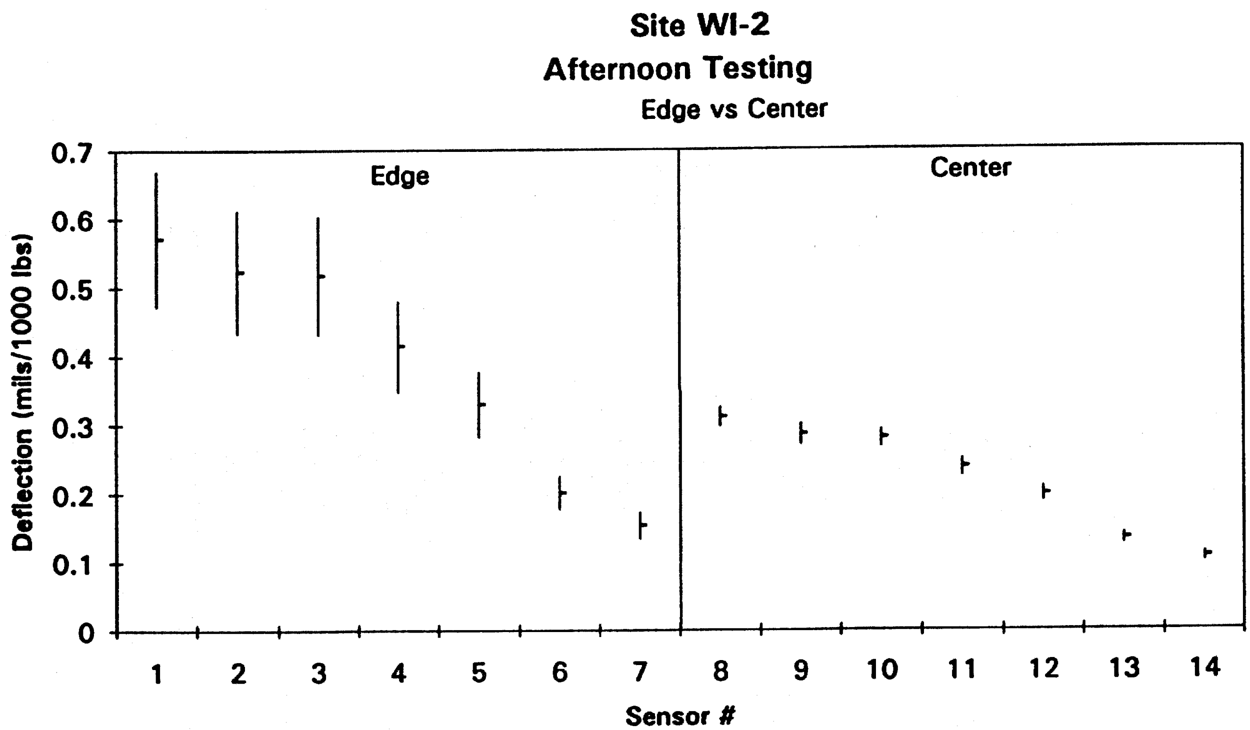
Figure 156. Deflections in inches along the section length at site WI-2.
[normalized per 4.45-kN [1,000-lb] load]



(25.4 mm = 1 in; 0.6°C = 1°F)

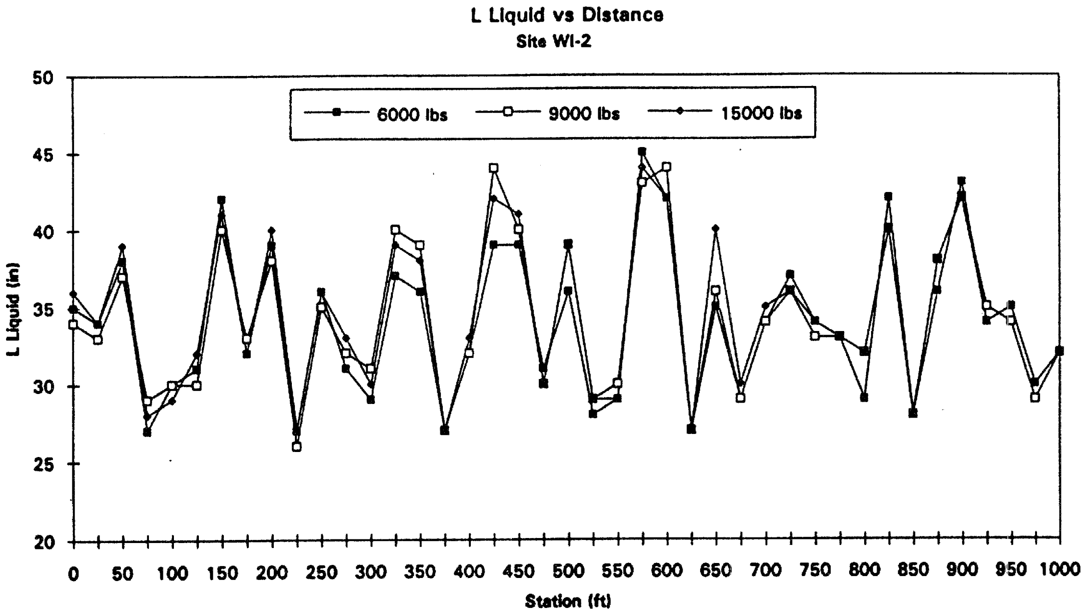
Figure 157. Temperature profile for site WI-2.





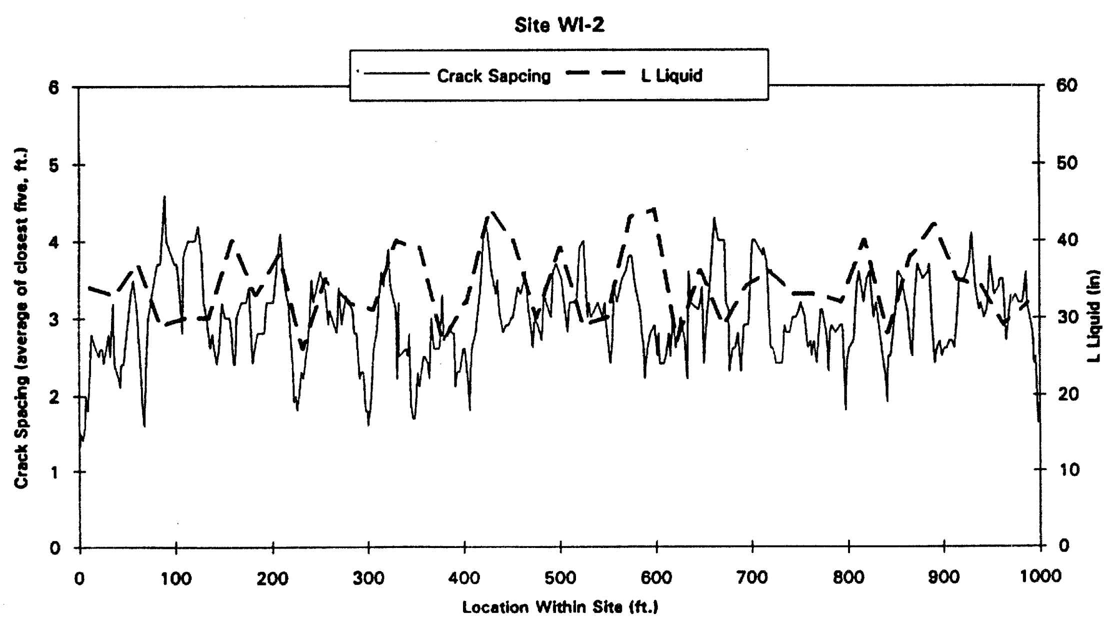
(0.0254 mm = 1 mil; 4.45 kN = 1,000 lb)

Figure 158. Center versus edge deflection for each of the seven sensors at site WI-2.
[40.03 kN (9,000 lb) load]



(25.4 mm = 1 in, 0.305 m = 1 ft)

Figure 159. Backcalculated values of radius of relative stiffness, ℓ (using program ILLI-BACK) for site WI-2.



(0.305 m = 1 ft)

Figure 160. Comparison of radius of relative stiffness values with average crack spacing (nearest five cracks) at site WI-2.

Section WI-3 - Interstate 90/94

Introduction

Section WI-3 was opened to traffic in 1984. WI-3 is a section of I-90/I-94 that runs northwest to southeast from the Wisconsin-Minnesota border to the Wisconsin-Illinois border as shown in figure 161. The length of the highway that was considered runs from milepost 136 west to Minnesota. A 305-m (1,000-ft) west bound section was selected for detailed inspection between mileposts 136 and 135, as illustrated in figure 161. This section is in the southern region of Wisconsin just north of Madison.

The climate in this region is typically continental with a large annual temperature range and frequent short-period temperature changes. Most of the precipitation, 60 percent, occurs during a 5-month period from May to September. The highest monthly average temperature is 21.4°C (70.6°F) during July, and the lowest monthly average is -9.1°C (15.6°F) during January. I-90/94 lies within the wet-freeze environmental zone.

Traffic

The 1991 ADT was 35,100 vpd, and the cumulative ESALs up to 1991 were estimated to be 3,961,000.

Structure

I-90/I-94 is a six-lane divided highway. The pavement lane width is 3.7 m (12 ft) with an outside shoulder width of 3.0 m (10 ft) and an inside shoulder width of 3.0 m (10 ft). The shoulders are constructed of PCC.

This section was designed as a four-layer system comprised of a 254-mm (10-in) CRCP, a 152-mm (6-in) crushed stone base course, and a 229-mm (9-in) sandy subbase over native subgrade. The CRC pavement was designed with a steel percentage of 0.67. The reinforcing steel are #6 bars spaced at 165 mm (6.5 in). The transverse steel are #4 bars spaced at 1 219 mm (48 in). The bars were placed on chairs as a single layer during the construction of the pavement. The bars are epoxy coated. The concrete shoulders (jointed) were tied to the mainline CRC pavement.

The average concrete pavement thickness for WI-2 was determined to be 254 mm (10 in) based on cores taken as part of this study. The concrete is considered well graded, and its consolidation is average. The aggregate type used was a stone (quartz) with a maximum size of 38 mm (1½ in).

The granular base course was determined to be 152 mm (6 in) and well graded with an AASHTO classification of A-1-a. The subbase material was over 178 mm (7 in) thick and uniformly graded. Its AASHTO classification was A-3. The subgrade material was not classified but was reported to be a silty clay of A-4 material.

Design/Construction

No design data were available for this section.

Performance

The performance of I-90/94 measured in 1990 according to the PSI scale, 0.0 to 5.0, was 3.9. A PDI survey was done in 1990, and the value was 43.

Historical Data (M&R)

No information about maintenance and rehabilitation work was available for this section.

Field Investigation Data

Visual Condition Survey

- Total number of cracks = 293
  - 0 were at high severity
  - 164 were at medium severity
  - 129 were at low severity
- Average crack spacing = 1.06 m (3.48 ft)
- Standard deviation for crack spacing = 0.48 m (1.58 ft)
- Coefficient of variation of the crack spacing = 45 percent
- Figure 162 shows average crack spacing distribution based on the closest five cracks
- Figure 163 shows actual crack spacing
- Extent of Y cracks = 7 percent
- Cluster cracking was apparent at stations 2+00, 3+50, 7+00, and 9+00
- Other distress types included:
  - 7.6 m (25 ft) longitudinal cracking (low severity)
  - Popouts
- Pavement shoulders were in good condition.

Drainage Survey

- No information was obtained

Windshield Survey

- 8.05 km (5 mi) of pavement surveyed was generally in good condition exhibiting about 58 m (190 ft) of longitudinal cracking condition, one PCC patch, and three AC patches.

Terminal Joint Survey

- Terminal joints used = Lug anchor

- Joints exhibited the following distresses:
  - Both joints exhibited about 0.2 m^2 (2 ft^2) of low severity spalling
  - One exhibited some faulting

Deflection Testing

- Figure 164 shows normalized/basin (per 4.45 kN (1,000 lb) load) deflections measured along the length of the section (for all seven sensors using the 40.03 kN (9,000 lb) load)
- Load transfer efficiency at cracks at mid-slab
  - 91 to 95 percent in the morning
  - 88 to 94 percent in the afternoon
- Load transfer efficiency at cracks along edge
  - 89 to 92 percent in the morning
  - 90 to 92 percent in the afternoon
- Figure 165 shows the distribution of slab temperature with depth and time of day.
- Figure 166 shows the average deflections and ranges of deflections for each sensor for center and edge loading, for morning and afternoon testing (using the 40.03 kN (9,000 lb) load within the selected 30.5-m (100-ft) subsection).
- Figure 167 shows the backcalculated radius of relative stiffness, ℓ, values for basin testing for each of the three nominal load levels of 40.03, 53.38, and 71.17 kN (9,000, 12,000, and 16,000 lb).
- ℓ value based on 40.03 kN (9,000 lb) load basin testing
  - Average = 762 mm (30 in)
  - Standard Deviation = 102 mm (4 in)
- ℓ value at crack locations at mid-slab (morning/afternoon)
  - Average = 686/838 mm (27/33 in)
  - Standard Deviation = 51/102 mm (2/4 in)
- ℓ value at crack locations at edge (morning/afternoon)
  - Average = 762/711 mm (30/28 in)
  - Standard Deviation = 76/51 mm (3/2 in)
- Estimated ℓ value using actual slab thickness and concrete modulus of elasticity and assumed modulus of subgrade reaction
  - Value = 838 mm (33 in)
- Figure 168 shows the variation of ℓ in relation to crack spacing (average of the closest five cracks)
- Average deflection under 40.03 kN (9,000 lb) load (0.000001 mm (0.001 in))
  - Basin test - 0.069 (2.7)
  - Mid-slab crack location (morning/afternoon) - 0.081/0.081 (3.2/3.2)
  - Edge crack location (morning/afternoon) - 0.188/0.112 (7.4/4.4)

Crack Width Measurements

- Number of cracks monitored within the 30.5-m (100-ft) subsection = 29
- Morning testing
    - Average temperature 102 mm (4 in) below surface = 11.7°C (53°F)
    - Average crack width = 0.54 mm (0.021 in)
- Afternoon testing
    - Average temperature 102 mm (4 in) below surface = 17.8°C (64°F)
    - Average crack width = 0.41 mm (0.016 in)
- Slab length change, mm/mm/°C (in/in/°F) = 20.2 millionth (11.2 millionth)

Reinforcing Steel Depths and Spacing

- Steel depth measurement offsets = 610, 1 219, 1 829 mm (24, 48, 72 in)
- Range of depth of cover = 64 to 124 mm (2.5 to 4.9 in)
- Average depth of cover = 102 mm (4.0 in)
- Standard deviation for depth of cover = 10 mm (0.4 in)
- Steel spacing was not measured

Corrosion Potential Testing (copper-copper sulfate half cell potentiometer)

- Testing was not performed as the steel bars were epoxy coated

Concrete Core Examination for Corrosion

- No cores over steel bars were obtained

Profile Testing

- Average IRI over the 305-m (1,000-ft) section = 1 280 mm/km (81 in/mi)

Laboratory Testing

The following data were obtained from testing of the concrete, base/subbase, and subgrade materials:

Concrete

Modulus of Elasticity	-	26 890 MPa (3,900,000 lbf/in^2)
Average Split Tensile Strength	-	3.10 MPa (450 lbf/in^2)
Split Tensile Strength Range	-	2.55 to 3.93 MPa (370 to 570 lbf/in^2)
Coefficient of Thermal Expansion	-	9.45 mm/mm/°C (5.25 in/in/°F)

Base

AASHTO Classification	-	Granular base (A-1-a)

Subbase

AASHTO Classification - Sandy subbase (A-3)

Subgrade

Subgrade was not classified because a sample was not obtained. It was reported to be A-4 silty clay material.

Chloride content testing was not conducted at this section. This section was built with epoxy coated steel bars.

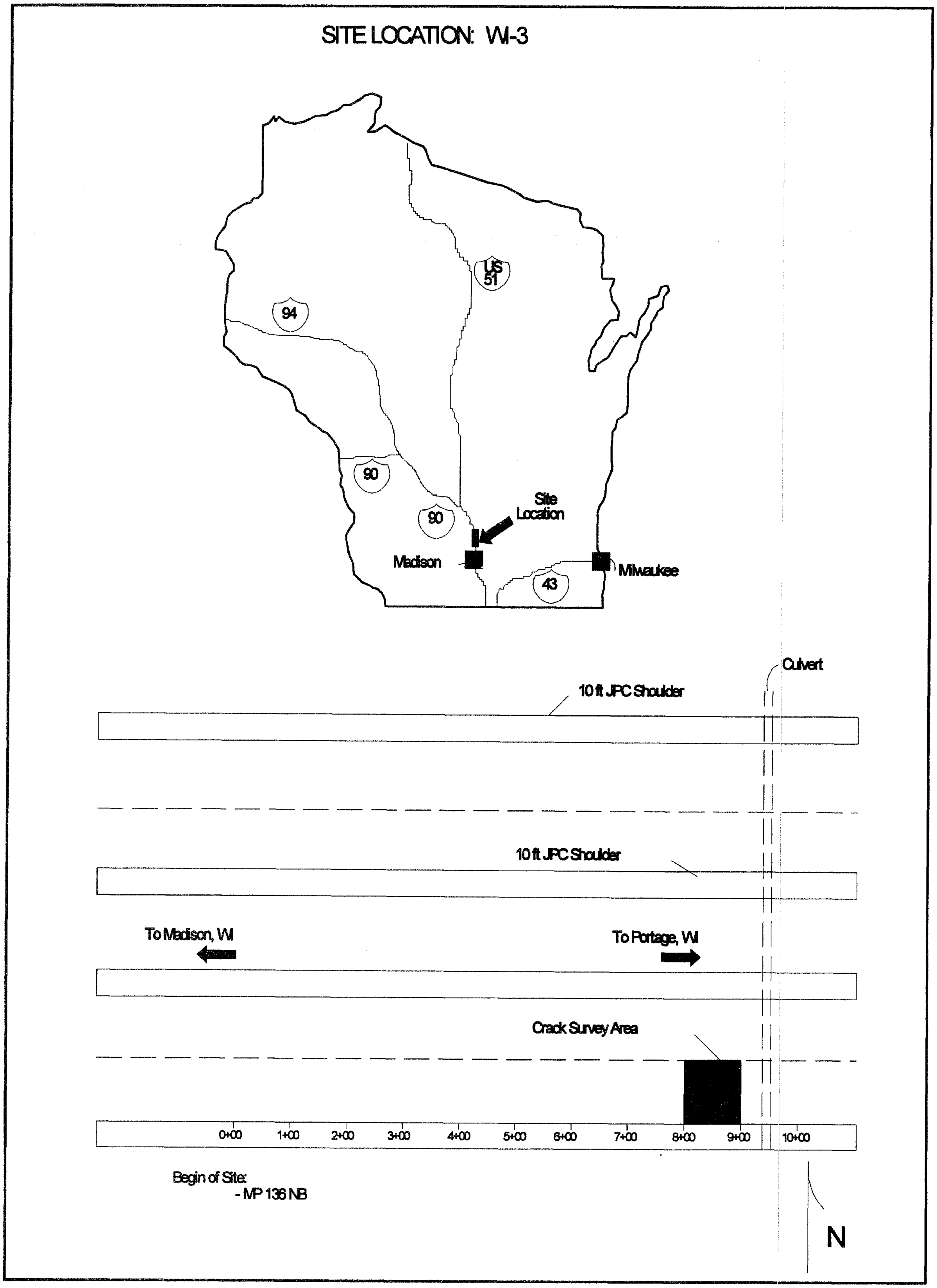
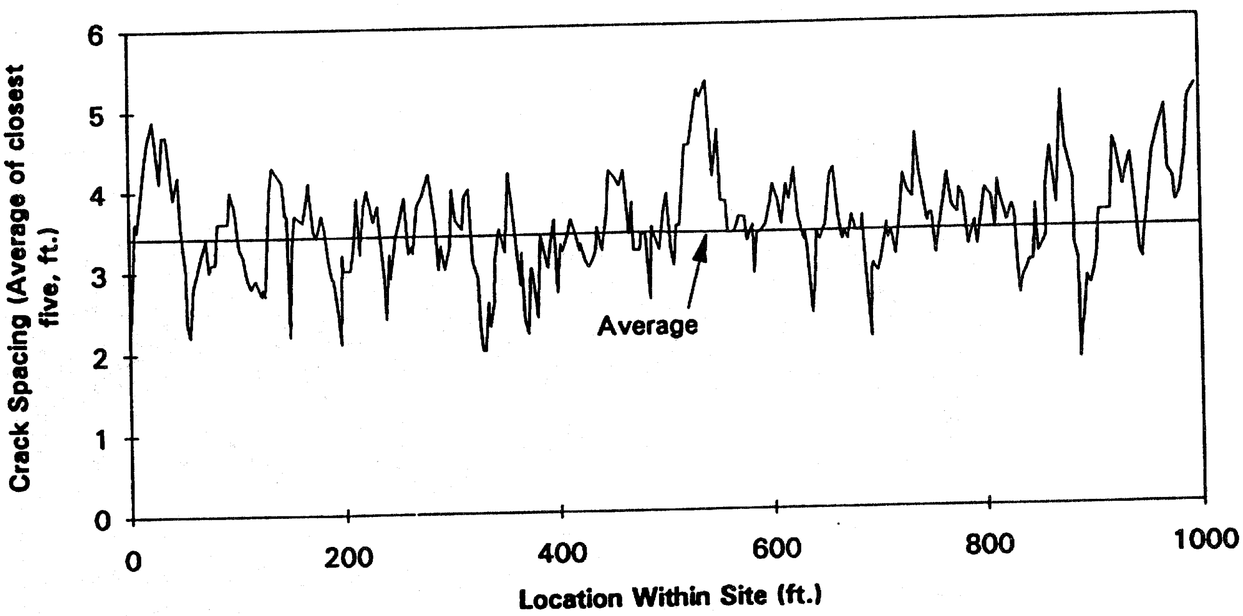


Figure 161. Site details for WI-3.



(0.305 m = 1 ft)

Figure 162. Crack spacing pattern at site WI-3.

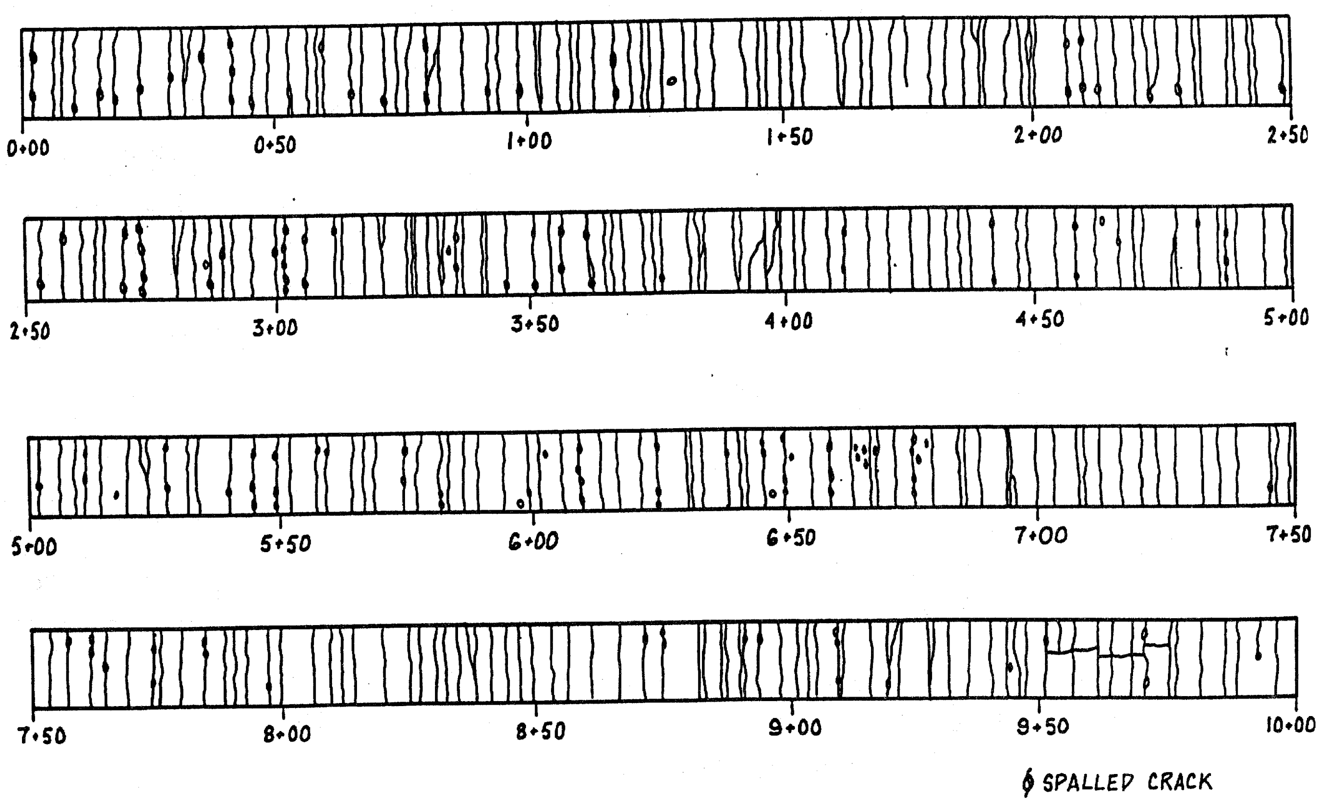
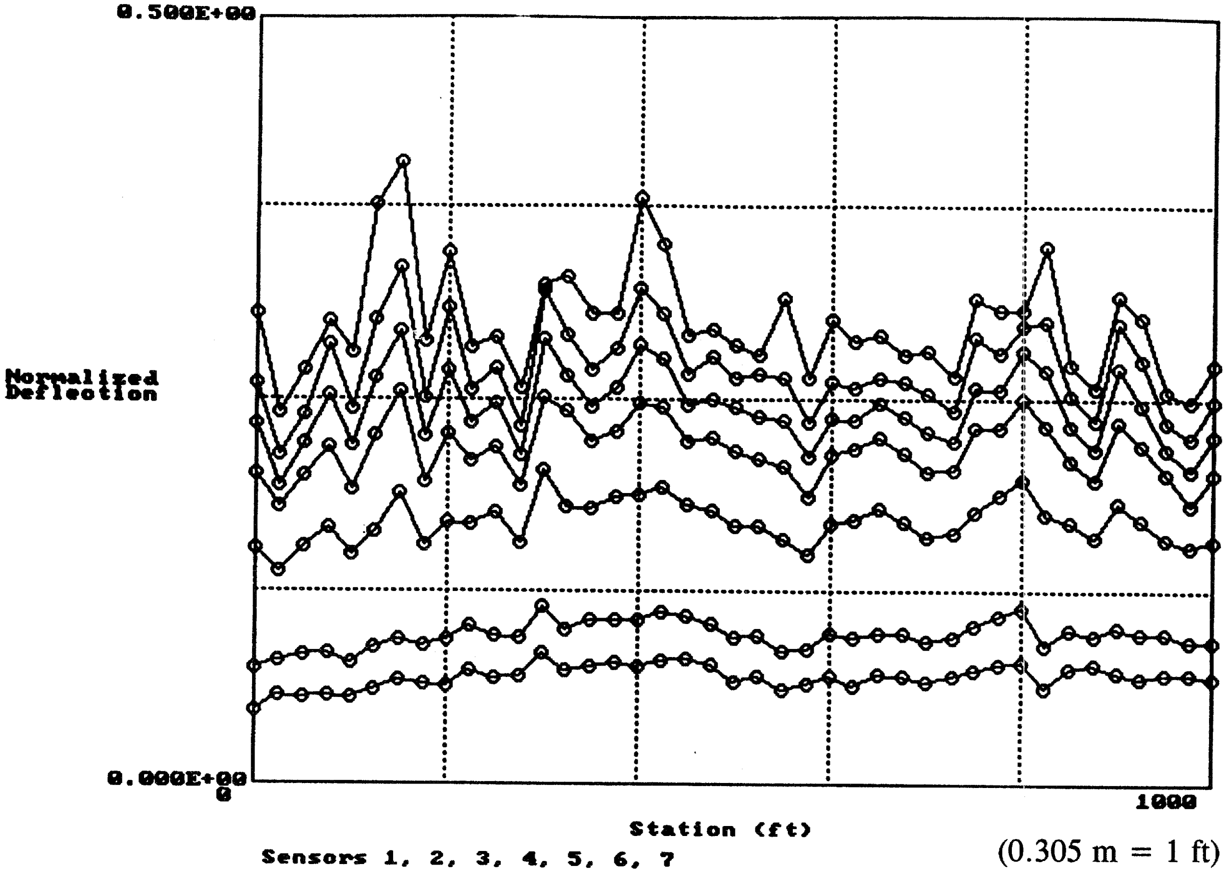
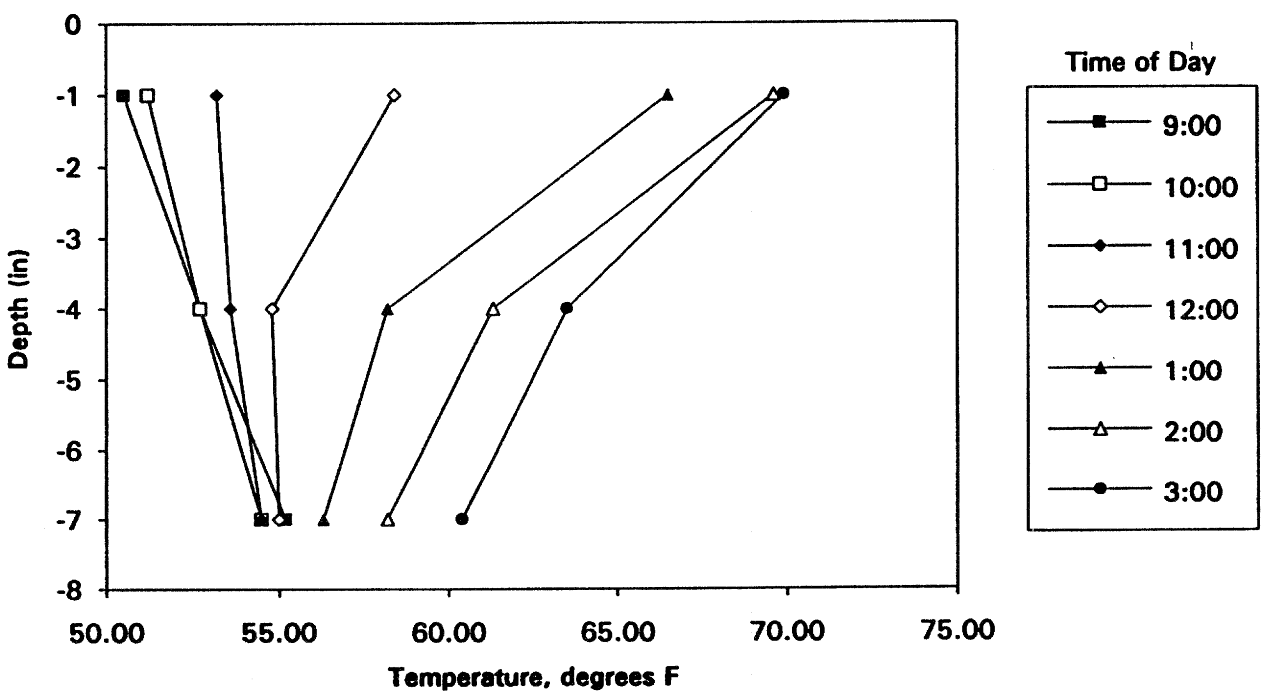


Figure 163. Crack pattern at site WI-3.



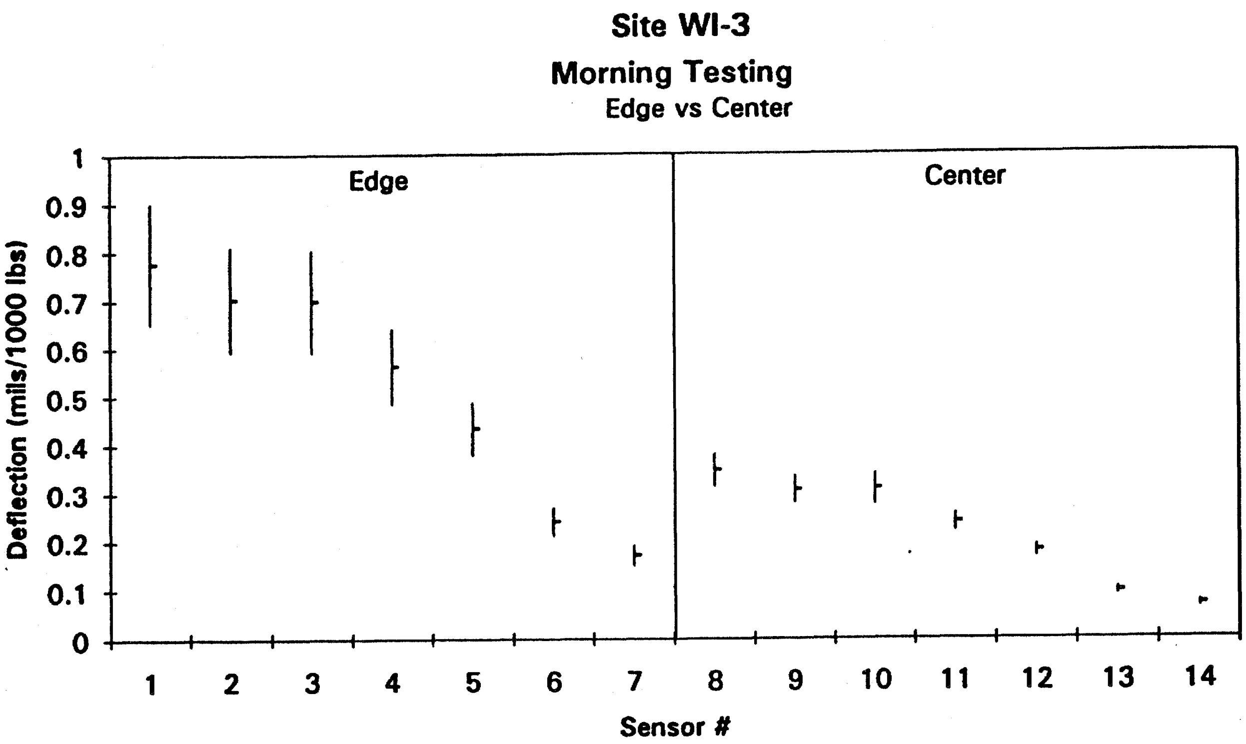
(0.305 m = 1 ft)

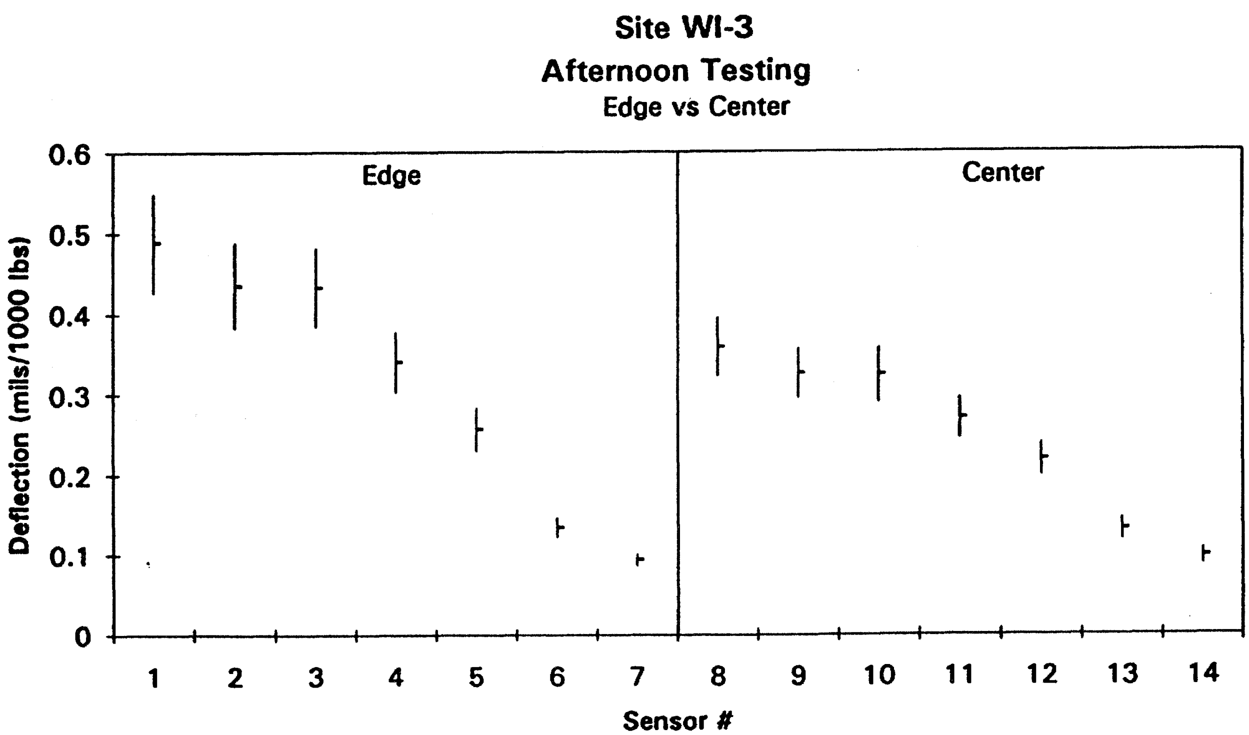
Figure 164. Deflections in inches along the section length at site WI-3.
[normalized per 4.45-kN [1,000-lb] load]



(25.4 mm = 1 in; 0.6°C = 1°F)

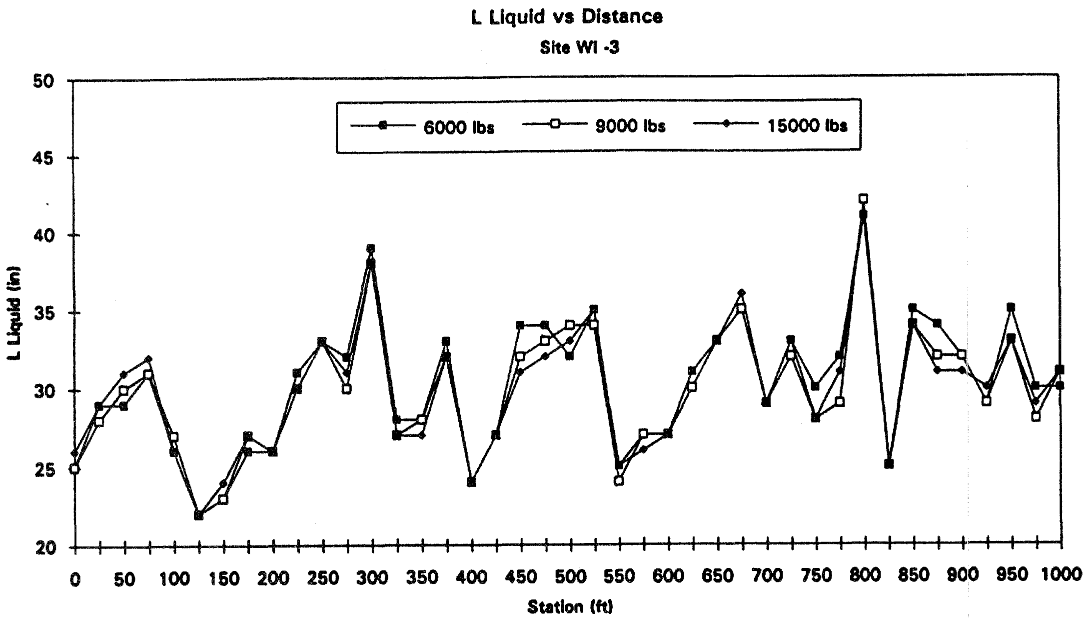
Figure 165. Temperature profile for site WI-3.





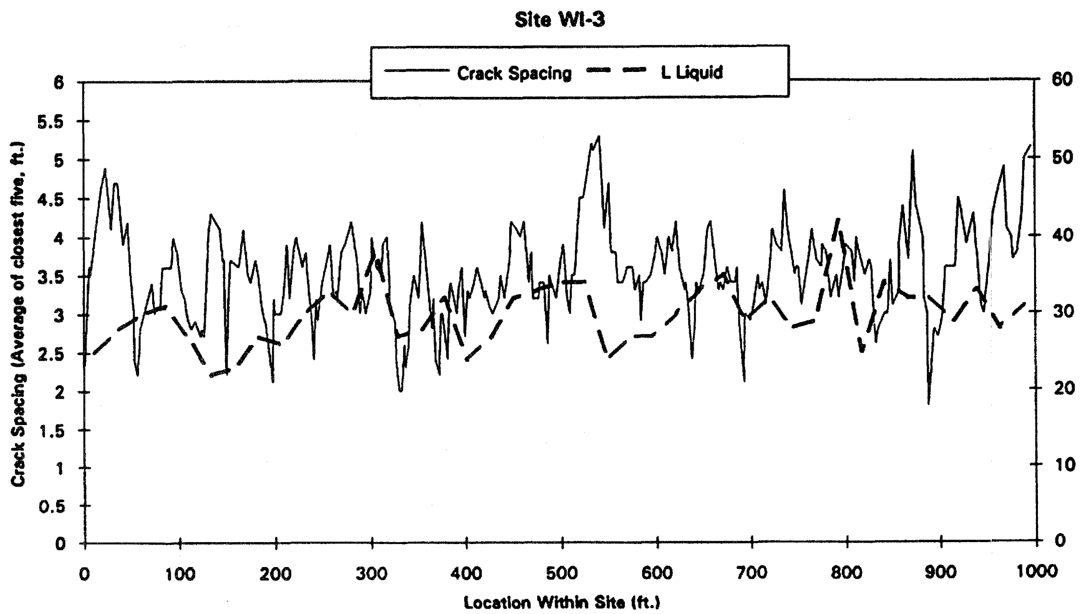
(0.0254 mm = 1 mil; 4.45 kN = 1,000 lb)

Figure 166. Center versus edge deflection for each of the seven sensors at site WI-3.
[40.03 kN (9,000 lb) load]



(25.4 mm = 1 in, 0.305 m = 1 ft)

Figure 167. Backcalculated values of radius of relative stiffness, ℓ (using program ILLI-BACK) for site WI-3.



(0.305 m = 1 ft)

Figure 168. Comparison of radius of relative stiffness values with average crack spacing (nearest five cracks) at site WI-3.

Section WI-4 - Interstate 90/94

Introduction

Section WI-4 was opened to traffic in 1984. WI-4 is a section of I-90/94 that runs northwest to southeast from the Wisconsin-Minnesota border to the Wisconsin-Illinois border as shown in figure 169. The length of the highway that was considered runs from milepost 111 west to Minnesota. A 305-m (1,000-ft) west bound section was selected as illustrated in figure 169. This section is in the southern region of Wisconsin just north of Madison.

The climate in this region is typically continental with a large annual temperature range and frequent short-period temperature changes. Most of the precipitation, 60 percent, occurs during a 5-month period from May to September. The highest monthly average temperature is 21.4°C (70.6°F) during July, and the lowest monthly average is -9.1°C (15.6°F) during January. I-90/94 lies within the wet-freeze environmental zone.

Traffic

The 1991 ADT was 42,550 vpd, and the cumulative ESALs up to 1991 was estimated to be 4,183,000.

Structure

I-90/94 is a six-lane divided highway. The pavement lane width is 3.7 m (12 ft) with an outside shoulder width of 3.0 m (10 ft) and an inside shoulder width of 3.0 m (10 ft). The shoulders are constructed of PCC.

This section was designed as a four-layer system comprised of a 254-mm (10-in) CRCP, a 152-mm (6-in) crushed stone base course, and a 229-mm (9-in) sandy subbase over a subgrade of in-situ material. The CRC pavement was designed with a steel percentage of 0.67. The reinforcing steel are #6 bars spaced at 165 mm (6.5 in). The transverse steel are #4 bars spaced at 1 219 mm (48 in). The bars were placed on chairs as a single layer during the construction of the pavement. The bars are cathodically protected. The concrete shoulders (jointed) were tied to the mainline CRC pavement.

The average concrete pavement thickness for WI-4 was determined to be 269 mm (10.6 in) based on cores taken as part of this study. The concrete is considered well graded and consolidated. The aggregate type used was a stone (quartz) with a maximum size of 51 mm (2 in).

The granular base course was determined to be 155 mm (6.1 in) and well graded with an AASHTO classification of A-1-a. The subbase material thickness was not determined. Its AASHTO classification was A-3. The subgrade material was not classified but was reported to be A-2-4 loess material.

Design/Construction

No design data were available for this section.

Performance

The performance of I-90/94 measured in 1990 according to the PSI scale, 0.0 to 5.0, was 3.4. A PDI survey was done in 1991, and the value was 47.

Historical Data (M&R)

No information about maintenance and rehabilitation work was available for this section.

Field Investigation Data

Visual Condition Survey

- Total number of cracks = 225
  - 0 were at high severity
  - 192 were at medium severity
  - 33 were at low severity
- Average crack spacing = 1.40 m (4.59 ft)
- Standard deviation for crack spacing = 0.68 m (2.22 ft)
- Coefficient of variation of the crack spacing = 48 percent
- Figure 170 shows average crack spacing distribution based on the closest five cracks
- Figure 171 shows actual crack spacing
- Extent of Y cracks = 10 percent
- Cluster cracking was apparent at stations 0+00, 4+50, 7+50, and 9+50
- Other distress types included:
  - One PCC patch (low severity)
- Pavement shoulders were in fair condition.

Drainage Survey

- Ditches line both sides of the pavement.
- Ditches were lined with vegetation. There was no sign of ponded water or wet ground.

Windshield Survey

- 8.05 km (5 mi) of pavement surveyed was generally in good condition exhibiting seven PCC patches.

Terminal Joint Survey

- Terminal joints were not surveyed because of time constraints

Deflection Testing

- Figure 172 shows normalized/basin (per 4.45 kN (1,000 lb) load) deflections measured along the length of the section (for all seven sensors using the 40.03 kN (9,000 lb) load)
- Load transfer efficiency at cracks at mid-slab
  - 89 to 97 percent in the morning
  - 86 to 93 percent in the afternoon
- Load transfer efficiency at cracks along edge
  - 90 to 97 percent in the morning
  - 92 to 95 percent in the afternoon
- Figure 173 shows the distribution of slab temperature with depth and time of day.
- Figure 174 shows the average deflections and ranges of deflections for each sensor for center and edge loading, for morning and afternoon testing (using the 40.03 kN (9,000 lb) load within the selected 30.5-m (100-ft) subsection).
- Figure 175 shows the backcalculated radius of relative stiffness, ℓ, values for basin testing for each of the three nominal load levels of 40.03, 53.38, and 71.17 kN (9,000, 12,000, and 16,000 lb).
- ℓ value based on 40.03 kN (9,000 lb) load basin testing
  - Average = 965 mm (38 in)
  - Standard Deviation = 178 mm (7 in)
- ℓ value at crack locations at mid-slab (morning/afternoon)
  - Average = 686/889 (27/35 in)
  - Standard Deviation = 76/76 mm (3/3 in)
- ℓ value at crack locations at edge (morning/afternoon)
  - Average = 787/686 mm (31/27 in)
  - Standard Deviation = 102/102 mm (4/4 in)
- Estimated ℓ value using actual slab thickness and concrete modulus of elasticity and assumed modulus of elasticity
  - Value = 914 mm (36 in)
- Figure 176 shows the variation of ℓ in relation to crack spacing (average of the closest five cracks)
- Average deflection under 40.03 kN (9,000 lb) load (0.000001 mm (0.001 in))
  - Basin test - 0.081 (3.2)
  - Mid-slab crack location (morning/afternoon) - 0.109/0.097 (4.3/3.8)
  - Edge crack location (morning/afternoon) - 0.406/0.130 (16/5.1)

Crack Width Measurements

- Number of cracks monitored within the 30.5-m (100-ft) subsection = 23

- Morning testing
  - Average temperature 102 mm (4 in) below surface = 10.0°C (50°F)
  - Average crack width = 0.63 mm (0.025 in)
- Afternoon testing
  - Average temperature 102 mm (4 in) below surface = 19.4°C (67°F)
  - Average crack width = 0.45 mm (0.018 in)
- Slab length change, mm/mm/°C (in/in/°F) = 14.4 millionth (8.0 millionth)

Reinforcing Steel Depths and Spacing

- Steel depth measurement offsets = 610, 1 219, 1 829 mm (24, 48, 72 in)
- Range of depth of cover = 99 to 147 mm (3.9 to 5.8 in)
- Average depth of cover = 122 mm (4.8 in)
- Standard deviation for depth of cover = 10 mm (0.4 in)
- Steel spacing was not measured

Corrosion Potential Testing (copper-copper sulfate half cell potentiometer)

- Range of measurements = -0.37 to -0.55 volts
- Average measurement = -0.46 volts
- Standard deviation for the measurement = -0.03 volts
- Potential for corrosion was indicated

Concrete Core Examination for Corrosion

- All four cores exhibited spread corrosion

Profile Testing

- Average IRI over the 305-m (1,000-ft) section = 1 991 mm/km (126 in/mi)

Laboratory Testing

The following data were obtained from the testing of the concrete, base/subbase, and subgrade materials:

Concrete

Modulus of Elasticity	- 35 160 MPa (5,100,000 lbf/in^2)
Average Split Tensile Strength	- 4.34 MPa (630 lbf/in^2)
Split Tensile Strength Range	- 3.93 to 5.38 MPa (570 to 780 lbf/in^2)
Coefficient of Thermal Expansion	- 9.05 mm/mm/°C (5.03 in/in/°F)

Base

AASHTO Classification	- Granular base (A-1-a)

Subbase

AASHTO Classification - Granular subbase (A-3)

Subgrade

The subgrade layer was not classified because a sample was not obtained. It was reported to be A-2-4 loess material.

Chloride content testing was conducted on pulverized concrete retrieved at various depths and locations. Total chloride content and water soluble chloride content values are listed below for WI-4.

Location	Depth, mm (in)	Total Chloride Content (% wt.)	Water Soluble Chlorides (% wt.)
No steel below sample	0-51 (0-2)	0.228	0.215
	51-102 (2-4)	0.092	0.086
Steel directly below sample	0-51 (0-2)	0.268	0.237
	51-102 (2-4)	0.095	0.088

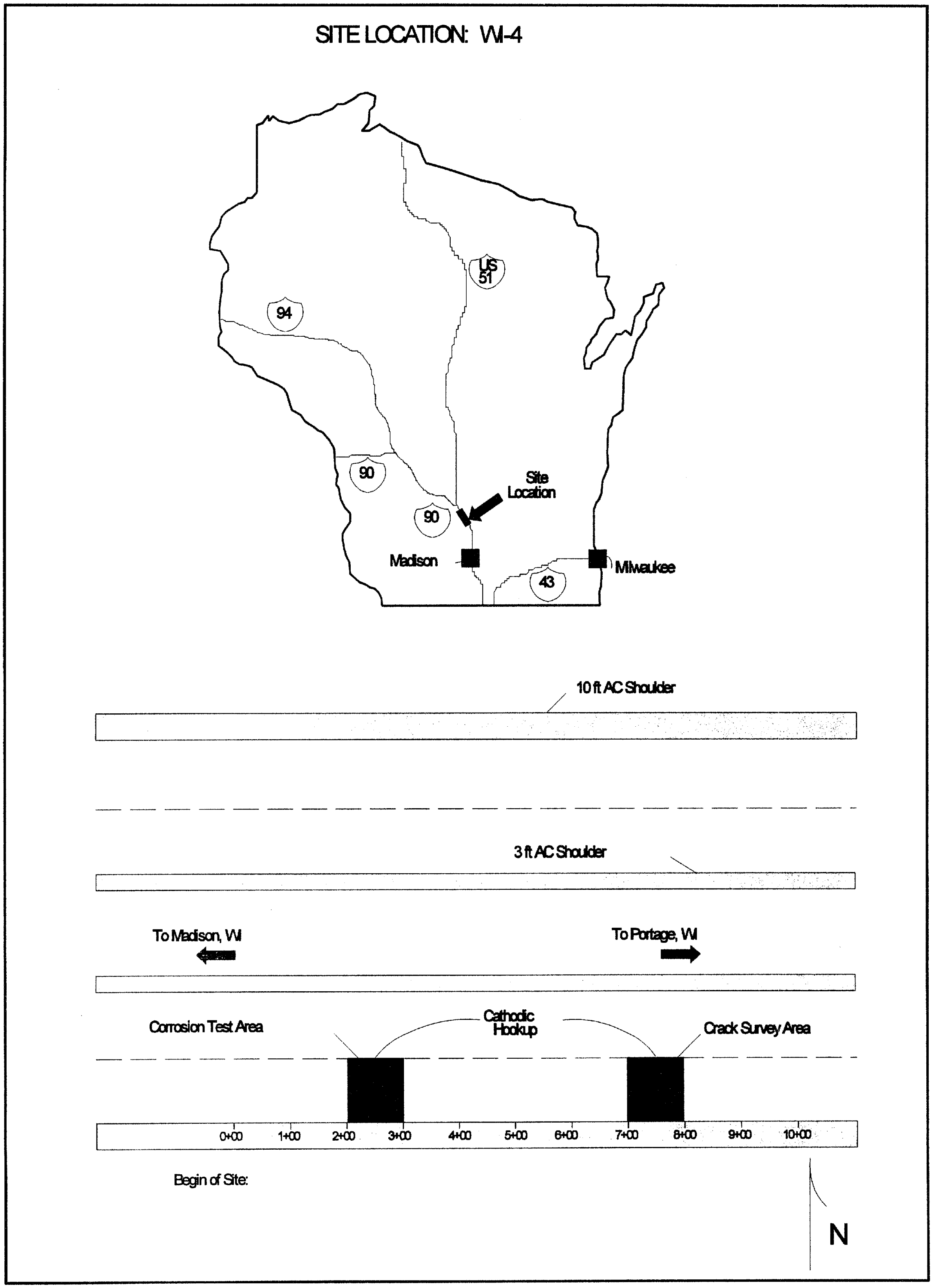
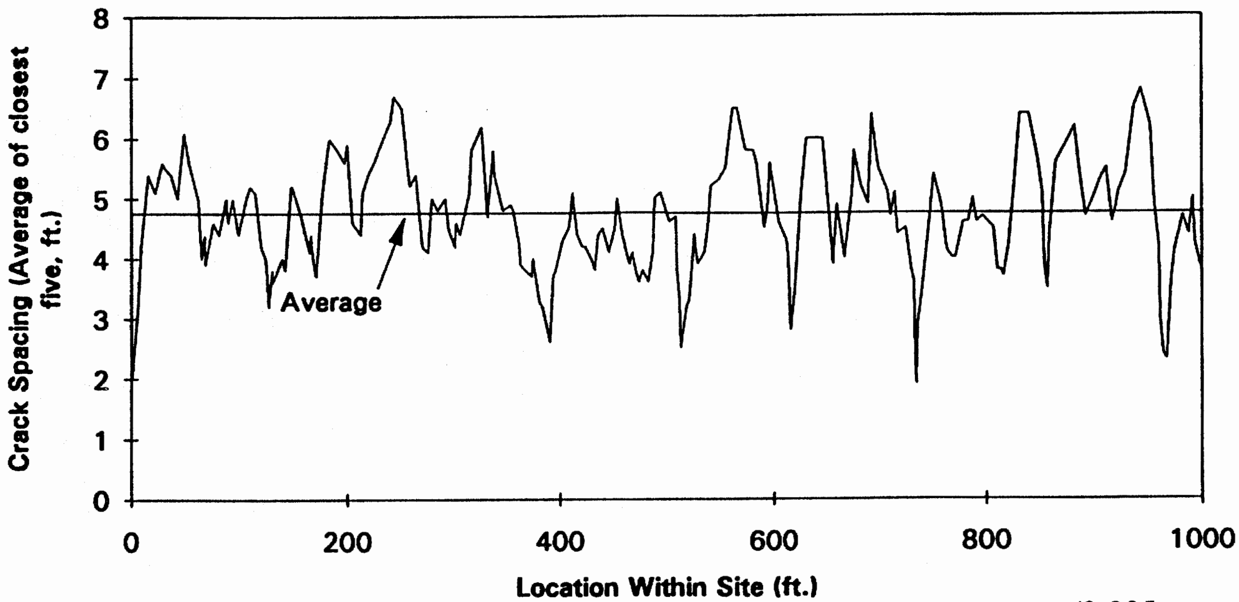


Figure 169. Site details for WI-4.



(0.305 m = 1 ft)

Figure 170. Crack spacing pattern at site WI-4.

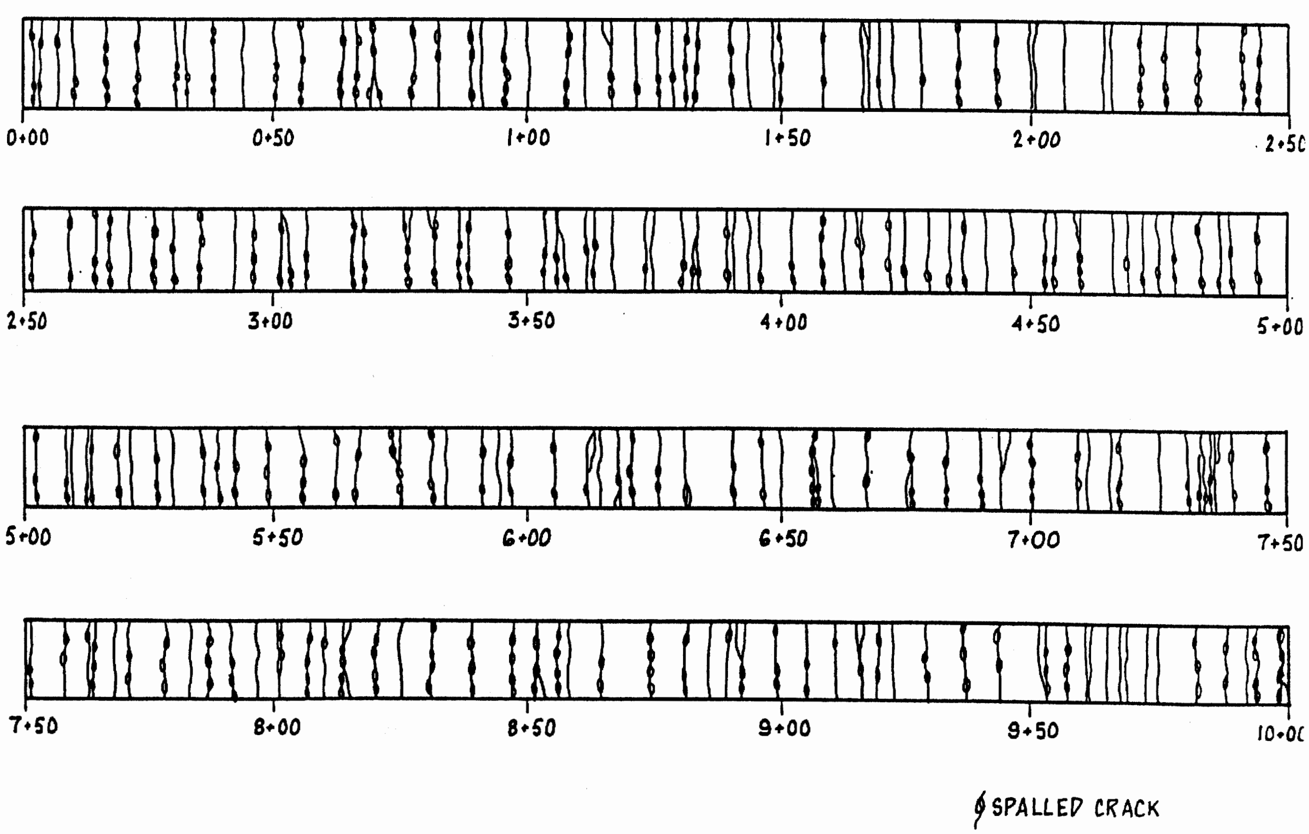
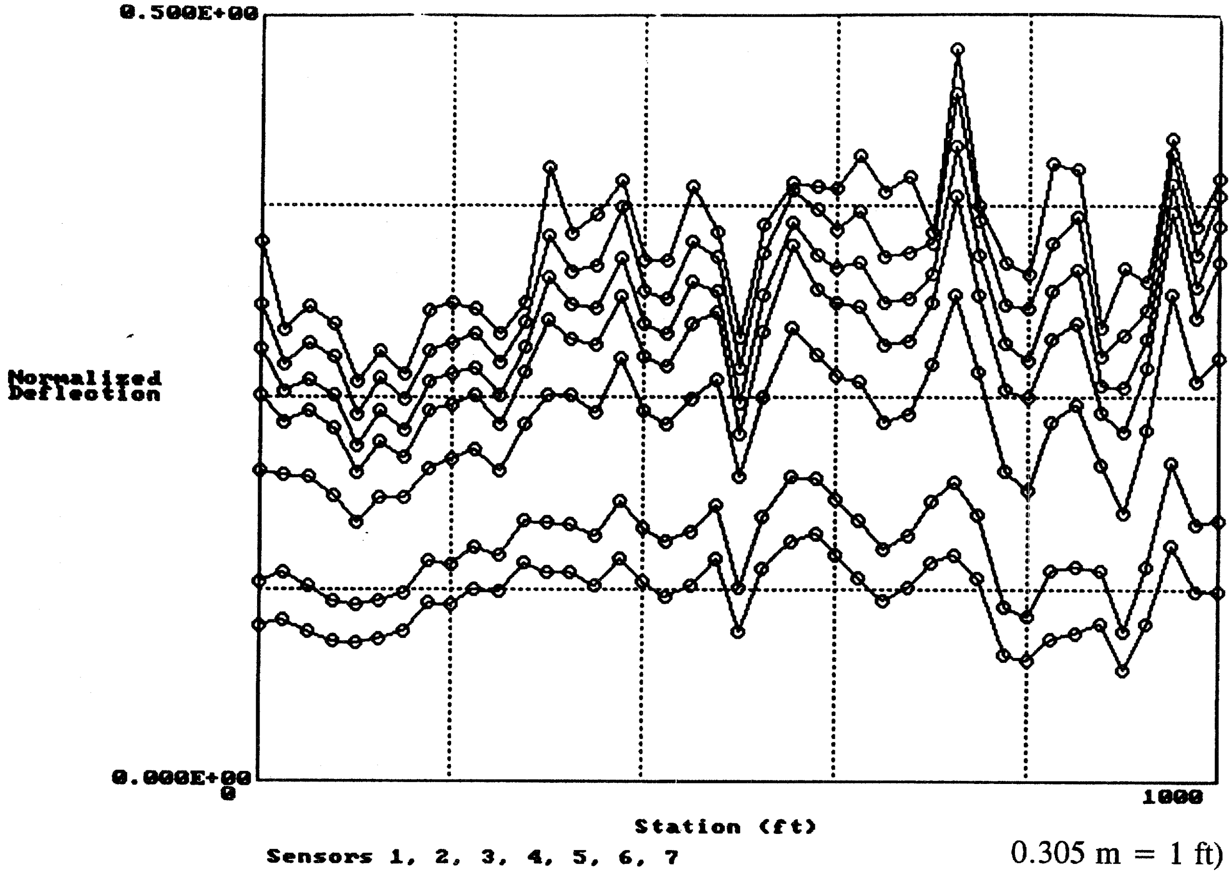
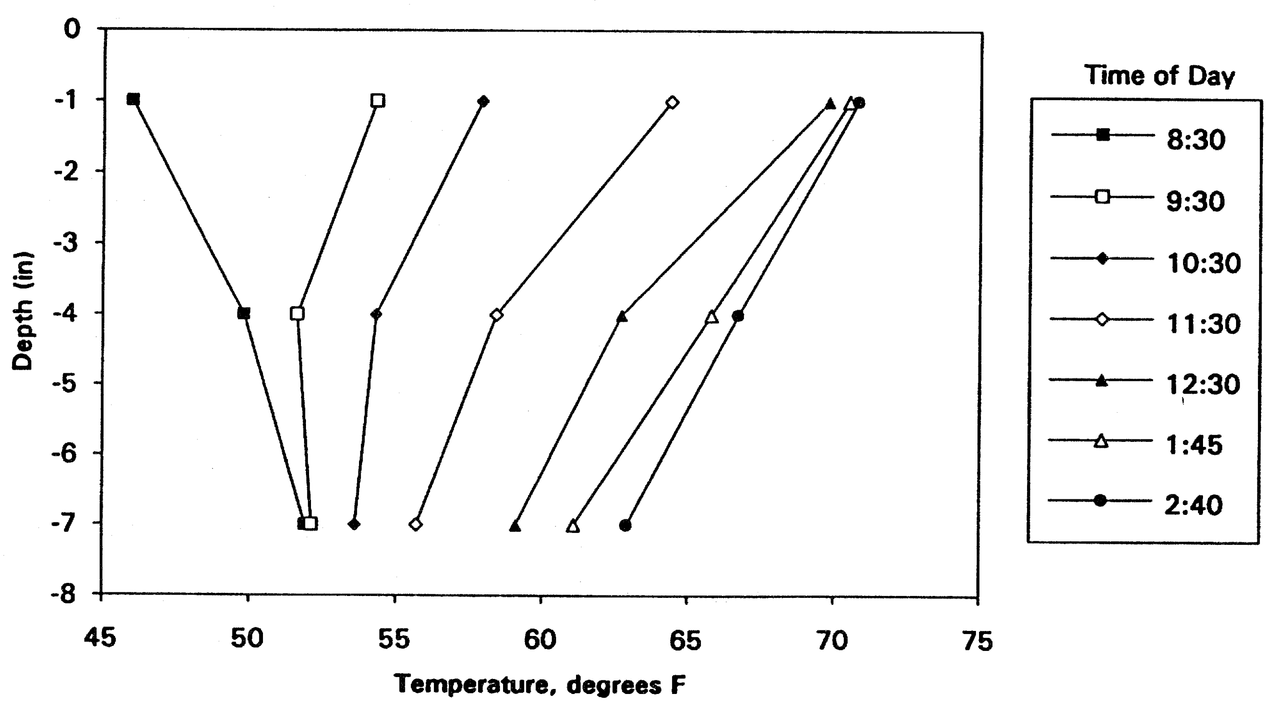


Figure 171. Crack pattern at site WI-4.



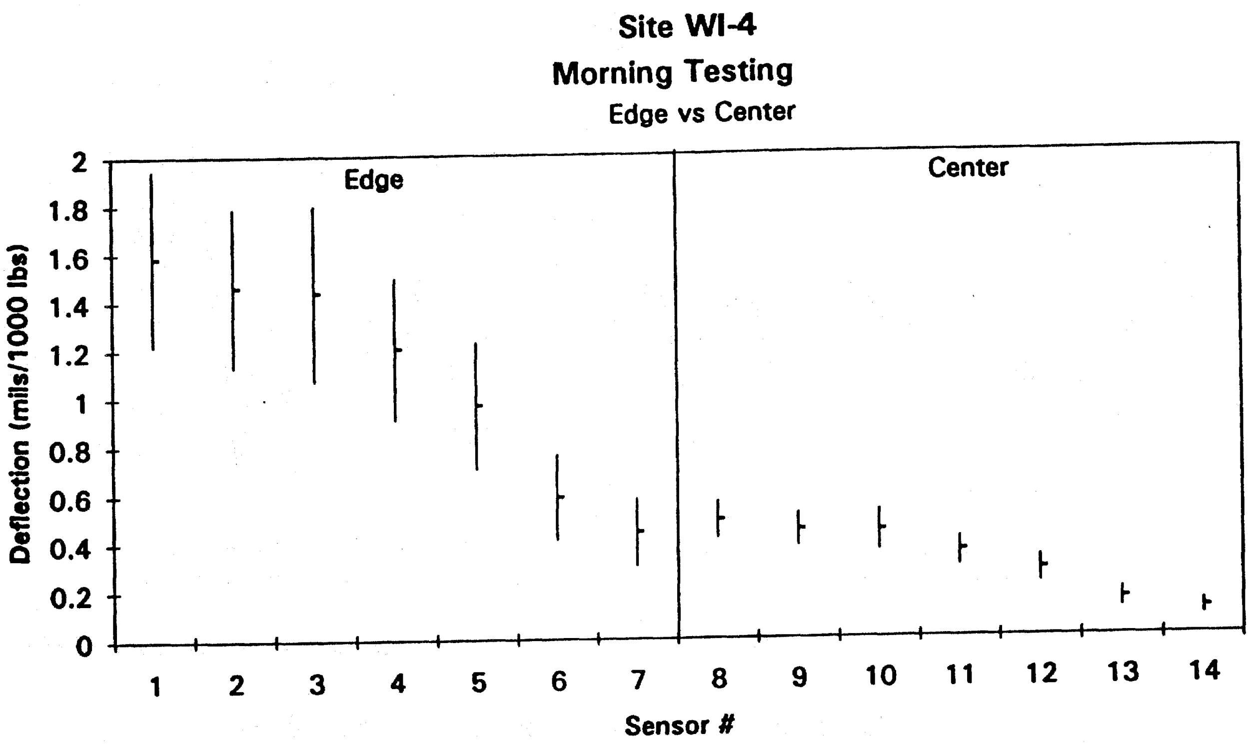
0.305 m = 1 ft)

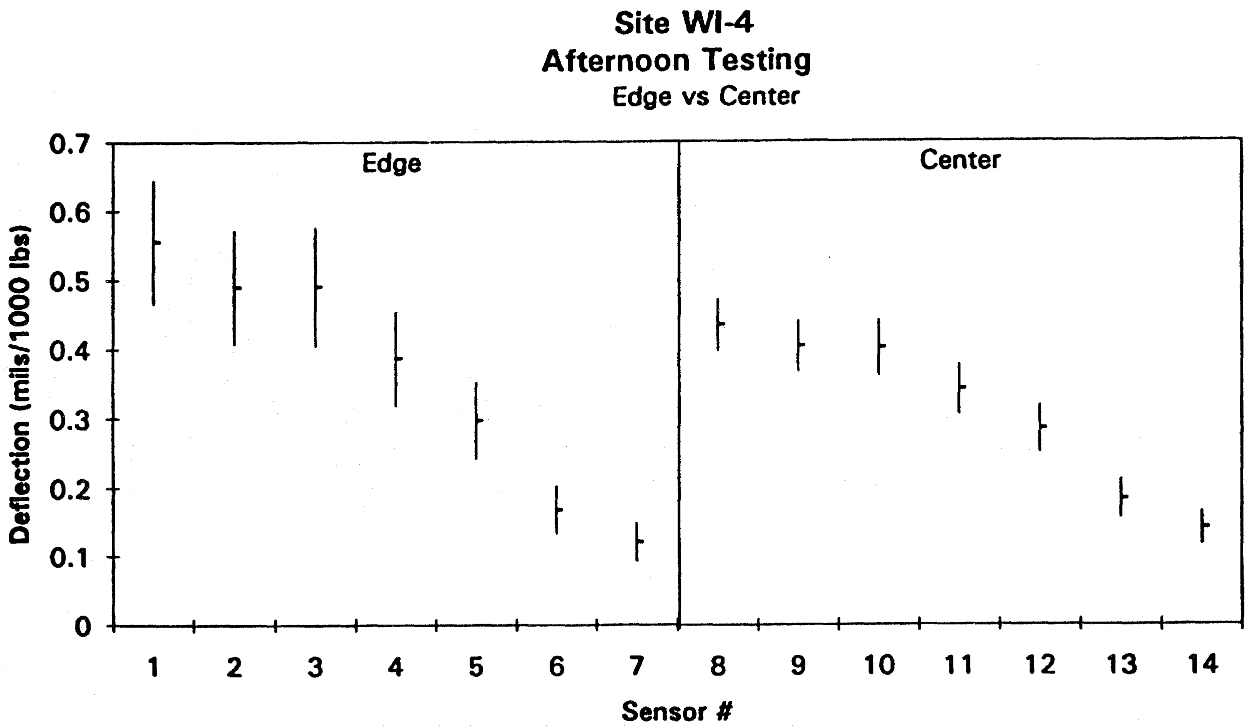
Figure 172. Deflections in inches along the section length at site WI-4.
[normalized per 4.45-kN [1,000-lb] load]



(25.4 mm = 1 in; 0.6°C = 1°F)

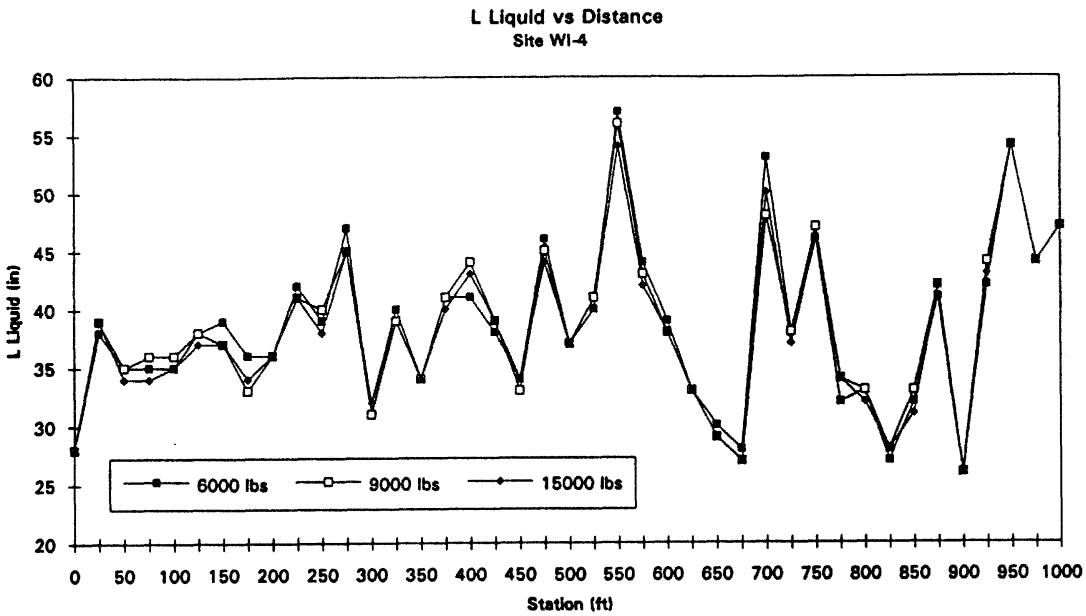
Figure 173. Temperature profile for site WI-4.





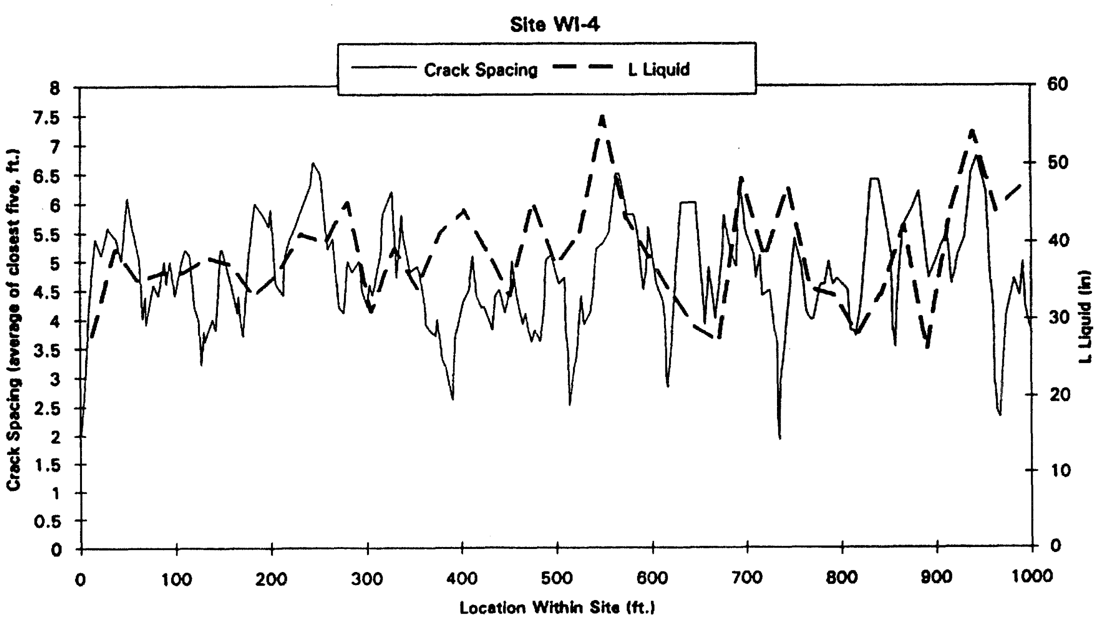
(0.0254 mm = 1 mil; 4.45 kN = 1,000 lb)

Figure 174. Center versus edge deflection for each of the seven sensors at site WI-4.
[40.03 kN (9,000 lb) load]



(25.4 mm = 1 in, 0.305 m = 1 ft)

Figure 175. Backcalculated values of radius of relative stiffness, ℓ (using program ILLI-BACK) for site WI-4.



(0.305 m = 1 ft)

Figure 176. Comparison of radius of relative stiffness values with average crack spacing (nearest five cracks) at site WI-4.

Section WI-5 - Interstate 90/94

Introduction

Section WI-5 was opened to traffic in 1975. WI-5 is a section of I-90/94 that runs northwest to southeast from the Wisconsin-Minnesota border to the Wisconsin-Illinois border as shown in figure 177. The length of the highway that was considered runs from milepost 99 east to Madison. A 305-m (1,000-ft) west bound section was selected for detailed inspection between mileposts 99 and 100, illustrated in figure 177. This section of I-90/94 is in the western region of Wisconsin.

The climate in this region is typically continental with a large annual temperature range and frequent short period temperature changes. Most of the precipitation, 60 percent, occurs during a 5-month period from May to September. The highest monthly average temperature is 22.8°C (73.0°F) during July, and the lowest monthly average is -10.0°C (14.0°F) during January. I-90/94 lies within the wet-freeze environmental zone.

Traffic

The 1991 ADT was 26,890 vpd. The estimate of cumulative ESALs to 1991 was not available.

Structure

This section of I-90/94 is a four-lane divided highway. The pavement lane width is 3.7 m (12 ft) with an outside shoulder width of 1.8 m (6 ft) and an inside shoulder width of 0.6 m (2 ft). The shoulders are constructed of asphaltic concrete.

This section was designed as a four-layer system comprised of a 203-mm (8-in) CRCP, a 152-mm (6-in) crushed stone base course, and a 229-mm (9-in) granular pit-run subbase over a sandy subgrade. The CRC pavement was designed with a steel percentage of 0.61. The reinforcing steel are #6 bars spaced at 229 mm (9 in). The steel was tube fed as a single layer during the construction of the pavement. The bars are not epoxy coated.

The average concrete pavement thickness for WI-5 was determined to be 203 mm (8 in) based on cores taken as part of this study. The concrete is considered well graded, and its consolidation is average. The aggregate type used was a stone (quartz) with a maximum size of 51 mm (2 in).

The granular base course was determined to be 178 mm (7 in) and well graded with an AASHTO classification of A-1-a. The subbase material could not be differentiated from the subgrade. The subgrade material was classified as a A-2-6 material with a liquid limit of 15 and a plastic limit of 0.

Design/Construction

No design data were available for this section.

Performance

The performance of I-90/94 measured in 1990 according to the PSI scale, 0.0 to 5.0, was 4.0. A PDI survey was done in 1991, and the value was 94.

Historical Data (M&R)

No information about maintenance and rehabilitation work were available for this section.

Field Investigation Data

Visual Condition Survey

- Total number of cracks = 306
  - 1 was at high severity
  - 291 were at medium severity
  - 14 were at low severity
- Average crack spacing = 1.03 m (3.39 ft)
- Standard deviation for crack spacing = 0.56 m (1.84 ft)
- Coefficient of variation of the crack spacing = 54 percent
- Figure 178 shows average crack spacing distribution based on the closest five cracks
- Figure 179 shows actual crack spacing
- Extent of Y cracks = 13 percent
- Cluster cracking was apparent throughout the test section
- Other distress types included:
  - 11 AC patches (low severity)
  - 13 PCC patches (low severity)
  - <u>Longitudinal cracking (128 m (416 ft) moderate and 76 m (249 ft) high severity)</u>
- Pavement shoulders were in fair to poor condition.

Drainage Survey

- No information was collected

Windshield Survey

- 8.05 km (5 mi) of pavement surveyed was generally in fair to poor condition exhibiting about <u>4 390 m (14,400 ft) of longitudinal cracking condition, 67 PCC patches, 36 AC patches, and 4 punchouts.</u>

Terminal Joint Survey

- Terminal joints used = Lug anchor (two surveyed)
- Joints exhibited the following distresses:
  - One exhibited no distress
  - Others exhibited 5.1 m^2 (55 ft^2) of low to medium severity spalling and 10.2 m^2 (110 ft^2) of medium severity AC patch deterioration.

Deflection Testing

- Figure 180 shows normalized/basin (per 4.45 kN (1,000 lb) load) deflections measured along the length of the section (for all seven sensors using the 40.03 kN (9,000 lb) load)
- Load transfer efficiency at cracks at mid-slab
  - 89 to 94 percent in the morning
  - 91 to 95 percent in the afternoon
- Load transfer efficiency at cracks along edge
  - 88 to 94 percent in the morning
  - 89 to 94 percent in the afternoon
- Figure 181 shows the distribution of slab temperature with depth and time of day.
- Figure 182 shows the average deflections and ranges of deflections for each sensor for center and edge loading, for morning and afternoon testing (using the 40.03 kN (9,000 lb) load within the selected 30.5-m (100-ft) subsection).
- Figure 183 shows the backcalculated radius of relative stiffness, ℓ, values for basin testing for each of the three nominal load levels of 40.03, 53.38, and 71.17 kN (9,000, 12,000, and 16,000 lb).
- ℓ value based on 40.03 kN (9,000 lb) load basin testing
  - Average = 889 mm (35 in)
  - Standard Deviation = 127 mm (5 in)
- ℓ value at crack locations at mid-slab (morning/afternoon)
  - Average = 711/940 mm (28/37 in)
  - Standard Deviation = 51/229 mm (2/9 in)
- ℓ value at crack locations at edge (morning/afternoon)
  - Average = 711/762 mm (28/30 in)
  - Standard Deviation = 51/102 mm (2/4 in)
- Estimated ℓ value using actual slab thickness and concrete modulus of elasticity and assumed modulus of subgrade reaction
  - Value = 737 mm (29 in)
- Figure 184 shows the variation of ℓ in relation to crack spacing (average of the closest five cracks)
- Average deflection under 40.03 kN (9,000 lb) load (0.000001 mm (0.001 in))
  - Basin test - 0.137 (5.4)

- Mid-slab crack location (morning/afternoon) - 0.109/0.132 (4.3/5.2)
- Edge crack location (morning/afternoon) - 0.216/0.163 (8.5/6.4)

Crack Width Measurements

- Number of cracks monitored within the 30.5-m (100-ft) subsection = 28
- Morning testing
  - Average temperature 102 mm (4 in) below surface = 8.9°C (48°F)
  - Average crack width = 0.45 mm (0.018 in)
- Afternoon testing
  - Average temperature 102 mm (4 in) below surface = 17.8°C (64°F)
  - Average crack width = 0.28 mm (0.011 in)
- Slab length change, mm/mm/°C (in/in/°F) = 17.6 millionth (9.8 millionth)

Reinforcing Steel Depths and Spacing

- Steel depth measurement offsets = 305, 762, 1 219 mm (12, 30, 48 in)
- Range of depth of cover = 53 to 91 mm (2.1 to 3.6 in)
- Average depth of cover = 66 mm (2.6 in)
- Standard deviation for depth of cover = 7.6 mm (0.3 in)
- Steel spacing was not measured

Corrosion Potential Testing (copper-copper sulfate half cell potentiometer)

- Range of measurements = -0.04 to -1.00 volts
- Average measurement = -0.53 volts
- Standard deviation for the measurement = -0.14 volts
- Potential for corrosion was indicated

Concrete Core Examination for Corrosion

- All four cores exhibited spread corrosion

Profile Testing

- Average IRI over the 305-m (1,000-ft) section = 1 469 mm/km (93 in/mi)

Laboratory Testing

The following data were obtained from the testing of the concrete, base/subbase, and subgrade materials:

Concrete

Modulus of Elasticity - 36 540 MPa (5,300,000 lbf/in^2)

Average Split Tensile Strength	-	3.59 MPa (520 lbf/in^2)
Split Tensile Strength Range	-	3.31 to 3.93 MPa (480 to 570 lbf/in^2)
Coefficient of Thermal Expansion	-	8.06 mm/mm/°C (4.48 in/in/°F)

Base

AASHTO Classification	-	Granular base (A-1-a)

Subbase

AASHTO Classification	-	Granular subbase (A-2-6)

Subgrade

Liquid Limit	-	15 percent
Plastic Limit	-	0 percent
Percent Passing #200 Sieve	-	12.1 percent
AASHTO Classification	-	A-2-6

Chloride content testing was conducted on pulverized concrete retrieved at various depths and locations. Total chloride content and water soluble chloride content values are listed below for WI-5.

Location	Depth, mm (in)	Total Chloride Content (% wt.)	Water Soluble Chlorides (% wt.)
No steel below sample	0-51 (0-2)	0.236	0.233
	51-102 (2-4)	0.115	0.108
Steel directly below sample	0-51 (0-2)	0.273	0.250
	51-102 (2-4)	0.113	0.108

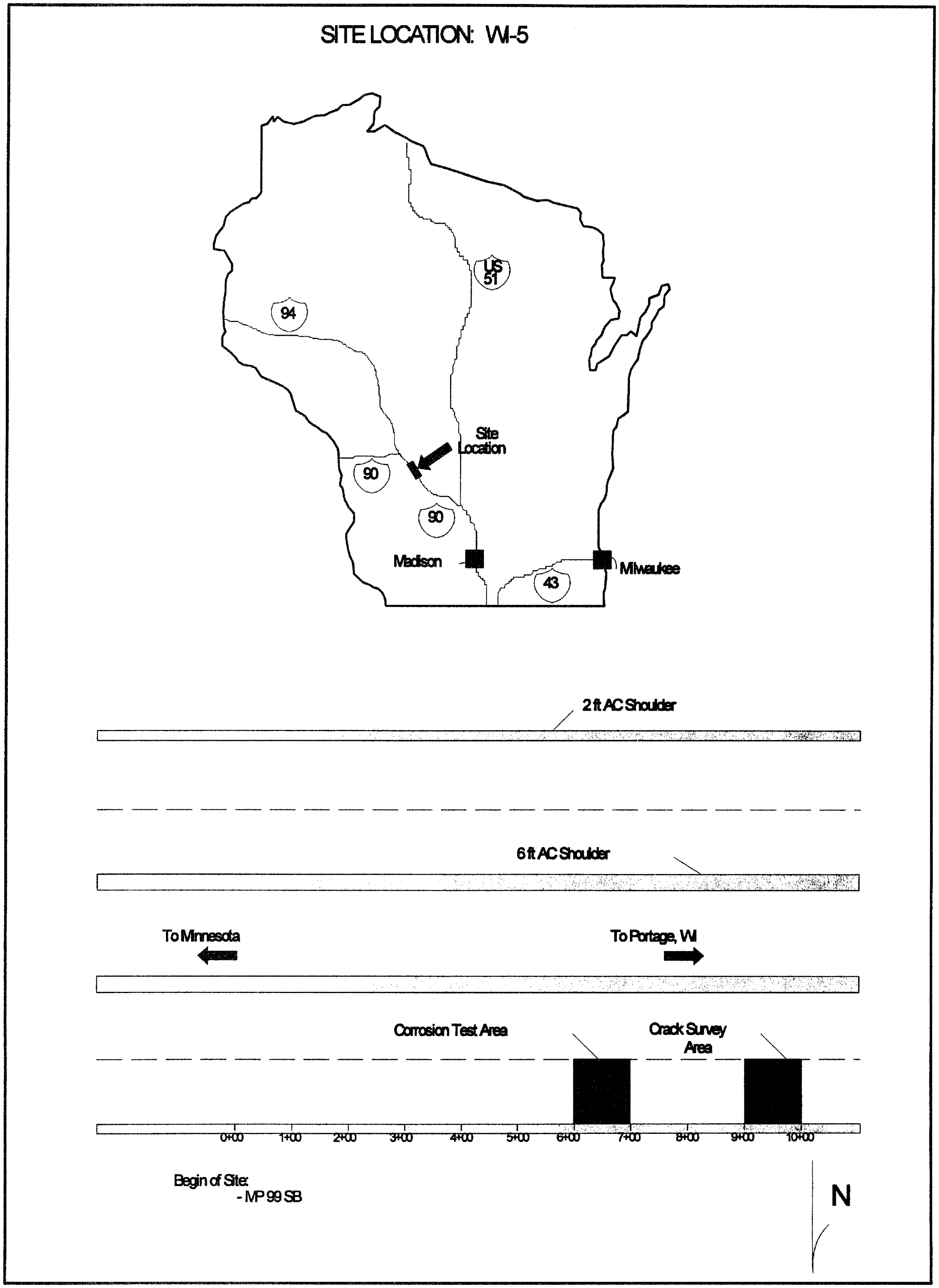
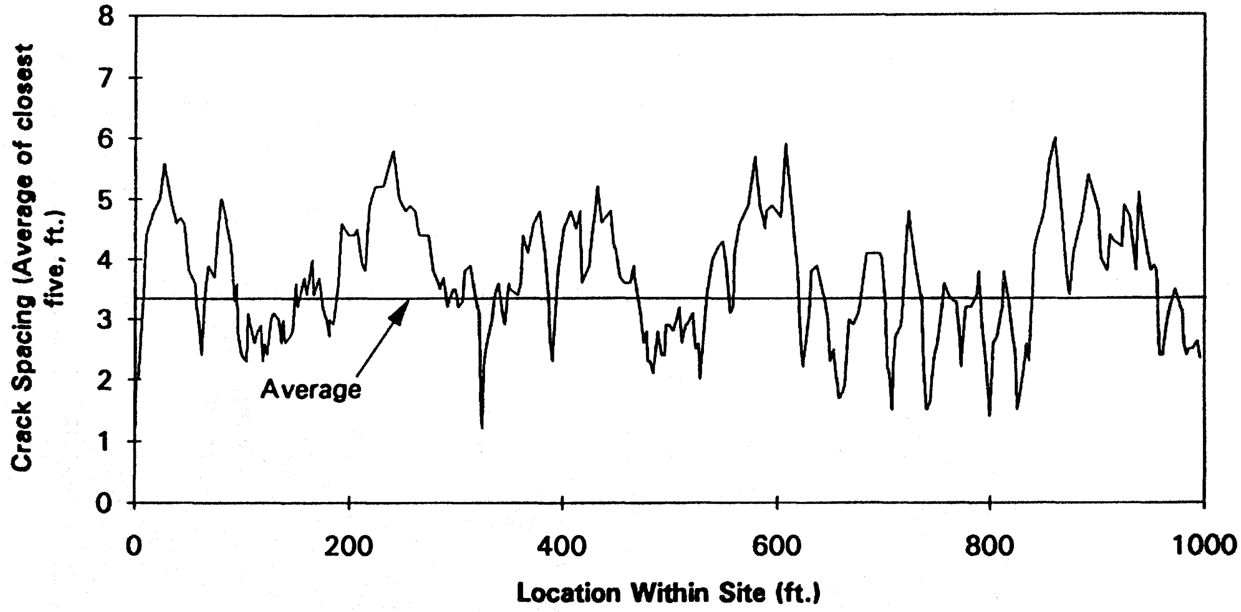


Figure 177. Site details for WI-5.



(0.305 m = 1 ft)

Figure 178. Crack spacing pattern at site WI-5.

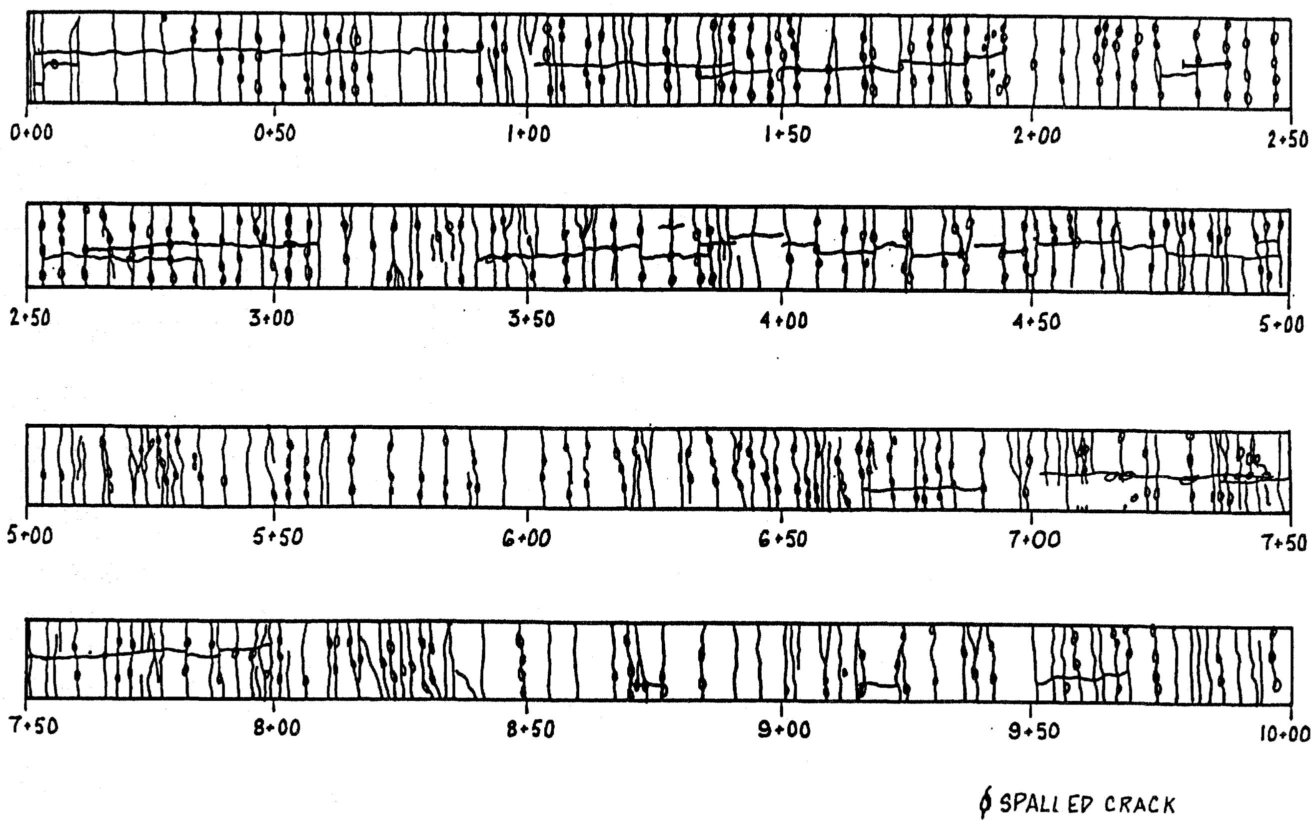
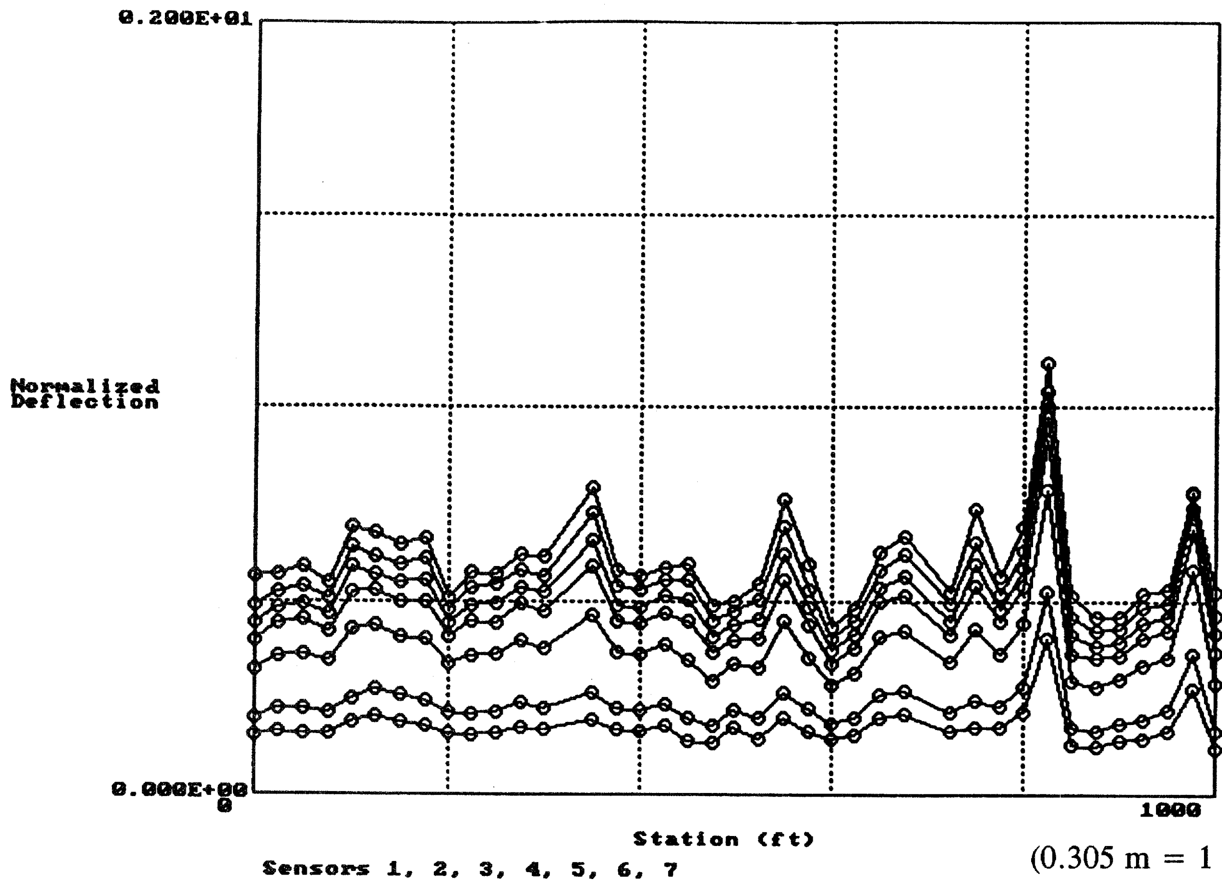
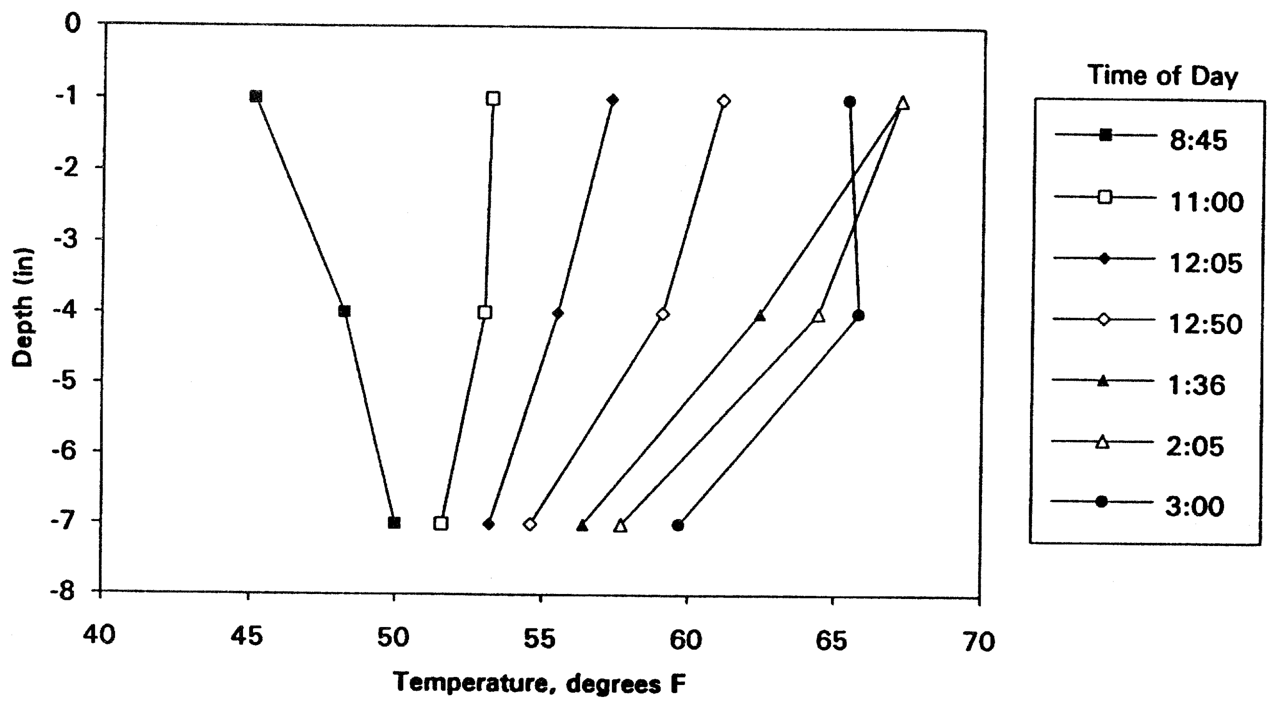


Figure 179. Crack pattern at site WI-5.



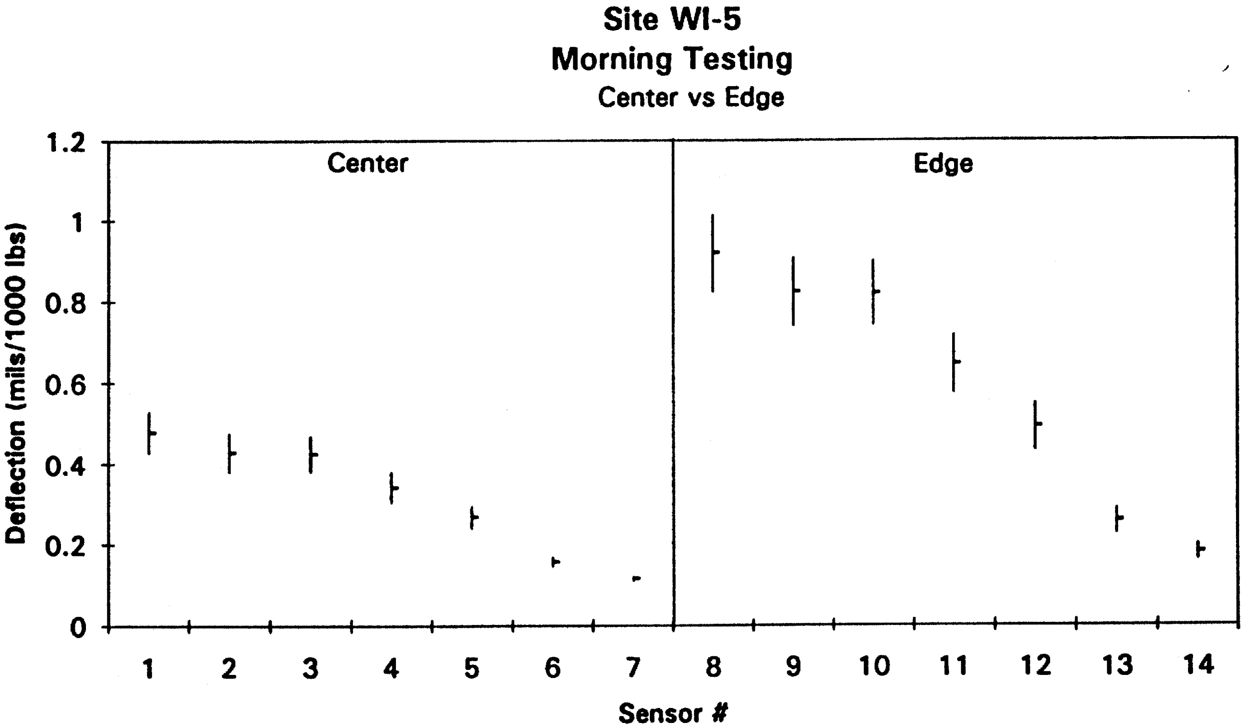
(0.305 m = 1 ft)

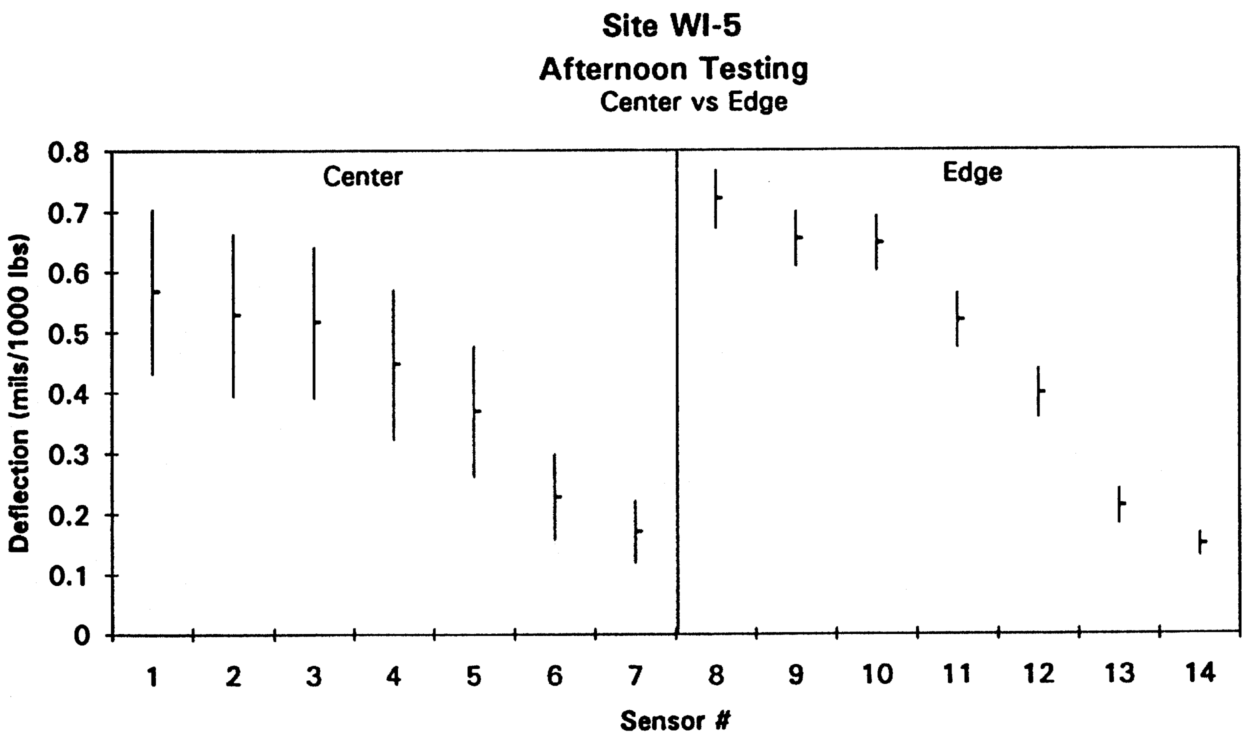
Figure 180. Deflections in inches along the section length at site WI-5.
[normalized per 4.45-kN [1,000-lb] load]



(25.4 mm = 1 in; 0.6°C = 1°F)

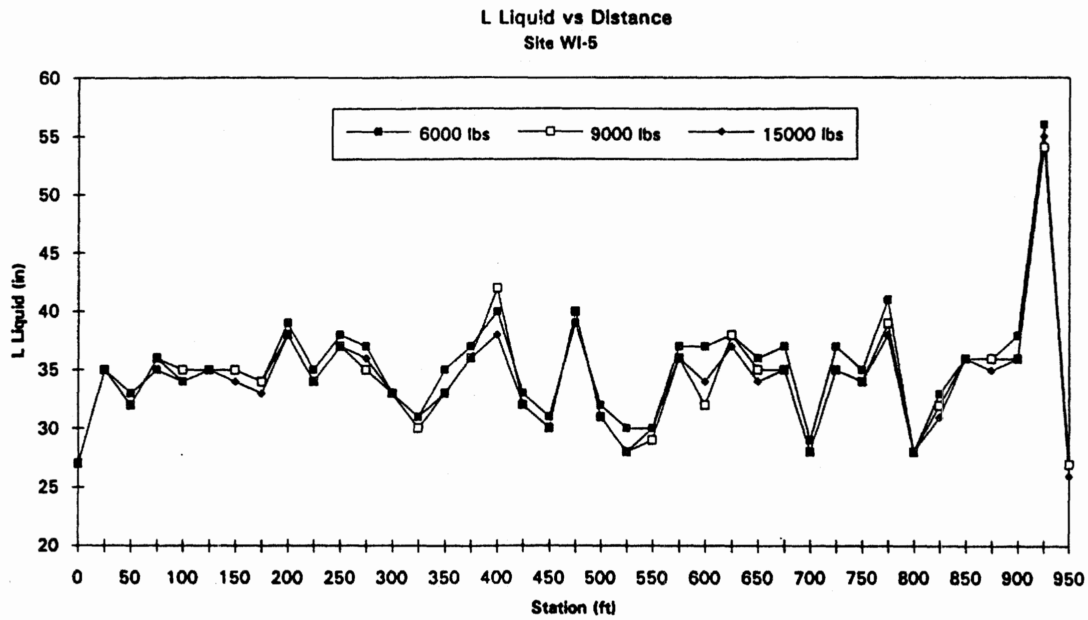
Figure 181. Temperature profile for site WI-5.





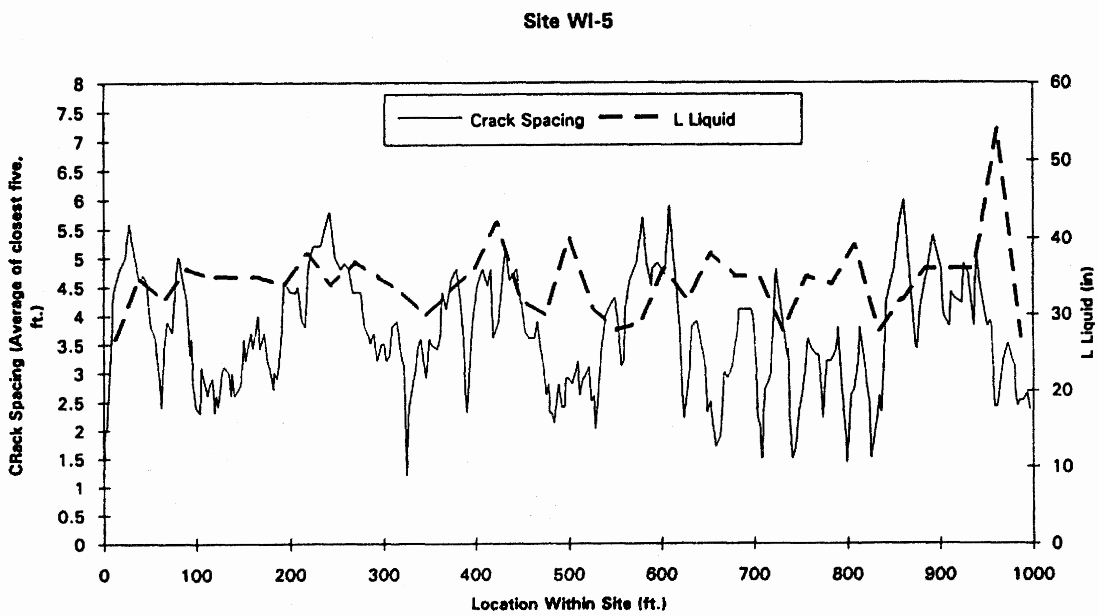
(0.0254 mm = 1 mil; 4.45 kN = 1,000 lb)

Figure 182. Center versus edge deflection for each of the seven sensors at site WI-5.
[40.03 kN (9,000 lb) load]



(25.4 mm = 1 in, 0.305 m = 1 ft)

Figure 183. Backcalculated values of radius of relative stiffness, ℓ (using program ILLI-BACK) for site WI-5.



(0.305 m = 1 ft)

Figure 184. Comparison of radius of relative stiffness values with average crack spacing (nearest five cracks) at site WI-5.

CHAPTER 9 - SUMMARY OF TEST DATA

General

As discussed earlier, this volume presents detailed inventory and field and laboratory test data for each of the 23 CRC pavement test sections included in the field investigation program. One of the major concerns at the beginning of the field study was the availability and reliability of data related to traffic along the test sections. Even though intensive interactions were made with appropriate State highway agencies, traffic data were not made available for many of the test sections because in most cases the reliable traffic data did not exist. This is not unusual as the same problem has been encountered on many similar pavement data collection programs including the LTPP program. For the LTPP program, the State highway agencies have initially provided the best estimates of the ESALs for the test sections while efforts are underway to perform more indepth traffic data collection using site-specific weigh-in-motion and automated vehicle counting equipment. Thus, for this project, traffic effects are indirectly incorporated by considering age (time) effects. However, it should be noted that based on the traffic data that were available, the estimated ESALs for the test sections ranged from a few hundred thousand ESALs to over 30 million ESALs.

Data Summaries

A summary of the key data elements for each of the 23 test sections is presented in table 4. The table includes both raw (as-measured) data and reduced data such as radius of relative stiffness and effective coefficient of thermal expansion (based on joint width change data).

Overall Summary

This report presented a test section by test section tabulation of CRC pavement performance data. The reader is referred to volume III in this series of reports for a more detailed evaluation of the field performance data.

Table 4. Summary of key data elements.

Test Section ID	Route	Nearest Mile Post	Direction	No. of Lanes	Age as of Fall 1991 Testing, years	Climatic Region	Terminal Joint Type	Design Thickness, in.	Subgrade Type (AASHTO)	Base Type	Outside Shoulder Type	Longitudinal Steel Details: Long. Steel Amount, %	Long. Steel Bar Size #	Long. Steel Spacing in.	Steel Placement Method	Epoxy Coated Steel
IL-1	US-51	na	SB	4	0.3	wet-freeze	wide flange	10	A-7-6	perm. ctb	pcc	0.70	6	6.25	chair	no
IL-2	I-72	45	WB	4	15	wet-freeze	lug	8	A-6	ctb	ac	0.59	5	6.50	tube	no
IL-3	US-36	st. 5+30	EB	4	20	wet-freeze	lug	8	A-7-5	atb	ac	0.60	5	6.50	chair	no
IL-4	I-55	86	EB	6	20	wet-freeze	lug	8	A-7-5	atb	ac	0.60	5	6.50	tube	no
IL-5	US-50	na	WB	4	5	wet-freeze	wide flange	8	A-7-5	lcb	pcc	0.70	6	9.12	chair	no
IA-1	I-29	18	NB	4	20	wet-freeze	lug	8	A-2-6	ctb	ac	0.65	6	8.50	tube	no
IA-2	I-80	15	WB	4	22	wet-freeze	lug	8	A-6	atb	ac	0.65	6	8.50	tube	no
IA-3	I-380	15	NB	4	15	wet-freeze	lug	8	A-6	atb	pcc	0.65	6	8.50	tube	no
OK-1	I-40	231	WB	4	4	wet-no freeze	wide flange	9	A-6	atb	pcc	0.50	5	6.88	chair	no
OK-2	US-69	na	NB	4	5	wet-no freeze	wide flange	9	A-6	atb	pcc	0.50	5	6.88	chair	no
OK-3	I-35	148	NB	4	3	wet-no freeze	wide flange	10	A-4	atb	pcc	0.50	5	6.88	chair	yes
OK-4	US-69	na	SB	4	7	wet-no freeze	wide flange	9	A-6	soil-asphalt	pcc	0.50	5	6.88	chair	no
OK-5	I-40	299	EB	4	2	wet-no freeze	wide flange	10	A-2-6	perm. ctb	pcc	0.61	6	7.25	chair	no
OR-1	I-5	184	SN	4	7	wet-no freeze	wide flange	13	A-4	granular	ac	0.60	6	4.75	tube	no
OR-2	I-5	184	NB	4	4	wet-no freeze	wide flange	10	A-4	ctb	ac	0.60	6	6.50	tube	no
OR-3	I-205	na	SB	4	20	wet-no freeze	lug	8	A-6	ctb	ac	0.54	5	6.50	tube	no
PA-1	I-180	seg. 374	EB	4	15	wet-freeze	wide flange	9	A-2-4	granular	ac	0.45	5	8.00	tube	no
PA-2	I-81	seg. 524	NB	4	22	wet-freeze	lug	9	A-2-4	granular	pcc	0.55	5	6.00	chair	no
WI-1	I-43	31	NB	4	18	wet-freeze	lug	8	A-2-4	granular	ac	0.65	4	6.00	chair	no
WI-2	I-90	180	EB	4	6	wet-freeze	wide flange	10	A-4	granular	pcc	0.67	6	6.50	tube	yes
WI-3	I-90/I-94	136	NB	6	7	wet-freeze	lug	10	A-4	granular	pcc	0.67	6	6.50	tube	yes
WI-4	I-90/I-94	111	WB	6	7	wet-freeze	na	10	A-2-4	granular	pcc	0.67	6	6.50	tube	no
WI-5	I-90/I-94	99	EB	4	16	wet-freeze	lug	8	A-1-a	granular	ac	0.61	6	9.00	chair	no
Average					11.3			9.0				0.60				
Std Dev					7.4			1.2				0.07				
Maximum					22.0			13.0				0.70				
Minimum					0.3			8.0				0.45				

(25.4 mm = 1 in)

Table 4. Summary of key data elements (continued).

Test Section ID	1991 2-Way AADT	Design Lane Cumul. ESALs upto 9/91	Average Maximum Deflection (sensor 1) Under 9,000 lb Load, 0.001 in.										% Change in Edge Deflection from Morn. to Afternoon	Edge Deflection as % of Basin Defl. (morning)	Edge Deflection as % of Basin Defl. (afternoon)
			Basin Testing		Morning Mid-Slab Crack Testing		Morning Edge Crack Testing		Afternoon Mid-Slab Crack Testing		Afternoon Edge Crack Testing				
			Average	Std. Dev.	Average	Std. Dev.	Average	Std. Dev.	Average	Std. Dev.	Average	Std. Dev.			
IL-1	na	180,000	2.2	0.4	3.1	0.5	5.2	0.8	2.8	0.3	3.8	0.5	27	236	173
IL-2	7,500	4,800,000	4.3	0.7	6.2	1.6	12	3.5	5.4	0.7	10.9	2.4	9	279	253
IL-3	17,700	4,800,000	4.9	0.5	5.2	0.2	10	0.9	5	0.1	9.3	0.5	7	204	190
IL-4	17,700	13,700,000	3.9	0.2	3.9	0.2	7.9	1.2	3.8	0.1	6.9	1.6	13	203	177
IL-5	na	300,000	4.4	0.7	4.3	0.6	9.3	1.2	4.1	0.4	6.6	0.7	29	211	150
IA-1	7,500	3,700,000	4.1	1.2	3.8	1.4	6.9	1.2	4.3	1.3	5.3	0.7	23	168	129
IA-2	12,700	8,850,000	5.0	0.4	5.3	0.5	13.4	1.6	4.7	0.3	9.6	1.2	28	268	192
IA-3	27,700	5,300,000	4.2	0.2	4.5	0.1	7.7	0.8	4.5	0.1	9.1	0.5	-18	183	217
OK-1	13,000	na	2.7	0.2	3.1	0.1	5.0	0.5	3.2	0.2	5.3	0.5	-6	185	196
OK-2	8,000	na	2.7	0.3	3.0	0.2	4.1	0.9	3.0	0.2	3.7	0.6	10	152	137
OK-3	21,000	na	2.9	0.3	3.3	0.2	5.2	0.4	3.3	0.2	4.6	0.5	12	179	159
OK-4	9,000	na	3.0	0.9	5.0	1.0	9.3	3.5	3.7	0.5	7.7	2.7	17	310	257
OK-5	12,000	na	3.0	0.6	4.0	0.3	5.0	0.7	3.5	0.2	4.3	0.5	14	167	143
OR-1	29,700	11,300,000	2.7	0.3	2.8	0.1	4.1	0.3	na	na	na	na		152	na
OR-2	30,300	3,000,000	1.9	0.4	2.1	0.3	4.9	0.6	2.1	0.3	3.5	0.2	29	258	184
OR-3	59,000	30,000,000	4.7	1.5	4.3	0.6	7.2	1.3	na	na	na	na		153	na
PA-1	9,000	5,000,000	2.2	0.4	2.3	0.1	4.4	0.7	2.4	0.1	na	na		200	na
PA-2	13,000	32,000,000	5.3	1.8	4.5	0.9	6.4	0.8	5.5	0.6	5.8	0.9	9	121	109
WI-1	10,900	2,290,000	2.8	0.6	3.4	0.9	6.1	1.3	3.2	0.8	4.4	0.9	28	218	157
WI-2	31,400	2,530,000	2.8	0.4	2.8	0.3	7.0	0.9	2.8	0.2	5.4	1.0	23	250	193
WI-3	35,100	3,960,000	2.7	0.3	3.2	0.4	7.4	1.4	3.2	0.4	4.4	0.7	41	274	163
WI-4	42,600	4,180,000	3.2	0.4	4.3	0.8	16.0	4.0	3.8	0.4	5.1	1.0	68	500	159
WI-5	26,900	na	5.4	1.1	4.3	0.4	8.5	0.8	5.2	1.3	6.4	0.4	25	157	119
Average			3.5	0.6	3.9	0.5	7.5	1.3	3.8	0.4	6.1	0.9	19.3	218.7	172.9
Std Dev			1.1	0.4	1.0	0.4	3.1	1.0	1.0	0.4	2.2	0.6	17.6	78.6	39.4
Maximum			5.4	1.8	6.2	1.6	16.0	4.0	5.5	1.3	10.9	2.7	68.1	500.0	256.7
Minimum			1.9	0.2	2.1	0.1	4.1	0.3	2.1	0.1	3.5	0.2	-18.2	120.8	109.4

(4.45 kN = 1,000 lb) (25.4 mm = 1 in)

Table 4. Summary of key data elements (continued).

Test Section ID	Measured E, million psi	Average Split Ten. Strength, psi	Average Core Thickness, in.	Estimated k, pci	Estimated l, in.	Basin Test l Average, in.	Basin Test l Std. Dev., in.	Morning Mid-slab Crack l Average, in.	Morning Mid-slab Crack l Std. Dev., in.	Morning Edge Crack l Average, in.	Morning Edge Crack l Std. Dev., in.	Afternoon Mid-slab Crack l Average, in.	Afternoon Mid-slab Crack l Std. Dev., in.	Afternoon Edge Crack l Average, in.	Afternoon Edge Crack l Std. Dev., in.	% Change in Mid-Slab Crack l from Morn. to Afternoon
IL-1	5.4	490	10.1	200	39.2	40	9	30	5	27	4	35	5	32	4	17
IL-2	5.7	578	8.8	200	35.9	37	7	25	3	24	5	28	2	25	5	12
IL-3	4.9	602	8.3	200	33.1	38	5	35	3	39	3	38	2	39	2	9
IL-4	4.3	472	9.3	200	34.8	42	3	40	2	41	2	39	2	39	3	-3
IL-5	4.9	483	8.5	200	33.7	38	6	35	4	38	7	36	2	39	4	3
IA-1	4.4	483	8.2	300	28.8	40	8	40	7	33	4	43	7	33	2	8
IA-2	4.1	509	8.0	250	29.1	41	5	36	4	42	2	40	2	40	1	11
IA-3	5.2	560	8.0	250	30.9	37	2	35	1	34	1	35	1	35	2	0
OK-1	5.8	478	9.2	250	35.2	35	4	24	4	25	3	23	4	24	4	-4
OK-2	6.6	573	9.2	250	36.4	40	6	31	3	29	4	31	3	30	4	0
OK-3	5.2	497	10.3	300	35.6	41	6	37	6	32	5	37	5	35	5	0
OK-4	6.4	475	9.4	250	36.7	33	8	24	3	26	5	29	3	25	3	21
OK-5	3.3	482	10.1	250	32.8	34	8	24	1	27	2	26	1	27	2	8
OR-1	3.6	528	12.4	200	41.4	38	8	33	4	32	2	na	na	na	na	na
OR-2	4.3	484	10.2	300	33.7	38	8	31	5	30	3	32	6	31	3	3
OR-3	4.7	448	7.6	200	30.6	35	8	29	3	32	6	na	na	na	na	na
PA-1	4.2	483	9.2	200	34.4	25	3	20	2	20	2	20	1	na	na	0
PA-2	4.9	545	9.4	300	32.8	42	12	28	5	23	2	37	10	26	4	32
WI-1	5.6	663	8.4	300	31.2	26	5	25	2	24	2	24	2	26	3	-4
WI-2	4.5	487	10.2	300	34.1	34	5	33	3	33	8	34	4	30	4	3
WI-3	3.9	447	10.2	300	32.9	30	4	27	2	30	3	33	4	28	2	22
WI-4	5.1	633	10.6	300	36.2	38	7	27	3	31	4	35	3	27	4	30
WI-5	5.3	517	7.9	300	29.4	35	5	28	2	28	2	37	9	30	4	32
Average	4.9	518.1	9.3	252.2	33.9	36.4	6.2	30.3	3.3	30.4	3.5	33.0	3.7	31.1	3.3	9.5
Std Dev	0.8	58.2	1.1	43.9	3.1	4.6	2.3	5.5	1.5	5.8	1.8	6.0	2.5	5.3	1.1	11.8
Maximum	6.6	663.0	12.4	300.0	41.4	42.0	12.0	40.0	7.0	42.0	8.0	43.0	10.0	40.0	5.0	32.1
Minimum	3.3	447.0	7.6	200.0	28.8	25.0	2.0	20.0	1.0	20.0	1.0	20.0	1.0	24.0	1.0	-4.2

Notes:

1. l = radius of relative stiffness (RRS)
2. Values of concrete modulus of elasticity and average splitting tensile strength were measured using cores obtained during field testing.

(25.4 mm = 1 in) (6.89 kPa = 1 psi) (0.27 MPa/m = 1 pci)

Table 4. Summary of key data elements (continued).

Test Section ID	Mid-Slab Crack l as % of Basin l (morning)	Mid-Slab Crack l as % of Basin l (afternoon)	% Change in Edge Crack l from Morn. to Afternoon	Edge Crack l as % of Basin l (morning)	Edge Crack l as % of Basin l (afternoon)	Load Transfer Efficiency at Cracks along Edge: Average %	Load Transfer Efficiency at Cracks along Edge: Std. Dev. %	Morning Mid-Depth Temp., F	Afternoon Mid-Depth Temp., F	Average Morning Crk Width, .01 mm	Average Afternoon Crk Width, .01 mm	Average Unit Length Change,per F (midslab temp.) (*0.000001)	No. of Cracks Included in Crack Width Study	Measured Coeff. of Thermal Expansion, 0.000001 in./in./F
IL-1	75	88	19	68	80	92	3	44	58	22	16	3.9	28	4.48
IL-2	68	76	4	65	68	83	17	58	73	55	44	6.7	28	4.87
IL-3	92	100	0	103	103	94	1	na	na	48	42	na	25	5.28
IL-4	95	93	-5	98	93	95	1	64	76	35	27	9.4	43	5.2
IL-5	92	95	3	100	103	94	2	62	75	29	22	6.2	35	4.14
IA-1	100	108	0	83	83	93	2	67	81	46	37	3.2	15	4.29
IA-2	88	98	-5	102	98	95	1	57	73	20	14	2.3	19	4.74
IA-3	95	95	3	92	95	93	1	65	72	47	34	17.7	29	5.22
OK-1	69	66	-4	71	69	88	7	65	82	63	44	3.7	10	4.94
OK-2	78	78	3	73	75	89	3	42	54	48	38	7.7	28	4.73
OK-3	90	90	9	78	85	93	3	61	78	54	44	3.7	19	3.98
OK-4	73	88	-4	79	76	76	22	40	54	76	70	2.4	17	4.86
OK-5	71	76	0	79	79	88	5	42	47	45	39	5.9	15	7.40
OR-1	87	na	na	84	na	93	2	50	53	31	na	na	21	4.91
OR-2	82	84	3	79	82	92	2	48	53	20	20	na	20	4.83
OR-3	83	na	na	91	na	93	2	49	50	84	na	na	18	4.22
PA-1	80	80	na	80	na	86	3	43	44	155	na	na	19	4.68
PA-2	67	88	13	55	62	92	2	47	58	216	211	3.6	24	4.10
WI-1	96	92	8	92	100	86	4	55	68	91	69	14.4	26	5.59
WI-2	97	100	-9	97	88	92	3	57	60	58	27	outlier	35	5.67
WI-3	90	110	-7	100	93	91	3	53	64	54	41	11.2	29	5.25
WI-4	71	92	-13	82	71	94	3	50	67	63	45	8.0	23	5.03
WI-5	80	106	7	80	86	91	2	48	64	45	28	9.8	28	4.49
Average	83.3	90.5	1.3	83.9	84.3	90.6	4.1			61.1	45.6	7.0	24	4.9
Std Dev	10.6	11.3	7.7	12.9	12.1	4.5	5.1			44.4	41.6	4.4	8	0.7
Maximum	100.0	110.0	18.5	102.6	102.6	95.0	22.0			216.0	211.0	17.7	43	7.4
Minimum	66.7	65.7	-12.9	54.8	61.9	76.0	1.0			20.0	14.0	2.3	10	4.0

Notes:

1. l = radius of relative stiffness (RRS)
2. Value of concrete coefficient of thermal expansion was measured using a core obtained during field testing.

(0.6°C = 1°F) (25.4 mm = 1 in) (1.8 mm/mm/°C = 1 in/in/°F)

Table 4. Summary of key data elements (continued).

Test Section ID	Average Depth of Cover Over Steel, in.	Steel Corrossion Level (4 cores)	Average Crack Spacing, feet	Std. Dev. Crack Spacing, feet	Average IRI, in./mile	Terminal Joint Distress Level	No. of Trans. Cracks			Longitud. Cracking Amount, ft & Severity	Test Section Survey		Windshield Survey		
							Low Severity	Moderate Severity	High Severity		No. of Patches & Severity	No. of Punchouts & Severity	No. of Miles Visually Surveyed	Total No. of Ptches & Punchouts	Total No. of Ptches & Punchouts per Mile
IL-1	5.1	none	5.1	3.51	93	none	178	0	0	0	0	0	5	0	0.0
IL-2	2.2	1spot	4.22	2.66	127	low	9	221	5	60H	4AC(M),3PCC(L)	0	5	62	12.4
IL-3	3.9	1spt/3sprd	3.58	2.1	152	medium	6	264	5	0	1AC(M),1PCC(L)	0	5	14	2.8
IL-4	3	2spt/2sprd	2.13	1.17	157	medium	9	463	0	0	0	0	5	9	1.8
IL-5	3.2	2spt/2sprd	3.02	2.11	141	none	344	12	0	0	0	0	3	3	1.0
IA-1	4.2	1spt/1sprd	5.89	3.87	72	medium	24	144	0	343L	0	0	5	0	0.0
IA-2	3.1	2spt/2sprd	2.98	2.24	82	medium	35	420	0	4L,49M,14H	1AC(L)	0	5	18	3.6
IA-3	3.2	3spt/1sprd	2.98	1.76	118	none	16	423	0	0	0	0	5	16	3.2
OK-1	3.9	3spt/1sprd	8.44	5.78	53	none	0	124	0	0	0	0	3	1	0.3
OK-2	4.5	1spt	4.57	3.37	na	medium	27	201	1	0	0	0	na	na	na
OK-3	4.8	none	4.75	2.99	74	none	14	210	0	0	0	0	4	0	0.0
OK-4	4.1	4spt	6.36	3.21	na	none	2	130	7	0	0	1L	na	na	na
OK-5	4.6	none	6.13	3.36	50	none	108	48	0	0	0	0	4	2	0.5
OR-1	7.5	none	4.02	2.10	na	low	4	301	0	0	0	0	na	na	na
OR-2	4.0	none	5.6	3.09	na	low	21	274	0	18L	0	0	na	na	na
OR-3	2.8	none	4.43	2.72	na	low	0	225	0	40M	0	0	na	na	na
PA-1	3.4	2sprd	4.8	2.67	75	low	0	238	84	0	0	0	5	0	0.0
PA-2	2.7	4sprd	4.32	2.47	75	medium	0	256	0	20M	1PCC(L),Map Crk.	3L	5	53	10.6
WI-1	2.8	na	2.88	2.27	112	medium	0	259	0	76L,70M	84AC(L)	0	5	31	6.2
WI-2	4.3	na	2.9	1.42	97	none	30	314	0	0	0	0	5	0	0.0
WI-3	4.0	na	3.46	1.59	81	low	129	164	0	25L	0	0	5	4	0.8
WI-4	4.8	4sprd	4.58	2.22	126	na	33	192	0	0	1PCC(L)	0	5	7	1.4
WI-5	2.6	4sprd	3.38	1.86	93	medium	14	291	0	416M,249H	11AC(L),13PCC(L)	0	5	107	21.4
Average	3.9		4.4	2.6	98.8										
Std Dev	1.1		1.4	1.0	32.3										
Maximum	7.5		8.4	5.8	157.0										
Minimum	2.2		2.1	1.2	50.0										

Notes:

1. spt = spot corrossion of steel bar
2. sprd = spread corrossion of steel bar
3. Severity levels - L = low; M = moderate; H = high
4. Patching data includes both partial and full depth patching

(25.4 mm = 1 in) (0.305-m = 1 ft) (15.8 mm/km = 1 in/mi) (1.6 km = 1 mi)

Table 4. Summary of key data elements (continued).

Test Section ID	Basin Test l Average in.	Morning Mid-slab Crack l Average in.	Morning Edge Crack l Average in.	Base Type	Basin Test k Average, pci	Basin Test D Average, million lb-in.	Morning Mid-slab Crack k Average, pci	Morning Mid-slab Crack D Average, million lb-in.	Morning Edge Crack k Average, pci	Morning Edge Crack D Average, million lb-in.	Afternoon Mid-slab Crack k Average, pci	Afternoon Mid-slab Crack D Average, million lb-in.	Afternoon Edge Crack k Average, pci	Afternoon Edge Crack D Average, million lb-in.	% Change in Mid-Slab Crack k from Morn. to Afternoon
IL-1	40.0	30.0	27.0	perm. ctb	288	737	415	336	287	153	337	506	273	286	81
IL-2	37.0	25.0	24.0	ctb	200	375	298	116	160	53	258	159	166	65	87
IL-3	38.0	35.0	39.0	atb	163	340	171	257	69	160	150	313	76	176	88
IL-4	42.0	40.0	41.0	atb	165	513	182	466	83	235	186	430	103	238	102
IL-5	38.0	35.0	38.0	lcb	179	373	211	317	85	177	205	344	108	250	97
IA-1	40.0	40.0	33.0	ctb	182	466	217	556	150	178	152	520	182	216	70
IA-2	41.0	36.0	42.0	atb	138	390	171	287	46	143	143	366	68	174	84
IA-3	37.0	35.0	34.0	atb	196	367	202	303	121	162	197	296	96	144	98
OK-1	35.0	24.0	25.0	atb	363	545	580	192	322	126	587	164	335	111	101
OK-2	40.0	31.0	29.0	atb	284	727	412	380	327	231	411	380	339	275	100
OK-3	41.0	37.0	32.0	atb	248	701	270	506	197	207	252	472	205	308	93
OK-4	33.0	24.0	26.0	soil-asphalt	425	504	445	148	204	93	356	252	245	96	80
OK-5	34.0	24.0	27.0	perm. ctb	362	484	474	157	297	158	452	207	317	168	95
OR-1	38.0	33.0	32.0	granular	294	613	336	398	238	250	na	na	na	na	na
OR-2	38.0	31.0	30.0	ctb	417	870	555	513	232	188	533	559	306	283	96
OR-3	35.0	29.0	32.0	ctb	230	345	306	216	155	163	na	na	na	na	na
PA-1	25.0	20.0	20.0	granular	808	316	1151	184	601	96	1138	182	na	na	99
PA-2	42.0	28.0	23.0	granular	161	501	355	218	313	88	174	326	287	131	49
WI-1	26.0	25.0	24.0	granular	603	276	552	216	318	106	585	194	389	178	106
WI-2	34.0	33.0	33.0	granular	365	488	358	425	166	197	354	473	241	195	99
WI-3	30.0	27.0	30.0	granular	463	375	504	268	182	147	323	383	329	202	64
WI-4	38.0	27.0	31.0	granular	265	553	360	191	78	72	240	360	323	172	67
WI-5	35.0	28.0	28.0	granular	179	269	316	194	167	103	182	341	198	160	58
Average	36.4	30.3	30.4		303.4	483.7	384.4	297.6	208.6	151.4	343.6	344.1	229.3	191.4	86.3
Std Dev	4.6	5.5	5.8		161.4	159.9	210.0	128.4	123.2	53.4	230.5	121.2	100.6	66.6	16.2
Maximum	42.0	40.0	42.0		808.0	869.5	1151.0	555.5	601.0	249.6	1138.0	558.9	389.0	307.6	106.0
Minimum	25.0	20.0	20.0		138.0	268.6	171.0	116.4	46.0	53.1	143.0	158.6	68.0	64.8	49.0

Notes:

1. l = radius of relative stiffness (RRS)
2. k = modulus of sugrade reaction
3. D = concrete slab rigidity

(25.4 mm = 1 in) (0.27 MPa/m = 1 pci)

Table 4. Summary of key data elements (continued).

Test Section ID	Mid-Slab Crack k as % of Basin k (morning)	Mid-Slab Crack k as % of Basin k (afternoon)	% Change in Edge Crack k from Morn. to Afternoon	Edge Crack k as % of Basin k (morning)	Edge Crack k as % of Basin k (afternoon)	% Change in Mid-Slab Crack D from Morn. to Afternoon	Mid-Slab Crack D as % of Basin D (morning)	Mid-Slab Crack D as % of Basin D (afternoon)	% Change in Edge Crack D from Morn. to Afternoon	Edge Crack D as % of Basin D (morning)	Edge Crack D as % of Basin D (afternoon)	Age as of Fall 1991 Testing, years
IL-1	144	117	95	100	95	150	46	69	188	21	39	0.3
IL-2	149	129	104	80	83	136	31	42	122	14	17	15.0
IL-3	105	92	110	42	47	122	75	92	110	47	52	20.0
IL-4	110	113	124	50	62	92	91	84	102	46	46	20.0
IL-5	118	115	127	47	60	109	85	92	141	47	67	5.0
IA-1	119	84	121	82	100	94	119	112	121	38	46	20.0
IA-2	124	104	148	33	49	127	74	94	122	37	45	22.0
IA-3	103	101	79	62	49	98	83	80	89	44	39	15.0
OK-1	160	162	104	89	92	85	35	30	88	23	20	4.0
OK-2	145	145	104	115	119	100	52	52	119	32	38	5.0
OK-3	109	102	104	79	83	93	72	67	149	29	44	3.0
OK-4	105	84	120	48	58	171	29	50	103	18	19	7.0
OK-5	131	125	107	82	88	131	33	43	107	33	35	2.0
OR-1	114	na	na	81	na	na	65	na	na	41	na	7.0
OR-2	133	128	132	56	73	109	59	64	150	22	33	4.0
OR-3	133	na	na	67	na	na	63	na	na	47	na	20.0
PA-1	142	141	na	74	na	99	58	58	na	30	na	15.0
PA-2	220	108	92	194	178	149	44	65	150	17	26	22.0
WI-1	92	97	122	53	65	90	78	70	168	38	65	18.0
WI-2	98	97	145	45	66	111	87	97	99	40	40	6.0
WI-3	109	70	181	39	71	143	71	102	137	39	54	7.0
WI-4	136	91	414	29	122	188	35	65	238	13	31	7.0
WI-5	177	102	119	93	111	176	72	127	156	38	60	16.0
Average	129.4	109.6	132.6	71.5	83.5	122.6	63.3	74.1	133.0	32.9	40.8	11.3
Std Dev	29.0	22.5	70.0	35.1	31.8	30.7	23.0	24.8	36.7	11.2	14.2	7.4
Maximum	220.5	161.7	414.1	194.4	178.3	188.2	119.2	127.0	238.3	47.5	66.9	22.0
Minimum	91.5	69.8	79.3	29.4	46.6	85.4	29.3	30.2	88.4	13.0	17.3	0.3

Notes:

1. l = radius of relative stiffness (RRS)
2. k = modulus of sugrade reaction
3. D = concrete slab rigidity

ACKNOWLEDGEMENTS

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Finally, the study could not have been possible without the financial support (as part of the pooled funding) by Arizona, Arkansas, Connecticut, Delaware, Illinois, Iowa, Louisiana, Oklahoma, Oregon, Pennsylvania, South Dakota, and Texas.

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APPENDIX A - FIELD DATA COLLECTION PROCEDURES

Site Information

At each site, detailed information about the site was noted to identify the location of the tested section as precisely as possible. A representative 305-m (1,000-ft) long section at each site was selected for testing. Each test section was then marked with 30.5-m (100-ft) stations. The location of the randomly selected 30.5-m (100-ft) subsection used for detailed crack survey (joint width measurements) and for load transfer testing at cracks was noted. Also noted were the following items:

1. Locations of cores obtained for laboratory strength testing. These were also the locations for the shallow borings performed to obtain samples of unbound base and subgrade materials.

2. Locations of cores obtained for identifying the extent (if any) of steel corrosion.

3. Locations where powdered concrete samples were obtained for chloride content determination.

A form used to note the above information is shown in figure 185.

Visual Condition Survey

Five visual condition surveys were performed at each test site location. They include a CRCP surface condition survey, drainage survey, detailed crack survey, "windshield" condition survey, and terminal joint survey. These survey procedures are outlined below.

CRCP Surface Condition Survey

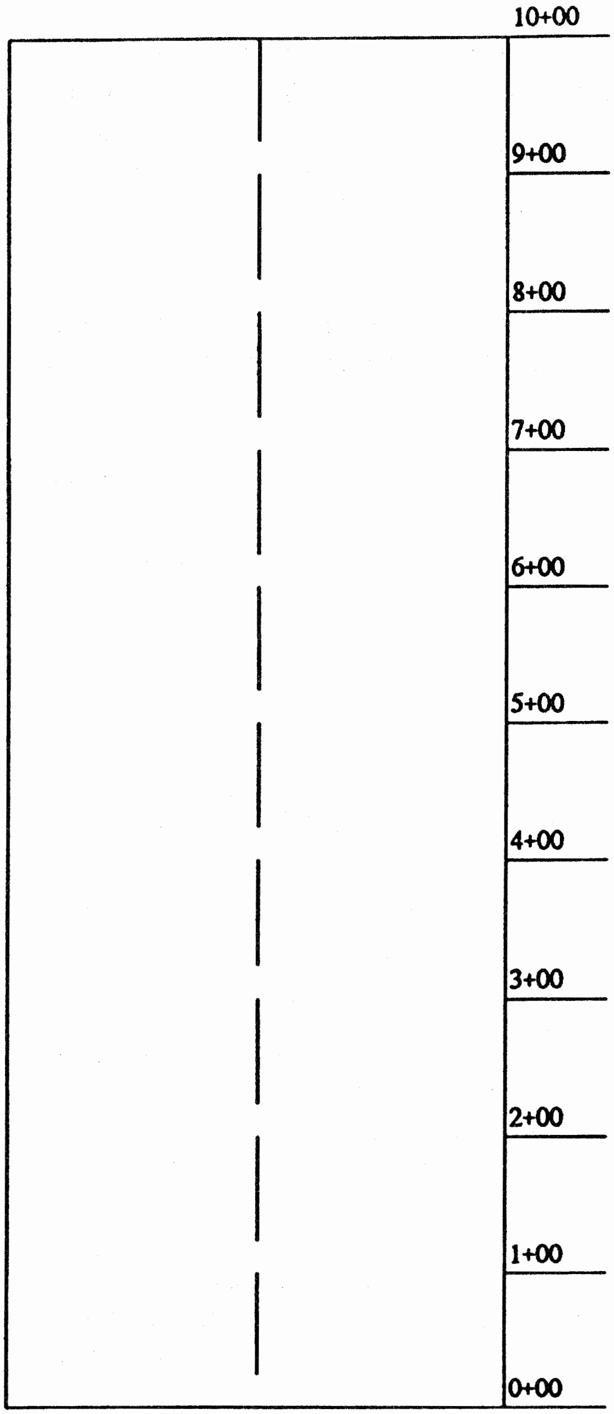
A visual condition survey was conducted over the entire 305-m (1,000-ft) length of the test section following the procedures and guidelines presented in the *SHRP Distress Identification Manual*. Additionally, distresses unique to a particular section, which were not defined by SHRP, were identified and noted by the field engineer. The CRCP distresses defined in the SHRP manual are categorized as shown in table 5.

The field engineer performed the condition survey with a notepad containing all applicable forms, pencils, a hard hat, a safety vest, a 35-mm camera, and a distance measuring wheel. The surveys were conducted by slowly walking along the right edge of the pavement surface while measuring longitudinal distance with the distance measuring wheel. The occurrence of surface distresses were identified by type, severity, location, and quantity. The type and severity were noted by sketching a symbol representing the appropriate distress as outlined in the *SHRP Distress Identification Manual*. The location and extent of the distress were defined by sketching the appropriate distress symbol on one of 10 distress maps that were

CRC SECTION LAYOUT

Section ID: ______ Date: ______ Surveyor: ________

Sketch of CRC Section
Sketch Locations of Coring/Boring, Corrosion Coring and Chloride Samples



Coring/Boring Locations

ID	Station	Offset

Corrosion Core Locations

ID	Station	Offset

Chloride Content Locations

Over Steel

ID	Station	Offset

No Steel

ID	Station	Offset

Figure 185. CRC section layout.

Table 5. CRCP distresses as defined by the *SHRP Distress Identification Manual*.

	Distress Type	Unit of Measurement	Defined Severity Levels
Cracking			
1.	Durability "D" Cracking	No., m^2 (ft^2)	Yes
2.	Longitudinal Cracking	Linear m (ft)	Yes
3.	Transverse Cracking	No., m^2 (ft^2)	Yes
Surface Defects			
4.	Map Cracking and Scaling	m^2 (ft^2)	Yes
5.	Polished Aggregate	m^2 (ft^2)	No
6.	Popouts	Number	No
Miscellaneous Distresses			
7.	Blowups	Number	No
8.	Construction Joint Deterioration	Number	Yes
9.	Lane-to-Shoulder Dropoff	mm (in)	No
10.	Lane-to-Shoulder Separation	mm (in)	No
11.	Patch/Patch Deterioration	m^2 (ft^2)	Yes
12.	Punchouts	Number	Yes
13.	Spalling of Longitudinal Joint	Linear m (ft)	Yes
14.	Water Bleeding and Pumping	No., Ln. m (ft)	Yes

CRC Site No. ____________ Surveyor ____________ Date ________

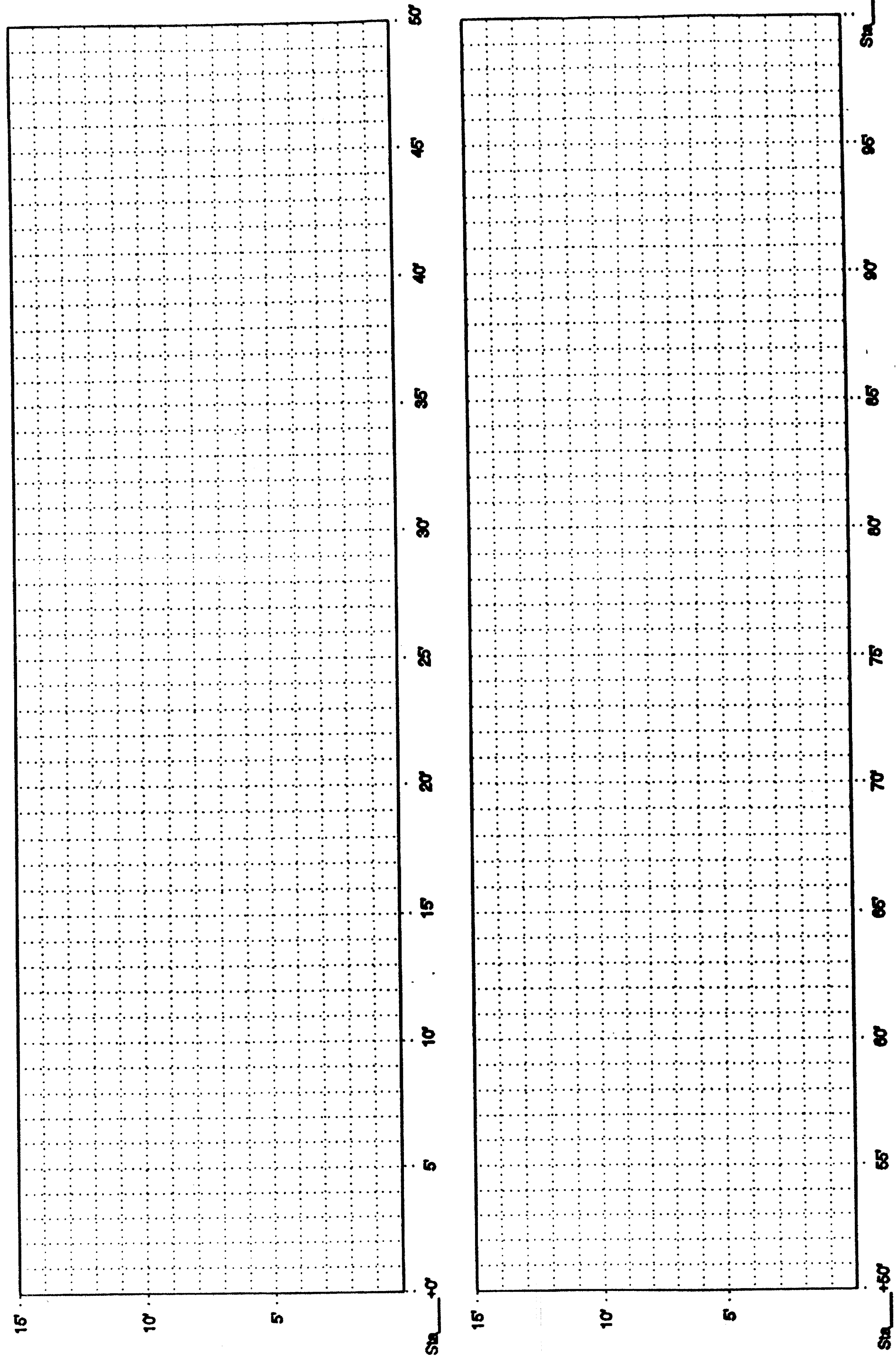


Figure 186. Distress map - typical.

scaled in both the longitudinal and transverse direction. Each distress map was used for 30.5-m (100-ft) of pavement as shown in figure 186.

The field engineer would quantify the identified distresses by type for the entire site at the end of the day and note the information in Form 2 shown in figure 187. Key distresses were also photographed.

Drainage Survey

A drainage survey was conducted for the entire 305-m (1,000-ft) test site. Drainage structures and facilities and moisture conditions were identified on a drainage map similar to the collection of surface distresses described previously. Drainage structures and facilities were identified by type, location, and condition with an alphanumeric code comprised of a type designation and condition designation. Moisture conditions were identified by location, severity, and extent with symbols representing various moisture conditions.

Detailed Crack Survey

A 30.5-m (100-ft) long subsection was selected at random for a detailed crack survey. A crack comparator was used to measure the crack width. Each transverse crack within the subsection was identified by location, extent, severity, and width. The location, extent, and severity was mapped in as much detail as possible on the crack detail map shown in figure 186. Each crack was also identified on the pavement and on the crack detail map. Crack widths were measured for each transverse crack within the subsection during both the early morning and mid-afternoon. A step-by-step process for crack width measurement is described below.

1. A longitudinal line was marked on the pavement by snapping a chalk line on the pavement surface. The line was generally about 0.3 m (1 ft) inside of the outside pavement edge. The lateral distance from the outside edge of the pavement was noted by the field engineer in the space provided on the crack detail map.

2. Crack widths and surface temperatures were measured with a crack comparator in the early morning for each transverse crack at the chalk line location. The engineer entered the crack width and surface temperature with the appropriate crack number on the form shown in figure 188.

3. Crack widths and surface temperatures were measured again in the mid-afternoon for the same transverse cracks measured in step 2 and entered on the form as described above.

Photographs were taken to include records of transverse cracks within the subsection.

VISUAL CONDITION SURVEY SUMMARY

Date: ____________ Time: ____________ Section ID: ____________

Surveyor: ____________

Distress Item	Unit of Measurement	Severity Level Low	Severity Level Medium	Severity Level High
Durability "D" Cracking	Number of Joints and Cracks			
	Square Feet			
Longitudinal Cracking	Linear Feet			
Transverse Cracking	Number of Cracks			
	Square Feet			
Map Cracking and Scaling	Square Feet			
Polished Aggregate	Square Feet			
Popouts	Number			
Blowups	Number			
Construction Joint Deterioration	Number of Joints			
Lane-to-Shoulder Dropoff	Inches *			
Lane-to-Shoulder Separation	Inches *			
Flexible Patch/Patch Deterioration	Square Feet			
	Number of Patches			
Rigid Patch/Patch Deterioration	Square Feet			
	Number of Patches			
Punchouts	Number			
Spalling of Longitudinal Joints	Linear Feet			
Water Bleeding and Pumping	Number of Joints and Cracks			
	Linear Feet			
Other: ____________				
____________				

* in inches to nearest 1/10" at 100' intervals along the lane-to-shoulder joint

General Drainage Condition:

____

____

____

(25.4 mm = 1 in, 0.305 m = 1 ft)

Figure 187. Form 2 - visual condition survey.

DETAILED CRACK SURVEY

Date: __________ Section ID: ____________ Surveyor: ____________

Time of Morning Survey: __________ Time of Afternoon Survey: _________

Distance measured from right edge of pavement, feet = ____________

Crack No.	Distance (feet)	Crack Width, inches		Crack Class
		Morning	Afternoon	

(25.4 mm = 1 in) (0.305 m = 1 ft)

Figure 188. Detailed crack survey data form.

Roadway Windshield Condition Survey

A general surface condition survey of the roadway was conducted independently of the other field data collection operations. The survey was extended for 16.1 km (10 mi) or the entire length of the roadway, whichever was lesser, that incorporated the 305-m (1,000-ft) test section. The profilometer technician independently performed the survey from within the profilometer van. This activity did not require traffic control. The survey was conducted entirely along the outside lane in the direction of the section being tested and at a speed of no greater than 48.3 km (30 mi) per hour. The van was equipped with flashing headboard lights to warn motorists of the slow moving operation.

Distresses were identified by location, type, and severity by predefined codes as shown in table 6 and were entered in real time directly into a computer by the operator. All distresses were referenced longitudinally by mile increments as collected by the profilometer distress measuring instrument (DMI). No reference to transverse location was identified.

Terminal Joint Survey

A survey of the roadway terminal joints was conducted for the two joints adjacent to a test section. Each terminal joint was identified by type, location, and condition. Photographic and video records were taken of each terminal joint.

Nondestructive Deflection Testing

Deflection testing was conducted using a Dynatest 8002 FWD. Testing was conducted generally in accordance with the procedures established by SHRP for General Pavement Studies testing and detailed in the *SHRP-LTPP Manual for FWD Testing — Operational Field Guidelines*, January 1989. Two types of testing were conducted as follows:

1. Basin testing.
2. Load transfer testing at cracks.

The FWD test data was used for layer material characterization, crack load transfer evaluation, and void detection. Three nominal load levels were used:

1. 40.03 kN (9,000 lb).
2. 53.38 kN (12,000 lb).
3. 71.17 kN (16,000 lb).

Each load level was applied twice after one seating load for a total of seven loadings per location.

Concrete temperatures were measured several times during the day. Temperatures were measured during the basin testing operation, during the early morning measurements of crack widths, and during the mid-afternoon measurements of crack width. Temperatures were

Table 6. Windshield survey distress codes.

Distress Type	Code
Transverse Cracking Spalling Begin	1
Transverse Cracking Spalling End	2
Longitudinal Cracking Begin	3
Longitudinal Cracking End	4
Blowup	5
Terminal Joint Deterioration	6
Concrete Patch	7
Flexible Patch	8
Punchout	9
Other	10

collected using holes drilled by a hand-held drill at three depths: 25 mm (1 in) below slab surface; slab mid-depth; and 25 mm (1 in) above slab bottom.

Basin Testing

Basin deflection testing was conducted by the FWD technician over the entire 305-m (1,000-ft) test site. Seven geophone sensors were used for measuring deflections over a 1 829-mm (72-in) basin using SHRP recommended sensor spacing shown in figure 189. Testing was conducted along the center of the outside pavement lane at a spacing of approximately 7.6 m (25 ft) over the whole length of the test section resulting in about 40 deflection tests per section. Generally, an attempt was made to apply the load on uncracked concrete (no cracks under load plate) while maintaining approximately 7.6-m (25-ft) spacing.

Load Transfer Testing at Cracks

Load transfer testing at cracks was conducted at approximately 10 predetermined cracks of the 30.5-m (100-ft) subsection selected for the detailed crack survey. The cracks selected for testing were recorded on the crack detail map and keyed in the deflection data file. The sensor spacing is shown in figure 189 in which only sensors 1 and 2 were used to collect deflection data. Each selected crack was tested in the early morning and in the mid-afternoon at both the approach and leave sides of the crack along the lane edge and at mid-lane. Lane edge refers to the actual longitudinal joint between the shoulder and pavement.

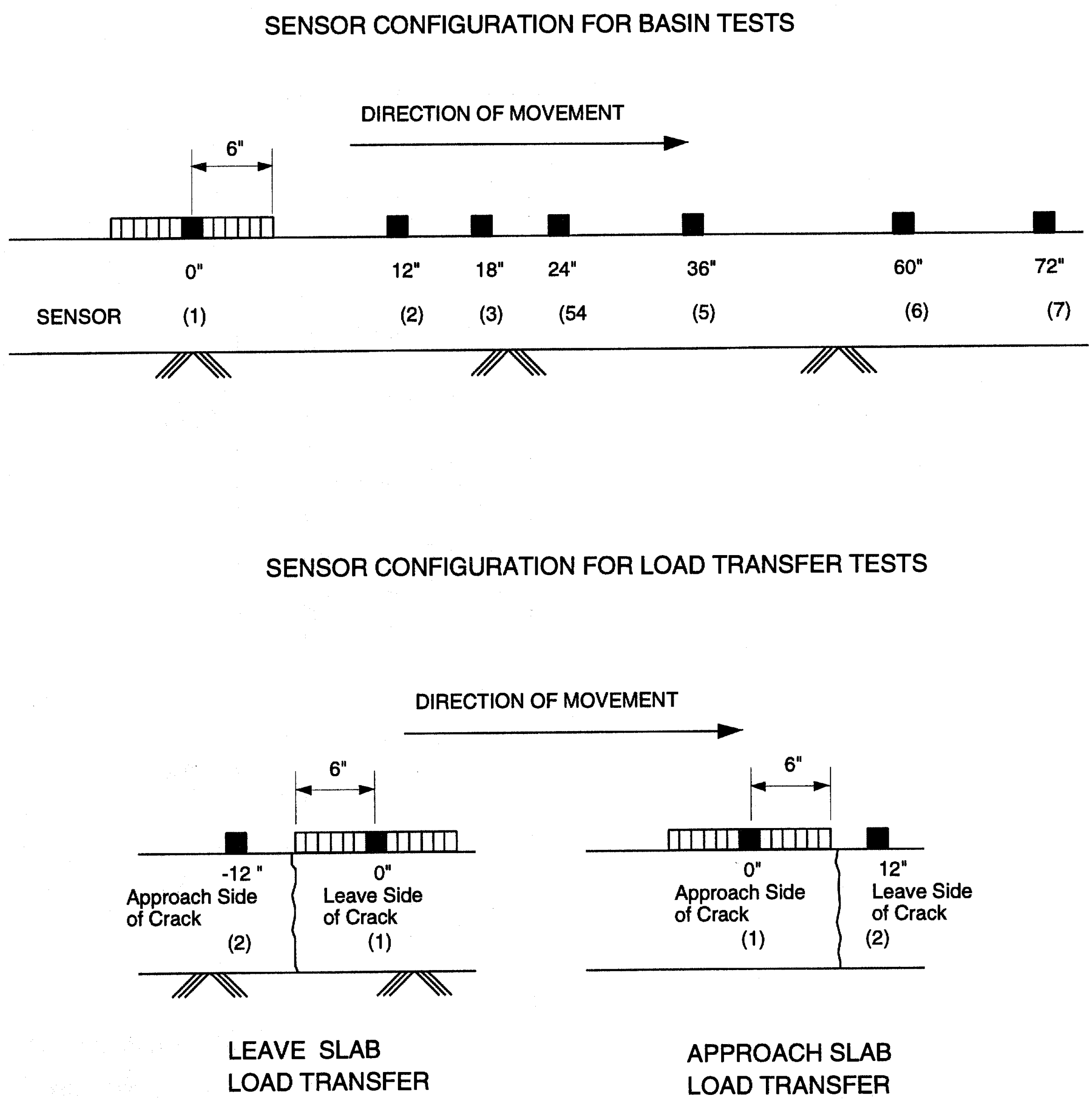
Load transfer testing was conducted by using two video cameras mounted on the back of the FWD to line up the loading plate adjacent to the cracks to be tested. All relevant FWD data were incorporated in the deflection files generated by Dynatest software. The FWD operator produced hardcopies of the deflections and loads for each basin and load transfer test at the end of each day. Locations for the basin and load transfer tests are shown in figure 190. Over 100 tests were conducted at each test section.

It should be noted that the locations for the load transfer testing do not correspond with those established by SHRP for CRCP testing. It was felt that the critical location for load transfer degradation with time is along the lane edge and this degradation plays a significant role in development of punchout related distresses. Thus, the lane edge testing was considered more critical than the wheel path testing for load transfer at cracks.

Corrosion Related Testing

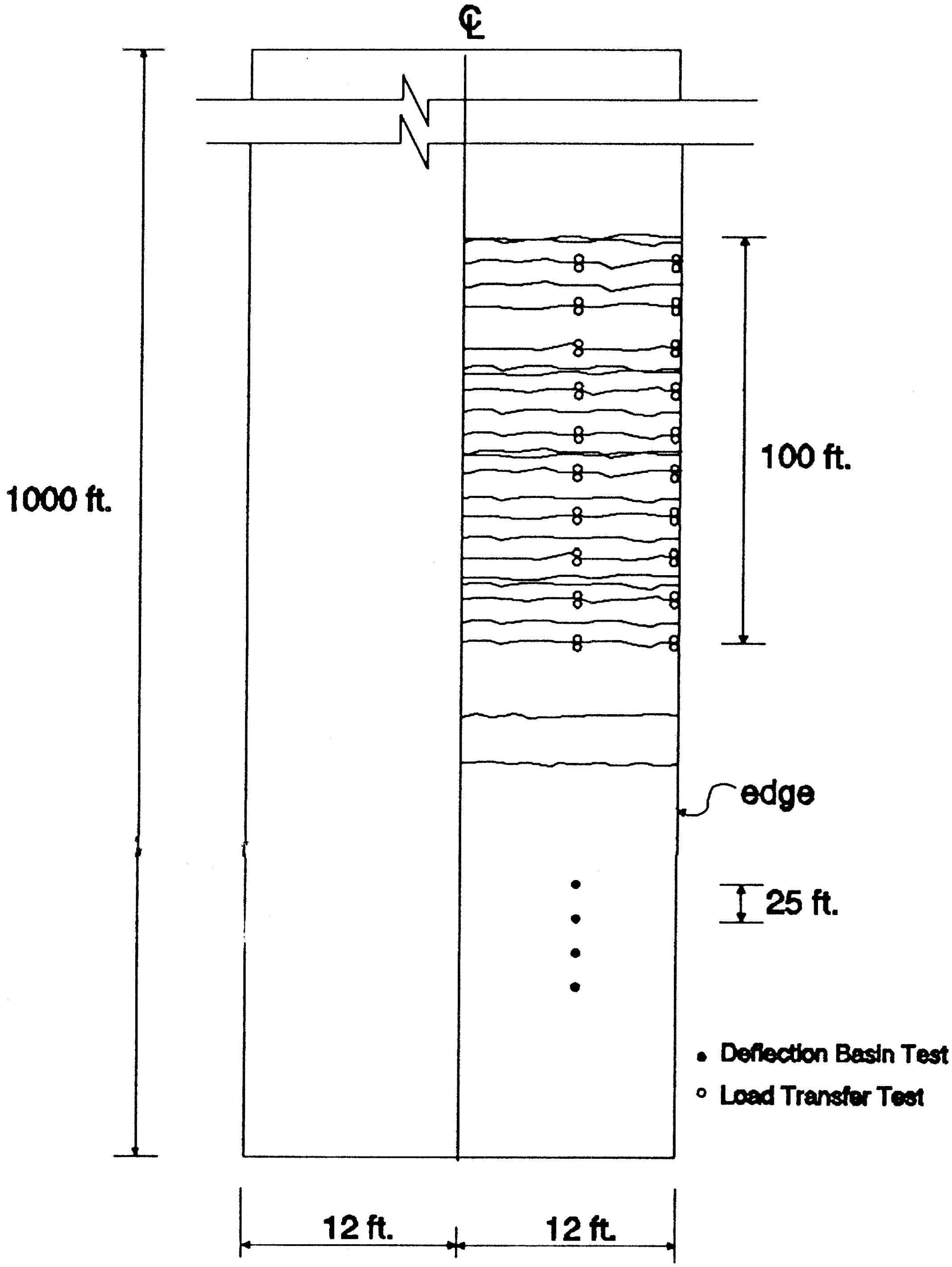
Corrosion related testing was conducted for test sections in States where potential for steel corrosion exists because of extensive deicer salt applications. Corrosion related testing included the following:

1. Corrosion potential testing.
2. Chloride content testing.
3. Visual examination of cores.



(25.4 mm = 1 in)

Figure 189. FWD sensor spacing.



(0.305 m = 1 ft)

Figure 190. Locations for basin and load transfer tests.

Corrosion Potential Testing

Corrosion potential testing was conducted on a randomly selected 30.5-m (100-ft) subsection within the 305-m (1,000-ft) test section. Testing was conducted in accordance with the procedures of ASTM C-876 — Standard Test Method for Half-Cell Potentials of Uncoated Reinforcing Steel in Concrete. Test measurements were used to estimate the level of corrosion activity in the steel reinforcement. Testing was conducted by the field engineer using a copper sulfate half cell and a potentiometer.

The field engineer mapped a testing grid on the pavement surface within the 30.5-m (100-ft) subsection where potential differences were to be measured. An initial grid spacing of 1.2 m (4 ft) was used and adjusted based on initial readings. The field engineer selected one location for steel connection to the lead wire if transverse steel existed in the pavement. If transverse steel did not exist in the pavement, three to four steel connections had to be made in the exposed longitudinal bars. Steel connections were made using a spring clip. Potential differences were measured at each grid location and recorded on the form shown in figure 191. Generally, potentials more negative than -0.35 V CSE (where CSE is the designation for copper-copper sulfate electrode) indicate that there is a greater than 90 percent probability that reinforcing steel corrosion is occurring at the time of testing.

Chloride Content Sample Collection and Testing

Chloride content samples were retrieved from six locations within the 30.5-m (100-ft) subsection selected for corrosion potential testing. Powdered samples were obtained by a technician using a hand-held drill at three depths below the pavement surface. Three of the six sample locations were collected directly over a steel bar. The remaining samples were collected in concrete where no steel was located. The location of the samples and identification of the samples were recorded by the technician on the form shown in figure 185. Samples were tested in the laboratory to estimate total and water soluble chlorides (AASHTO T-260 procedure).

Visual Inspection of Cores

Four 102- or 152-mm (4- or 6-in) diameter cores were obtained by the drill crew from locations where transverse cracks intersected the reinforcing steel within the 30.5-m (100-ft) subsection selected for corrosion potential evaluation. The cores were visually inspected for evidence of corrosion and necking in the steel by the field engineer. The following rating scheme, developed by the Illinois Department of Transportation, was used to classify the steel corrosion:

CORROSION TESTING RESULTS

Date: ________ Section ID: ________________ Surveyor: ________________

Time Start: ________________ Time Finish: ________________

Record potentials to nearest 0.01 V

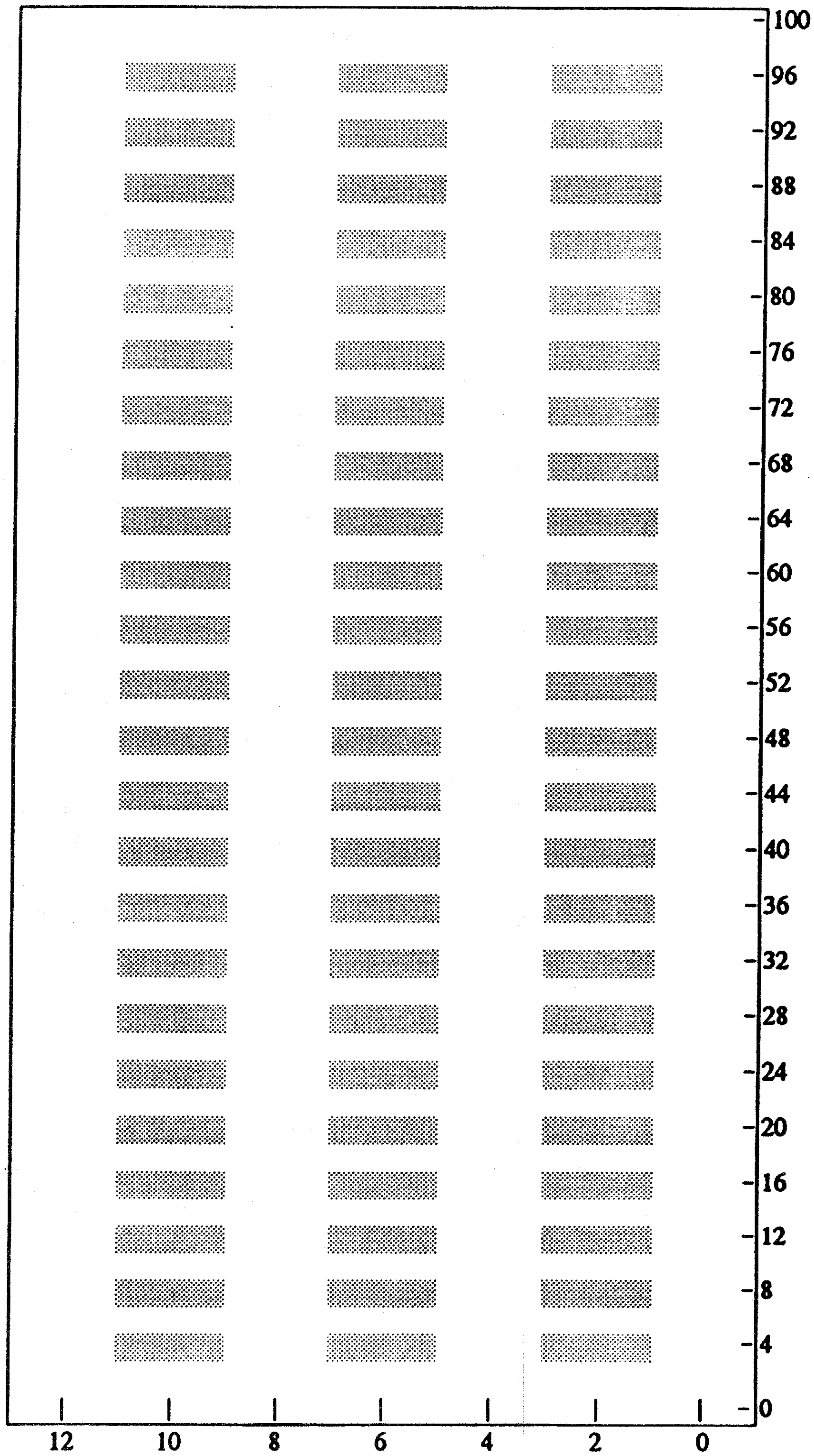


Figure 191. Corrosion test data form.

Level	Description
1	Clear or free of rust
2	Slight rust with no appreciable reduction in cross-sectional area
3	Moderate rust with no appreciable reduction in cross-sectional area
4	Heavy rust with a marked reduction in cross-sectional area

Crack widths at several depths from within core holes were measured and recorded by a technician with calipers.

Coring and Shallow Borings for Material Characterization

Four locations within the 305-m (1,000-ft) test section were selected by the field engineer for coring and boring. The coring and boring operation was conducted to obtain test samples and determine layer thickness of the pavement structure. A brief description of the operation follows.

Concrete Pavement Coring and Testing

Four 102-mm (4-in) diameter cores were obtained by the drill crew from within the 305-m (1,000-ft) test site. The coring locations were located in concrete areas that were free of surface cracks and steel. The four cores were labeled and tested in the laboratory as follows:

1.	Core No. 1	For Coefficient of Thermal Expansion (FHWA Procedure) and Modulus of Elasticity (ASTM C469).
2.	Core Nos. 2, 3, 4	For Splitting Tensile Strength (ASTM C496).

Coring and Boring of Base/Subbase and Subgrade

The underlying pavement material was removed below the concrete pavement for determining layer thicknesses. The coring/boring log used to record the material details is shown in figure 192. As the material was removed, the operator assessed the frictional resistance at layer interfaces by a rating of none, low, medium, or high. Descriptions of these ratings are listed below:

None: No bond exists between pavement layers.

Low: Small particles of the underlying material are bonded to the overlying material.

Medium: Medium to large particles of the underlying material are bonded to the overlying material.

CORING/BORING LOG SHEET

Section ID: ________________

Surveyor: ________________

Date: ________________

Station: ________________

Offset: ________________

Depth		Material	Sample/Core	DCP	
From	To	Code	ID	Drops	Comments

1–Concrete 2–CTB 3–ATB 4–Granular Base 5–Granular Subbase 6–Soil (Describe type in comments)

Figure 192. Coring/boring log sheet.

High: 100% of the underlying material surface is bonded to the overlying material.

Samples were retrieved and tested for various material types as follows:

Cement-treated bases: The 102-mm (4-in) diameter cores obtained from cement-treated bases were tested for compressive strength, if the cores were recovered intact.

Unbound base and subbase: Sufficient samples of unbound base and subbase layer materials were obtained in a sealed bag or container to allow for visual classification of these materials.

Subgrade: Sufficient samples of subgrade material were obtained in a sealed bag or sealed container for visual classification tests and Atterburg Limits tests.

Dynamic cone penetration tests were conducted by the drill crew to characterize the strength/stiffness properties of the subgrade. The total number of blows to drive the penetrometer 305-mm (12 in) into the subgrade was recorded.

Reinforcing Steel Location

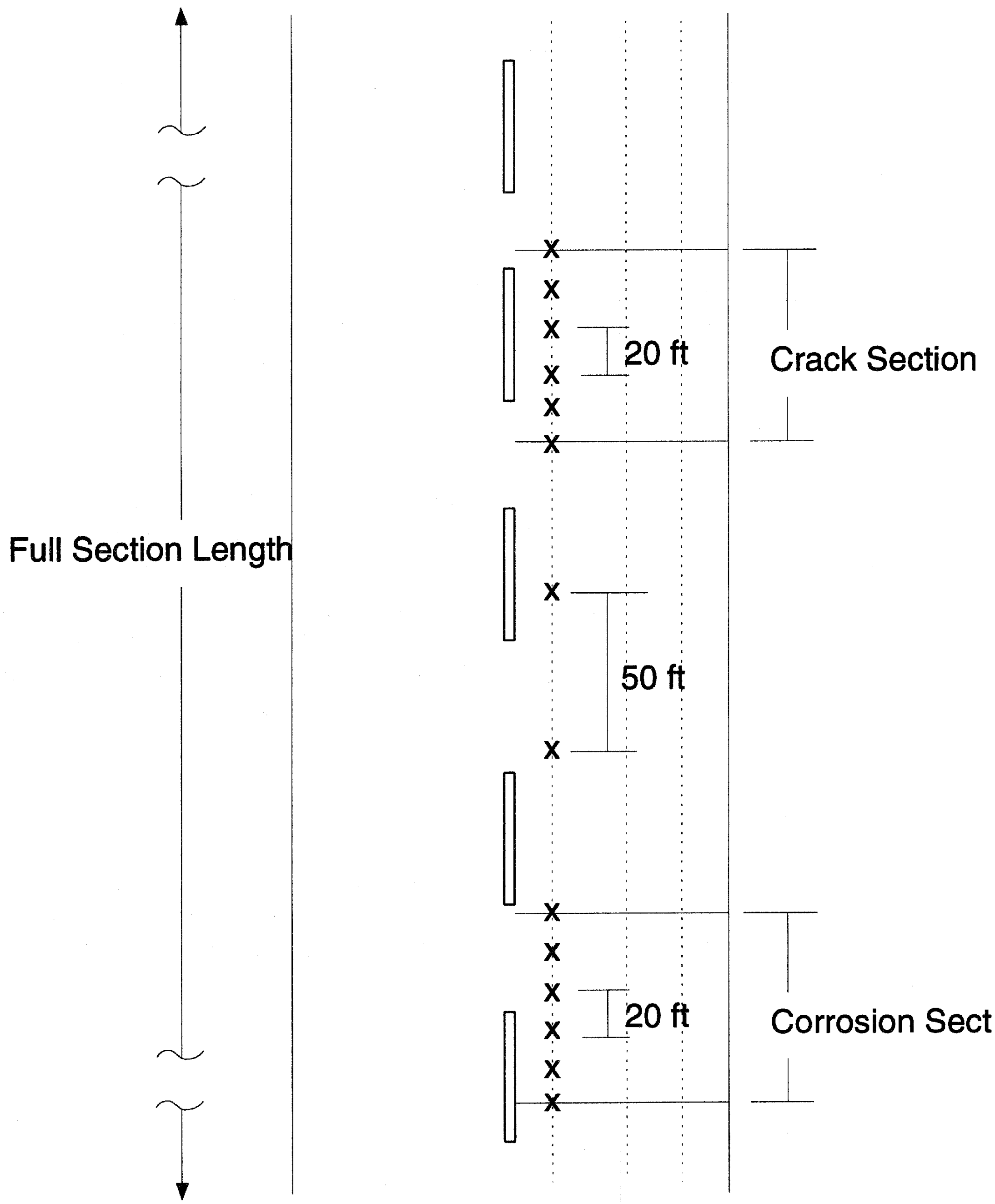
A survey using a pachometer was conducted to establish the location of the reinforcing steel with respect to the pavement surface. The survey was conducted by the field engineer or the field technician along the whole length of the test section at a spacing of about 15.2 m (50 ft) along at least three lines. However, a spacing of 6.1 m (20 ft) was used within both the 30.5-m (100-ft) section selected for detailed crack survey and the 30.5-m (100-ft) section selected for corrosion related testing as shown in figure 193. The depths to steel and the lateral offsets from the right edge of the pavement were recorded on a form shown in figure 194.

Photographic and Video Records

As stated previously, visual records were obtained using photographs and video of all key distresses and unique features. Additionally, a video of the entire 305-m (1,000-ft) test site location was made by the field engineer. The video included narration describing key features of the test site.

Profile Survey

A profile survey was conducted along each test section using a South Dakota type profiler. The testing was conducted independently by a profiler technician. Testing was conducted in the outside lane of the test site at a speed of approximately 64.4 km/h (40 mi/h). Profile data was recorded at 0.3-m (1-ft) intervals from 91.4 m (300 ft) before to 91.4 m (300 ft) after the section. All pertinent data were keyed in by the operator directly to computer disk, therefore, no forms were required for this operation.



(0.305 m = 1 ft)

Figure 193. Reinforcing steel location layout.

REINFORCING STEEL LOCATION SURVEY

Date: ________ Section ID: ________ Surveyor: ________________

Time Start: ________________ Time Finish: ________________

Offset of Line 1 (feet): ________
Offset of Line 2 (feet): ________
Offset of Line 3 (feet): ________

Section Length Steel Depth

	Line		
Station	1	2	3
0+00			
0+50			
1+00			
1+50			
2+00			
2+50			
3+00			
3+50			
4+00			
4+50			
5+00			
5+50			
6+00			
6+50			
7+00			
7+50			
8+00			
8+50			
9+00			
9+50			
10+00			

Crack Subsection Steel Depth

	Line		
Location	1	2	3
0			
20			
40			
60			
80			
100			

Corrosion Subsection Steel Depth

	Line		
Location	1	2	3
0			
20			
40			
60			
80			
100			

(0.305 m = 1 ft)

Figure 194. Reinforcing steel location survey form.

The procedures established by SHRP for profile measurement were generally followed. However, only three passes (repeat runs) were made. The profile data for the right-hand and left-hand wheel paths were analyzed to determine the half-car simulated values. The average profile data presented in terms of the IRI are the averages of the half-car simulated values for the three passes.

Recycled
Recyclable