U.S. Department
of Transportation

**Federal Highway
Administration**

Publication No. FHWA-RD-96-164
March 1997

Performance of Concrete Pavements Containing Recycled Concrete Aggregate

Research and Development
Turner-Fairbank Highway Research Center
6300 Georgetown Pike
McLean, Virginia 22101-2296

FOREWORD

This report documents the investigation of the field performance of a series of concrete pavements which incorporated recycled concrete aggregate (RCA) as the coarse aggregate in the concrete. In total, 17 pavement sections were evaluated on 9 different pavements from around the country. As a part of the field evaluation, condition and drainage surveys were conducted, falling weight deflectometer tests were run, serviceability was evaluated, and cores were taken for laboratory study. Laboratory testing of these cores included compressive strength, split tensile strength, dynamic elastic modulus, static elastic modulus, thermal coefficient of expansion, petrographic examination, and evaluation of crack and joint face roughness.

This report will be of interest to those involved in concrete pavement mix design, as well as the design and construction of concrete pavements. Sufficient copies are being distributed to provide two copies to each FHWA Region, and three copies to each FHWA Division and State highway agency. Direct distribution is being made to the FHWA Division Offices. Additional copies may be purchased from the National Technical Information Service (NTIS), 5285 Port Royal Road, Springfield, Virginia 22161.

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

NOTICE

Technical Report Documentation Page

1. Report No. FHWA-RD-96-164	2. Government Accession No.	3. Recipient's Catalog No.
4. Title and Subtitle PERFORMANCE OF CONCRETE PAVEMENTS CONTAINING RECYCLED CONCRETE AGGREGATE		5. Report Date March 1997
		6. Performing Organization Code
7. Author(s) M. J. Wade, G. D. Cuttell, J. M. Vandenbossche, H. T. Yu, K. D. Smith, and M. B. Snyder		8. Performing Organization Report No.
9. Performing Organization Name and Address University of Minnesota Department of Civil Engineering 500 Pillsbury Drive SE Minneapolis, MN 55455		10. Work Unit No. (TRAIS) 3E2C
		11. Contract or Grant No. DTFH61-93-C-00133
12. Sponsoring Agency Name and Address Office of Engineering and Highway Operations R & D Federal Highway Administration 6300 Georgetown Pike McLean, VA 22101–2296		13. Type of Report and Period Covered Interim Report October 1993–October 1996
		14. Sponsoring Agency Code

15. Supplementary Notes

Interim Report for FHWA Contract DTFH61-93-C-00133, *Physical and Mechanical Properties of Recycled PCC Aggregate Concrete*

FHWA Contracting Officer's Technical Representative (COTR): *Steve Forster, HNR–20*

Special thanks are given to the following highway agencies for their assistance in the conduct of this study: *Connecticut, Kansas, Minnesota, Wisconsin, and Wyoming.*

16. Abstract

This interim report documents the field performance of nine concrete pavement projects that incorporate recycled concrete aggregate (RCA) in the construction of the pavement. Multiple sections were evaluated on many of the nine projects, due to perceived differences in performance levels or variations in pavement design (such as the use of virgin aggregate or the inclusion of dowel bars). All told, a total of 17 sections (of which 12 contain RCA) were subjected to an extensive field testing program, consisting of pavement condition surveys, drainage surveys, falling weight deflectometer (FWD) testing, coring, and serviceability assessments. A minimum of eight cores were retrieved from each section for laboratory evaluation of compressive strength, split tensile strength, dynamic elastic modulus, static elastic modulus, and thermal coefficient of expansion, as well as for volumetric surface testing and petrographic analyses.

Each of the 17 sections included in the investigation is described in detail. Performance observations and results from the FWD and laboratory testing are presented, with emphasis on evaluating the effect of RCA on pavement performance. An overall summary is provided that synthesizes the findings and conclusions of the field testing program.

A laboratory-based research effort is currently being planned to provide additional insight on the behavior of concrete mixtures using RCA. Those laboratory-based results, taken in conjunction with the results of the field testing program, will be used to produce guidelines for the design of recycled concrete mixtures and to develop recommendations for the design of concrete pavements using RCA.

17. Key Words Concrete pavement, pavement recycling, recycled aggregate, pavement design, pavement performance, pavement evaluation, field testing.	18. Distribution Statement No restrictions. This document is available to the public through the National Technical Information Service, Springfield, Virginia 22161.

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No of Pages	22. Price
Unclassified	Unclassified	296	

Form DOT F 1700.7 (8-72) Reproduction of completed page authorized

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)

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF FIGURES (continued)

LIST OF FIGURES (continued)

LIST OF TABLES

LIST OF TABLES (continued)

1. INTRODUCTION

Background

In the last two decades, recycling of highway pavement materials has received widespread interest and acceptance as a viable rehabilitation option. In this process, paving materials from the existing pavement are reclaimed and used back into some part of the reconstructed pavement (or in some part of a new pavement constructed elsewhere).

Both existing asphalt and existing concrete pavements have been successfully recycled. The primary factors cited by most State Highway Agencies (SHA) for considering pavement recycling generally include the following:

- Dwindling landfill space.
- Increased disposal costs.
- Conservation of materials.
- Scarcity of high-quality, virgin aggregates.
- Overall reduction in project costs.

In concrete pavement recycling, the existing concrete pavement is broken into smaller, more manageable pieces and transported to a crushing plant. At the crushing plant, the material is run through several crushing operations to produce aggregate materials of specified sizes. These aggregate materials, referred to as recycled concrete aggregate (RCA), are then stockpiled for use in the reconstructed pavement. A few of the many applications in which RCA has been used as an aggregate source include new concrete surfaces, new asphalt surface courses, lean concrete or cement-treated bases, granular bases, backfill material, and rip-rap.

The use of RCA products in new PCC pavement surfacing is not new, as many pavements have been constructed using recycled concrete aggregate in portland cement concrete since the 1940's. Most of these pavements have performed well, although a few have performed so poorly as to be noteworthy or have developed conditions that have caused highway agencies some concern. The performance of these unsatisfactory pavements has been highly publicized, and little information has been published concerning the performance of the pavements that have performed well, resulting in a decrease in the use of crushed concrete materials in pavement structures.

If crushed concrete products are to be used successfully in the production of portland cement concrete pavements, then there is a need to better characterize the properties of recycled concrete aggregate and concrete mixtures so that the materials produced are suitable for paving applications. Furthermore, research is needed to identify the material and pavement design factors that have resulted in both good and unacceptable performance so that design guidelines and guide specifications can be developed that will allow crushed concrete products to be utilized with confidence in concrete pavement structures.

Project Objectives and Scope

To fulfill the research needs described above, the Federal Highway Administration (FHWA) launched this research study in 1993. The overall objectives of this study are to:

- Determine the causes of pavement distresses found to be related to the use of recycled concrete coarse aggregate in concrete pavements.
- Develop a set of practical and reliable guidelines for concrete mixture design using recycled aggregate.
- Develop pavement structural and geometric designs for which recycled concrete aggregates are appropriate.

The results of this research will be used to develop guidelines for concrete mixture and pavement structural designs that allow the use of recycled concrete aggregate materials without the future occurrence of distresses that have been associated with the use of RCA materials in the past. The final product of the study will be a set of practical guidelines for concrete mixture design using recycled concrete aggregates, as well as pavement designs for which recycled aggregate concrete is appropriate.

Interim Report

The objectives listed above are to be met through a combination of a comprehensive literature review on RCA and RCA pavements, an evaluation of the performance of in-service concrete pavements constructed with RCA, and a laboratory investigation of RCA concrete aggregate properties. An interim report previously submitted to the FHWA documents the literature review, and includes a state-of-the-art review of work that has been conducted in the area of RCA, specifically in the application of using RCA in a new concrete surface.

This second interim report has been prepared to document the results of field investigations that were conducted under this project. A total of 9 concrete pavement projects constructed with RCA were evaluated during the fall of 1994. Extensive field testing (including pavement condition surveys, falling weight deflectometer [FWD] deflection testing, and slab coring) was conducted to characterize the condition of each pavement. Many of the projects included pavement sections constructed with virgin aggregate but were otherwise of similar design; when present, these sections were also tested and evaluated to provide direct comparisons of the effects of using RCA products in PCC pavement surfaces on pavement performance when all other factors are held constant. All told, 17 pavement sections were included in the field investigation.

This report consists of three chapters in addition to this one. Chapter 2 describes the data collection procedures that were followed during the field surveys, including a detailed description of the field testing and data processing and reduction procedures used for each section. It also describes the laboratory testing procedures that were developed and used to determine the properties of the RCA and RCA concrete contained in the cores retrieved from the field sections. Chapter 3 provides a detailed description of each project included in the study and presents performance observations and testing results from both the lab and the field. Chapter 4 summarizes and synthesizes the findings and conclusions that can be drawn from the studies of the field test sites and the results of the lab tests of field specimens. This work is considered in the context of the previously-completed literature review to identify areas that would benefit from additional laboratory-based research; and forms the basis for the laboratory test work plan, which is being published under separate cover.

This report is concluded with a two-part appendix. Appendix A contains a complete summary of all of the data elements that were collected for each section, while appendix B contains the detailed results of petrographic examinations performed on cores obtained from each section.

2. DATA COLLECTION ACTIVITIES

Introduction

During the fall of 1994, a comprehensive field data collection program was conducted on 9 concrete pavement projects, representing a total of 16 pavement sections. These pavement sections represent a broad range of pavement designs, traffic loads, and environmental conditions for pavements that have performed acceptably, as well as those that have not performed acceptably. Strong efforts were made to select test sites that also included control sections for contrast and comparison. A more detailed description of the site selection criteria and process is presented in chapter 3 of this report. The identification and selection of these sections are described in chapter 3 of the first interim report for this project.[1] Each of these nine projects contained at least one section whose concrete surface course was constructed with RCA. Six of the projects also contained adjacent "control" sections that were built at the same time but were constructed with virgin aggregate materials. Two other projects contained alternate structural designs or exhibited a region of distinctly different performance. When present, these control and alternate design/performance sections were also tested and evaluated, thereby allowing direct comparisons of the effects of RCA and selected design features on pavement performance.

The projects included in the field data collection program are listed in table 1, which indicates that all three of the most common rigid pavement types—jointed plain concrete pavements (JPCP), jointed reinforced concrete pavements (JRCP), and continuously reinforced concrete pavements (CRCP)—were included in the field evaluation. Figure 1 illustrates the general location of the various projects, and shows that the majority of the sections are located in the upper Midwest.

The first two letters in the project identification code used in table 1 and figure 1 indicate the State in which the project is located, while the first number refers to the project number. Sections within each project are designated as section 1 (always indicating the recycled section) and section 2 (if present, indicating the control or alternate design/performance section). For example, MN 4-1 indicates the recycled section in Minnesota project 4.

This chapter describes the data collection activities that were conducted at the nine field sites. Details of the pavement testing and evaluation procedures are described, along with information on the processing and analysis of the data obtained from that testing. Summary tables containing the design, construction, and performance data for the various pavement projects are found in appendix A.

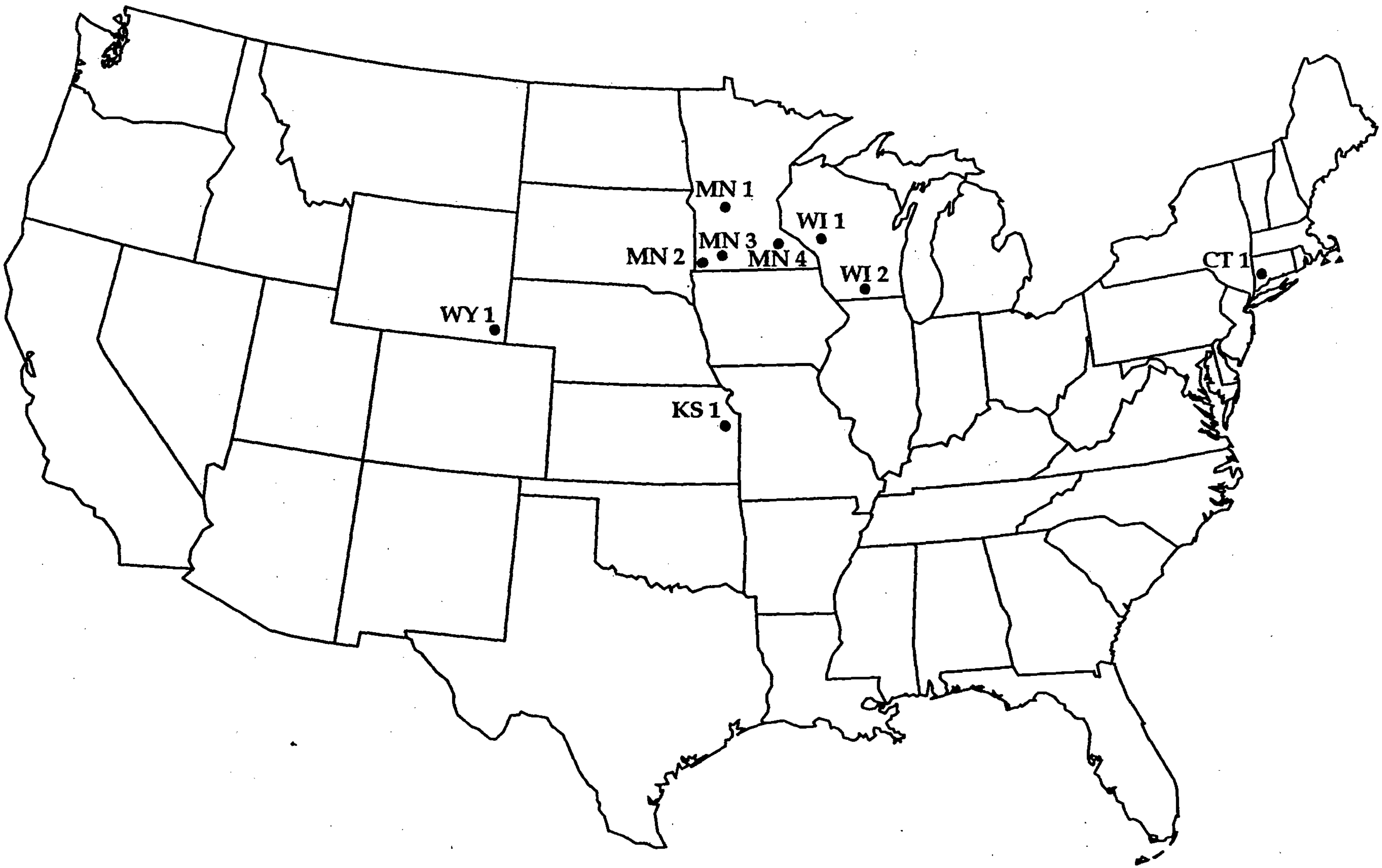


Figure 1. General location of projects evaluated in study.

Table 1. Listing of projects evaluated in study.

Project	Location*	Pavement Type	Climatic Region	No. of Sections
CT 1	I-84, Waterbury	JRCP	Wet-Freeze	2
KS 1	State Hwy. K-7, Johnson Co.	JPCP	Wet-Freeze	2
MN 1	I-94, Brandon	JRCP	Dry-Freeze	2
MN 2	I-90, Beaver Creek	JRCP	Dry-Freeze	1
MN 3	US 59, Worthington	JPCP	Dry-Freeze	1
MN 4	US 52, Zumbrota	JRCP	Wet-Freeze	2
WI 1	I-94, Menomonie	JPCP	Wet-Freeze	2
WI 2	I-90, Beloit	CRCP	Wet-Freeze	2
WY 1	I-80, Pine Bluffs	JPCP	Dry-Freeze	2

* Note: Refer to table 72 within appendix A for milepost location and related direction.

Office Data Collection

Prior to the conduct of the field studies, a comprehensive data collection plan was devised to ensure the efficient and effective collection of data. Included in the data collection plan were recommendations on the RCA projects to be evaluated under the study; those projects were selected from a master list of RCA projects compiled by the research team. Preliminary design and construction information (e.g., age, slab thickness, joint spacing, load transfer design, base type) for the projects was obtained from research reports and through discussions with SHA contacts. This information was used in the selection of the projects for evaluation and in the conduct of the field investigations.

After the field studies were completed, key missing data elements were identified and requested from the participating highway agencies. Information requested typically included traffic data, mix design data, strength data, recycled aggregate data, and joint design data. Most agencies were able to fulfill the requests for additional information, although not all of the requested information was always available.

Field Data Collection

A variety of testing activities were conducted on the 16 pavement sections evaluated under this study. These activities consisted of the following:

- A pavement condition and drainage survey.
- Measurement of slab deflections and joint/crack load transfer using a falling weight deflectometer (FWD).
- Retrieval of pavement cores.
- Photographic (35-mm slides) documentation of the pavement condition.
- An estimate of the present serviceability rating (PSR) of the pavement.

Traffic control for the field testing was provided by the participating SHA's. Each SHA was notified about 1 month prior to the field testing in their State, which provided the maintenance crews sufficient time to work the request into their schedule. A followup phone call was made about 1 week prior to the testing to discuss specific meeting times and lane closure requirements. On the whole, the SHA's were very cooperative in setting up the traffic control for the field testing, and in many cases went out of their way to accommodate the needs of the testing crews.

Site Selection

Most of the projects included in the study were several kilometers long, and it was necessary to identify a "representative" section within that length for testing. The day before the field testing was conducted, the survey team visited the project site to establish the testing location. During this visit, a 305-m (1,000-ft) pavement section was identified within which the field testing would be conducted. Considerations in the selection of the specific section location included:

- Horizontal curves less than 3 degrees and vertical grades less than 4 percent.
- A minimum of cut/fill transitions, either longitudinally or transversely.
- No culverts, pipes, or other substructures within the section (if possible).
- Uniform traffic flow throughout the project.
- Identification of any other factors that may in some way compromise the safety of the field survey teams.

In one case (CT 1), the section lengths were less than 305 m (1,000 ft), so the locations of the sections were fixed and the entire sections were tested and evaluated.

Pavement Condition and Drainage Surveys

Detailed pavement conditions surveys were conducted on each pavement section. The condition survey recorded the presence of all visible distresses and conditions that were observed on the pavement. Among the distress items noted or measured were:

- Cracking (transverse, longitudinal, and compression cracks).
- Corner breaks.
- Spalling.
- Punchouts (CRCP only).
- Localized areas of scaling.
- Concrete durability distress (D-cracking or reactive aggregate).
- Joint faulting.
- Crack faulting.
- Joint width.
- Lane-shoulder separation and drop-off.
- Other distinguishing pavement (patches, core holes, etc.) or roadway (signs, culverts, bridges, etc.) features.

In the conduct of the distress surveys, the SHRP *Distress Identification Manual for the Long-Term Pavement Performance Studies* was followed to ensure that the data collected is consistent with the data collected under the LTPP studies.[2] Joint faulting was measured using both mechanical and digital read-out (Georgia-type) faultmeters at both the outside pavement edge (0.3 m [1 ft] from the lane/shoulder joint) and in the outer wheelpath of the outside lane. The mechanical and digital faultmeters provided measurements that were accurate to the nearest 0.025 mm (0.001 in) and 0.1 mm (0.004 in), respectively.

The CRCP sections evaluated under this study were not mapped. Instead, within a designated 305-m (1,000-ft) section, the total number of transverse cracks were counted and the number of deteriorated cracks, punchouts, and other failed areas were recorded. In addition, the widths of several transverse cracks were measured.

During the pavement condition surveys, every effort was made to record the exact location of each pavement section to allow the possibility of future monitoring. The location information was recorded at two levels: project and section. The project-level information was intended for locating the general location of the pavement sections at highway speeds. Either starting milepost or, if mileposts were not available, the direction and distance from the nearest fixed object (such as an overpass) were used for this purpose. The section-level information was used to establish the exact location of the test sections; station numbers stamped in the pavement were used for this purpose. This information, along with other general information identifying the section, was recorded on the form shown in figure 2.

The survey form shown in figure 3 was used to record the pavement condition information. The location of features of interest, such as distress and other information, were tracked using relative stationing. The beginning of each section was assigned the station number 0+00 and all referencing within that section was done using station numbers that increased in the direction of traffic. At the beginning of the survey, the starting location of the section was painted on the pavement and the survey section length was marked off with a measuring wheel. The measured section length was then

Field Survey: ***General Information***

Project ID: ________________

Date of Survey (mm/dd/yy): ___/___/___

Surveyors' Initials: _____/_____/_____

PAVEMENT TYPE: AC (Conv. or FD) PCC (JPC, JRC, or CRC) AC/AC AC/PCC

DRAINAGE TYPE: None Perm Base/Edge Drains Edge Drains Only Daylighted

DRAINAGE RETROFITTED? Yes No

Test Section Location:

	Milepost (MP)	Station (STN)	Section Length, ft
Start Point		+	
End Point		+	

If no MP or STN: Distance from the nearest landmark: __________ ft

Direction FROM the landmark: Line of Traffic / Against Traffic

Landmark description (type/name of structure/interchange/crossroad):

____

____

Shoulder: **General Condition:** Good / Fair / Poor / Failed

Shoulder Surface Type

1. Turf
2. Granular
3. AC
4. Concrete
5. Surface Treatment
6. Other:____________

	Type	Width
Outside Shoulder:		ft
Inside Shoulder:		ft

PCC AND AC/PCC PAVEMENTS ONLY

Contraction Joint: Joint Spacing: _____._ ft Skewed? y / n

Random Spacing: ____-____-____-____ ft ft/Lane: ______

Sealant Type: None HP SI Preform Other: ____________

Lane-Shoulder Joint: **Sealant Type:** None HP SI Preform Other

Longitudinal Joint: Method Used to Form Centerline Joint: Saw Cut / Plastic Insert

Sealant Type: None HP SI Preform Other: ____________

Roughness and Serviceability:

Roughness Device Used:__________

		Lane 1*	Lane 2	Lane 3
Roughness Index	Trial 1			
	Trial 2			
Roughness Measurement Speed (mph)				
Present Serviceability Rating (mean)				

*Lane 1 is outer lane, Lane 2 is next lane 1, etc.

Figure 2. General field survey form.

To Be Sketched	Note on Sketch	
Corner Breaks	1	L M H
D-Cracking	2	L M H
Longitudinal Cracking	3	L M H
Transverse Cracking	4	L M H
Spalling (L & T)	6/7	L M H
Blowups	11	
Crack Faulting (M & H cracks)	in	
All shoulder distresses		

Field Survey: *Data Collection Form*

Date of Survey (mm/dd/yy): ____/____/____

Surveyors' Initials: ______/______/______

Project ID: ____________________

Page No: _______ of ______

Note:
Map Cracking/Scaling
Polished Aggregate
Patches/Replaced Slabs
Improper Joint Construction

Transverse Joint Type Code
1. Contraction Joint
2. Construction Joint
3. Patch Approach Joint
4. Pressure Relief Joint

	OUTER LANE							INNER LANE						
STATION														
Trans Joint Type														
Transverse Joint Spalling	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H
Transverse Joint Seal Damage	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H
Longitudinal Joint Seal Damage	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H
Pumping	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H
Patch/Slab Replacement Deterioration	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H
Slab Deterioration Adjacent to Patch	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H	N L M H
Lane to Shoulder Dropoff (in)														
Lane to Shoulder Separation (in)														
Faulting (in) 1-ft from edge														
2.5-ft from edge														
Joint Width — Outer Lane Only (in)														
Joint Depth (in)														

Figure 3. Distress field survey form.

recorded and the end of the section was painted on the pavement. The measured section length was later used in normalizing the distress quantities (e.g., cracks per km).

As part of the field studies, a comprehensive drainage survey was conducted to assess the drainability of each section. This survey consisted of evaluating the depth and condition of the drainage ditches, examining the transverse and longitudinal joint sealant, examining the condition of drainage outlets, looking for signs of poor drainage (pumping, cat-tails in the ditches, standing water, etc.), and measuring transverse pavement and shoulder slopes. The form used to record the drainage conditions is shown in figure 4.

FWD Deflection Testing

In conjunction with the distress surveys, deflection testing was performed on each section. This testing was conducted to estimate the concrete elastic modulus and the effective subgrade support (*k*-value), and also to estimate the load transfer efficiency (LTE) across transverse joints and cracks.

Although there is a SHRP LTPP protocol for pavement deflection testing, it calls for an extensive level of testing that was considered to be beyond the needs for this project. Therefore, the following deflection testing scheme was conducted for each section:

- 5 center-of-slab locations (to backcalculate concrete elastic modulus values and effective k-values).
- 20 transverse joint locations at 10 transverse joints (load placement on both sides of the 10 joints to determine load transfer efficiencies and void detection).
- 20 transverse crack locations at 10 transverse cracks (load placement on both sides of the 10 cracks to determine load transfer across transverse cracks).
- 10 midslab edge locations (to consider slab fatigue damage and to calculate LTE for sections with tied PCC shoulders).

On pavements that did not exhibit any transverse cracks, only center-of-slab, transverse joint, and slab edge deflection testing were performed.

The same sensor spacings (-305, 0, 305, 457, 610, 914, and 1524 mm [-12, 0, 12, 18, 24, 36, and 60 in]) and drop sequence (5430, 4070, 5430, and 7240 kg [12 kip, 9 kip, 12 kip, and 16 kip]) used under the SHRP LTPP program were employed, although only one drop was made at each load instead of three. The FWD testing pattern used in the study is illustrated in figure 5, with the sensor spacings depicted in figure 6. The standard load transfer sensor configuration was used to determine load transfer efficiencies at transverse joints and cracks.

Field Survey: ***Drainage Information***

Project ID: ____________________

____

Date of Survey (mm/dd/yy): ____/____/____

Surveyors' Initials: ______/______/______

Slope Measurements:

	Station	Slope	
		Outer	Inner
Longitudinal Slope (nearest 1/16") 3 measurements, equally spaced along project.	+	/	/
	+	/	/
	+	/	/
Transverse Slope (nearest 1/16") 3 measurements, equally spaced along project.	+	/	/
	+	/	/
	+	/	/
Shoulder Slope (nearest 1/16") 3 measurements, equally spaced along project.	+	/	/
	+	/	/
	+	/	/

Cut/Fill and Ditch Line Depth:

Circle, if Cut/Fill Depth Uniform	Cut/Fill Depth	Station(s)		Depth of Ditch Line
1.	Fill > 40 ft	+	+	ft
2.	Fill 16 - 40 ft	+	+	ft
3.	Fill 6 - 16 ft	+	+	ft
4.	At Grade (5 ft fill to 5 ft cut)	+	+	ft
5.	Cut 6 - 15 ft	+	+	ft
6.	Cut 16 - 40 ft	+	+	ft
7.	Cut > 40 ft	+	+	ft

Lane/Shoulder Joint Integrity:

	Outer Shoulder	Inner Shoulder
Sealant Damage	N L M H	N L M H
Blowholes	N L M H	N L M H
Sealant Type	None HP SI Preform Other:	

Subsurface Drainage (visual):

Type of drainage system present: _____

1. None. 3. Transverse Drains.
2. Longitudinal Drains. 7. Other: ____________

Condition of Drainage Outlets:________________________________

Indicators of Poor Drainage:

Cattails or willows growing in ditch: y / n

Drainage outlets clogged: y / n

Drainage outlets below ditch line: y / n

Non-continuous cross section, crown to drainage ditch: y / n

Pumping: N L M H

Other: ________________________________

Figure 4. Drainage survey form.

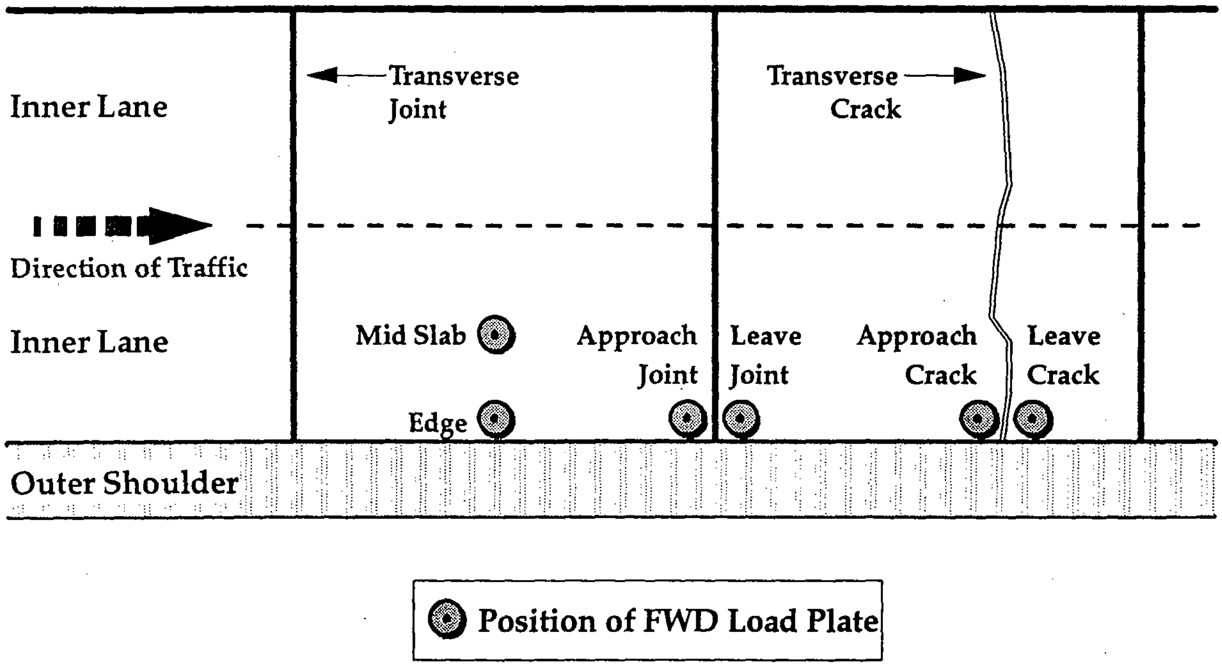


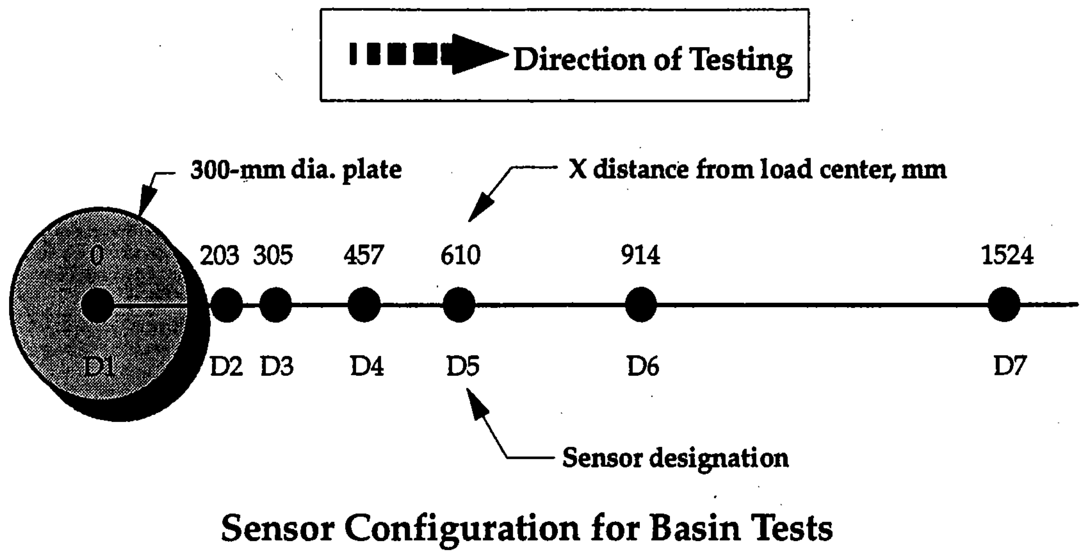
Figure 5. FWD testing pattern.

A Dynatest model 8081 heavyweight falling weight deflectometer was used for the deflection testing. Representative slabs within each section were selected for the FWD testing, and the 5 center-of-slab locations, 10 transverse joint locations, and 10 midslab edge locations were distributed over the entire length of the section.

For each test section, slab temperatures were monitored throughout the conduct of the FWD testing to evaluate joint lockup and slab curling conditions. Following the SHRP LTPP protocol, 13-mm (0.5-in) diameter holes were drilled 203 mm (8 in) apart to obtain temperatures at the top, middle and bottom of the PCC slabs. Three holes were drilled to different depths in the slab (as shown in figure 7) and "filled" with mineral oil; the temperature of the mineral oil was then checked every 30 min using a temperature probe. The slab surface temperature was closely monitored to ensure that all testing was conducted at temperatures less than 27 °C (80 °F) to avoid joint/crack lockup.

Pavement Coring

Pavement coring was conducted as part of the distress surveys and deflection testing to help characterize the properties of the pavement. Each pavement section was placed in one of three different categories, which dictated the type and amount of coring conducted on each section:



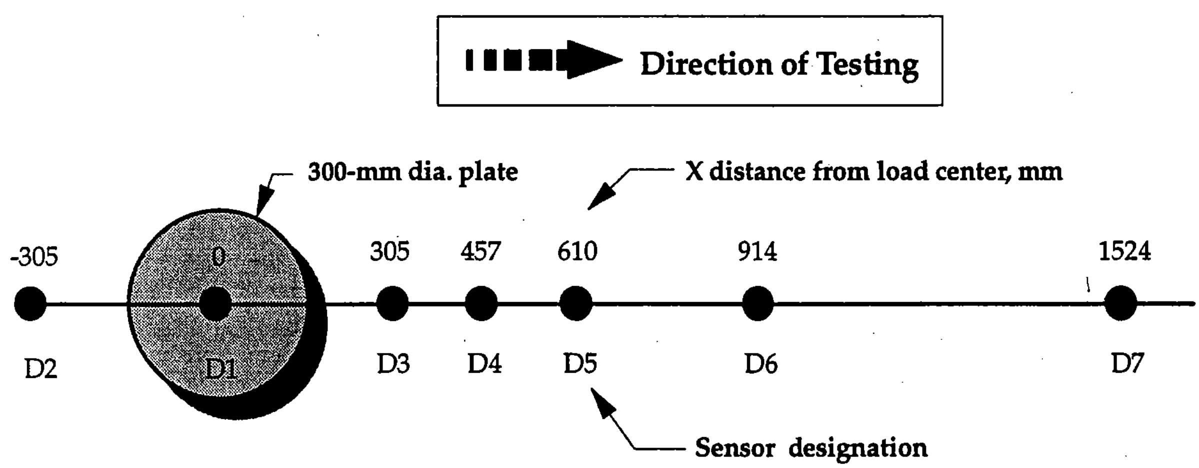


Figure 6. FWD sensor spacing.

- Category 1 (*pavements in good condition*)
  - Five cores taken at (uncracked) midslab locations (for strength, elastic modulus, and thermal coefficient testing).
  - Three cores taken across transverse joints (for quantification of joint face texture).

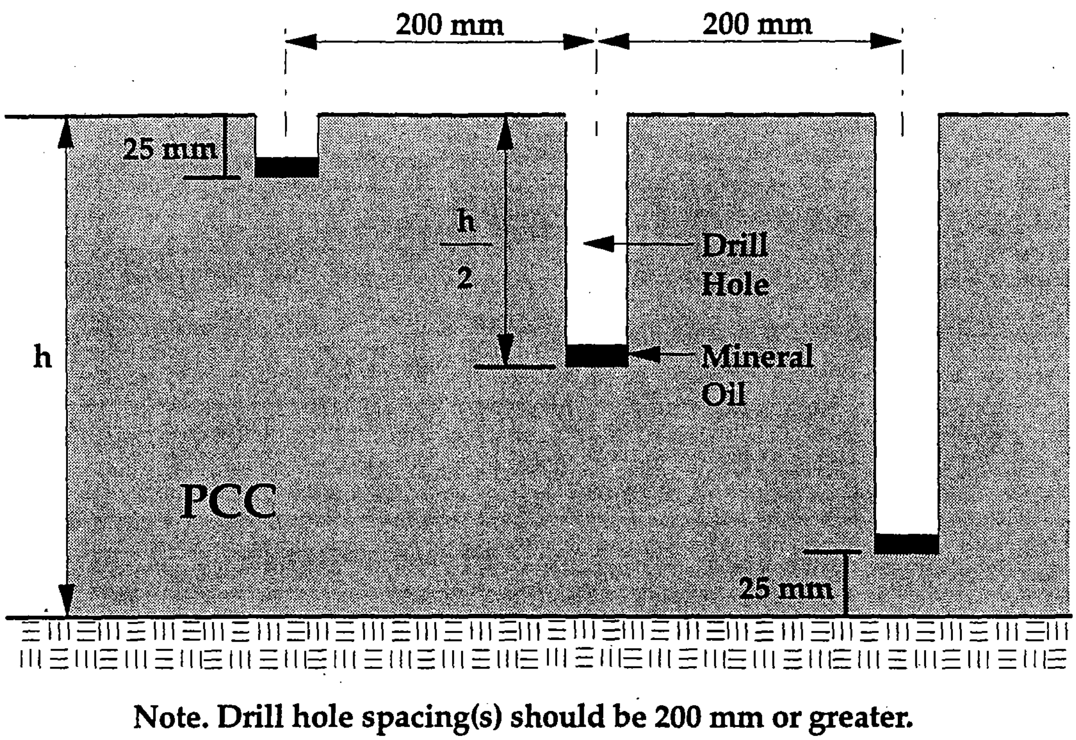


Figure 7. Location of temperature monitoring holes.

- Category 2 (*pavements exhibiting midpanel slab cracking*)
  - Five cores taken at (uncracked) midslab locations (for strength, elastic modulus, and thermal coefficient testing).
  - Three cores taken across transverse joints (for quantification of joint face texture).
  - Three cores taken across transverse cracks (for quantification of crack face texture).
- Category 3 (*pavements exhibiting other distresses*)
  - Five cores taken at (uncracked) midslab locations (for strength, elastic modulus, and thermal coefficient testing).
  - Three cores taken across transverse joints/cracks (for quantification of joint/crack face texture) (D-cracked and ASR sections).
  - Two cores taken at 0.3 and 0.6 m (1 and 2 ft) away from transverse joints (to determine extent of D-cracking) (D-cracked sections only).
  - Three cores taken across deteriorated transverse cracks (for quantification of crack face texture) (CRCP only).
  - Two cores taken across nondeteriorated transverse cracks (for quantification of crack face texture) (CRCP only).

Petrographic analyses and uranyl acetate testing (for detection of alkali-silica reactivity) were also performed on the cores retrieved from the joints and cracks of each project.

Thus, the number of cores retrieved from each section varied significantly, depending upon the type of pavement and the type of distress it was exhibiting. Table 2 summarizes the type and amount of coring performed on each section.

A combination of 100-mm (4-in) and 150-mm (6-in) diameter cores was retrieved from the sections, the size depending upon both the type of laboratory testing that was to be performed on the core and the maximum top size of the coarse aggregate. The following general criteria were followed in selecting the core sizes for each section:

- For compression testing and linear traverse testing, generally 100-mm (4-in) diameter cores were sufficient unless the top size of the coarse aggregate exceeded 25 mm (1 in).
- For indirect tensile strength testing, 150-mm (6-in) diameter cores were specified.
- For joint and crack cores, 150-mm (6-in) diameter cores were specified to provide a crack or joint face of larger surface area for quantification of surface texture.

For pavements containing dowels at the transverse joints, a pachometer was used to locate the dowel so that they could be avoided during the coring operation. Similarly, the reinforcing steel in CRCP was located and avoided when retrieving cores from that pavement type. However, in a few cases, cores were intentionally taken through the steel to look for "socketing" around the dowel bars or to inspect the steel for corrosion and check the condition of the steel coating.

Cores were generally retrieved using a Milwaukee portable core drill unit powered by a generator housed in the accompanying van, although some SHA's assisted in coring operations using their own equipment. Prior to the coring, the pavement section was evaluated, and suitable coring locations were marked. The locations of the cores were distributed over the entire length of the sections to the extent possible.

The location and condition of each core were documented in a core log, similar to that shown in figure 8. This core log documents the location and orientation of all cores, and also contains a summary of any core losses or fractures both before and after the coring operation, and serves as a permanent record of the coring operations.

After the cores were retrieved, their length was measured and their location and condition noted in the log book. The holes were cleaned of excess moisture and filled with a cementitious patching material.

Photographic Survey

After the pavement condition survey was completed, a battery of 35-mm slides was taken to fully document the pavement condition and environment. The photo survey consisted of an initial set of slides to provide an overview of the project, followed by photos of typical pavement conditions (e.g., cracking, joints, shoulders, ditches).

Table 2. Summary of coring operations for each section.

Section	Location	Pavement Design	No. of Joint Cores	No. of Crack Cores	No. of Midslab Cores
CT 1-1 (*recycled*)	I-84, Waterbury	230 mm JRCP 12 m joints 38 mm dowels 51 mm top size	3 (150 mmϕ) [6 inϕ]	3 (150 mmϕ) [6 inϕ]	5 (150 mmϕ) [6 inϕ]
CT 1-2 (*control*)	I-84, Waterbury	230 mm JRCP 12 m joints 38 mm dowels 38 mm top size	3 (150 mmϕ) [6 inϕ]	3 (150 mmϕ) [6 inϕ]	5 (150 mmϕ) [6 inϕ]
KS 1-1 (*recycled*)	K-7, Johnson Co.	230 mm JPCP 4.7 m joints No dowels 38 mm top size	3 (150 mmϕ) [6 inϕ]	—	5 (150 mmϕ) [6 inϕ]
KS 1-2 (*control*)	K-7, Johnson Co.	230 mm JPCP 4.7 m joints No dowels 19 mm top size	3 (150 mmϕ) [6 inϕ]	—	5 (150 mmϕ) [6 inϕ]
MN 1-1 (*recycled*)	I-94, Brandon	280 mm JRCP 8.2 m joints 32 mm dowels 19 mm top size	3 (150 mmϕ) [6 inϕ]	1 (150 mmϕ) [6 inϕ]	4 (100 mmϕ) [4 inϕ] 1 (150 mmϕ) [6 inϕ]
MN 1-2 (*control*)	I-94, Brandon	280 mm JRCP 8.2 m joints 32 mm dowels 19 mm top size	3 (150 mmϕ) [6 inϕ]	—	4 (100 mmϕ) [4 inϕ] 1 (150 mmϕ) [6 inϕ]
MN 2-1 (*recycled*)	I-90, Beaver Creek	230 mm JRCP 8.2 m joints 25 mm dowels 19 mm top size	3 (150 mmϕ) [6 inϕ]	3 (150 mmϕ) [6 inϕ]	4 (100 mmϕ) [4 inϕ] 1 (150 mmϕ) [6 inϕ]
MN 3-1 (*recycled*)	US 59, Worthington	200 mm JPCP 4.0-4.9-4.3-5.8 m joints No dowels 19 mm top size	3 (150 mmϕ) [6 inϕ] 2 (100 mmϕ)	—	4 (100 mmϕ) [4 inϕ] 1 (150 mmϕ) [6 inϕ]

Table 2. Summary of coring operations for each section (continued).

Section	Location	Pavement Design	No. of Joint Cores	No. of Crack Cores	No. of Midslab Cores
MN 4-1 (*recycled*)	US 52, Zumbrota	230 mm JRCP 8.2 m joints 25 mm dowels 25 mm top size	3 (150 mmϕ) [6 inϕ]	3 (150 mmϕ) [6 inϕ]	4 (100 mmϕ) [4 inϕ] 1 (150 mmϕ) [6 inϕ]
MN 4-2 (*control*)	US 52, Zumbrota	230 mm JRCP 8.2 m joints 25 mm dowels 38 mm top size	3 (150 mmϕ) [6 inϕ]	3 (150 mmϕ) [6 inϕ]	4 (100 mmϕ) [4 inϕ] 1 (150 mmϕ) [6 inϕ]
WI 1-1 (*recycled*)	I-94, Menomonie	280 mm JPCP 3.7-4.0-5.8-5.5 m joints No dowels 38 mm top size	3 (150 mmϕ) [6 inϕ]	3 (150 mmϕ) [6 inϕ]	5 (150 mmϕ) [6 inϕ]
WI 1-2 (*recycled*)	I-94, Menomonie	280 mm JPCP 3.7-4.0-5.8-5.5 m joints 35 mm dowels 38 mm top size	3 (150 mmϕ) [6 inϕ]	1 (150 mmϕ) [6 inϕ]	5 (150 mmϕ) [6 inϕ]
WI 2-1 (*recycled*)	I-90, Beloit	250 mm CRCP 38 mm top size	—	3 (150 mmϕ) [6 inϕ]	4 (150 mmϕ) [6 inϕ]
WI 2-2 (*recycled*)	I-90, Beloit	250 mm CRCP 38 mm top size	—	4 (150 mmϕ) [6 inϕ]	5 (150 mmϕ) [6 inϕ]
WY 1-1 (*recycled*)	I-80, Pine Bluffs	250 mm JPCP 4.3-4.9-4.0-3.7 m joints No dowels 25 mm top size	3 (150 mmϕ) [6 inϕ]	—	4 (100 mmϕ) [4 inϕ] 1 (150 mmϕ) [6 inϕ]
WY 1-2 (*control*)	I-80, Pine Bluffs	250 mm JPCP 4.3-4.9-4.0-3.7 m joints No dowels 38 mm top size	3 (150 mmϕ) [6 inϕ]	—	4 (100 mmϕ) [4 inϕ] 1 (150 mmϕ) [6 inϕ]

Section Identification Number:

Date:

Surveyed By:

Coring By:

FWD Testing By:

Core ID #	Station	Offset	Thickness	Comments

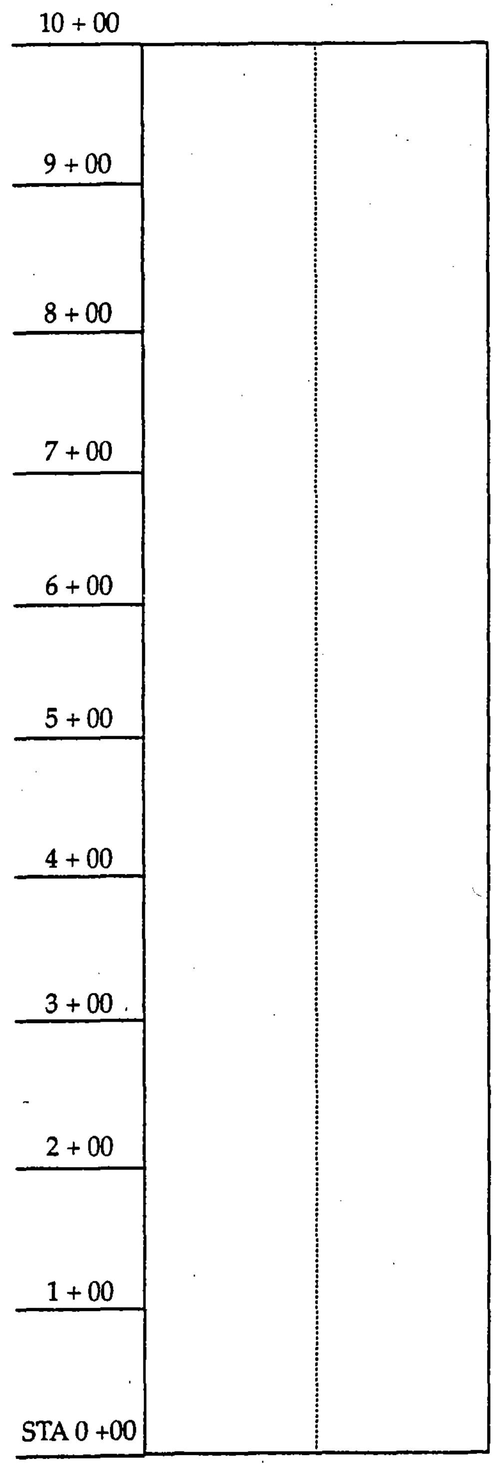


Figure 8. Section core log.

Present Serviceability Rating

At the conclusion of the field surveys, the survey team drove over the pavement section at posted speeds and rated the serviceability of the pavement using the AASHTO scale (0 to 5, 0 being impassable, 5 being perfectly smooth). The average of the individual rater values was then computed and reported as the estimated present serviceability of the section.

Data Reduction Procedures

Reduction of Pavement Distress Data

After each field trip, the pavement distress data collected in the field were first entered into spreadsheet files, one file per section. All of the data on the field survey sheets (figures 2, 3, and 4), including the data extracted from the distress maps, were transferred to the individual spreadsheet files; hence, a very detailed data base was created that contains distress data for individual slabs. Simple data reduction that could be performed on the raw data (such as determining the average values, maximum values, minimum values, percentages, and normalizing distress quantities) was performed on the individual spreadsheets. The summary information from each file was then aggregated into a large flat file that consists of rows of sections and columns of reduced performance data (i.e., each row in the flat file contains all of the reduced performance data for a pavement section). The pavement performance data shown in the summary tables were extracted from the flat table.

Reduction of FWD Testing Results

All of the FWD data collected in the field were stored on computer diskette. In the office, these data were checked to remove any bad data (for example, deflections that increase as the distance from the load increases) and then evaluated using several different computer programs for backcalculation, load transfer efficiencies, and void detection.

Backcalculation

Backcalculation of the FWD testing results provide an estimate of the elastic modulus of the concrete slab (E) and the modulus of subgrade reaction (k). While several different backcalculation methods are available, a procedure developed and verified in a concurrent FHWA study was used (Smith, et al. 1995). This procedure is based on a theoretically rigorous approach utilizing the closed-form solution for the plate on a Winkler foundation (as proposed by Russian researcher Korenev) and effective plate concepts (as presented by Ioannides, et al. and by Ioannides and Khazanovich).[3,4,5]

The backcalculation method finds a pavement system elastic parameters that provide the least discrepancy between the calculated and measured deflection basins.

That is, a set of E and k values are sought whose calculated deflection profile closely matches the measured profile. The problem can be formulated as a minimization of the error function, F, defined as follows:

$$F(E, k) = \sum_{i=0}^{n} \alpha_i \left(w(r_i) - W_i\right)^2 \tag{1}$$

where:

α_i = Weighting factors.
w_i = Calculated deflection.
W_i = Measured deflection.

This routine forms a system of two nonlinear equations for k and l where l is the radius of relative stiffness, defined as:

$$\ell = \left[\frac{E h^3}{12(1 - \mu^2)k}\right]^{0.25} \tag{2}$$

where:

E = Modulus of elasticity of PCC, lbf/in^2.
h = Slab thickness, in.
μ = Poisson's ratio.
k = Modulus of subgrade reaction, $lbf/in^2/in$.

The two nonlinear equations are solved to produce a minimum difference between the measured and calculated deflections. The procedure is based on four sensors (at distances of 0, 305, 610, and 914 mm [0, 12, 24, and 36 in] from the center of the load plate). The weighting factors might be set equal to 1, $(1/W_i)^2$, or any other numbers.

In addition, the new methodology also permits the evaluation of two-layer systems. This ability is particularly valuable because the bonding condition between the slab and base can have a significant effect on the backcalculation results (as well as on the performance of the pavement). The procedure allows for the identification of bonded and unbonded structures, which results in a more accurate representation of the pavement structure. In this study, all sections were determined to be unbonded except the KS 1 project, which contained a cement-treated base course.

The backcalculation procedure described herein has been evaluated and validated using deflection data obtained on 303 concrete pavement sections throughout the country. The methodology using a four-sensor configuration was found to produce results similar to those obtained from a seven-sensor, AREA-based procedure. More details on the backcalculation methodology, its theoretical basis, and overall validity can be found in a recent FHWA report.[6] The procedure has been computerized to allow for efficient and rapid analyses.

Load Transfer Efficiency

The deflection LTE at the PCC pavement joints and cracks is defined as the ratio of the deflection of the unloaded side to that of the loaded side:

$$LTE = \frac{\delta_U}{\delta_L} * 100 \tag{3}$$

where:

LTE = Load transfer efficiency, percent.
δ_U = Deflection of the unloaded side.
δ_L = Deflection of the unloaded side.

The LTE given by equation 3 is valid if the deflection measurements are taken at the points immediately to either side of the joint or crack; however, because of equipment limitations, this is not possible. In reality, the measurements are taken 150 mm (6 in) away from the joint, resulting in the ability of slab bending over the 310-mm (12-in) separation to affect the measurements. To compensate for the slab bending, a correction factor is applied to equation 3:

$$LTE = \frac{\delta_U}{\delta_L} * B * 100 \tag{4}$$

where B is the ratio of the deflection at the load plate to the deflection 310 mm (12 in) away from the load plate under a center loading condition (basin testing).

The LTE values determined from the FWD testing data are given in appendix A. The column labeled "Approach" refers to the results from testing conducted with the load placed at the outer approach corner of the slab, and the column labeled "Leave" refers to the results from testing conducted with the load plate placed at the outer leave corner of the slab.

Void Detection

The FWD data were also used to determine the extent of loss of support at the slab corners. The loss of materials from beneath the transverse joints, especially near the slab corners, can lead to faulting and corner breaks. This is an important design consideration to prevent loss of serviceability.

The loss of support at the slab corners can be determined by examining the corner deflections from the FWD testing conducted at various load levels. The step-by-step procedure for void detection presented by Crovetti and Darter is summarized below:[7]

1. Measure deflection at the slab corner under a range of load levels that includes a 40-kN (9,000-lb) load. The SHRP load sequence used in FWD testing—40 kN (9 kip), 53 kN (12 kip), and 71 kN (16 kip)—satisfies this requirement. The testing should be conducted in the air temperature range between 10 °C (50 °F) to 27 °C (80 °F).

2. Plot the results on a deflection (y-axis) versus load (x-axis) graph and draw a best-fit line through the points. Extend this line to determine the y-intercept.

3. A corner that has full support will show the y-intercept very close to zero. Any line that shows the y-intercept greater than 0.76 mm (0.03 in) is considered an indication of loss of support.

The concept behind this procedure is simple: if a void exists under a slab, the slab will exhibit a non-linear response and show a higher deflection at lower load levels, causing the y-intercept to plot higher in the graph. In general, a larger y-intercept may be expected for larger voids.

The loss of support is expressed as the percent of corners with voids. The percent corners with voids is taken as the fraction of the corners with voids, as determined using the above procedure, among the corners tested. The results of this analysis showed that the testing conducted with the load plate placed on the leave slab indicates a greater percentage of the corners with voids than the testing conducted with the load plate placed on the approach slab. This result is expected, since voids typically form under the leave side of a joint. The results of the void evaluation are given in appendix A. The percent corners with voids given in appendix A are the results of the testing conducted with the load plate placed on the leave slab.

Laboratory Testing of Pavement Cores

Substantial laboratory testing was performed on the cores that were retrieved from the field sites. A brief description of the various laboratory tests is given below:

Compression Testing—Cores were prepared and capped as per the guidelines provided in ASTM C 42, "Standard Test Method for Obtaining and Testing Drilled Cores and Sawed Beams of Concrete." Testing was performed in accordance with guidelines provided in ASTM C 39, "Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens."

Split Tensile Testing—Testing was performed in accordance with guidelines provided in ASTM C 496, "Standard Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens."

Dynamic Elastic Modulus—Testing was performed in accordance with guidelines provided in ASTM C 215, "Standard Test Method for Fundamental Transverse, Longitudinal, and Torsional Frequencies of Concrete Specimens."

Static Elastic Modulus—Testing was performed in accordance with guidelines provided in ASTM C 469, "Standard Test Method for Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression."

Thermal Coefficient of Expansion—The testing procedure used to determine the thermal coefficient of expansion and contraction is based on that which was developed by FHWA and described in the RFP (request for proposals) for the project. The measuring apparatus used to test the cores is depicted in figure 9. The following test procedure was used:

1. Place the measuring apparatus in the controlled temperature bath (CTB). Set the CTB at 10.0 °C (50.0 °F).

2. Soak the specimens in lime water at 23.0 ± 2 °C (73.4 ± 4 °F) for a sufficient period of time to bring them to a saturated condition. Determine the length of the concrete specimen to the nearest 1 mm while it is submerged in the bath.

3. Place the 10.2 by 20.3 cm (4 by 8 in) cylindrical specimen into the apparatus once the temperature of the water is at 10.0 °C (50.0 °F). Gently wiggle the specimen back-and-forth in the apparatus to ensure that it is resting in the center of the rollers and that it is in immediate contact with the bullet head. Record the time. *From this point forth, the specimen is not to be moved (or touched) until after length readings have been recorded at both 10.0 and 40.0 °C (50.0 and 104.0 °F).*

4. Wait 1 h (or 1.5 h for a 15.2 by 30.5 cm [6 by 12 in] cylindrical specimen) for the specimen to stabilize at 10.0 °C (50.0 °F). Then record the time and readings from the thermometer and LVDT connected to the apparatus. (See footnote for the required accuracy for each reading to be recorded.) The length of the specimen can be determined by adding the difference between the LVDT reading just recorded for the concrete specimen and the LVDT reading taken while the stainless steel calibration specimen was at 10.0 °C (50.0 °F). The difference is then added to the length of the calibration specimen at 10.0 °C. The length of the calibration specimen was determined to the nearest micron at 10.0 °C (50.0 °F).

5. Set the CTB to 40.0 °C (104.0 °F).

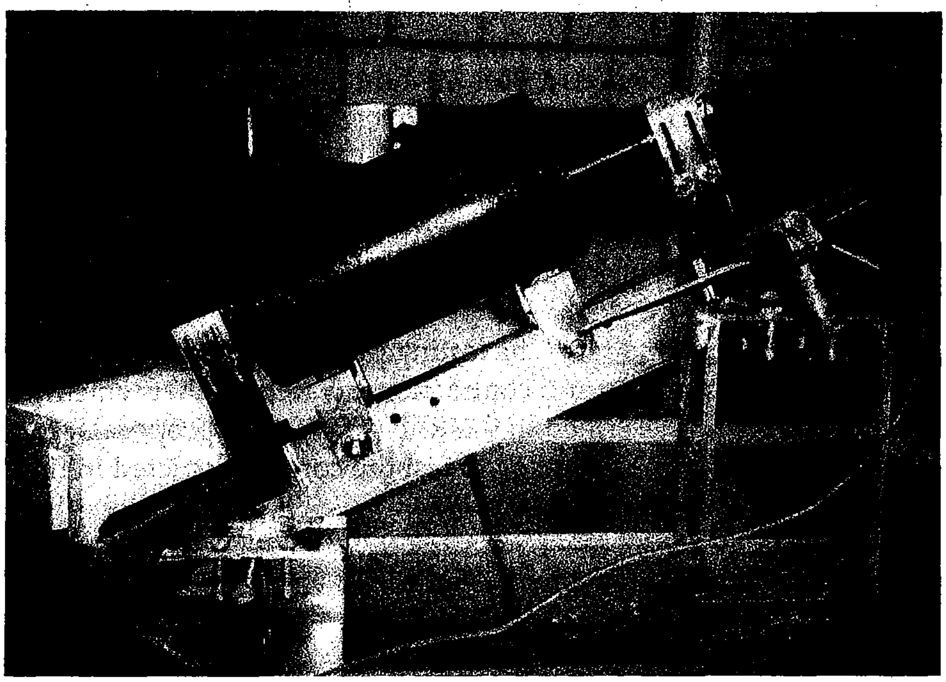


Figure 9. Thermal coefficient apparatus stand.

6. Record the time at which the temperature of the CTB reaches 40.0 °C (104.0 °F). After the temperature of the CTB has reached 40.0° C (104.0 ° F), wait 45 min (for either the 10.2 by 20.3 cm (4 by 8 in) or 15.2 by 30.5 cm (6 by 12 in) cylindrical specimens), and then record the time and the readings from the thermometer connected to the apparatus and the LVDT. Check the two temperatures recorded at the beginning of the 45 min and after the 45 min has expired to ensure that they are the same.

Once a week, the testing apparatus should be removed from the CTB and allowed to dry for a minimum of 12 h. Lubricate the rollers and LVDT before placing the apparatus back into the CTB. The apparatus should be placed back into the CTB for a minimum of 2 h before beginning a test.

The suggested times are based on calibrations of the equipment used in the laboratory at the University of Minnesota. These times may be equipment-dependent and it is suggested that each laboratory validate these procedures on their own equipment.

Volumetric Surface Texture—The Volumetric Surface Texture (VST) test procedure was developed at the University of Minnesota to provide an estimate of the load transfer potential available through aggregate interlock across a fractured concrete surface. It may also provide an indication of the degree of surface abrasion that has taken place since fracture. The test apparatus consists of a spring-loaded probe with a digital readout that is mounted on a frame over a computer-controlled microscope stage of the type typically used for performing linear traverse or other measurements of concrete air void systems. The digital readout measures the distance from an arbitrarily established datum to the fractured surface at any chosen point (d_i). These distances are recorded electronically for each point in a predetermined grid pattern across the fractured surface (a 3.18-mm [0.125-in] grid was used for this work) to define the 3-dimensional profile of the fractured surface. The average measurement area was about 161 cm^2 (25 in^2). The volumetric surface texture measuring device that was used to test the cores is depicted in figure 10.

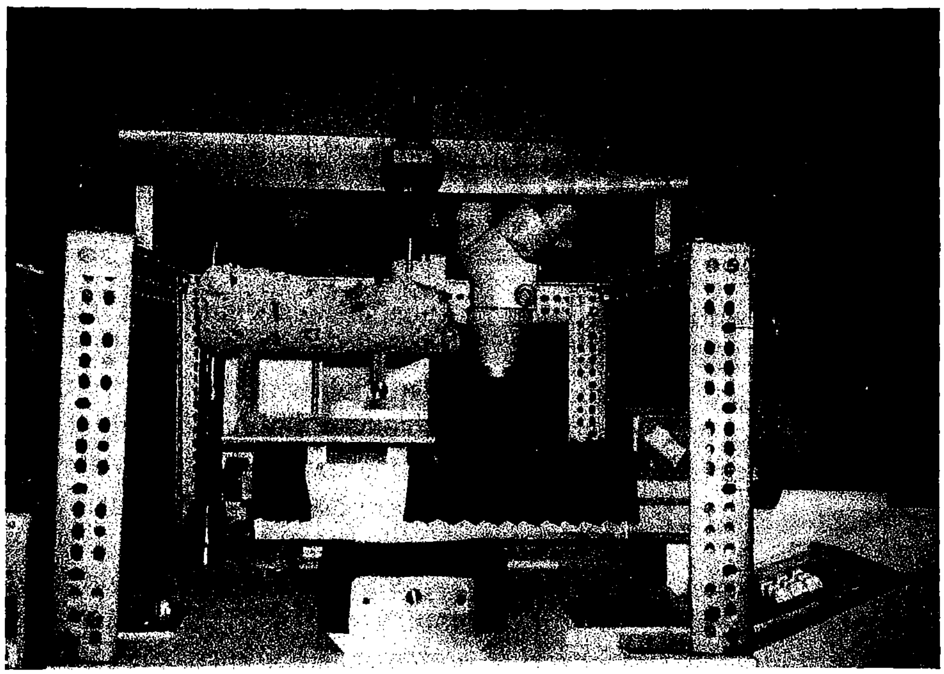


Figure 10. Volumetric surface texture measuring device.

Once the VST testing is completed, the surface texture is quantified by a volumetric surface texture ratio (VSTR). The VSTR is the ratio of the volume of texture per surface area in units of cm^3/cm^2. To calculate the VSTR, the distances (d_i, such that i represents the individual square being measured from the predetermined grid) measured from the datum to the fractured surface are averaged (d_{ave}). See figure 11. The difference between the average distance and each individual distance ($r_i = d_i - d_{ave}$) is calculated and then multiplied by the area of the individual square (A). The resulting volume for each individual distance ($V_i = r_i A$) represents the volume of solid material above (if the volume is positive) or the volume of the void below (if the volume is negative) the plane

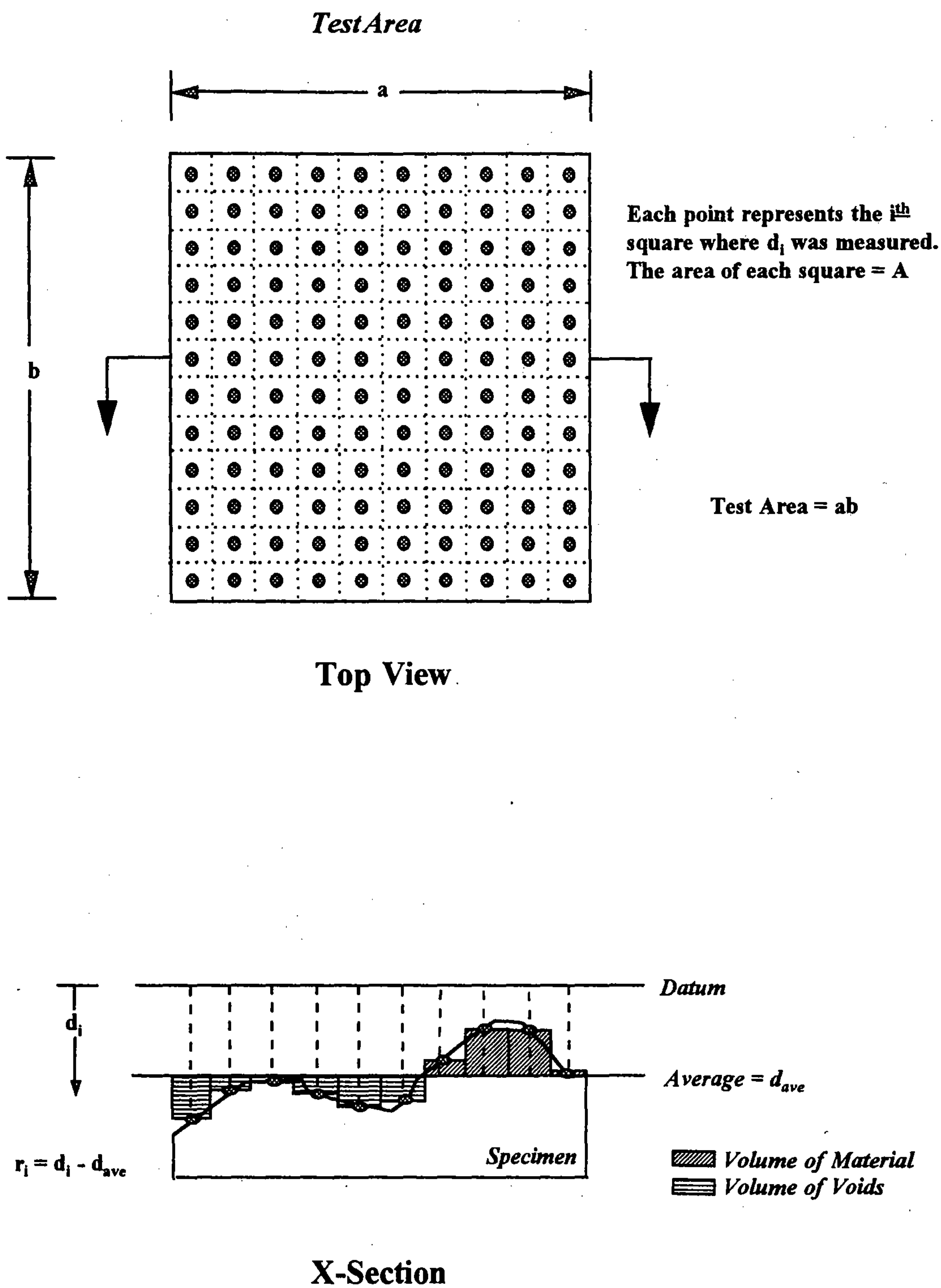


Figure 11. Pictorial representation of VSTR calculation.

established by the average distance (d_{ave}). A summation of the absolute values of the volumes (VST = $\sum$ abs[r_iA]) yields the total volume of solid material above the average distance plane plus the volume of voids below the plane. This summation represents the total volume of surface texture. Dividing the VST by the test area produces the VSTR. This normalization allows comparisons to be made between VST values when the size of the total area tested varies between specimens. Typically, a VSTR below 0.2700 cm^3/cm^2 indicates poor surface texture while a VSTR above 0.3000 cm^3/cm^2 indicates good surface texture. All surface texture measurements are accurate to four significant digits (e.g., 0.2700 cm^3/cm^2).

The VSTR can be related to the load transfer efficiency of a crack or an undoweled joint by multiplying the ratio by the effective pavement thickness. The effective pavement thickness refers to only the portion of the fractured slab face which contains crack texture. For instance, the effective pavement thickness is reduced at a joint because the texture provided by the propagation of the crack starts at the bottom of the saw cut. The effective slab thickness is also reduced when the top and/or bottom of the slab is spalled off. The ratio multiplied by the effective thickness is referred to as the VST and it represents the volume of surface texture per cm width of the cracked slab face.

A visual examination was also performed on the cores. A visual rating was given to both the gross and macro texture. The gross texture is the texture provided by the path in which the crack propagated along. The macro texture refers to texture provided by the coarse aggregate. The following subjective rating scale was used for rating the gross and macro texture: VG-very good; G-good; F-fair; P-poor; VP-very poor.

Petrographic Analysis—Petrographic examination was conducted on the core specimens in accordance with guidelines provided in ASTM C 856, "Standard Practice for Petrographic Examination of Hardened Concrete."

The coarse aggregate and mortar content of the specimens was determined by linear measurement, following a modified linear traverse procedure adapted from ASTM C 457, "Standard Test Method for Microscopical Determination of Parameters of the Air-Void System in Hardened Concrete."

Tests for indications of alkali-silica reactivity were conducted on the core specimens, using uranyl acetate procedure described in SHRP-C/FR-91-101, "Handbook for the Identification of Alkali-Silica Reactivity in Highway Structures."

Project Data Base

All of the information collected for the 16 pavement sections evaluated under this project is stored in EXCEL™ spreadsheet format. This type of format provides tremendous flexibility in manipulating and sorting the data, performing queries, and in analyzing the data.

Several different spreadsheets are currently available. One spreadsheet contains all of the data in one large file. This is useful in analyzing the data and reviewing performance trends.

Several other spreadsheets have been developed to present the data in report format. These are essentially the sheets presented in appendix A. The data are presented in the following categories:

- General and climatic data.
- Structural design data.
- Joint design data.
- Reinforcement design and construction data.
- Outer shoulder and drainage design data.
- Aggregate data.
- Aggregate gradation data.
- PCC mixture design data.
- PCC strength data.
- Traffic data.
- Deflection data.
- Laboratory testing results.
- Performance data.

Summary

This chapter has summarized the data collection and data reduction activities that were conducted under this project. Detailed descriptions of the various field testing activities are provided, including summaries of the condition surveys, drainage surveys, FWD testing, and coring operations. In addition, information is furnished on how those data were reduced for the preparation of the project summary tables and subsequent later analysis.

3. SUMMARY OF FIELD TESTING RESULTS

Introduction

In order to maximize the results of the field testing, much effort was devoted to selecting the projects to be included in this study. This chapter provides detailed descriptions of the nine projects that were selected for inclusion in this study. Five of the nine projects included both a recycled section and a control section. In addition, two recycled sections were evaluated on three projects due to differences in design or performance. For example, WI 1 (I-94 near Menomonie) featured recycled sections with and without dowel bars at the transverse joints. Similarly, MN 2 (I-90 near Beaver Creek) and WI 2 (I-90 near Beloit) both contained sections that were exhibiting different levels of performance, so two sections were evaluated in each of these projects.

This chapter begins with a review of the project identification and selection process, and briefly introduces the nine projects that were selected for inclusion in this study. Detailed summaries of the design, construction, performance and evaluation of each section follow.

Background

One of the principal objectives of this study is the determination of the causes of pavement distresses associated with the use of recycled concrete coarse aggregate in jointed concrete pavements. The specific problem of midpanel cracking with smooth crack face surfaces (and attendant load transfer problems) is a primary focus. However, other concrete pavement distresses have been identified as being either possibly caused or exacerbated by the use of recycled concrete aggregate:

- Reinforcing mesh failures.
- Foundation pumping.
- Joint and crack spalling.
- D-cracking (including recurrent forms).
- Alkali-silica reactivity (including recurrent forms).
- Joint and crack faulting.
- Distresses related to unusual thermal and moisture expansion characteristics of recycled concrete aggregate (e.g., blowups and other forms of joint or crack deterioration).

A review of previous research has indicated that the areas of greatest general concern in practice involve the failure of mesh reinforcing, faulting of transverse cracks and undoweled transverse joints, and the potential for recurrent D-cracking.[1] It has been hypothesized that these distresses may be attributed, at least in part, to the increased volumetric instability that is generally associated with RCA concrete (i.e., the increased expansion and contraction of the RCA concrete in response to changes in temperature and moisture). Published reports have cited no direct evidence of a link between the use of recycled concrete aggregates and foundation pumping or crack spalling, and only Wyoming has documented experience in recycling ASR-damaged pavements.

Based upon the literature review, the resulting synthesis, and the structural properties and performance characteristics of the candidate projects, the following distresses were selected for study under this project:[1]

- Midpanel cracking.
- Reinforcing mesh failures.
- D-cracking.
- Faulting of cracks and joints.
- ASR damage.

The effects of recycled aggregate thermal and moisture expansion and contraction properties on these distresses were also studied.

Project Identification and Selection

<u>Identification of Potential Projects</u>

A list of candidate RCA concrete highway pavement sections was developed based upon personal contacts with SHA personnel, a project advisory panel meeting, and a literature review. The final list of candidate sections included nearly 100 pavement projects located throughout the United States. The majority of these projects were JPCP projects, and exhibited the following characteristics:

- Still in service without major rehabilitation.
- Constructed more than 6 years ago.
- Information readily available for structural design, concrete mix design, pavement construction, and traffic data.

<u>Selection Criteria</u>

The project team was charged with selecting nine recycled aggregate concrete pavement projects for the field studies. The selection of these projects involved the consideration of the following factors:

- Pavement age (selected sections to be at least 8 years old).
- Pavement type (a balance between JPCP and JRCP was desired).
- Joint spacing (a range of joint spacings was desired).
- Accumulated traffic loadings and current traffic levels (a range was desired).
- Climate (a range of climatic conditions was desired).
- Availability of detailed information on the projects.

- Availability of past performance data.
- Anticipated level of cooperation from the responsible highway agency.
- Relative condition of the existing pavement.

The last factor, the relative condition of the existing pavement, became a key part of the selection process. The selected pavement sections were placed into one of the following three categories:

1. "Good" performance - JRCP with nonworking transverse cracks and little or no distress; or JPCP without transverse cracks and exhibiting little or no distress.
2. Structural problems - JRCP with deteriorated transverse cracks, or JPCP exhibiting any transverse cracking.
3. Other distresses - JRCP and JPCP exhibiting other distresses possibly related to the use of recycled concrete aggregate.

The selection of three suitable pavement sections for each of these three categories was desired. In addition, because this research project focuses on the problem of midpanel cracking in recycled aggregate concrete pavements, the project selection process concentrated on suitable JPCP and JRCP designs, although CRCP projects constructed with recycled concrete aggregate were also considered.

Selection Process

With these criteria and guidelines in mind, the project team reviewed the candidate sites and developed a list of preliminary site selections for the field study. This list was presented to the project advisory panel at a February 1994 panel meeting. The list was discussed and panel recommendations for revisions were solicited. The panel also provided clarifications, corrections, and additional data that would be useful in finalizing the site selection list. One additional project selection criterion that was strongly recommended by the advisory panel was that the selected projects should include adjacent, nonrecycled control sections wherever possible.

Following the panel meeting, numerous followup phone calls and several informal field site visits were made to obtain additional information on the most promising candidate sections. The purpose of this effort was to:

- Verify and ensure that the identified sections were still in service and had received no major rehabilitation.
- Ascertain the general condition of the pavement (including types and amount of distress).
- Confirm fundamental design data.
- Determine the availability of mix design, construction, and curing data.

Proposed Field Test Sites

Based on the acquired information and the suggestions of the advisory panel, a list of field study sites was finalized. Table 3 presents an overview of the sites that were proposed for the project field studies. This list was later approved by the FHWA COTR. Since few of the candidate sites were located in climates other than wet-freeze or dry-freeze, the selected project sites were also located only in these regions.

The category 1 (good performance) sites offered an excellent range of design, traffic, and environmental variables in three projects. In addition, all of the projects in this category included control sections that could be surveyed and sampled for direct comparisons of performance and materials effects. The MN 1 site (I-94 near Brandon) is only 6 years old, but was included because its level of performance to date is much better (practically no cracking observed despite the 8.2-m [27-ft] joint spacing) than that of comparable pavements of approximately the same age that were constructed with virgin aggregates.

The three sites included in category 2 (structural problems) are all located in the upper Midwest and represent a more narrow, but significant, range of design variables. MN 4 (U.S. 52 near Zumbrota) offers a control section, and WI 1 (I-94 near Menomonie) offers both doweled and undoweled sections that have exhibited very different amounts of joint faulting in spite of the use of relatively large coarse aggregate and a short joint spacing.

The three sites belonging to category 3 (other distresses) represent three different failure types. The distress associated with each site is as follows:

- MN 3 (U.S. 59 JPCP near Worthington) has significant faulting levels on the recycled concrete pavement and was the first major attempt to recycle an extensively D-cracked PCC pavement into a new PCC pavement surface.
- WI 2 (I-90 CRCP near Beloit) is showing signs of early failure (deteriorated cracks and punchouts), possibly due to poor foundation support.
- WY 1 (I-80 JPCP near Pine Bluffs) was constructed using recycled concrete that was showing severe damage from alkali-silica reactivity (ASR).

Although CRCP were originally excluded from the scope of the project, the WI 2 site was selected for inclusion in this project because it offered the potential to evaluate the effects of foundation support on RCA concrete pavement performance through performance comparisons with other nearby CRCP that were the subject of another FHWA-sponsored study.

A number of sites in Wyoming were also considered for inclusion in this study because they included RCA obtained from ASR-damaged pavements. The site selected (WY 1) was the oldest of these sections. The Wyoming DOT incorporated a number of mix design features that have been successful thus far in preventing recurrent ASR damage, including the use of a blend of RCA and virgin coarse aggregate, and the use

Table 3. Proposed projects for field testing.

Category	Location	Climatic Region	Age, years	Control Section	Pavement Type (% long. reinf.)	Shoulder Type	Joint Spacing, m	Dowel Diam., mm	Agg. Top Size, mm
1 (Good)	CT 1, I-84 near Waterbury	W-F	14	yes	230-mm JRCP (0.09 %)	AC	12	38 (I-beam)	51 / 38
	MN 1, I-94 near Brandon	W-F Transition	6	yes	280-mm JRCP (0.054 %)	AC	8.2	32	19
	KS 1, K-7 in Johnson County	W-F	9	yes	230-mm JPCP	AC	4.7	none	38 / 19
2 (Structural Problems)	MN 4, U.S. 52 near Zumbrota	W-F	10	yes	230-mm JRCP (0.065 %)	AC	8.2	25	25 / 38
	MN 2, I-90 near Beaver Creek	W-F Transition	10	no	230-mm JRCP (0.065 %)	AC	8.2	25	19
	WI 1, I-94 near Menomonie	W-F	10	no	280-mm JPCP	PCC	3.7-4.0-5.8-5.5	none / 35	38
3 (Other Distresses)	MN 3, U.S. 59 near Worthington	W-F Transition	14	no	200-mm JPCP	AC	4.0-4.9-4.3-5.8	none	19
	WI 2, I-90 near Beloit	W-F	8	no	250-mm CRCP (0.67 %)	PCC	n/a	n/a	38
	WY 1, I-80 near Pine Bluffs	D-F	9 / 10	yes	250-mm JPCP	PCC	4.3-4.9-4.0-3.7	none	25 / 38

of fly ash. A study recently completed by the Wyoming DOT indicated the surface cracking that has been observed on some of these projects is due to concrete placement and finishing problems, not recurrent ASR. Although the use of a blended aggregate makes it more difficult to determine the effects of ASR-damaged RCA on pavement performance, this project was selected to provide an indication of the effectiveness of aggregate blending in improving crack face texture and reducing associated distresses, as well as some insight into the recycling potential of ASR-damaged pavements.

Project Summaries

Detailed summaries of each of the nine selected projects are provided subsequently in alphabetically-ordered sections. These summaries include the following subsections:

- Project information (general).
- Design information (structural).
- Mix design (composition, proportions, and properties).
- Construction information.
- Climatic conditions.
- Concrete properties.
- Traffic loadings.
- Selection of distress survey section.
- Drainage survey.
- Pavement distress survey.
- FWD testing.
- Coring and core test results.
- Project summary.

This information has been used to develop the field study conclusions and recommendations presented in chapter 4 of this report. They also serve as the foundation for the laboratory study work plan that is presented under separate cover.

Connecticut 1, I-84 in Waterbury

In 1978, the Connecticut Department of Transportation (CTDOT) initiated a two-phase research project to explore the feasibility of concrete pavement recycling. The Phase I objectives were to determine the relative energy requirements for different construction methods, to make environmental assessments, and to optimize the design of recycled pavements.[8] The objectives of Phase II were to develop the technical expertise to remove and crush the salvaged concrete and to place the recycled concrete pavement, to evaluate the performance characteristics of a recycled concrete pavement, and to determine the cost-effectiveness of recycling.[9] The study concluded that crushed concrete could be substituted for conventional aggregate and that a workable mix could be produced. As a result, CTDOT proposed the construction of its first recycled concrete pavement in 1979.

Project Information

The site selected for recycling was a section of the westbound lanes of I-84 in Waterbury. The existing pavement was a 180-mm (7-in) JRCP that was originally constructed in 1956. It was 7.3-m (24-ft) wide and was reinforced with wire mesh. Transverse joints were spaced at 12-m (40-ft) intervals and were provided with I-beam load transfer devices. The existing pavement had never received an overlay and did not exhibit major distress.

The recycled concrete pavement research project extends from South Main Street to the Hamilton Avenue overpass. A 180-m (600-ft) control section and a 305-m (1,000-ft) recycled section were established within the project. The control section began at milepoint 33.94 and extended westward to milepoint 33.82. The recycled section extended from milepoint 33.71 to milepoint 33.52. The control and recycled sections were separated by about 150 m (500 ft) of concrete pavement extending across the Washington Street overpass.

Design Information

Both the control and the recycled sections are 230-mm (9-in) JRCP with 0.09 percent longitudinal reinforcing steel. The reinforcement design employs a smooth welded wire fabric (WWF) consisting of No. 4 gauge wires spaced at 310-mm (12-in) intervals in the transverse direction and No. 2 gauge wires spaced at 150-mm (6-in) intervals in the longitudinal direction. The transverse joints are spaced at 12-m (40-ft) intervals and contain 38-mm (1.5-in) dowel bars. The recycled section contains a 250-mm (10-in) aggregate base. The control section contains a 460-mm (18-in) aggregate base for the first 122 m (400 ft) and a 250-mm (10-in) aggregate base for the remaining 61 m (200 ft). Both sections contain AC shoulders and no provisions for drainage.

Mix Design

Preliminary mix designs were developed using concrete samples taken from I-91 in North Haven.[10] The results of these preliminary investigations were used as guidelines for developing mix designs for the recycled concrete section. Tables 4 and 5 show the gradation of the 9.5- and 51-mm (3/8- and 2.0-in) RCA of four aggregate samples taken from I-84, as well as the average of the four samples.

The coarse aggregate used in the recycled section consisted of 20 percent of the 9.5-mm (3/8-in) crushed material and 80 percent of the 51-mm (2.0-in) crushed material; the coarse aggregate used in the control section was more finely graded, consisting of 50 percent crushed stone passing the 38-mm (1.5-in) sieve and 50 percent passing the 13-mm (1/2-in) sieve. A natural sand fine aggregate was used in both the recycled and control sections. The gradations of the coarse and fine aggregate blends are provided in table 6 for both the recycled and control sections. This table indicates that the recycled concrete coarse aggregate is slightly finer than the crushed stone used in the control section.

Table 4. Gradation of 9.5-mm (3/8-in) recycled aggregate.[11]

Sieve	Percent Passing				
	Sample A	Sample B	Sample C	Sample D	Average
13 mm (1/2 in)	100	100	100	100	100
9.5 mm (3/8 in)	99	100	98	99	99
4.75 mm (No. 4)	44	55	23	43	41
2.36 mm (No. 8)	24	33	10	26	23
0.150 mm (No. 100)	6.2	6.7	4.2	5.8	5.7
0.075 mm (No. 200)	4.7	5.7	2.9	4.8	4.5

Table 5. Gradation of 51-mm (2.0-in) recycled aggregate.[11]

Sieve	Percent Passing				
	Sample A	Sample B	Sample C	Sample D	Average
51 mm (2.0 in)	100	100	100	100	100
38 mm (1.5 in)	100	97	100	97	98
32 mm (1.25 in)	98	90	100	92	95
25 mm (1.0 in)	93	70	96	74	83
19 mm (3/4 in)	71	38	80	41	58
12.7 mm (1/2 in)	24	8	41	10	21
9.53 mm (3/8 in)	7	3	16	3	7
4.75 mm (No. 4)	3	2	4	2	3
2.36 mm (No. 8)	2	1	3	1	2

Table 6. Aggregate gradations (percent passing each sieve) of recycled and control sections.[11]

Sieve	Recycled		Control	
	Coarse	Fine	Coarse	Fine
51 mm (2.0 in)	100		100	
38 mm (1.5 in)	98		100	
25 mm (1.0 in)	86		80	
19 mm (3/4 in)	66		55	
12.7 mm (1/2 in)	37		48	
9.53 mm (3/8 in)	25		16	
4.75 mm (No. 4)		100		100
2.36 mm (No. 8)		93		93
1.18 mm (No. 16)		75		75
0.600 mm (No. 30)		51		51
0.300 mm (No. 50)		11		11
0.150 mm (No. 100)		4		4
0.075 mm (No. 200)		0.8		0.8

The recycled concrete and natural coarse aggregates were tested in the laboratory to determine some of their properties. The results of these tests are shown in table 7. The recycled aggregate has a lower specific gravity, higher absorption, and lower wearing resistance than the virgin aggregate, trends that are typical of results from other studies of RCA.

The mix designs for the recycled and control sections are shown in table 8. The recycled concrete section (CT 1-1) contains about 6 percent more coarse aggregate by weight, but about 18 percent more by absolute volume. This difference is offset by 25 percent less fine aggregate and about 12 percent less water in CT 1-1. Cement contents for the two sections are identical, so CT 1-1 also has a lower water-to-cement ratio. The water content of the control mix is significantly higher than that of the recycled mix. This is probably done in response to the increased fine aggregate content of the control mixture, which would drive up the water demand. When these factors are considered, one would expect the recycled mixture to be much more difficult to work with; however, records indicate that CT 1-1 had a slump of 76 mm (3 in), compared with a slump of 64 mm (2.5 in) for the control mixture (CT 1-2). This may be explained, in part by the higher air content of the recycled mixture (5.0 percent vs. 4.0 percent), and the slightly finer gradation of the recycled coarse aggregate.

Table 7. Properties of recycled and virgin aggregate.[11]

Property	Recycled	Virgin
Class A wear, %	29.3	14.0
Bulk specific gravity	2.53	2.81
Sulfate soundness, % loss	2.58	2.98
Absorption, %	3.53	1.64
Weight, kg/m^3 (lb/ft^3)	1690 (105.5)	1680 (104.8)
% passing 0.075-mm (No. 200) sieve	1.8	

Table 8. Mix design for CT 1.[11]

Material	Recycled	Control
Coarse Aggregate	1302 kg/m^3	1225 kg/m^3
Fine Aggregate	476 kg/m^3	641 kg/m^3
Cement	362 kg/m^3	362 kg/m^3
Fly Ash	0 kg/m^3	0 kg/m^3
Water	144 kg/m^3	163 kg/m^3
w/c Ratio	0.40	0.45

Construction Information

Approximately 610 m (2,000 ft) of the existing 7.3-m (24-ft) pavement was removed to provide enough aggregate for the three-lane, 305-m (1,000-ft) recycled section. The crushing operation used three crushers that handled different size materials. Most of the steel was removed before the crushing operation, although some steel was embedded in the concrete and had to be removed after initial crushing. The crushed material was then stockpiled until construction of the recycled section.

The concrete for the recycled and control sections was placed in two lifts. The first lift was placed, the wire mesh was spread across the initial lift, and then the second lift was placed. The concrete was consolidated using paver-mounted vibrators. The slump on both the recycled and control section ranged between 64 and 76 mm (2.5 and 3.0 in). The temperature at the time of placement ranged from 21 to 29 °C (70 to 85 °F). After placement, the concrete was textured transversely using a broom, and a liquid membrane curing compound was applied.

Concrete Properties

A series of concrete beam specimens were cast on each day of paving. These beams were tested for flexural strength in the laboratory after 7 days of curing; the results of these tests are illustrated in figure 12.[11] The flexural strengths of the recycled and control concrete mixtures appear to be comparable, although the recycled concrete mixture seems slightly stronger (3.4 MPa vs. 3.0 MPa [490 lbf/in^2 vs. 440 lbf/in^2]).

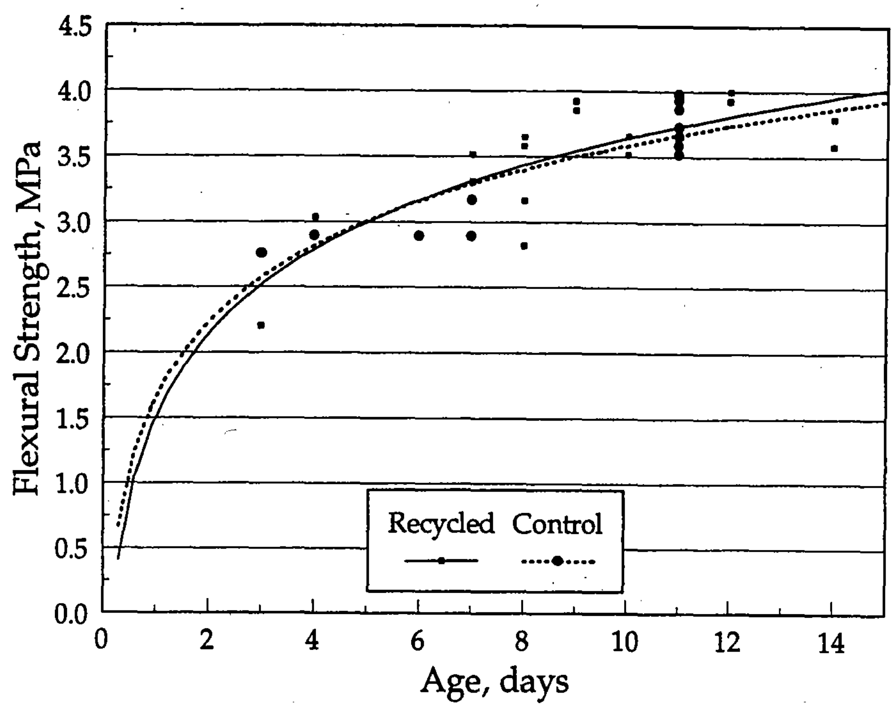


Figure 12. Flexural strength gain for CT 1.

Climatic Conditions

The CT 1 test sections are located in the wet-freeze environmental region. The area experiences about 138 days of precipitation per year and average annual precipitation of 1190 mm (47 in), resulting in a Thornthwaite moisture index of 70. The freezing index is 140 °C-days (250 °F-days), and the sections are exposed to about 90 freeze-thaw cycles per year. The minimum and maximum average monthly temperatures are -2 and 22 °C (28 and 71 °F).

Traffic Loadings

When this highway was opened to traffic in 1980, the two-way ADT was about 56,000 vehicles per day. As of 1994, the two-way ADT has increased to 75,000 vehicles per day. Although the percentage of truck traffic varies from year to year, it averages around 10 percent per year. Through 1994, this highway has sustained about 15.9 million 80-kN (18-kip) equivalent single-axle load (ESAL) applications.

Selection of Distress Survey Section

As previously mentioned, the length of the recycled section was 305 m (1,000 ft), and the length of the control section was 180 m (600 ft). Thus, the survey sections encompassed the entire length of each of these sections. The control section was constructed on about 3.0 m (10 ft) of fill material. The recycled section was constructed on a cross slope, with the initial 270 m (900 ft) nearly at grade, and the remaining 30 m (100 ft) in a slight cut section.

Drainage Survey

Neither section contains any elements for controlling subsurface drainage, such as longitudinal edge drains or a permeable base layer. However, pumping of moisture or fines was not observed on either section. The recycled section is on a horizontal curve with transverse slopes ranging from 1.0 percent at the east end to 6.2 percent at the west end. The control section has transverse slopes ranging from 3.6 to 6.2 percent.

Pavement Distress Survey

The pavement condition survey was conducted over the entire length of the recycled and control sections. A complete summary of the results of the survey are provided in appendix A. A summary of the average results for the key variables are shown in table 9. In general, the results indicate that the sections are performing similarly, as most of the distress measurements are about the same.

Transverse Joint Faulting

Neither section has developed significant joint faulting (≤ 0.5 mm [0.02 in] average). This can be attributed to the fact that both sections contain 38-mm (1.5-in) dowel bars and do not rely on aggregate interlock as the major mechanism of joint load transfer. For this reason, the development of faulting on these pavement sections appears to be independent of aggregate type (i.e., recycled concrete vs. natural).

Table 9. Summary of performance data (average values) for CT 1.

Performance Measurement	Recycled	Control
Corner Faulting, mm (Manual)	0.5	0.5
Wheelpath Faulting, mm (Manual)	0.5	0.3
Wheelpath Faulting, mm (Digital)	0.3	0.3
Deteriorated Transverse Cracks/km	26.8	32.8
Cracks/km	63.5	114.8
Longitudinal Cracking, m/km	0	0
Transverse Joint Spalling, % Joints	92	37
Joint Width, mm	14	13
Crack Width, mm	14	15
PSR	3.4	3.5

Transverse Cracking

At the time of survey, the driving lane of the control section contained nearly twice as many transverse cracks per km than the recycled section (114.8 vs. 63.5); however, the two sections contained comparable numbers of deteriorated cracks (32.8 vs. 26.8 per km). Some of the deteriorated crack widths in either section were very large, exceeding 25 mm (1 in) on some high-severity cracks. The number of deteriorated transverse cracks includes all transverse cracks of medium and high severity. Low-severity cracking is expected on JRCP (L/ℓ ratio is 16.6 and 15.2 for the control and recycled sections, respectively) and is not included in the computation of deteriorated cracks for this project. It is believed that the cracks were allowed to open by the proximity of expansion joints adjacent to nearby bridge approach slabs and by corroded or "frozen" mechanical load transfer systems.

The Connecticut DOT has monitored the performance of this pavement since its construction. The project team was provided with detailed maps that documented the progression of cracking within both paving sections. The following observations are based on an examination of those cracking maps and consideration of project construction records:

- Although both sections cracked at similar rates during the first 2 years of service, cracking developed at different rates after that. Furthermore, cracking developed at different rates in different lanes within either section.
- In the RCA concrete section, the inner and middle lanes developed most of their cracking during the first 6 years of service. Most of the cracking in the outer lane developed *after* 6 years of service, when it rapidly exceeded the amounts of cracking observed in the other two lanes.
- Cracking in the control section developed much differently, with the middle lane developing most cracks during the first 6 years of service, the inner lane developing cracks most rapidly *after* 6 years of service, and the outer lane cracks developing continuously throughout the service life. As with the RCA section, the control section outer lane eventually developed more cracks than the middle and inner lanes.
- The difference in cracking during the first 6 years of service may be attributed to the order in which traffic loads were imposed on the inner, middle and outer lanes shortly after construction. Construction records indicate that the outer lane for the recycled section was paved last and remained closed for a longer time period after construction. Therefore, it is believed that the inner and middle lanes were exposed to traffic loads (including loads associated with the construction of the outer lane) at a relatively early age, resulting in more rapid accumulation of fatigue damage than in the outer lane. As the traffic flow became less interrupted due to completion of nearby construction, it is believed that the outer lane gradually was exposed to heavier truck traffic, which may explain greater crack development in the outer lane after the first 6 years.

There is no readily apparent explanation for the differences in the development and deterioration of cracks on this project. All physical and structural aspects of the two sections are comparable, except that the control section was constructed on fill material which apparently produced a relatively low foundation stiffness (backcalculated k=68.4 kPa/mm [252 lbf/in^2/in]) compared with the recycled section (backcalculated k=105 kPa/mm [387 lbf/in^2/in]), which was largely constructed at grade. It is possible that some of the differences between sections and lanes can be attributed to this difference in foundation stiffness, differences in construction traffic, or the sequence of construction of the lanes. However, this hypothesis can not be verified at this time.

The failure of the steel reinforcement at the transverse cracks in these sections is not unexpected, due to the relatively low steel content (0.09 percent). This has allowed the cracks to open; many are now completely filled with incompressibles and other debris.

Longitudinal Cracking

Longitudinal cracking was nonexistent on both the recycled and control sections.

Transverse Joint Spalling

Transverse joint spalling was abundant on the recycled section, occurring at 92 percent of the transverse joints, which included 40 percent medium-severity spalls and 36 percent high-severity spalls. Spalling on the control section was limited to 37 percent of the transverse joints and included 25 percent medium-severity spalls and no high-severity spalls. Thus, spalling of the transverse joints is more common and more severe on the recycled section. This phenomenon may be a result of the slightly greater thermal expansive properties of the recycled mix (which would produce higher compressive stresses at the joints in the hot weather) and the lower abrasion resistance of the recycled aggregate particles. It is difficult to attribute the spalling of the recycled concrete pavement to differences in concrete strength or stiffness because laboratory tests of cores retrieved from the project indicate that the recycled material is stronger than the control concrete (as discussed below).

Present Serviceability Rating (PSR)

The average PSR of the driving lanes of the recycled and control sections are approximately 3.4 and 3.5, respectively. The difference between these values is probably insignificant, and they can be considered to be providing the same level of service at this time.

<u>FWD Testing</u>

Pavement deflection testing was performed in 1994 using a Dynatest model 8081 FWD. The testing pattern typically included 5 slab centers, 10 transverse joints (testing on both the approach and leave sides), 10 transverse cracks (testing on both the approach and leave sides), and 10 edges. Only 7 transverse cracks were present in the driving lane of the recycled concrete section; these were all tested, since 10 cracks were not available. FWD testing was used to determine PCC elastic modulus, subgrade modulus of support, load transfer efficiencies across joints and cracks, and loss of support. A summary of the average values obtained from the deflection data testing is provided in table 10.

PCC Elastic Modulus

The elastic modulus (E) of the concrete slab was backcalculated using the center-of-slab deflection measurements. Figure 13 shows a profile of the elastic modulus for the recycled section obtained using four mass drops at each of five different locations. The average elastic modulus is 37.0 GPa (5,360,000 lbf/in^2), although the values range from 28 to 50 GPa (4,100,000 to 7,300,000 lbf/in^2). At each location, the elastic modulus values obtained from each of the four drops exhibit little variability, although some variation was observed between the different locations.

Table 10. Deflection testing results for CT 1.

Property	Recycled	Control
Elastic Modulus, GPa	37.0	44.9
k-value, kPa/mm	105.1	68.4
Joint Load Transfer, %	90	86
Crack Load Transfer, %	76	84
Average Midslab Deflection, μm	82	89
Average Edge Deflection, μm	148	114
Corners With Voids, %	50	0
Maximum Air Temperature During Testing, °C	20	23

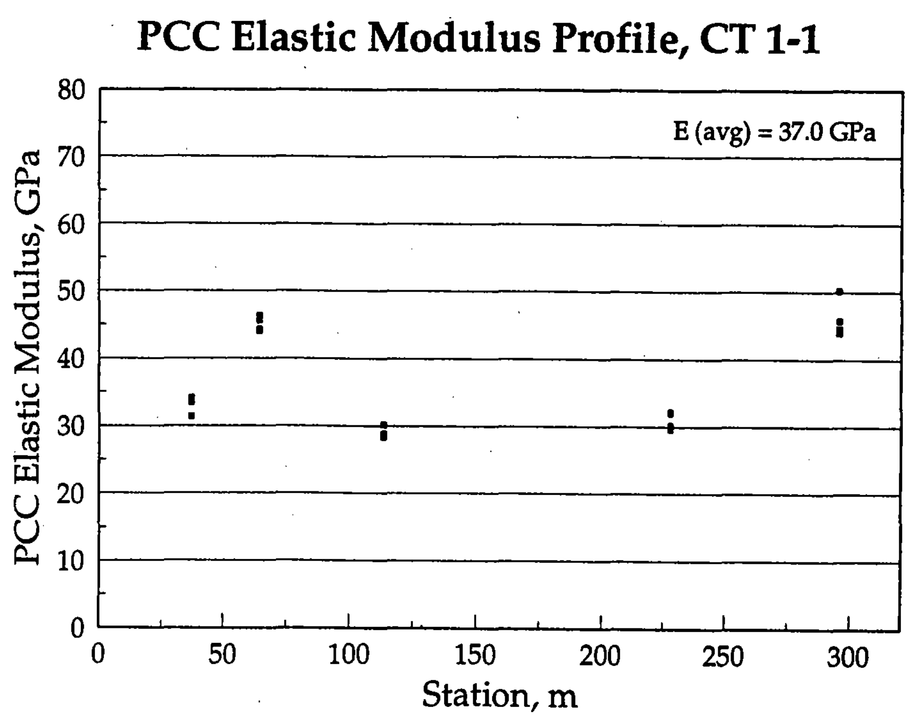


Figure 13. PCC elastic modulus profile for CT 1-1 (recycled section).

Figure 14 shows a similar plot for the control section. The average elastic modulus of the concrete slab was backcalculated as 44.9 GPa (6,500,000 lbf/in^2), with values ranging from 32 to 52 GPa (4,600,000 to 7,600,000 lbf/in^2). Again, little variability was observed between the four drops at any particular location; values at about the middle of the test section are the lowest and increase toward the east and west ends of the test section

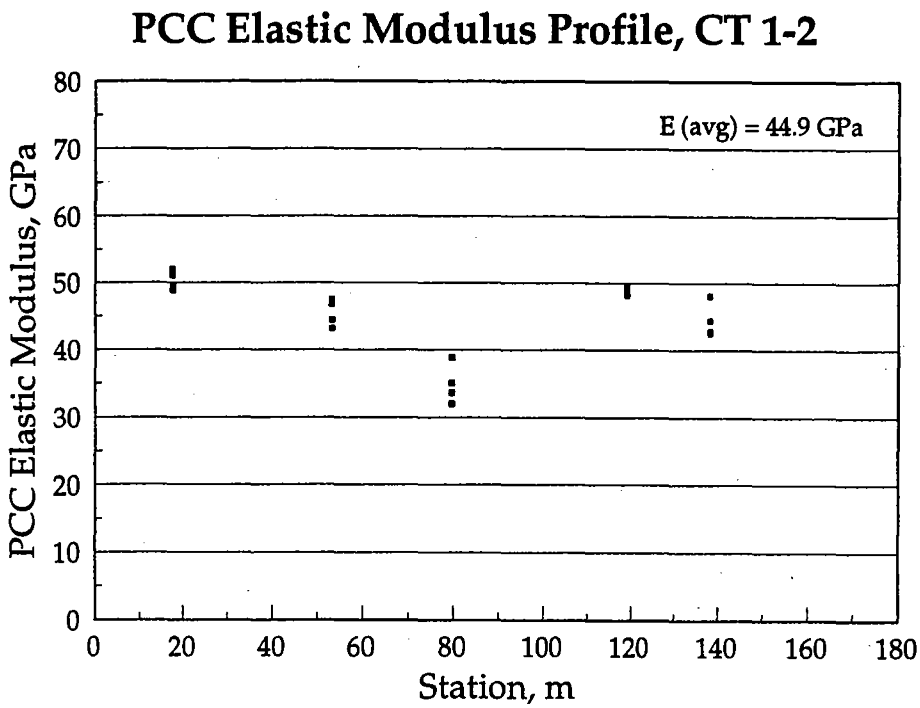


Figure 14. PCC elastic modulus profile for CT 1-2 (control section).

On average, the backcalculated elastic modulus of the recycled section is 18 percent lower than that of the control section, which agrees with results of other research studies comparing the properties of conventional and recycled concrete, which have found that the elastic modulus of a recycled mix is typically 20 to 40 percent lower than that of conventional concrete at the same water-cement ratio. (See references 12 through 15.) However, neither the water-cement ratios or the coarse aggregate contents of these two mixes are the same, as discussed previously. The lower water-to-cement ratio and higher coarse aggregate content of the recycled concrete mixture might be expected to compensate (at least partially) for the lower modulus that often accompanies the use of recycled aggregate.

Cores retrieved from these sections during the field survey were subjected to dynamic tests of elastic modulus in the laboratory. The results of these tests suggest that the two sections have equivalent concrete modulus values (31.7 GPa [4,600,000 lbf/in^2] average for the recycled section and 32.8 GPa [4,760,000 lbf/in^2] average for the control section). These values seem to be more consistent with the mix designs and field conditions present on these pavement sections, reflecting some compensation for the use of recycled aggregate with higher coarse aggregate content and lower water-to-cement ratio.

Modulus of Subgrade Reaction (k-value)

A profile plot of the effective k-values for the recycled section are illustrated in figure 15. The average k-value is 105 kPa/mm (387 lbf/in^2/in), with values ranging from 61 to 150 kPa/mm (324 to 554 lbf/in^2/in). A similar plot for the control section is shown in figure 16, where the k-values range from 52 to 90 kPa/mm (192 to 332 lbf/in^2/in) and the average of all tests is 68 kPa/mm (252 lbf/in^2/in). The k-values are subject to changes in moisture and temperature conditions and can fluctuate significantly throughout the year. Thus, the values presented here are representative only of the time and conditions during which the tests were performed (mid-October, 1994).

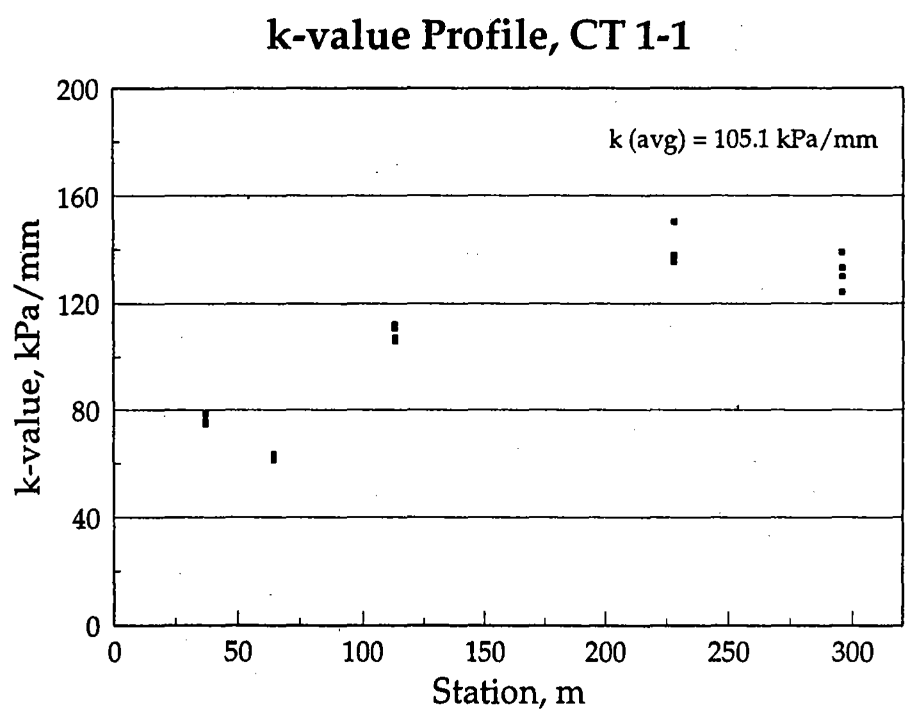


Figure 15. K-value profile for CT 1-1 (recycled section).

The k-values of the control section are generally lower and show less variability than those of the recycled section. Most likely, the differences are a result of the levels of cut and fill on the sections. The control section was constructed on about 3.0 m (10 ft) of fill material. The recycled section, on the other hand, was constructed on a cross slope and contained a transition from cut to fill. It is possible that these apparent differences in foundation stiffness contributed to some of the performance differences that were observed between the two sections.

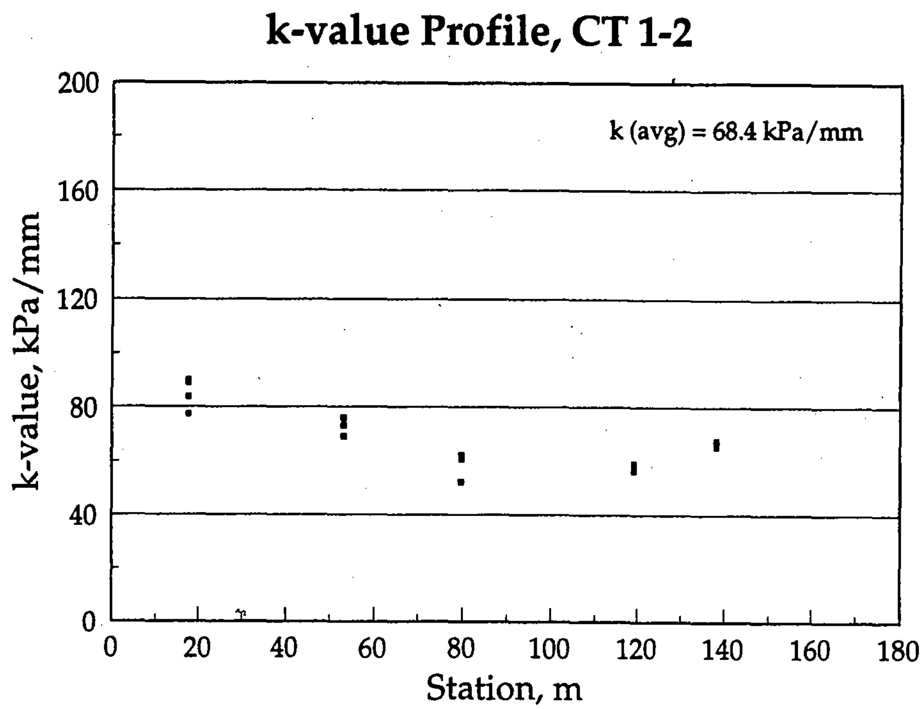


Figure 16. K-value profile for CT 1-2 (control section).

Joint Load Transfer

The load transfer efficiencies at both the approach and leave joints of the recycled section are shown in figure 17. The load transfer efficiencies represent the ratio of the deflection on the loaded side of the joint to the deflection on the unloaded side of the joint. The deflection load transfer efficiencies were greater than 80 percent at each test location, and the values measured on the approach and leave sides of each joint were generally comparable.

Figure 18 illustrates the joint load transverse efficiencies for the control section. The plot resembles the one for the recycled section, although some values do fall slightly below 80 percent. Since both pavement sections contain mechanical load transfer devices (38-mm [1.5-in] I-beam dowels), the transverse joint load transfer efficiencies do not rely on aggregate interlock to provide load transfer and are, therefore, essentially independent of the type of coarse aggregate used in the concrete.

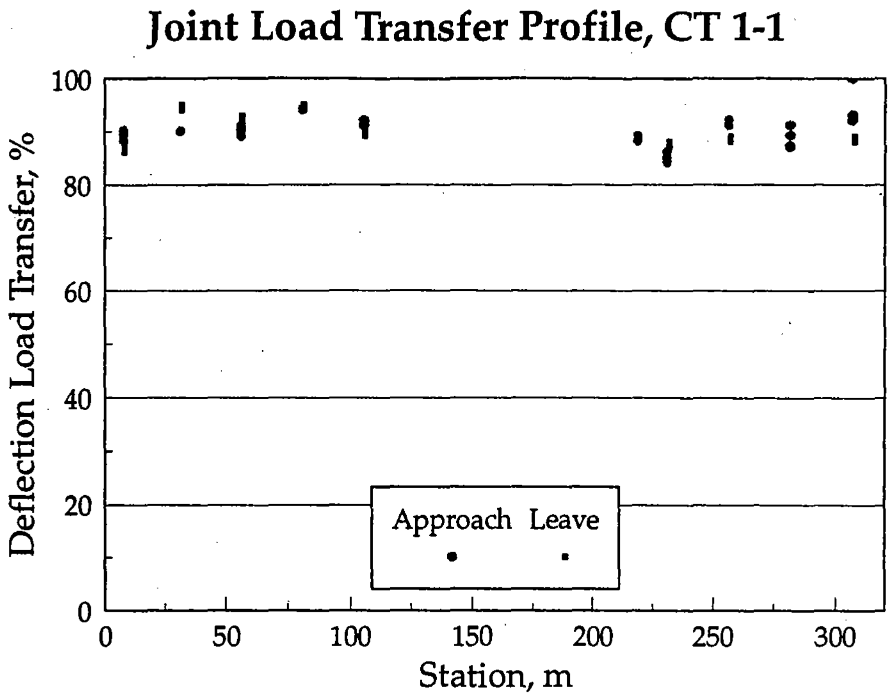


Figure 17. Joint load transfer profile for CT 1-1 (recycled section).

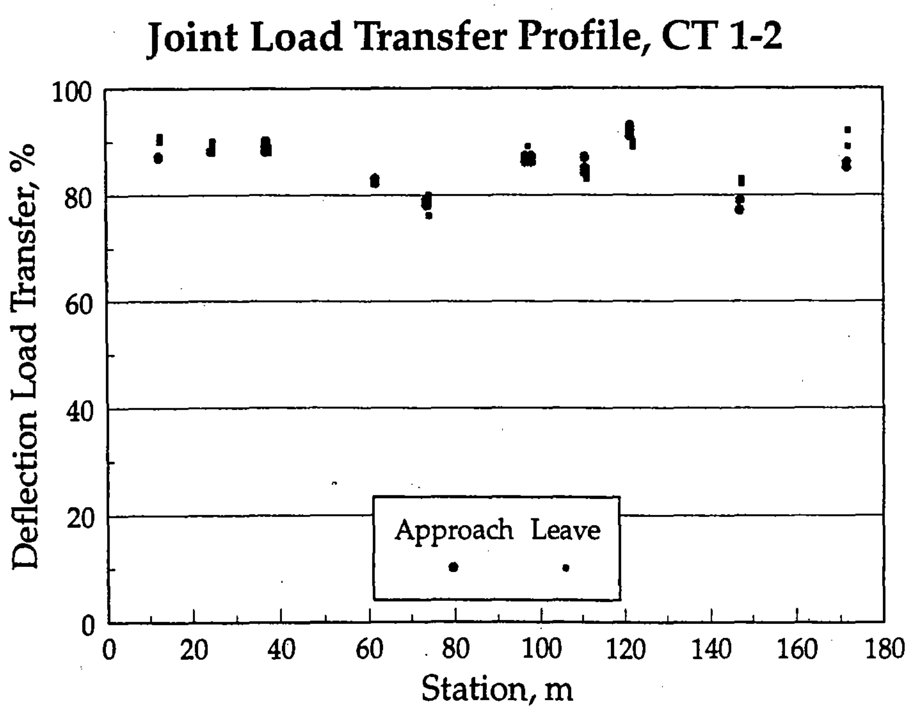


Figure 18. Joint load transfer profile for CT 1-2 (control section).

Crack Load Transfer

Load transfer at transverse cracks is accomplished almost entirely through the interlock of the aggregate particles across the concrete fracture plane. As a result, the load transfer efficiency at cracks is often lower than that of joints containing dowel bars. Furthermore, it is often believed that recycled aggregate concrete is less resistant to abrasion than conventional concrete due to a reduction of natural aggregate content in typical recycled aggregate concrete. This property does not necessarily result in more cracking but may result in more rapid deterioration of the crack. These beliefs are validated by the performance of the Connecticut test sections.

The load transfer efficiencies at the transverse cracks of the recycled section are illustrated in figure 19. The average crack load transfer efficiency is 76 percent (compared to 90 percent at the transverse joints), although values range from 29 to 100 percent, reflecting the range of crack widths present in the test section. The same type of plot for the control section cracks is shown in figure 20. The average crack load transfer efficiency in the control section is 84 percent (compared with 86 percent at the joints), with values ranging from 65 to 94 percent. Thus, the crack load transfer efficiencies for the control section are generally higher and less variable than those measured in the recycled section. They are also generally lower than the load transfer efficiencies measured at the doweled joints. Both observations support the findings of previous studies.

In figures 19 and 20, the letter above or below the group of data points indicates the crack severity. The correlation between the severity of the crack and load transfer effectiveness is clear, as the high-severity cracks have the lowest load transfer efficiencies and the medium- and low-severity cracks have higher load transfer efficiencies. The relationship appears most pronounced in the recycled concrete section, but is true for the control sections as well.

It can also be seen that the average load transfer efficiencies for any given crack severity level are higher for the control section than for the recycled concrete section. For example, the high-severity cracks in the control section have load transfer efficiencies around 70 to 80 percent, compared to 50 percent in the recycled section. This is presumably due to the increased quantity of large, abrasion-resistant natural coarse aggregate particles that are present in the control section.

Loss of Support

The detection of voids was performed using the corner deflections on the leave side of transverse joints and cracks. Figures 21 and 22 show the loss of support profile for the recycled and control sections, respectively. The joints and cracks at the east end of the recycled section demonstrate strong evidence of loss of support, while those at the west end do not appear to have any voids. The test results do not suggest loss of support on the control section. The cracks on the recycled section have lower load transfer efficiencies, and thus, more vertical movement, which can result in pumping

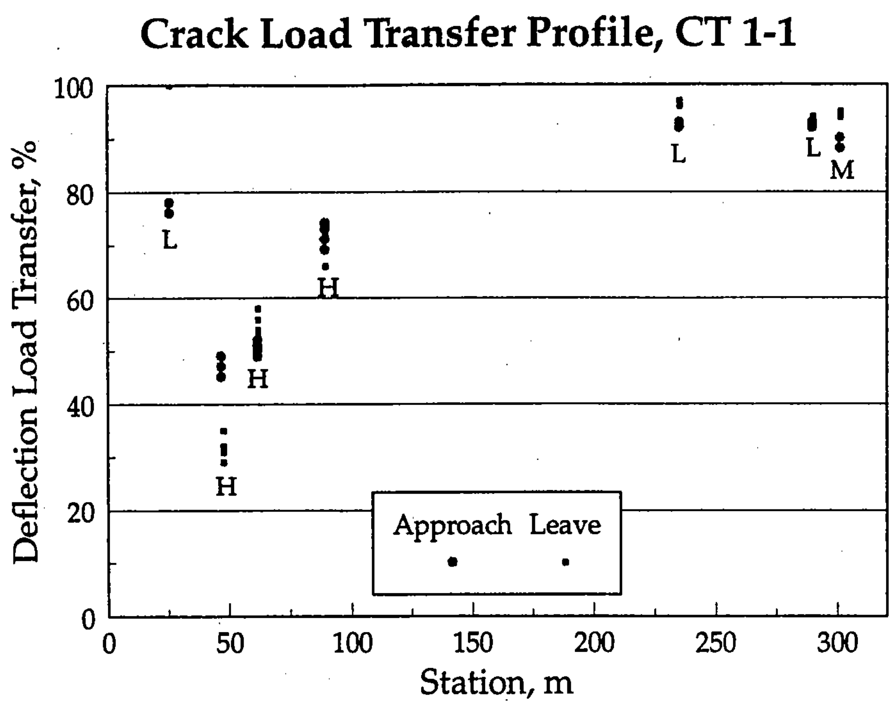


Figure 19. Crack load transfer profile for CT 1-1 (recycled section).

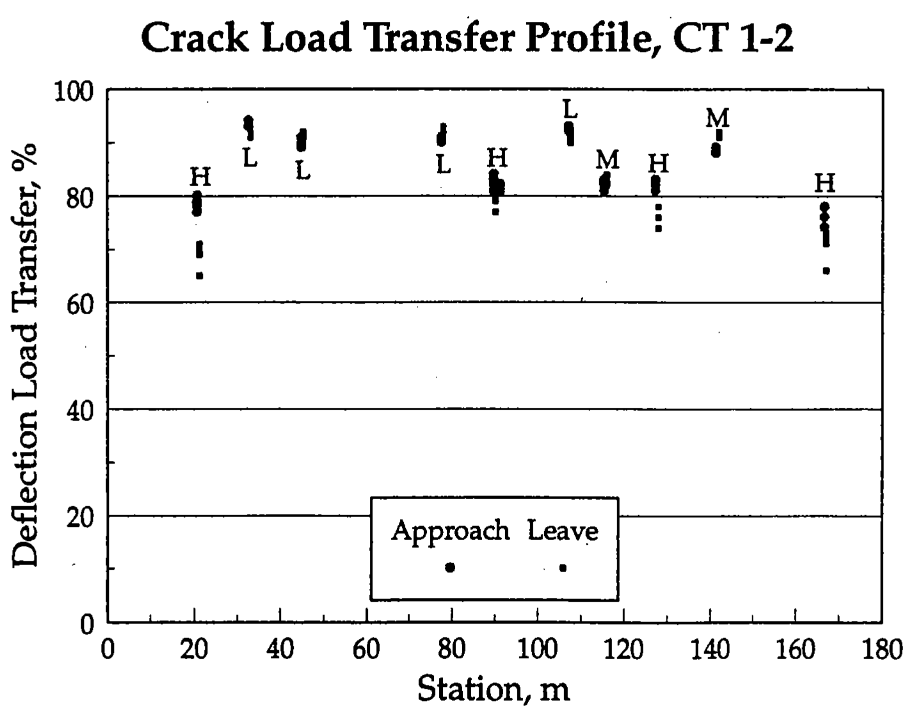


Figure 20. Crack load transfer profile for CT 1-2 (control section).

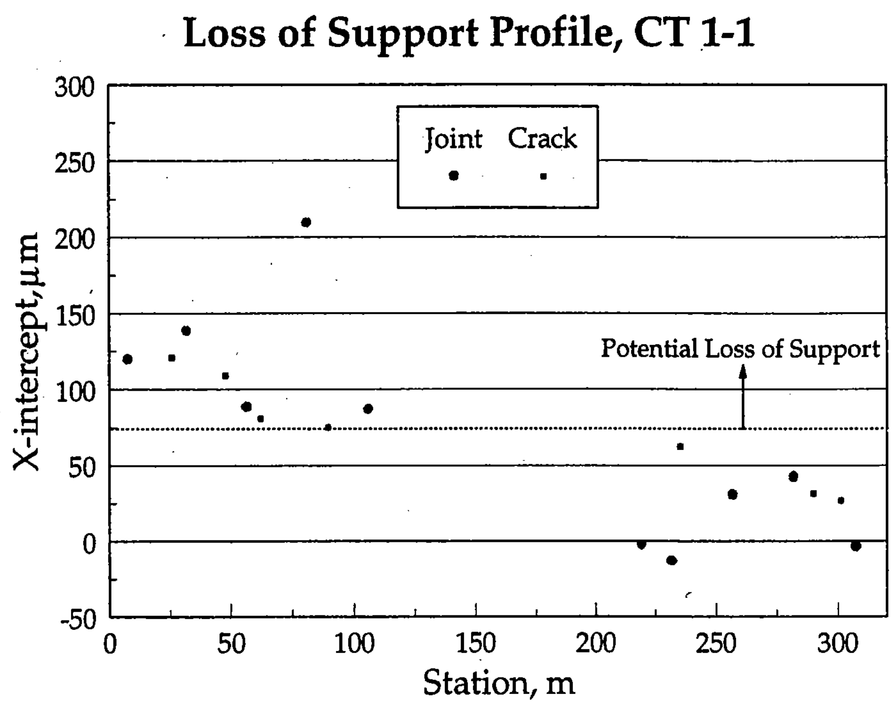


Figure 21. Loss of support profile for CT 1-1 (recycled section).

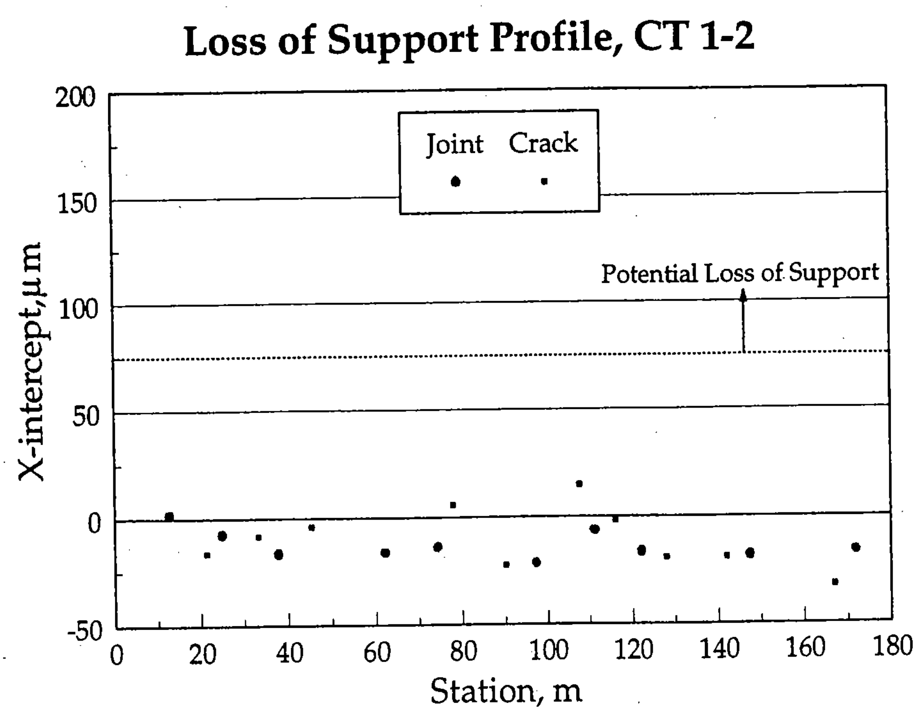


Figure 22. Loss of support profile for CT 1-2 (control section).

and loss of support. The apparent loss of support at the transverse joints, which developed in spite of apparently high load transfer efficiencies at these locations, may be due to differential movement across the lane-shoulder joint that has resulted in erosion under the driving lane transverse joints.

Coring

Eleven cores were taken from each of the two pavement sections: five from midpanel, three from transverse joints, and three from transverse cracks. All cores were 150 mm (6 in) in diameter and extended through the thickness of the concrete slab. Cores were not taken through the aggregate base course. The average thickness of the recycled and control section concrete cores were 226 and 231 mm (8.9 and 9.1 in), respectively, which compared well with the design thickness of 230 mm (9.0 in). These cores were tested in the laboratory to determine the properties of the concrete used in the two sections.

Core Testing

Table 11 indicates the number of cores for each laboratory test. A summary of the average values that were obtained during the laboratory testing of the field cores is presented in table 12. Observations made during the testing, and comparisons between the performance of the control and recycled sections are also provided below.

Petrographic Examination Summary

The coarse aggregates used in the recycled and control sections both contain highly angular particles of fine-grained, crushed trap rock. The aggregate particles in the recycled section are unevenly distributed throughout the cement paste, while those in the control section are distributed more uniformly. The specific gravity of the recycled coarse aggregate was only 2.53 (compared to 2.81 for the control section aggregate); however, this was the highest recycled concrete aggregate specific gravity of the nine recycled concrete projects studied in this project.

The recycled concrete section was also found to have a slightly greater mortar content than that of the control section (see table 13). Very little of this mortar was determined to be recycled concrete mortar, suggesting that the concrete crushing operation was effective in removing most of the old mortar from the original aggregate. This helps to explain why many of the concrete properties (e.g., thermal coefficient, strength, elastic modulus) were similar for the two test sections.

Uranyl acetate testing of cores obtained from both pavement sections indicate the presence of minor amounts of silica gel in the mortar and around some of the aggregate particles. While this may indicate the presence of alkali-silica reactivity, no such distress was identified during the field surveys.

Table 11. Number of cores for each laboratory test in CT 1.

Laboratory Tests	Recycled Section	Control Section
Thermal Coefficient	3	3
Split Tensile Strength	1	2
Dynamic Modulus of Elasticity	3	3
Static Modulus of Elasticity	0	0
Compressive Strength	2	1
Volumetric Surface Texture	6	6

Table 12. Core testing results for CT 1.

Property	Recycled	Control
Compressive Strength, MPa	39.2	35.4
Split Tensile Strength, MPa	3.8	3.3
Dynamic Elastic Modulus, GPa	31.7	32.8
Static Elastic Modulus, GPa	n/a	n/a
Thermal Coefficient, $(1\text{x}10^{-6})/\,°C$	11.6	10.6
VSTR (for Failed Split Tensile Core), cm^3/cm^2	0.4479	0.3209
VSTR (for Slab Faces at the Joints), cm^3/cm^2	0.6016	0.4933
VSTR (for Slab Faces at the Cracks), cm^3/cm^2	0.3467	0.5376

Table 13. Coarse aggregate and mortar contents for CT 1.

	Recycled	Control
Coarse Aggregate, %	32.2	38.6
New Mortar, %	63.7	61.5
Recycled Mortar, %	4.2	n/a

Mid-Panel Cores

The compression and split tensile strengths were 10 to 15 percent higher for the recycled section than for the control section and the dynamic elastic modulus values for the two sections were not significantly different. (Static elastic modulus tests could not be performed because the cores were shorter than the 1.5 aspect ratio required by ASTM C 469.) Thermal coefficients ranged from 10.9×10^{-6}/ °C to 12.3×10^{-6}/ °C (6.1×10^{-6}/ °F to 6.9×10^{-6}/ °F) for the recycled section, with an average of 11.6×10^{-6}/ °C (6.4×10^{-6}/ °F). Thermal coefficients ranged from 10.3×10^{-6}/ °C to 10.8×10^{-6}/ °C (5.7×10^{-6}/ °F to 6.0×10^{-6}/ °F) for the control section, with an average of 10.6×10^{-6}/ °C (5.9×10^{-6}/ °F).

These test results seem consistent with the fact that the recycled concrete mixture included an increased volume of a relatively clean RCA (with only small quantities of old mortar present), and a lower water-to-cement ratio than was used in the control section.

Joint Cores

VSTR's determined for the doweled joints and cracks of the recycled section are higher than those for the control section (0.6016 vs. 0.4933 cm^3/cm^2). This can be attributed, at least in part, to the larger aggregate top size used in the recycled section. VSTR's for the doweled joints provide an indication of the surface texture that exists at cracks before they are subjected to abrasion under heavy traffic loadings. Deposits of fines up to 13 mm (0.5 in) thick were found caked on the fractured faces of the core halves pulled from the joints.

Crack Cores

VSTR's across the cracks are slightly higher for the control section (0.5376 cm^3/cm^2 for the control vs. 0.3467 cm^3/cm^2 for the recycled), although experience suggests that these values are probably large enough to provide good aggregate interlock when the cracks are tight. The difference might be attributable to previous abrasion damage to the recycled concrete crack face when the crack was more closed (the crack widths for both sections were slightly more than 30 mm [1.2 in] when the cores were retrieved, so it can be assumed that aggregate interlock was not playing a major role in transferring loads across the cracks at that time). Because of the large crack widths, the VSTR's are poor indicators of the measured load transfer efficiencies. Deposits of fines up to 13 mm (0.5 in) thick were found caked on the fractured faces of the core halves pulled from the cracks, also indicating that aggregate interlock is not playing a major role in load transfer. Cracks tended to propagate around the aggregate particles thereby increasing surface texture and VSTR's.

For the CT test sections, VSTR's were higher at cracks which initiated earlier in the pavement's life than for those cracks which occurred later, regardless of the distress survey rating given to a crack. This is because the initial VSTR of cracks which occur early in the pavement life are generally higher than those that develop in mature concrete because they tend to meander *around* the aggregate particles instead of through them. In addition, the aggregate used in this section is very strong (stronger than any other aggregate included in the study), so little deterioration of the crack face occurred under traffic. Weaker aggregates at the crack faces are often subject to abrasion. The effect of pavement age at time of cracking on crack texture is illustrated in figure 23. This figure also reflects the influence of aggregate particle strength on the surface texture of the fractured concrete faces.

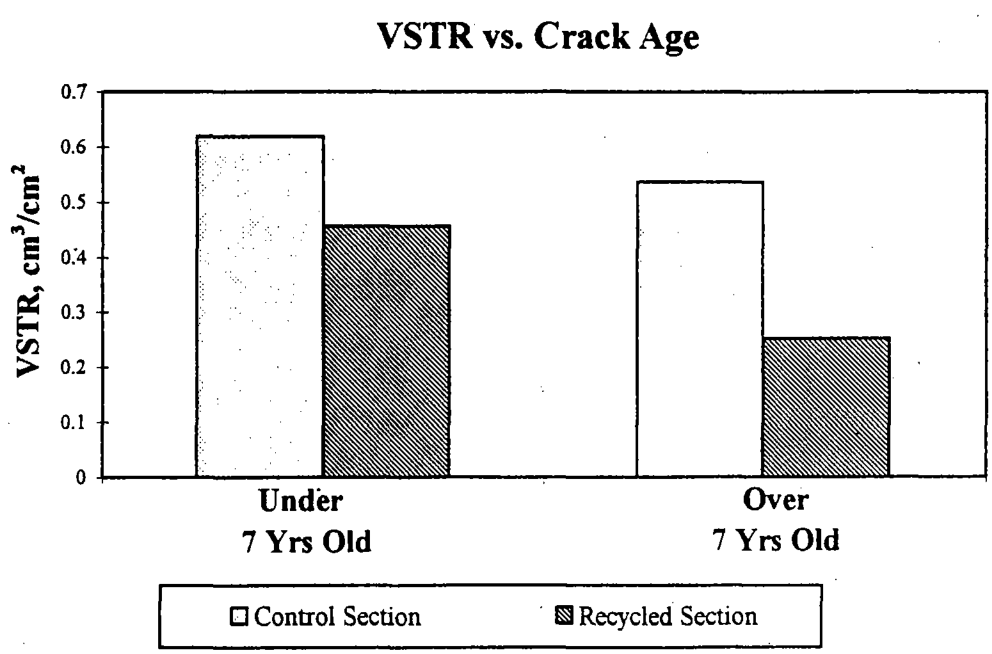


Figure 23. The effect of pavement age at time of cracking on crack texture.

All longitudinal steel exhibited severe corrosion. The longitudinal steel in the recycled section was typically located between 108 and 121 mm (4.3 and 4.8 in) from the pavement surface, but was located 32 mm (1.3 in) from the bottom of one recycled core and 50 mm (2 in) from the bottom of one control core. VST testing was performed on only two of three recycled cores extracted at the cracks because one contained a crack which did not propagate through the full thickness of the pavement. This core contained two layers of reinforcing mesh steel: one 146 mm (5.7 in) from the pavement surface, and the other 178 mm (7 in) from the pavement surface.

Project Summary

Both the control and recycled sections consist of 230 (9-in) JRCP with 0.09 percent reinforcing steel and 38-mm (1.5-in) I-beam dowel bars at the transverse joints. Both sections have been subjected to about 15.9 million ESAL applications since being opened to traffic in 1980. Overall, the sections are nearly identical with the exception of foundation support conditions, the coarse aggregates used in the mixtures and the corresponding mix designs. These differences, and the resulting differences in performance, are highlighted below:

- CTDOT reported that the recycled concrete aggregate had a lower specific gravity, higher absorption capacity, and lower wearing resistance than the conventional aggregate.
- The recycled concrete mix contains a significantly greater weight and volume of coarse aggregate than does the control mix. It also features a lower water-to-cement ratio. Petrographic examination suggests that the recycled aggregate contained only small quantities of recycled mortar, which might be expected in a case such as this where the relatively weak mortar would tend to fracture and separate from the relatively hard trap rock. These factors seem to account for the fact that the physical and mechanical properties of the recycled concrete (i.e., strength, elasticity, thermal coefficient, etc.) were comparable to (or better than) those of the control section concrete.
- The backcalculated dynamic modulus of elasticity of the recycled concrete is about 18 percent lower than that of the control section concrete. However, laboratory testing of cores indicated that the recycled concrete modulus was only 3.5 percent lower than that of the control section concrete.
- The thermal coefficient of the recycled concrete is slightly higher than that of the control section concrete ($11.6 \times 10^{-6}/\ ^\circ$C vs. $10.6 \times 10^{-6}/\ ^\circ$C [$6.4 \times 10^{-6}/\ ^\circ$F vs. $5.9 \times 10^{-6}/\ ^\circ$F]). The difference between these two average values is small, but statistically significant at the 90 percent level.
- Seven-day flexural strength results (taken from historical data) indicated that the recycled concrete was stronger than the control mixture. Split tensile testing of cores retrieved in 1994 indicates that the recycled concrete is still slightly stronger than the control section concrete.
- The strong, dense coarse aggregate used in both the recycled and control sections on this project helped produce excellent VSTR's for cores extracted from

both the joints and cracks, although the surface texture of the control section joints was somewhat less than that of the recycled concrete section joints. This result may have been due to the larger top size of aggregate present in the recycled concrete (51 mm [2 in] versus 38 mm [1.5 in]). In either case, the I-beam dowels appeared to be providing comparable joint load transfer capacity in either section.

- There was no significant difference in joint and crack faulting and overall project serviceability between these two pavement sections.
- Many high-severity cracks are present in both sections. While cracking is expected with long panels, it appears that the reinforcement was under-designed (0.09 percent longitudinal steel) and probably failed due to a combination of corrosion (both dowels and mesh reinforcing), heavy traffic, the close proximity of bridge expansion joint, and pavement response to variations in temperature and moisture. It should be noted that the steel dowels were observed to be corroded to the point of possibly creating joint lock-up effects.
- The control section has a slightly higher density of deteriorated transverse cracks, but the load transfer capacity of these cracks is higher than those in the recycled concrete section. The higher density of the determined transverse cracks in the control section may be due, in part, to the apparently reduced support provided by the construction fill.
- Considerably more joint spalling is apparent on the recycled section, although most is of low-severity. Joint seal damage was observed to be greater for the recycled section than that of the control section and the recycled concrete was found to have a slightly higher coefficient of thermal expansion, as described previously. These two factors may help to explain the increased incidence of joint spalling on the recycled concrete section.
- Uranyl acetate testing of cores obtained from both sections indicate the presence of minor amounts of silica gel in the mortar and around some of the aggregate particles. While this may indicate the presence of alkali-silica reactivity, no such distress was identified during the field surveys.

Kansas 1, K-7 in Johnson County

In the mid-1980's, recycling of AC pavements was common practice in Kansas, but recycling of PCC pavements was a novel procedure. Economic and environmental considerations at that time led to the design and construction of two projects in which the original concrete pavement was recycled into new concrete pavement. The most notable of these was on State Highway K-7 in Johnson County between State Highway K-10 and the Kansas River.

Project Information

The original pavement was a two-lane roadway that was constructed in 1960. It was a 230-mm (9-in) doweled PCC pavement with wire mesh reinforcement. The pavement was experiencing moderate D-cracking near the joints, although the concrete between the joints appeared to be sound.[16] The original concrete aggregate blend consisted of 58 percent coarse aggregate and 42 percent fine aggregate.

Design Information

The original two lanes were removed and a new four-lane highway was constructed in 1985 with two 3.7-m (12-ft) traffic lanes in each direction. The structural design features a 230-mm (9-in) JPCP, a 100-mm (4-in) cement-treated base (CTB), and a 150-mm (6-in) lime-treated subgrade. The transverse joints are spaced at 4.7-m (15.5-ft) intervals and are skewed counter-clockwise 0.17 m ahead per m width (2 ft/12-ft lane). The longitudinal centerline joint is tied with 610-mm (24-in) long, 13-mm (No. 4) deformed bars spaced at 760-mm (30-in) intervals. The recycled pavement was used as aggregate in all of the CTB, in 1.6 km (1 mi) of the JPCP, and in 0.8 km (0.5 mi) of the bituminous shoulder.

Mix Design

Portions of the original PCC slab were removed for laboratory testing and the development of recycled concrete mix designs. The concrete was crushed and transported to the laboratory, where sieve analyses of the crushed concrete were performed. The results of the sieve analyses are presented in table 14. The final aggregate blend selected for use consisted of 50 percent coarse aggregate and 50 percent natural sand. The coarse aggregate portion was composed of 75 percent crushed concrete sized between the 38-mm (1.5-in) and 9.53-mm (3/8-in) sieves and 25 percent 9.53-mm (3/8-in) top size crushed concrete. The control section coarse aggregate was more finely graded, with a top size of approximately 19 mm (3/4 in). The aggregate gradations used in the recycled concrete and control sections are provided in table 15. Note that the crushed concrete contains more material passing the 4.75-mm (No. 4) sieve (16 percent vs. 4 percent), which may be due to degradation of the recycled concrete aggregate particles during production and handling.

The crushed concrete had a bulk specific gravity of 2.38, an absorption capacity of 5.0 percent and a Los Angeles abrasion test result of 45 percent mass loss. The coarse aggregate portion of the conventional mix had a bulk specific gravity of 2.60, about 8 percent higher than that of the crushed concrete.

Trial mix designs were developed and tested in the laboratory. The resulting mix designs for the recycled and control sections are provided in table 16. The mix designs are nearly the same for both sections, with the only difference being the addition of slightly more coarse and fine aggregate (by weight) in the conventional mix. It should be noted that, on a volumetric basis, the recycled concrete mixture actually contains about 5 percent more coarse aggregate and 4 percent less fine aggregate than the control mixture. The amount of water and cement used in the two mixes are identical. An air-entraining agent was also added to both mixes to maintain the air content at 6 ± 2percent (air content of the fresh PCC mixture was measured by the "Roll-O-Meter").

Table 14. Gradation of crushed concrete for recycled mix.[16]

Sieve	Percent Passing		
	Crushed Concrete, 38-mm (1.5-in) minus	Crushed Concrete, 9.5-mm (3/8-in) minus	Crushed Concrete Aggregate Blend
38 mm (1.5 in)	100	100	100
25 mm (1.0 in)	75	100	81
19 mm (3/4 in)	46	100	62
12.7 mm (1/2 in)	22	100	42
9.53 mm (3/8 in)	6	100	30
4.75 mm (No. 4)	1	60	16
2.36 mm (No. 8)	1	27	8
1.18 mm (No. 16)	1	17	5
0.600 mm (No. 30)	0	12	3
0.300 mm (No. 50)	0	7	2
0.150 mm (No. 100)	0	5	1
0.075 mm (No. 200)	0	3	1

Table 15. Aggregate gradations of recycled and control sections.[16]

Sieve	Recycled		Control	
	Coarse	Fine	Coarse	Fine
51 mm (2.0 in)	100		100	
38 mm (1.5 in)	100		100	
25 mm (1.0 in)	81		100	
19 mm (3/4 in)	62		100	
12.7 mm (1/2 in)	42		74	
9.53 mm (3/8 in)	30		41	
4.75 mm (No. 4)	16	98	4	98
2.36 mm (No. 8)		91		87
1.18 mm (No. 16)		75		69
0.600 mm (No. 30)		47		32
0.300 mm (No. 50)		12		9
0.150 mm (No. 100)		2		2
0.075 mm (No. 200)		1		0

Table 16. Mix design for KS 1.[16]

Material	Recycled	Control
Coarse Aggregate	848 kg/m^3	884 kg/m^3
Fine Aggregate	848 kg/m^3	884 kg/m^3
Cement	357 kg/m^3	357 kg/m^3
Fly Ash	0 kg/m^3	0 kg/m^3
Water	147 kg/m^3	147 kg/m^3
w/c Ratio	0.41	0.41

Construction Information

Demolition of the pavement was achieved using a 520 Link-Belt Diesel Hammer pulled by a D-7 Caterpillar tractor. The concrete was easily removed from the deteriorated portions of the pavement. However, in areas where sound concrete was present, the breaker was unable to break the wire mesh. Other attempts were made, with limited success, in an attempt to break the mesh. Bolt cutters were eventually used to sever the mesh.

The crushing operation employed a Cedar Rapids Impact Crusher and a Cedar Rapids Triple Roll as the primary and secondary crushers, respectively. Again, problems were encountered as the wire mesh often plugged the primary crusher. This problem was remedied by changing the flow of the impact crusher. Additional problems were encountered at the screening unit, and the screen sizes had to be altered. Yet another problem was the large amount of steel collected by the electro-magnet. The crusher had to be completely shut down about once per hour to remove the steel.

Construction of the pavement proceeded in a much smoother fashion. The lime treatment of the subbase was completed without difficulty. Likewise, the CTB and JPCP (both of which included recycled aggregate) were constructed with the same degree of ease as a conventional PCC paving operation. Portions of the AC shoulder also incorporated recycled concrete aggregate and were constructed without complications.

Temperature at the time of placement varied from 12 to 24 °C (53 to 75 °F). The air contents of the recycled and conventional mixes were about 6. 2 percent. The slump of the mixes did vary, however. The average slump of the recycled mix was 38 mm (1.5 in), whereas the average slump of the conventional mix was 64 mm (2.5 in). Both sections were tined transversely, and a liquid membrane curing compound was applied to the surface. Joints were sawed within 6 to 8 h of concrete placement.

Concrete Properties

Flexural testing of concrete beam specimens was performed on both the recycled concrete and the conventional concrete. The 6-day flexural strengths of the recycled concrete averaged 3.9 MPa (562 lbf/in^2), with 11-day average strengths increasing to 4. 2 MPa (606 lbf/in^2). For the conventional concrete, the 6-day and 7-day average flexural strengths were 4. 2 and 4.4 MPa (607 and 632 lbf/in^2), respectively. Comparison of the 6-day flexural strengths indicates a reduction of about 7 percent with the use of the recycled concrete aggregate, a difference consistent with the results of other studies.

Climatic Conditions

The KS 1 test sections are located in the wet-freeze environmental region. The minimum and maximum average monthly temperatures are -2 and 27 °C (29 and 80

°F), respectively. The freezing index is 56 °C-days (100 °F-days), and the sections are exposed to about 80 freeze-thaw cycles per year. The area experiences about 100 days of precipitation per year for a total annual precipitation of 860 mm (34 in) and a Thornthwaite moisture index of 22.

Traffic Loadings

Both sections were opened to traffic in 1985 and have sustained approximately the same traffic loadings and ESAL applications. The two-way ADT has increased from around 7,300 vehicles per day in 1985 to approximately 12,100 vehicles per day in 1994. The percentage of trucks on the highway has varied from 7 to 12 percent. The cumulative ESAL applications in the driving (outer) lane is around 2. 2 million through 1994.

Selection of Distress Survey Section

Several different criteria were considered when selecting the sections to be surveyed within each project. Obviously, the first criterion was that one section contain recycled concrete aggregate and the other section contain virgin aggregate. However, the selection was not that easy. Some sections of the highway contained a cement-treated base (CTB) that was constructed using recycled concrete aggregate. Another criterion was traffic. Sections in which equal traffic loadings were applied would provide better comparisons. The grade of the section was also a concern, as some portions of the highway were constructed in cut areas, while others were constructed on fill material.

After much deliberation, the research team selected the sections which were thought to best satisfy the various criteria. The recycled and control sections that were selected were both constructed on fill material and both employed a CTB containing recycled concrete aggregate. These sections carry traffic in opposite directions on the highway; however, traffic data provided by the Kansas Department of Transportation indicates no significant difference in directional traffic volumes.

Drainage Survey

The PCC pavement in both sections is constructed on a cement-treated base. The research team did not identify any drainage outlets that would suggest the presence of longitudinal collector drains, and available literature did not mention the use of transverse or longitudinal drains. Signs of pumping of moisture and fines were present on both sections. The recycled section displayed low-severity pumping, while medium-severity pumping was observed on the control section. The transverse pavement slopes in both sections varied from about 1.0 to 3.5 percent, with typical shoulder slopes of about 6 percent. The recycled section was constructed on a 1 percent longitudinal grade; the control section was essentially flat.

Pavement Distress Survey

The pavement condition survey was conducted over sections approximately 305 m (1,000 ft) in length. A complete summary of the results of the survey are provided in appendix A; a summary of the average results for some key performance measures is shown in table 17. With the exception of faulting, the results indicate that control and recycled pavement sections are performing comparably. The most significant difference is that faulting of the control section was consistently (if only slightly) greater than that of the recycled section.

Table 17. Summary of performance data (average values) for KS 1.

Performance Measurement	Recycled	Control
Corner Faulting, mm (Manual)	2.3	3.8
Wheelpath Faulting, mm (Manual)	2.3	3.3
Wheelpath Faulting, mm (Digital)	2.3	3.3
Transverse Cracking, % Slabs	0	0
Longitudinal Cracking, m/km	0	0
Transverse Joint Spalling, % Joints	29	26
Joint Width, mm	13	13
PSR	3.8	3.8

D-cracking

Neither section exhibited any signs of new or recurrent D-cracking. This is somewhat remarkable for the RCA section because of the large coarse aggregate top size (38 mm [1.5 in]) that was used. Possible explanations include: 1) the distress in the original pavement that was identified as D-cracking may have actually been caused by freeze-thaw damage in the mortar, which was largely removed during recycling (a hypothesis that might be verified through investigation of air entrainment records from the original pavement construction); 2) D-cracking may yet occur (the pavement was only about 10 years old at the time of survey); and 3) the previously-identified D-cracking may have been alkali-aggregate reactivity (indicated by petrographic examinations of the recycled concrete cores and discussed below), which is now somewhat mitigated through the use of a Type II cement with an alkali content of 0.47 percent.

Transverse Joint Faulting

Average faulting levels at the control section transverse joints were consistently higher than those in the recycled concrete section. While neither section exhibited excessively large faults, the faulting in both sections (especially the control section) is beginning to approach levels that will require treatment. Neither section contains dowels at the transverse joints, and signs of pumping were observed in both sections (low-severity in the recycled concrete section and medium-severity in the control section). The higher severity of pumping observed in the control section helps to explain the observed increase in faulting there.

The lack of edge drains may be allowing water to accumulate in the CTB; this water could then be pumped out (along with foundation fines) by heavy traffic loads.

Transverse Cracking

No transverse cracks were present on either the recycled or control section. Both sections employed a 4.7-m (15.5-ft) joint spacing. The ratio of slab length to radius of relative stiffness (based on backcalculated k and dynamic E) is approximately 5.5 for each section. This short joint spacing (and relatively low L/l) limits thermal curling stresses, inhibiting the initiation of transverse cracks.

Longitudinal Cracking

No longitudinal cracks were apparent on either section.

Transverse Joint Spalling

Spalling of the transverse joints was apparent to a small degree on both sections. Spalling occurred at 29 and 26 percent of the joints for the recycled and control sections, respectively. However, the majority of the spalls were low severity with the remaining being medium-severity spalls (7 and 4 percent for the recycled and control sections, respectively). The slight differences in transverse joint spalling between the recycled and control sections are statistically insignificant.

Present Serviceability Rating (PSR)

The average PSR values of the recycled and control sections are both 3.8. Thus, although the control section has slightly higher faulting levels, it is still providing a level of serviceability comparable to that of the recycled section.

<u>FWD Testing</u>

FWD testing was used to determine material properties (PCC elastic modulus and subgrade k-value), joint load transfer efficiencies, and loss of support. Pavement deflection testing typically included 5 slab centers, 10 transverse joints (testing on both approach and leave sides), 10 transverse cracks (both approach and leave sides), and 10 lane-shoulder edges. However, no transverse cracks were present on either Kansas section, so none were tested. Table 18 provides a summary of the results of the deflection testing program.

Table 18. Deflection testing results for KS 1.

Property	Recycled	Control
PCC Elastic Modulus, GPa	38.6	40.6
CTB Elastic Modulus, GPa	9.7	10. 2
k-value, kPa/mm	67.6	69.0
Joint Load Transfer, %	30	37
Crack Load Transfer, %	n/a	n/a
Average Midslab Deflection, µm	74	69
Average Edge Deflection, µm	143	109
Corner With Voids, %	40	0
Maximum Air Temperature During Testing, °C	12	11

Elastic Modulus

The elastic modulus of the PCC slab and the CTB were backcalculated using the center-of-slab deflection measurements. The moduli of the two layers were calculated assuming a fully-bonded condition between the layers. Three observations provide the basis for this assumption: 1) a bonding material was observed between these layers in cores retrieved from these sections; 2) the layers in retrieved cores often remained bonded; and 3) unreasonable modulus values were obtained when backcalculations were performed assuming an unbonded condition.

Figure 24 shows a profile of the elastic modulus values for the recycled section using four drops at five different locations. The average backcalculated PCC elastic modulus is 38.6 GPa (5,600,000 lbf/in^2), which agrees well with laboratory test values of 35.3 GPa (5,120,000 lbf/in^2). Backcalculated values range from 23 to 53 GPa (3,400,000 to 7,700,000 lbf/in^2). The CTB has an average elastic modulus of 9.7 GPa (1,400,000 lbf/in^2). The backcalculated PCC elastic modulus values do not vary significantly between load drops at any given location. However, large variations in the backcalculated PCC elastic modulus are observed between different locations. Variations in the CTB elastic modulus are not nearly as large.

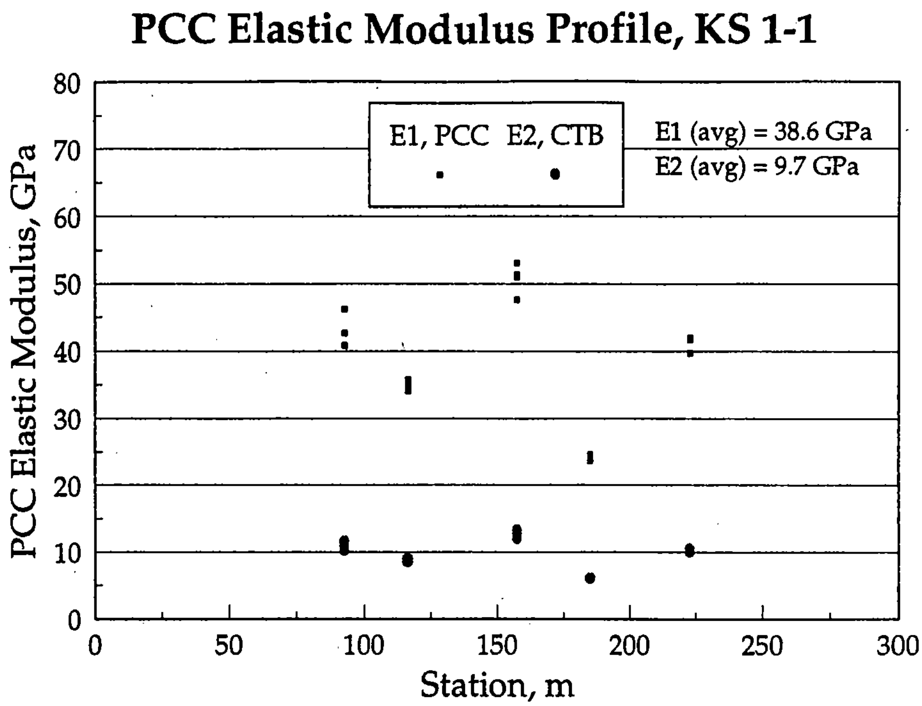


Figure 24. PCC elastic modulus profile for KS 1-1 (recycled section).

Figure 25 shows a similar plot for the control section. The average elastic modulus of the concrete slab is 40.6 GPa (5,890,000 lbf/in^2), which agrees well with laboratory test values of 35.8 GPa (5,190,000 lbf/in^2). Backcalculated values range from 30 to 50 GPa (4,400,000 to 7,300,000 lbf/in^2). The average elastic modulus of the CTB is 10.2 GPa (1,480,000 lbf/in^2). Again, deflection basin measurements at a given location were highly repeatable and consistent, and variations in the elastic modulus values at different locations are greater for the PCC slab than for the CTB.

On average, the elastic modulus of the recycled section PCC is about 5 percent lower than that of the control section (1.4 percent lower using lab test data). The results of other research studies commonly indicate elastic modulus values between 20 to 40 percent lower for RCA concrete prepared using the same water-cement ratio as the control mixture. (See references 12 through 15.)

The same mix design was used for the CTB under both the recycled and control sections. Consequently, the backcalculated elastic modulus values for these layers are about the same for both sections.

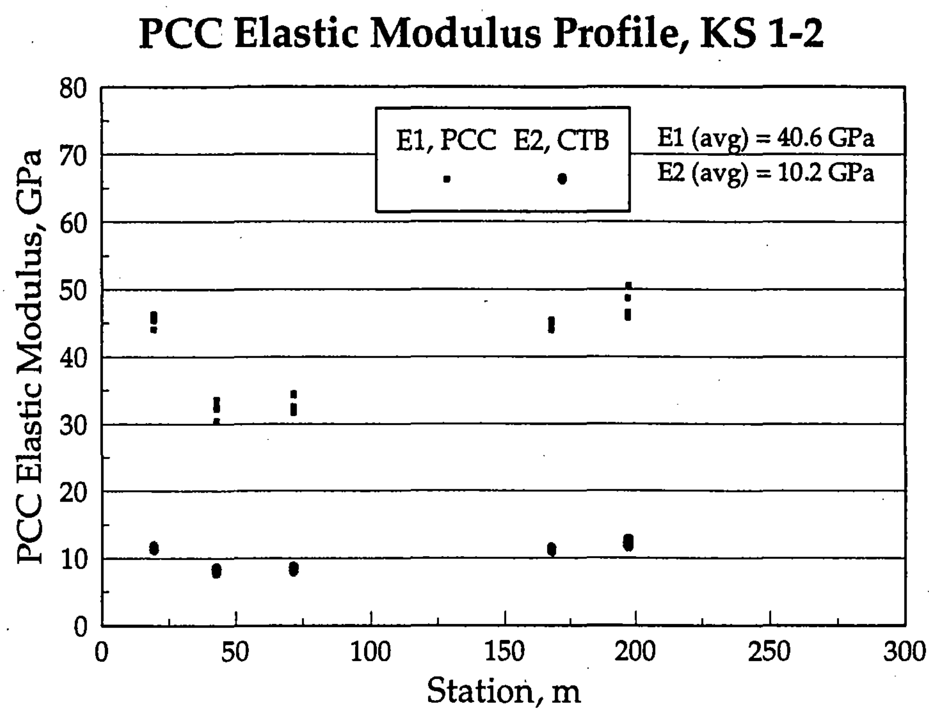


Figure 25. PCC elastic modulus profile for KS 1-2 (control section).

Modulus of Subgrade Reaction (k-value)

Figure 26 presents a profile plot of the backcalculated k-values for the recycled concrete pavement section. The average k-value is 68 kPa/mm (249 $lbf/in^2/in$), with values ranging from 43 to 87 kPa/mm (157 to 322 $lbf/in^2/in$). Similar data for the control section is shown in figure 27. The control section k-values range from 54 to 83 kPa/mm (200 to 306 $lbf/in^2/in$), with an average of 69 kPa/mm (254 $lbf/in^2/in$). These test results indicate relatively uniform slab support both within and between test sections, although the recycled section does exhibit slightly more variability in foundation support. It should be noted that backcalculated k-values fluctuate seasonally with variations in base and soil moisture and temperature. The values presented here were determined from test data obtained in November 1994.

Joint Load Transfer

The load transfer efficiencies at both the approach and leave joints of the recycled section are shown in figure 28. The load transfer efficiencies represent the ratio of the deflection of the loaded side of a transverse joint to the deflection of the unloaded side of the same joint. The average deflection load transfer efficiency of the recycled concrete pavement section is only 30 percent. At some locations, load placement

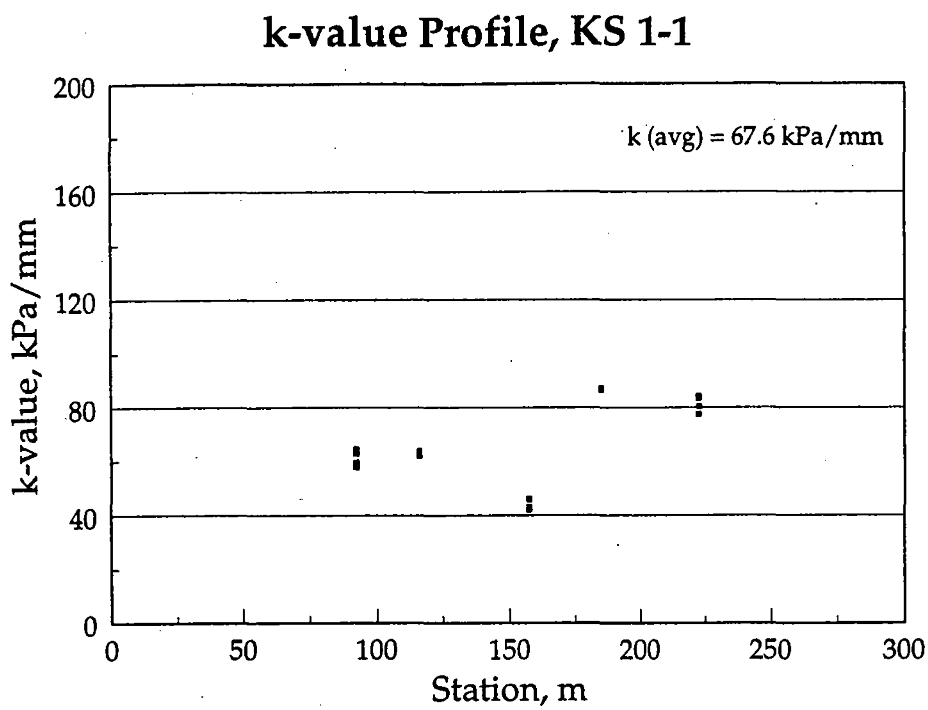


Figure 26. K-value profile for KS 1-1 (recycled section).

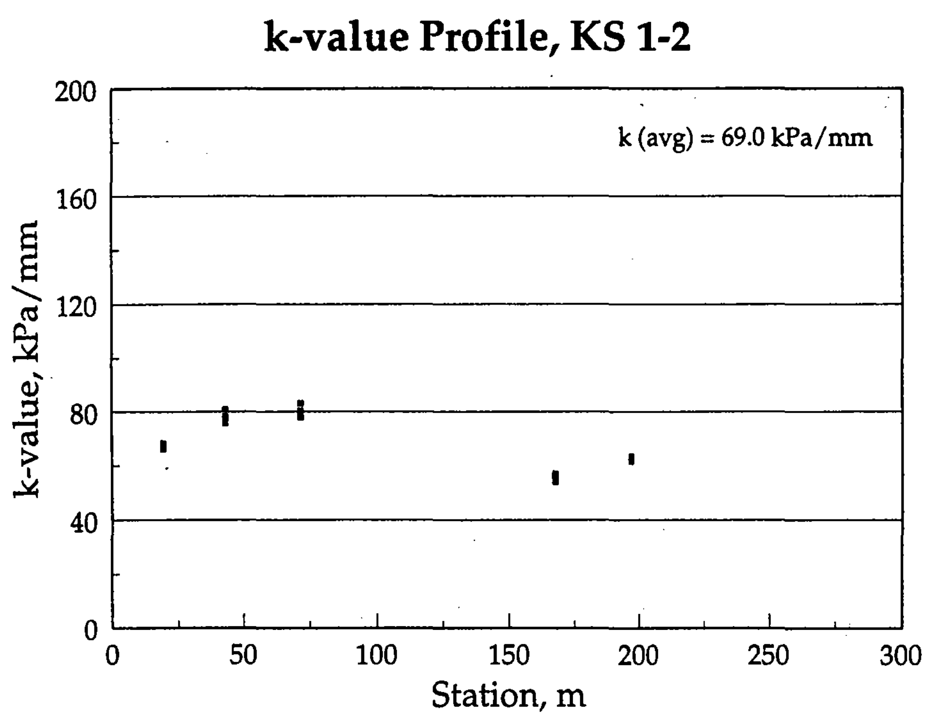


Figure 27. K-value profile for KS 1-2 (control section).

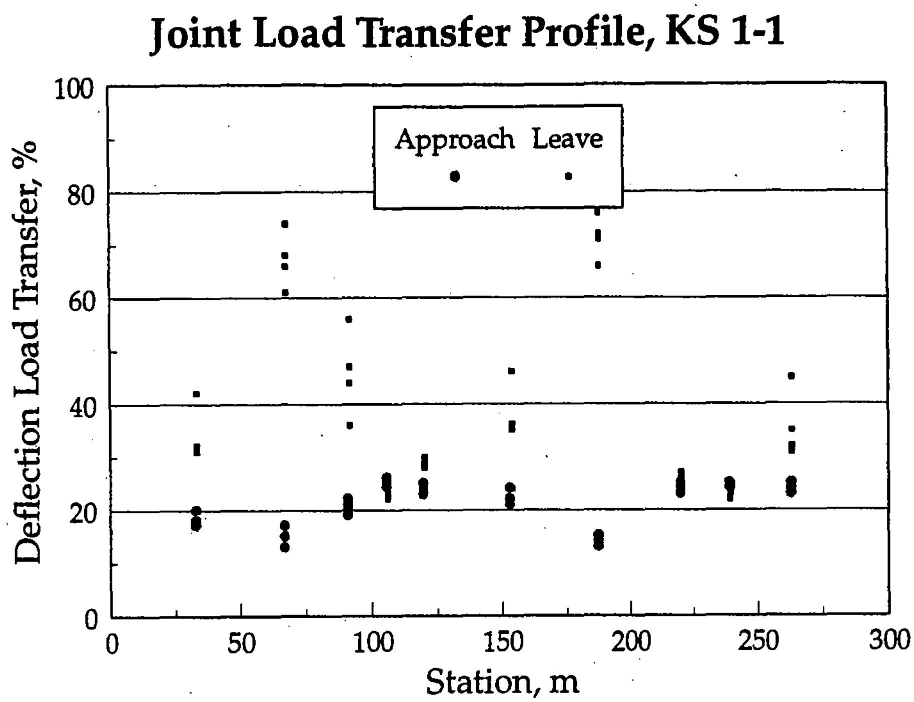


Figure 28. Joint load transfer profile for KS 1-1 (recycled section).

strongly influences load transfer efficiency, with lower average efficiencies observed when the load is placed on the approach side of the joint (21 percent) than on the leave side (39 percent). Differences between approach and leave joint load transfer efficiency were as great as 50 percent. Large differences might be attributable to the inclination of the concrete fracture plane beneath the sawed joint.

Figure 29 illustrates the joint load transfer efficiencies for the control section. The average deflection load transfer efficiency is 37 percent, with average values for approach and leave side load placement of 33 and 40 percent, respectively. Thus, the average transverse joint load transfer efficiency for the control section is 7 percent higher than for the recycled concrete section. The control section load transfer measurements are also less variable.

Loss of Support

Void detection analyses were performed using the corner deflection measurements from the leave side of transverse joints. Figures 30 and 31 show the potential loss of support profiles for the recycled concrete and control sections, respectively. Some locations within the recycled concrete section appear to have suffered slight losses of support, which is consistent with the higher deflections and lower joint load transfer efficiencies that were observed, as discussed previously. The control section presents no clear evidence of loss of support, although it has developed slightly more faulting than the recycled section.

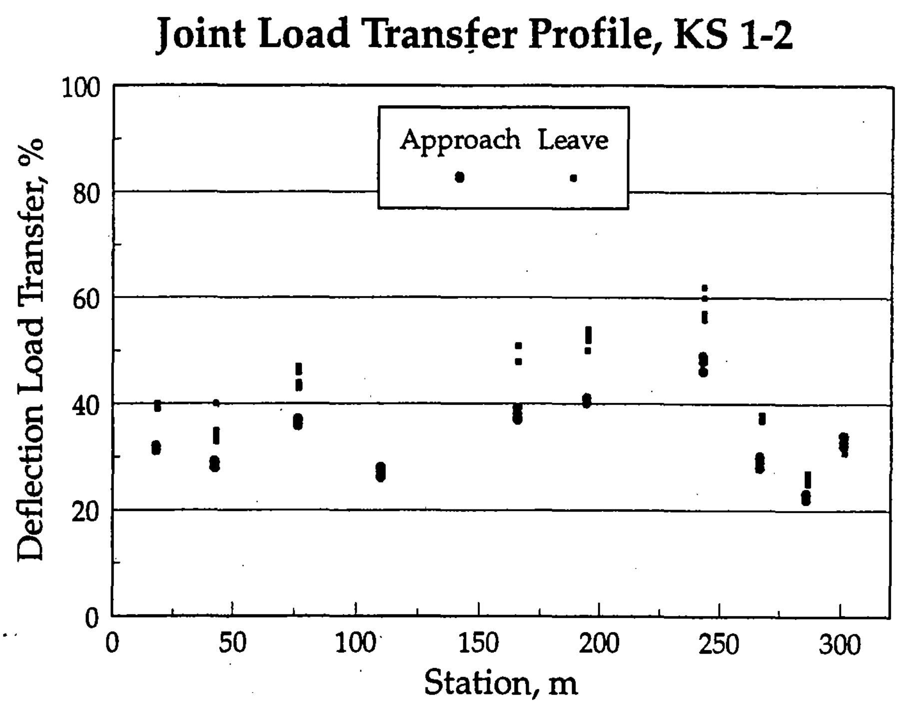


Figure 29. Joint load transfer profile for KS 1-2 (control section).

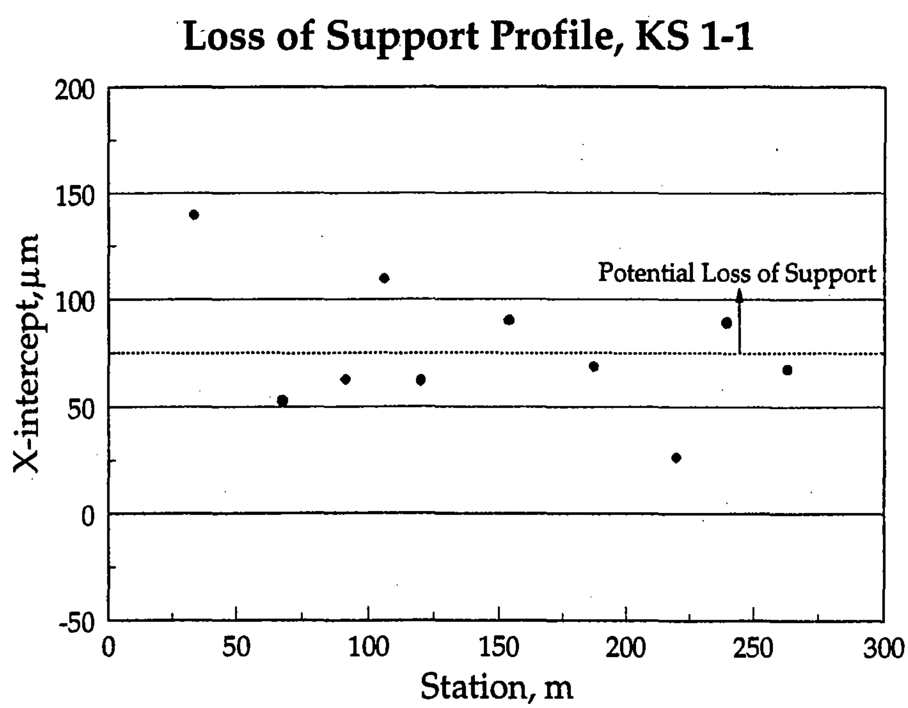


Figure 30. Loss of support profile for KS 1-1 (recycled section).

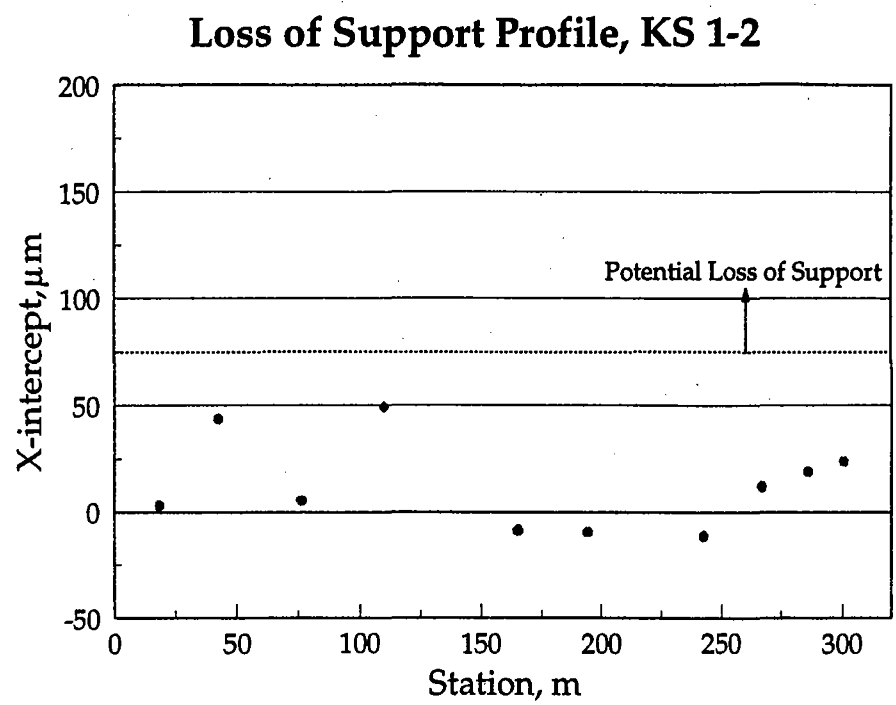


Figure 31. Loss of support profile for KS 1-2 (control section).

Coring

Eight cores were retrieved from each pavement section, including five from midpanel locations and three from transverse joints. Transverse cracks were not present in either section, so cores could not be taken at transverse cracks. All cores were 150 mm (6 in) in diameter and extended through the thickness of the concrete slab. In many cases, the cores included both surface concrete and fully-bonded CTB layers. Although the design slab thickness was 230 mm (9.0 in), core lengths averaged 239 and 246 mm (9.4 and 9.7 in) for the recycled and control sections, respectively.

Core Testing

Cores retrieved from the field sections were subjected to tests of compressive strength, split tensile strength, modulus of elasticity (static and dynamic determinations) and surface texture of the joint and crack faces. The number of cores for each laboratory test are indicated in table 19. The results of these tests are summarized in table 20. Observations made during the testing and comparisons between the results of tests on specimens obtained from the control and recycled sections are also provided below.

Table 19. Number of cores for each laboratory test in KS 1.

Laboratory Tests	Recycled Section	Control Section
Thermal Coefficient	3	3
Split Tensile Strength	1	1
Dynamic Modulus of Elasticity	3	3
Static Modulus of Elasticity	0	0
Compressive Strength	3	3
Volumetric Surface Texture	3	3

Table 20. Core testing results for KS 1.

Property	Recycled	Control
Compressive Strength, MPa	47.9	43.7
Split Tensile Strength, MPa	3.2	3.6
Dynamic Elastic Modulus, GPa	35.3	35.8
Static Elastic Modulus, GPa	n/a	n/a
Thermal Coefficient, $(1x10^{-6})/ °C$	10.5	9.4
VSTR (for Failed Split Tensile Core), cm^3/cm^2	0.2613	0.2595
VSTR (for Slab Faces at the Joints), cm^3/cm^2	0.2678	0.3321
VSTR (for Slab Faces at the Cracks), cm^3/cm^2	n/a	n/a

Petrographic Examination

The coarse aggregate used in both the recycled concrete and control sections consists of angular particles of very fine-grained limestone. These particles were uniformly distributed throughout the cement matrix. A higher mortar content was measured in the recycled concrete specimens (see table 21), which would be expected even if the two mixtures have comparable amounts of cement, sand, and water because the recycled mixture also contains some mortar from the old concrete, clinging to natural aggregate particles. Many large voids are present in the cement paste in both sections.

Table 21. Coarse aggregate and mortar contents for KS 1.

	Recycled	Control
Coarse Aggregate, %	10.8	17.7
New Mortar, %	74.1	77.9
Recycled Mortar, %	15.1	4.4

Uranyl acetate testing indicated the presence of moderate amounts of silica gel in the recycled concrete aggregate mortar and around some aggregate particles, possibly indicating an alkali-aggregate reaction. Only minor fluorescence was observed in the control concrete core.

Mid-Panel Cores

The compressive strength of the recycled concrete section exhibited little variability and averaged 47.9 MPa (6,950 lbf/in^2), which was slightly higher than the 43.7 MPa (6,340 lbf/in^2) average strength of the control section concrete. Possible explanations for the apparent greater compressive strength of the recycled concrete include the use of larger coarse aggregate particles (38 mm vs. 19 mm [1.5 in vs. 0.75 in]) and the use of a slightly greater volume fraction of coarse aggregate in the recycled mix. In addition, the introduction of 25 percent recycled fines may have contributed to the higher strength of the recycled mixture, as suggested by Fergus' study.[17]

Split tensile tests were performed on only one specimen from each section, with slightly higher strength observed for the control section (3.6 MPa [520 lbf/in^2]) than for the recycled concrete section (3.2 MPa [470 lbf/in^2]). These results agree with those of previous studies that indicate reduced tensile strength in RCA concrete can be attributed, at least in part, to weaker particle strength due to weakened bond between the natural aggregate and original concrete mortar.

There was no significant difference in the average dynamic modulus of elasticity values for the specimens obtained from the two Kansas sections (35.3 GPa [5,120,000 lbf/in^2] for the recycled section vs. 35.8 GPa [5,190,000 lbf/in^2] for the control section). The results of these tests exhibited little variability. Static tests of elastic modulus were not performed on specimens from these sections.

The coefficient of thermal expansion/contraction of the RCA concrete ranged from 9.9 x 10^{-6}/ °C to 11.0 x 10^{-6}/ °C (5.5 x 10^{-6}/ °F to 6.1 x 10^{-6}/ °F) averaging 10.5 x 10^{-6}/ °C (5.8 x 10^{-6}/ °F). The control section results ranged from 8.9 x 10^{-6}/ °C to 9.5 x 10^{-6}/ °C (4.9 x 10^{-6}/ °F to 5.4 x 10^{-6}/ °F) averaging 9.4 x 10^{-6}/ °C (5.2 x 10^{-6}/ °F). The higher coefficients of the RCA concrete specimens is probably attributable, at least in part, to the slightly higher mortar content in the RCA concrete (old mortar plus new).

In summary, the laboratory strength test results indicate that both concrete pavements are of comparable strength. This finding is consistent with expectations based upon mix design parameters and is supported by comparable field performances.

Joint Cores

Table 20 shows that the average VSTR for the control section joints is greater than that of the recycled section joints (0.3321 vs. 0.2678 cm^3/cm^2), in spite of the smaller aggregate top size in the control section. One possible explanation is an initial difference in surface texture caused by the inclusion of fewer natural aggregate particles in the RCA mixture and the separation of those particles from the old mortar, resulting in smaller and fewer surface irregularities than would be expected with comparable amounts of all natural aggregate. Another possible explanation is that the surface texture of the recycled concrete joint face is more easily abraded, resulting in a more rapid loss of texture over time. Given that both sections have undoweled joints and probably exhibit at least seasonal periods of poor load transfer, there is strong potential for joint face abrasion due to differential vertical joint movements.

Additional testing with laboratory mixtures and specimens (surface texture measurement and abrasion resistance) is necessary to determine the degree to which these mechanisms may be taking place.

Project Summary

The RCA concrete and control sections were designed to minimize the structural and material differences between the sections, thereby allowing direct measures of the effects of using recycled concrete and natural coarse aggregates. Both sections are undoweled 230-mm (9-in) JPCP with a 100-mm (4-in) CTB containing recycled concrete aggregate (although no signs of any edge drains could be found) and a 150-mm (6-in) lime-treated subgrade. Both sections were constructed on fill material and have experienced the same number of ESAL applications.

The two concrete mix designs include comparable batch weights of all components (although the recycled mixture contains a greater *volume* of coarse aggregate and a slightly reduced volume and weight of fine aggregate). Other mix design differences included the use of a larger coarse aggregate top size in the recycled mix (38 mm vs. 19 mm [1.5 in vs. 0.75 in]) and the partial replacement of natural fine aggregate with RCA fines (25 percent by weight). The larger RCA aggregate size is particularly unusual because recycled concrete is normally crushed to a *reduced* size to reduce the potential for recurrent D-cracking.

The reported air content for both mixtures was 6.2 percent, although the average slump of the RCA concrete mix was 38 mm (1.5 in) compared to 64 mm (2.5 in) for the conventional mix. The difference in slump can probably be attributed to the inclusion of larger aggregate and RCA fines in the recycled mixture, both of which would contribute to a reduction in mix consistency.

Tests of concrete strength, elasticity and thermal properties indicate that both concrete pavements have comparable properties in these areas. Furthermore, these test results were generally consistent with expectations based upon mix design parameters.

- Modulus of elasticity test results are not significantly different for the two concrete mixtures.
- Limited split tensile testing of cores and original flexural beam testing indicates reduced tensile strength in the RCA concrete, which might be attributed, at least in part, to weaker particle strength due to weakened bond between the natural aggregate and original concrete mortar.
- Compressive strength of the RCA mixture was about 10 percent greater than that of the control concrete, which may be due to the larger RCA aggregate size or the use of some RCA fines in the mixture.
- The average coefficient of thermal expansion/contraction of the RCA concrete was higher than that of the control concrete (10.5×10^{-6}/ °C vs. 9.4×10^{-6}/ °C [5.8×10^{-6}/ °F vs. 5.2×10^{-6}/ °F]). The higher coefficient of the RCA concrete is probably attributable, at least in part, to the slightly higher mortar content in the RCA concrete (old mortar plus new).
- The performances of the two pavement sections are comparable. The following performance-related observations are worth noting:
- Neither section has developed serious faulting at the transverse joints, in spite of the lack of dowel load transfer devices at the transverse joints. The control section does exhibit consistently higher (but still minor) levels of faulting than does the recycled section.
- Despite the differences in faulting on the two sections, the PSR of both sections was estimated at 3.8.
- Neither section exhibited transverse or longitudinal cracking.
- Transverse joint spalling was observed on fewer than 30 percent of the joints in both section and was of low severity in all cases.
- The RCA concrete pavement section exhibits generally lower deflection load transfer efficiencies and a greater potential for loss of support than does the control section, in spite of the use of larger coarse aggregate particles. VSTR tests of joint faces contained within cores found greater surface texture in the control section. This may be attributable to poorer abrasion resistance of RCA concrete or a reduction in the initial fracture plane texture caused by a the inclusion of fewer natural aggregate particles and the debonding of these particles from the old mortar.
- Uranyl acetate testing indicates the presence of moderate amounts of silica gel deposits in the RCA concrete core specimens, indicating the possibility of a past or present alkali-aggregate reaction. The same test indicated only minor amounts of silica gel in the control section.

- No new or recurrent D-cracking has developed in either pavement section, in spite of the large coarse aggregate size used in the RCA mixture. Possible explanations include: 1) the original D-cracking may have originated in the mortar (which was largely removed during recycling) rather than in the aggregate; 2) D-cracking may yet occur (the pavement was only about 10 years old at the time of survey); 3) the previously-identified D-cracking may have been alkali-aggregate reactivity (indicated by petrographic examinations of the recycled concrete cores and pavement evaluations), which is now somewhat mitigated through the use of a Type II cement with an alkali content of 0.47 percent; 4) the most susceptible aggregate already cracked or had cracked and was fractured in the recycling process; and 5) enough reduction in size occurred in the RCA to delay D-cracking for more than the 15 to 20 years it took to develop originally.

Minnesota 1, WB I-94 near Brandon

This project is the first of three Minnesota JRCP sections included in this study that were constructed using RCA concrete. The project includes both recycled concrete and control sections with the same structural designs and traffic.

Project Information

The project is located in the two westbound lanes of I-94 near Brandon. It is a four-lane divided highway. The recycled concrete section incorporated concrete from the original pavement for use as aggregate for the reconstructed pavement. The original pavement was built in 1960.

Design Information

The recycled and control sections were constructed in 1988 and employ the same basic design, consisting of 280-mm (11-in) JRCP over a 150-mm (6-in) aggregate base and an A-6 subgrade. The subgrade was prepared (bladed) and compacted to a depth of 150 to 760 mm (6 to 30 in), but no stabilizing agent was added. The transverse joints are skewed, spaced at 8.2-m (27-ft) intervals, and contain 32-mm (1.25-in) epoxy-coated dowel bars. The transverse joints are sealed with a preformed joint sealant.

The outer traffic lane (driving lane) was paved 4.3 m (14 ft) wide with rumble strips included within the outer 0.6 m (2 ft) to alert traffic that unintentionally wanders from the main 3.7-m (12-ft) travel lane. The outer shoulder extends 2.4 m (8 ft) further and consists of a 150-mm (6-in) AC surface layer over an aggregate base. The inside shoulder has the same structural design as the outer shoulder and extends 0.9 m (3 ft) from the edge of the 3.7-m (12-ft) inner travel lane. Longitudinal edge drains are also present within both sections, with outlets spaced at 150-m (500-ft) intervals.

Slab reinforcement consists of an uncoated deformed welded wire fabric with 8-mm (0.30-in) diameter longitudinal wires spaced at 310 mm (12 in) center-to-center, resulting in a longitudinal steel content of 0.054 percent of the slab cross-sectional area. Transverse wires are 6 mm (0.23 in) in diameter and are also spaced at 310 mm (12 in) center-to-center. The longitudinal centerline joint is equipped with 910-mm (36-in) long, 16-mm (No. 5) epoxy-coated deformed tie bars spaced 910 mm (36 in) on center.

Mix Design

Little information is available regarding the aggregate gradations and properties for the recycled and control mixes. However, the recycled section is known to contain recycled concrete aggregate as the coarse aggregate and a natural sand as the fine aggregate while the control section contains virgin material for both the coarse and fine aggregate. The stated maximum coarse aggregate size in both the recycled and control sections is 19 mm (0.75 in). However, petrographic examination of cores retrieved from these sections found no particles larger than 12.5 mm (0.50 in) in the control section core.

The mix design used for the recycled section is provided in table 22. The corresponding mix design for the control section is not available. However, the mix design for the recycled section is similar to that used for MN 4-1, and it is believed that the control section has a mix design similar to that used for MN 4-2 (see table 79 in appendix A).

Table 22. Mix designs for MN 1.

Material	Recycled
Coarse Aggregate	976 kg/m^3
Fine Aggregate	712 kg/m^3
Cement	288 kg/m^3
Fly Ash	51 kg/m^3
Water	160 kg/m^3
w/c+p Ratio	0.47

The aggregate for the recycled section was composed of 58 percent recycled concrete aggregate and 42 percent natural sand. Type I cement and Class C fly ash were used in proportions that produced a water-cement ratio of 0.56 and a water-cementitious (cement plus fly ash) ratio of 0.47. The mix also contained an air-entraining agent in proportions selected to provide an air content of 5.5 percent.

Construction Information

The 28-year-old concrete slab was removed and crushed to provide aggregate for the recycled concrete pavement. The control section used a virgin crushed diorite coarse aggregate material. The recycled and control sections were both placed using the same construction techniques. Surface texture was provided using an astroturf drag followed by transverse tining. A curing compound was also applied to the surface.

Climatic Conditions

The MN 1 test sections are located in the transition zone between the dry-freeze and wet-freeze environmental regions. The minimum and maximum average monthly temperatures are -12 and 22 °C (11 and 72 °F). The area experiences about 105 freeze-thaw cycles annually, and the freezing index is 1170 °C-days (2100 °F-days). The Thornthwaite moisture index is 5, which reflects an average of 106 days of precipitation per year totaling an average of 610 mm (24 in).

Traffic Loadings

The reconstructed pavement was opened to traffic in 1988 and, through the survey date in 1994, had been exposed to an estimated 3.7 million ESAL applications. In 1988, the two-way ADT was estimated at 8,200 vehicles per day. As of 1994, the two-way ADT had increased to about 9,500 vehicles per day, including about 32 percent heavy trucks. The corresponding ESAL applications in the opening year (1988) and the survey year (1994) are estimated at 462,000 and 595,000, respectively.

Selection of Distress Survey Section

The survey sections were selected to minimize variations in cut and fill within and between sections, and to minimize inclusion of horizontal and vertical curves. Although these potential sources of variability were not totally eliminated from this project, the selected survey sections were believed to be representative of the RCA and control sections. The recycled concrete survey section began at milepoint 90.9 (station 2924+35) and extended approximately 305 m (1,000 ft) westward. The beginning of the control section was located at milepoint 87.0 (station 2760+88) and also extended approximately 305 m (1,000 ft) westward.

Drainage Survey

Both sections contain longitudinal edge drains with outlets spaced every 150 m (500 ft); the drain systems appear to be functioning properly. Signs of pumping, such as accumulations of water or fines along the joints, were not observed on either section. Both sections feature crowned cross sections with transverse slopes of approximately 1.5 percent on the traffic lanes and approximately 3.5 percent on the shoulders.

Pavement Distress Survey

The pavement condition survey was performed over the recycled and control survey sections. A complete summary of the survey results is provided in tables 84 and 85 of appendix A, and a summary of the average results for key distress and performance variables is presented in table 23. The results of the distress survey indicate that the recycled and control sections are performing comparably. It is important to keep in mind, however, that these pavement sections were only 6 years old at the time of the survey and had been subjected to only about 3.7 million ESAL applications.

Table 23. Summary of performance data (average values) for MN 1.

Performance Measurement	Recycled	Control
Corner Faulting, mm (Manual)	0.5	0.5
Wheel Path Faulting, mm (Manual)	0.5	0.3
Wheel Path Faulting, mm (Digital)	0.5	0.5
Deteriorated Transverse Cracks/km	3.2	0.0
Cracks/km	3.2	0.0
Longitudinal Cracking, m/km	0	0
Transverse Joint Spalling, % Joints	49	41
Joint Width, mm	10	9
PSR	3.9	4.0

Transverse Joint Faulting

Transverse joint faulting was measured in the driving lane outer wheel path and at the panel corners closest to the outer shoulder of the recycled and control sections. No significant difference in faulting was measured between the two sections, with average faulting levels of about 0.5 mm (0.02 in) at each location on either project. These low faulting levels indicate that little pumping has taken place on this project, suggesting one or more of the following: good transverse joint load transfer exists through a combination of dowel bars and aggregate interlock; foundation drainage is adequate, or the foundation and base materials are resistant to erosion and pumping.

Transverse Cracking

At the time of the survey, transverse cracking was not a problem on either section. Only one transverse crack was observed on the recycled section. This crack was a medium-severity crack with about 3.8 mm (0.15 in) of faulting. The one crack on the test section corresponds to about 3 cracks per kilometer (5 cracks per mile). Transverse cracks were not observed within the control section. However, the sections were only 6 years old and had been exposed to only 3.7 million ESAL applications, so additional cracking may develop as with increasing traffic and age.

Low-severity cracking is generally expected in JRCP, and these sections are no exception since the L/ℓ ratio is 7.3 for both the recycled and control sections (computed using the average laboratory-determined value of the concrete elastic modulus and the average backcalculated subgrade modulus). These values are far in excess of the threshold value of 5.0 that is often considered the limit for preventing uncontrolled cracking.

Longitudinal Cracking

No longitudinal cracks were observed on either section.

Transverse Joint Spalling

Low-severity joint spalling was observed along 49 and 41 percent of the transverse joints in the recycled and control sections, respectively. This spalling does not significantly affect the ride quality of the two sections (see below). Medium- and high-severity joint spalling was not observed anywhere in either survey sections.

Present Serviceability Rating (PSR)

The project survey team estimated the average PSR values of the recycled and control sections at 3.9 and 4.0, respectively. The very slight difference between these two values might be attributable to the slightly greater joint spalling and wheel path faulting on the recycled concrete section, but the PSR estimates for the two sections are not significantly different. Both sections can be considered to be performing well so far.

FWD Testing

Pavement deflection testing was performed using a Dynatest model 8081 FWD. The typical project test pattern included 5 slab centers, 10 transverse joints (testing on both the approach and leave sides of each joint), 10 transverse cracks (testing on both the approach and leave sides of each crack), and 10 panel edges at midpanel adjacent to the outer shoulder. However, only one transverse crack was present on the recycled section and the control did not exhibit any transverse cracks, so only one crack was tested.

FWD testing was used to estimate pavement material properties (PCC elastic modulus and subgrade modulus of support or "k-value"), load transfer efficiencies across joints and cracks, and loss of support. A summary of the average values for these parameters is provided in table 24 and test results are discussed below.

Table 24. Deflection testing results for MN 1.

Property	Recycled	Control
Elastic Modulus, GPa	42.1	52.2
k-value, kPa/mm	36.7	36.7
Joint Load Transfer, %	91	91
Crack Load Transfer, %	75	n/a
Average Midslab Deflection, µm	87	85
Average Edge Deflection, µm	142	107
Corners With Voids, %	0	0
Maximum Air Temperature During Testing, °C	23	27

PCC Elastic Modulus

The elastic modulus (E) of the concrete slab was backcalculated using the center-of-slab deflection measurements. Figure 32 presents a plot of the concrete elastic modulus at five different locations along the recycled section (4 load tests per location). Backcalculated elastic modulus values range from 36 to 49 GPa (5,200,000 to 7,100,000 lbf/in^2) with an average of 42.1 GPa (6,100,000 lbf/in^2). These values were generally slightly higher than the results of dynamic tests of elastic modulus performed on cores, which averaged 36.2 GPa (5,250,000 lbf/in^2), as discussed below. The results of multiple tests at each location are consistent, although there is some variation in results obtained at the different locations.

A similar plot of backcalculated concrete modulus values for the control section is presented in figure 33. These values range from 43 to 59 GPa (6,200,000 to 8,600,000 lbf/in^2) and average 52.2 GPa (7,570,000 lbf/in^2). These values were also generally higher than the results of laboratory-based dynamic tests of cores from the section, which averaged 41.0 GPa (5,950,000 lbf/in^2). These test results were more variable (between tests at a given location and between different locations) than those obtained from the recycled section, but were still considered reasonable.

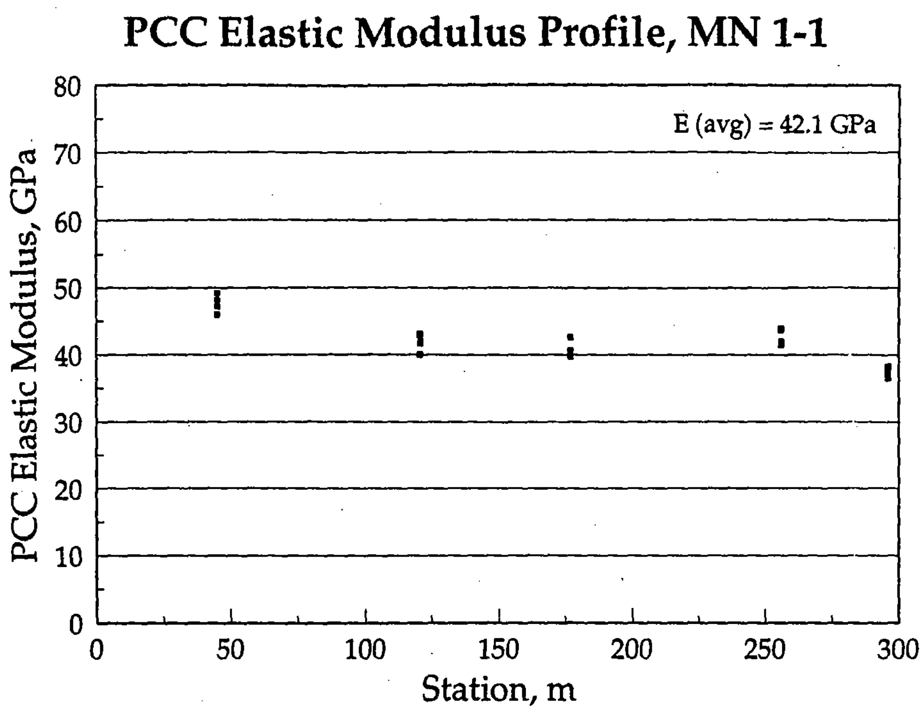


Figure 32. PCC elastic modulus profile for MN 1-1 (recycled section).

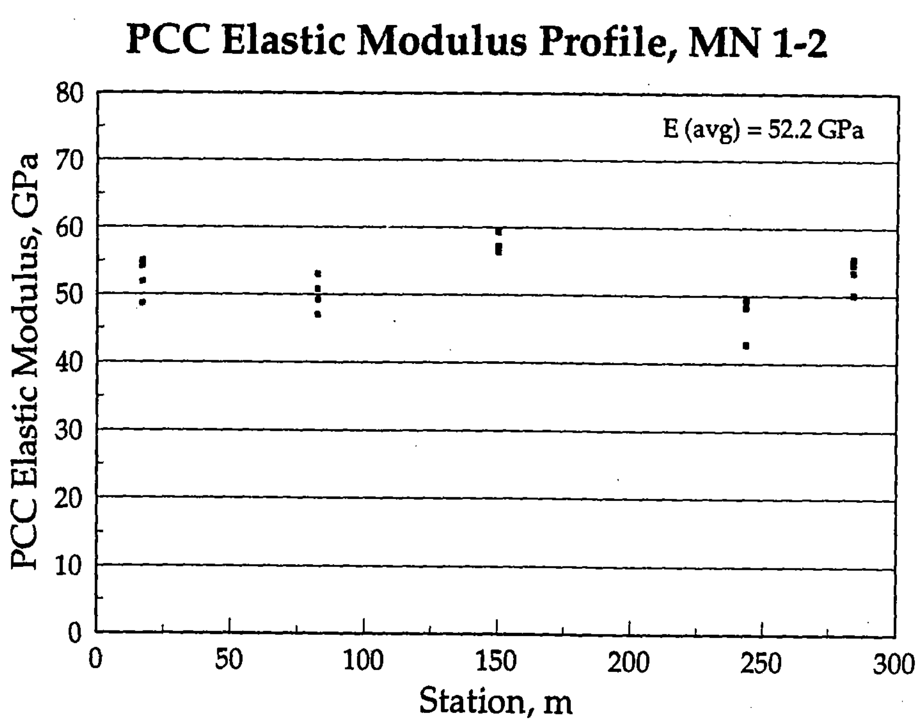


Figure 33. PCC elastic modulus profile for MN 1-2 (control section).

The average backcalculated elastic modulus of the control section concrete was 24 percent higher than that of the recycled section. This difference is consistent with the results of previous studies, which have found that the elastic modulus of conventional aggregate concrete is typically 20 to 40 percent higher than the elastic modulus of recycled aggregate concrete when all other mix design and curing parameters are held constant.[15] The lack of verifiable mix design data for the control section makes it difficult to examine the specific causes for these differences on this project. However, it is generally believed that the inclusion of increased quantities of relatively soft mortar and decreased quantities of hard natural aggregates in recycled concrete mixtures gives these materials a lower modulus of elasticity than conventional concrete mixtures.

Modulus of Subgrade Reaction (k-value)

The modulus of subgrade reaction (k) was also backcalculated using the center-of-slab deflection measurements. Figure 34 presents a plot of the modulus of subgrade reaction at five different locations along the recycled section (four load tests per location). Backcalculated subgrade modulus values range from 34 to 39 kPa/mm (124 to 144 lbf/in^2/in) with an average of 37 kPa/mm (135 lbf/in^2/in). A similar profile plot for the control section is illustrated in figure 35. These values range between 33 and 43 kPa/mm (122 and 158 lbf/in^2/in) with an average of 37 kPa/mm (135 lbf/in^2/in), the same as on the recycled section.

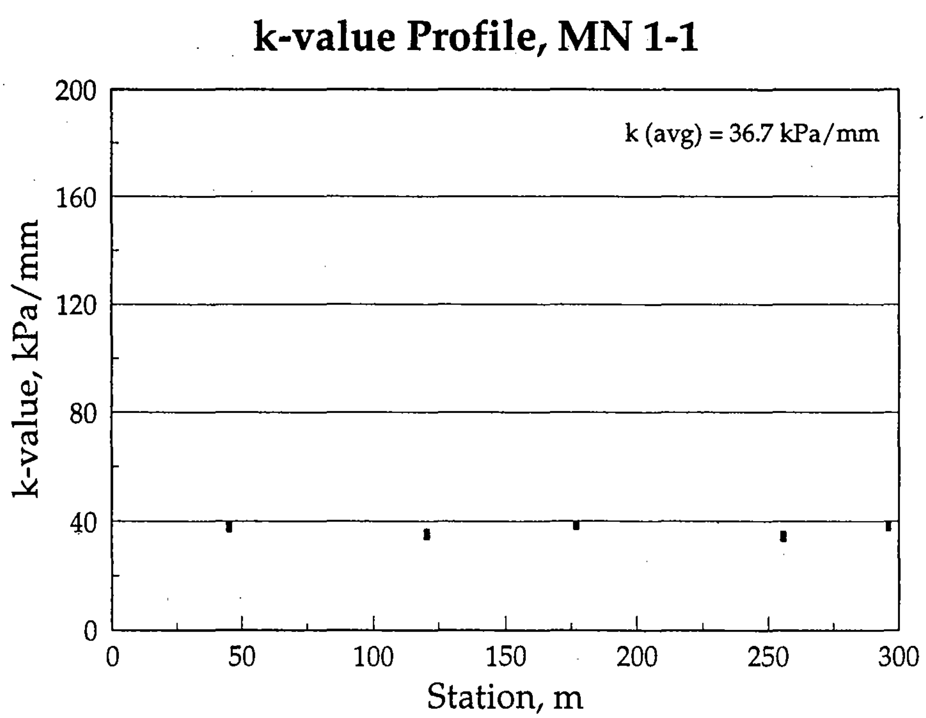


Figure 34. K-value profile for MN 1-1 (recycled section).

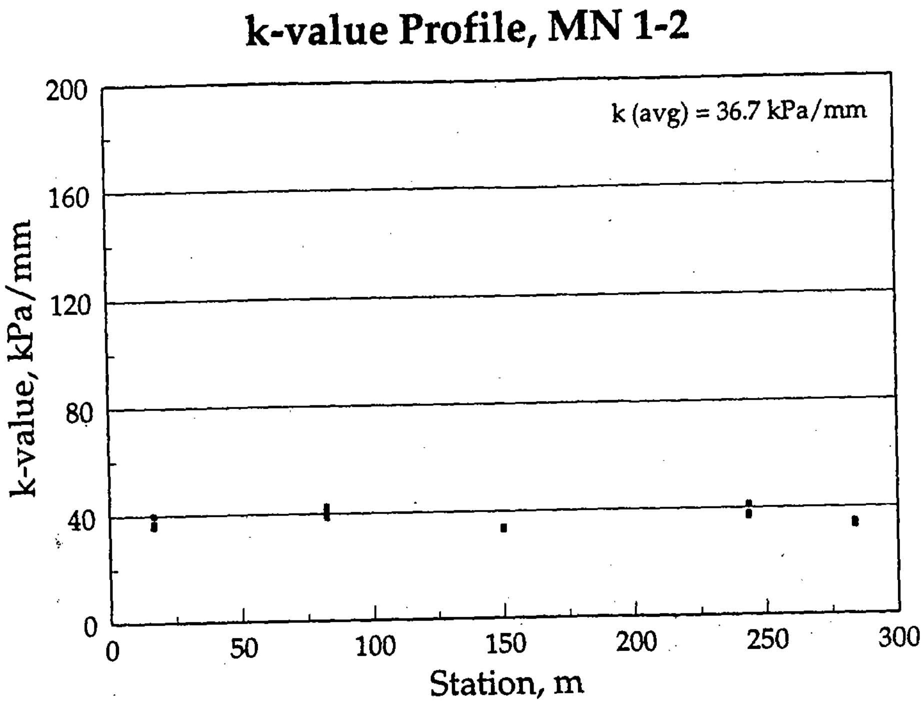


Figure 35. K-value profile for MN 1-2 (control section).

The profile plots for the recycled and control sections are nearly identical, with both sections exhibiting identical average subgrade modulus values. These similarities are not surprising because the two sections are located across the median from each other and the only known differences between them are in the concrete materials and mix design, factors which do not affect the subgrade modulus.

Joint Load Transfer

The load transfer efficiencies measured with the load placed on both the approach and leave sides of the recycled section transverse joints are shown in figure 36. These values are computed as the ratio of the deflection on the unloaded side of the joint to the deflection on the loaded side of the joint. The average deflection load transfer efficiency for this section is 91 percent, with little difference observed between the average load transfer efficiencies were measured with the load placed on the approach or leave side of the joints (90 and 92 percent, respectively). However, significant differences between the approach and leave load transfer efficiencies *were* measured at a few joints, where the approach value was less than the leave value. There was also some variation in load transfer efficiency between joints, although almost all of the joints tested exhibited load transfer efficiencies greater than 80 percent.

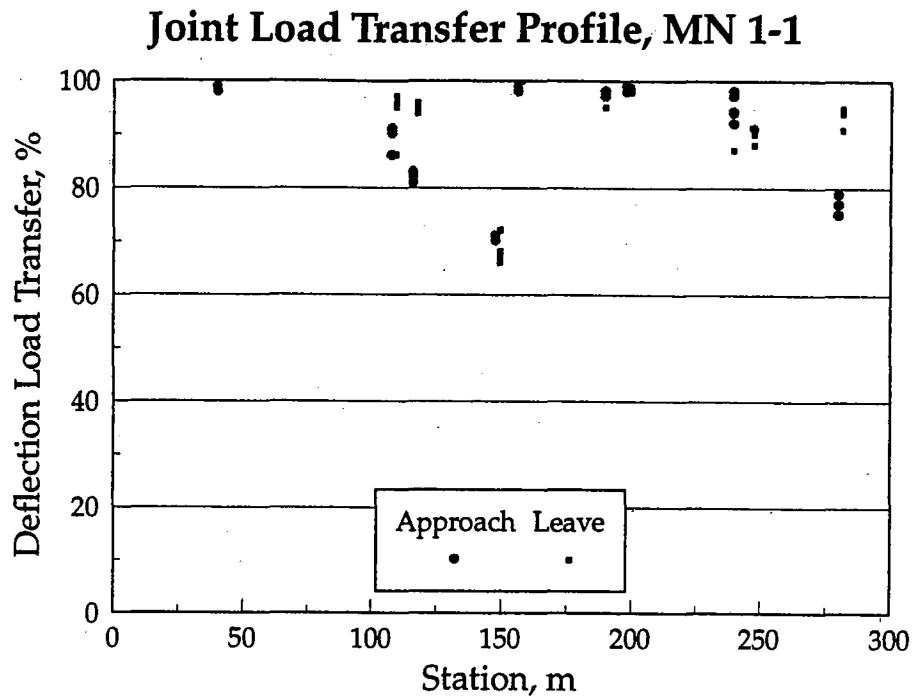


Figure 36. Joint load transfer profile for MN 1-1 (recycled section).

Figure 37 illustrates the joint load transverse efficiencies for the control section. The average load transfer efficiency is 91 percent, with values measured on the approach and leave sides of the joints averaging 88 and 94 percent, respectively. Again, significant differences between the approach and leave load transfer efficiencies *were* measured at a few joints, where the approach value was less than the leave value. However, the variation between in load transfer efficiency between test locations was generally not significant and only a few tests produced values lower than 80 percent.

In summary, the recycled and control sections are currently exhibiting about the same level of load transfer and are performing well. Both sections are equipped with 32-mm (1.25-in) dowel bars, which might, therefore, be considered adequate for these sections on the basis of their performance to date. However, the sections are only 6 years old and have been subjected to only about 3.7 million ESAL applications. A re-evaluation in another 5 years may show some degradation in the overall performance and a difference between the sections. Currently, these sections are both performing well and there is no evidence of any difference between the two at this time.

Crack Load Transfer

Only one crack was present on the recycled section and none were observed in the control section. The lone crack was rated at medium severity and had developed 3.8 mm (0.15 in) of faulting. The average load transfer efficiency at that crack was 75

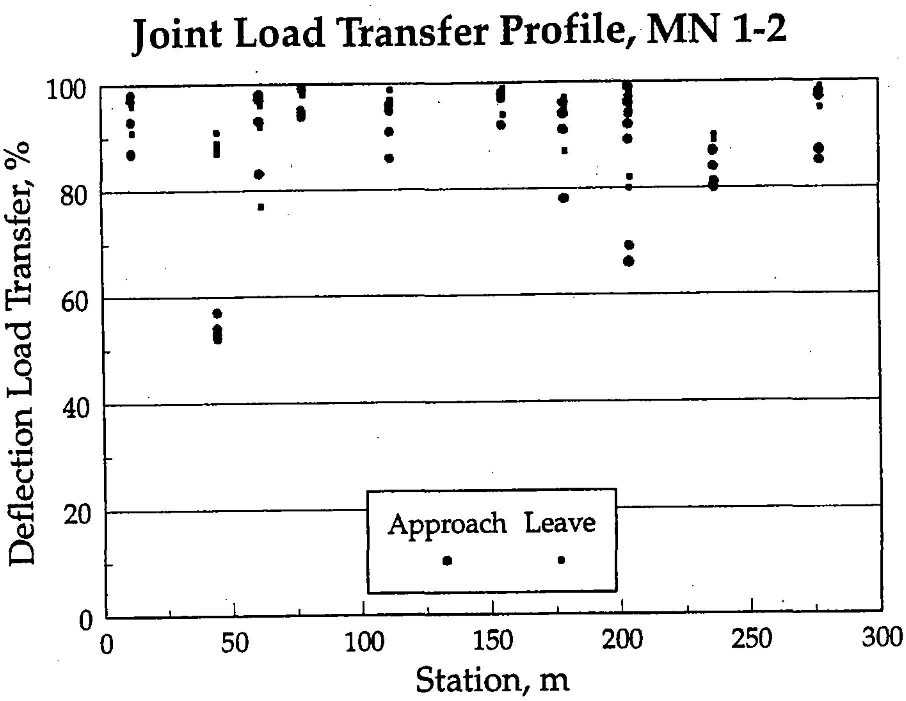


Figure 37. Joint load transfer profile for MN 1-2 (control section).

percent, varying from 68 percent when the load was placed on the approach side of the joint to 83 percent with leave side placement. Although the measured values of load transfer might be considered fair, they are clearly low enough to have produced some faulting under the traffic, foundation and moisture conditions present at this test site.

Loss of Support

The detection of voids was performed data using the corner deflections on the leave side of transverse joints and cracks and procedures described in the final report for NCHRP 1-21. Figures 38 and 39 illustrate the potential for loss of support along the recycled and control sections, respectively. Neither section shows significant potential for loss of support at transverse joints or cracks. These results are consistent with the lack of observed pumping and significant faulting throughout the sections.

Coring

The coring plan called for 11 cores to be taken from both the recycled and control sections: 5 at midpanel, 3 at transverse joints, and 3 at transverse cracks. However, only one transverse crack was present on the recycled section, and no transverse cracks were present on the control section. Thus, only nine cores were retrieved from the recycled section, and only eight were taken from the control section. On both sections, the four midpanel cores intended for strength and elastic modulus tests were 100 mm

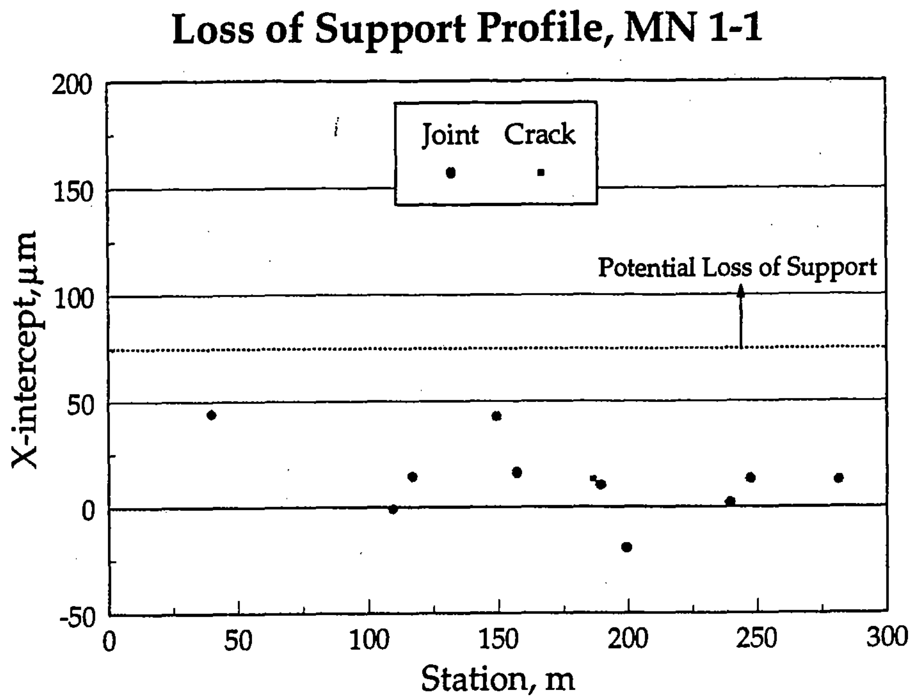


Figure 38. Loss of support profile for MN 1-1 (recycled section).

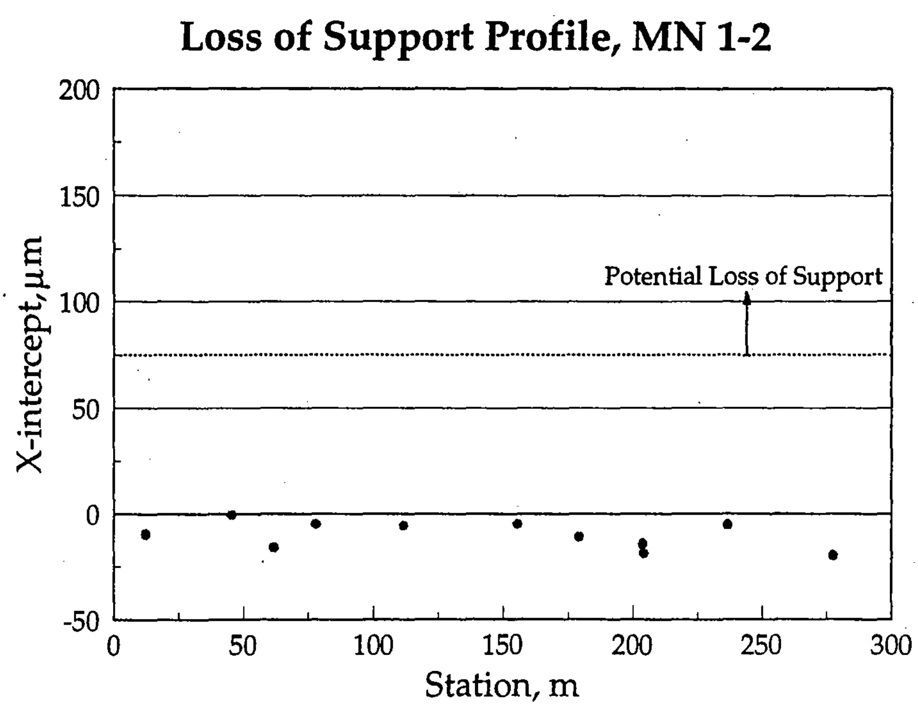


Figure 39. Loss of support profile for MN 1-2 (control section).

(4 in) in diameter; all other cores were 150 mm (6 in) in diameter. The average thickness of the cores retrieved from the recycled and control sections were 290 and 280 mm (11.4 and 11.0 in), respectively, which compares favorably with the nominal design thickness of 280 mm (11.0 in). These cores were tested in the laboratory to determine the physical and mechanical properties of the two concrete mixtures used on this project, as described in more detail below.

Core Testing

The number of cores for each laboratory test is indicated in table 25. A summary of the average values that were obtained during the laboratory testing of the field cores is presented below in table 26 and in table 83 of appendix A. Observations made during the testing and comparisons between the performance of the control and recycled sections are also provided below.

Table 25. Number of cores for each laboratory test in MN 1.

Laboratory Tests	Recycled Section	Control Section
Thermal Coefficient	3	3
Split Tensile Strength	1	1
Dynamic Modulus of Elasticity	4	3
Static Modulus of Elasticity	1	1
Compressive Strength	3	3
Volumetric Surface Texture	4	3

Table 26. Core testing results for MN 1.

Property	Recycled	Control
Compressive Strength, MPa	47.3	46.5
Split Tensile Strength, MPa	3.9	4.6
Dynamic Elastic Modulus, GPa	36.2	41.0
Static Elastic Modulus, GPa	31.4	32.1
Thermal Coefficient, (1×10^{-6})/ °C	11.2	11.3
VSTR (for Failed Split Tensile Core), cm^3/cm^2	0.2487	0.3805
VSTR (for Slab Faces at the Joints), cm^3/cm^2	0.2586	0.2766
VSTR (for Slab Faces at the Cracks), cm^3/cm^2	0.6043	n/a

Petrographic Examination Summary

The coarse aggregate for the recycled section contains angular and rounded gravel rock particles that were observed to be evenly distributed throughout the cement paste. The gravel rock is further characterized as original coarse aggregate containing igneous and sedimentary particles that are predominately carbonate. The coarse aggregate for the control section contain angular crushed diorite that was also observed to be evenly distributed throughout the cement paste. A Class C fly ash was included in both the recycled and control concrete mixtures. The new mortar contents of both the recycled and control materials were estimated using linear traverse techniques and were found to be comparable (about 65 percent); however, the RCA concrete also contained an additional 12 percent old mortar. This is reflected in the difference in natural coarse aggregate content, which was estimated (using linear traverse techniques) at 23 and 34 percent for the RCA and control sections, respectively. These data are summarized in table 27.

Table 27. Coarse aggregate and mortar contents for MN 1.

	Recycled	Control
Coarse Aggregate, %	23.3	34.2
New Mortar, %	65.0	65.8
Recycled Mortar, %	11.7	n/a

Uranyl acetate testing of cores obtained from the eastbound lanes indicate the presence of minor amounts of silica gel in the mortar and around some of the aggregate particles. While this may indicate the presence of alkali-silica reactivity, no such distress was identified during the field surveys.

Mid-Panel Cores

The compressive strengths of the RCA concrete cores ranged between 46.5 and 48.5 MPa (6,740 and 7,030 lbf/in^2), with an average of 47.3 MPa (6,860 lbf/in^2). Compressive strengths for the control section cores ranged between 45.0 and 49.0 MPa (6,530 and 7,100 lbf/in^2), averaging 46.5 MPa (6,740 lbf/in^2). Diametral or split cylinder tensile testing was performed on only one core from each section; strengths of 3.9 and 4.6 MPa (570 and 670 lbf/in^2) were obtained for the recycled and control sections respectively. While the RCA concrete exhibited slightly higher compressive strengths and a slightly lower tensile strength than the control section concrete, the differences cannot be shown to be statistically significant.

The dynamic elastic modulus for the RCA concrete cores ranged from 35.1 to 38.0 GPa (5,090,000 to 5,510,000 lbf/in^2), with an average of 36.2 GPa (5,250,000 lbf/in^2). Control section values ranged from 40.3 to 42.5 GPa (5,840,000 to 6,160,000 lbf/in^2), with an average of 41.0 GPa (5,950,000 lbf/in^2). The static elastic moduli for these sections were estimated using one core from each section; the elastic moduli of RCA concrete and control concrete cores were 31.4 and 32.1 GPa (4,550,000 and 4,650,000 lbf/in^2), respectively. Thus, the results of the dynamic testing suggest that the use of the RCA aggregate resulted in the production of a lower modulus concrete than was obtained using concrete that included only natural coarse aggregate. This conclusion is neither supported or disproved by the results of the static elastic modulus tests, which are based on only one test per survey section.

The thermal coefficient of expansion ranged from 10.6 x 10^{-6}/ °C to 12.0 x 10^{-6}/ °C (5.9 x 10^{-6}/ °F to 6.7 x 10^{-6}/ °F) for the recycled section, with an average of 11.2 x 10^{-6}/ °C (6.2 x 10^{-6}/ °F). The control section thermal coefficients ranged from 10.9 x 10^{-6}/ °C to 11.9 x 10^{-6}/ °C (6.1 x 10^{-6}/ °F to 6.6 x 10^{-6}/ °F) for the control section, with an average of 11.3 x 10^{-6}/ °C (6.3 x 10^{-6}/ °F). The higher total mortar content of RCA concrete would have been expected to produce significantly higher thermal expansion coefficients; this was not the case for the samples that were obtained for this project, however. It is possible that the effects of mortar content were offset by differences in the thermal expansion coefficients and restraining effects of the natural aggregate included in the RCA and control sections. It is also possible that the thermal characteristics of the mortar were not sufficiently different from those of the coarse aggregate particles to produce significant changes in thermal expansion for the two concrete samples. Additional testing would be required to investigate these issues more completely.

In general, the laboratory tests of concrete strength, elasticity and thermal coefficient of expansion showed little difference in the physical properties of the two concrete mix designs used on this project. It appears that the RCA concrete may have a slightly lower elastic modulus and lower tensile strength than the control section concrete. However, most test results indicated that the properties of the concrete in both sections are in the range of values expected for typical paving concrete.

Joint and Crack Cores

Volumetric surface texture ratios (VSTR's) obtained for cores retrieved from joints in the RCA and control sections are approximately the same (0.2586 cm^3/cm^2 vs. 0.2766 cm^3/cm^2). While this difference is probably not statistically significant, a reduced VSTR would be expected for the RCA section because the higher mortar content of the RCA concrete (11 percent higher, see table 27) would be associated with fewer coarse aggregate particles and, therefore, a straighter fracture plane. In any event, all joints on these sections included steel dowels that appear to be functioning adequately (see previous discussion of FWD test results), so the contribution of fracture plane surface texture is minimal to joint load transfer on this project.

The VSTR obtained for the core pulled from the lone crack in the recycled section was much higher than the average obtained for transverse joints on the same project ($0.6043\ cm^3/cm^2$ vs. $0.2586\ cm^3/cm^2$). Some of this difference in texture can be attributed to a difference in aggregate particle sizes, which ranged between 13 and 19 mm (0.5 and 0.75 in) at the crack while those measured at the fractured face of the joints had a top size of approximately 13 mm (0.5 in). However, the crack did meander somewhat vertically through this single core, and the test results are skewed accordingly.

Slight indications of dowel-concrete bearing failure were observed on the bottom and one side of the dowel bar included in one of the joint cores retrieved from the RCA concrete section. In addition, a very small amount of corrosion was found on the bottom side of the dowel in one core retrieved from the control section, although none was present on the dowel retrieved from the recycled section. Corrosion was observed on the longitudinal steel found in the core pulled from the crack in the recycled section.

In summary, no significant mechanical differences were observed between the cores retrieved from the RCA concrete and control sections. The RCA section should be monitored for future evidence of dowel-concrete bearing failure.

Project Summary

This project provides a direct comparison of the performances of recycled concrete and traditional concrete pavement sections constructed in 1988 using identical structural designs (280-mm [11-in] JRCP with an effective steel content of 0.054 percent; widened outer lanes, 8.2-m [27-ft] transverse joint spacing; 32-mm [1.25-in] epoxy-coated dowel bars; longitudinal edge drains) and subjected to identical traffic (3.7 million ESAL through 1994) and environmental conditions. The only known difference between the two sections is the type of coarse aggregate used in the two sections. The recycled section contains 19-mm (0.75-in) top size, recycled concrete aggregate produced from the pre-existing 28-year-old concrete pavement. The control section contains only natural concrete aggregate. Both sections contain a natural sand as the fine aggregate. Unfortunately, detailed information regarding the control section aggregate properties and mix design are not available.

The results of a condition survey, deflection testing and laboratory tests on retrieved cores indicate the recycled and control sections are constructed of similar materials and are exhibiting similar performances. A summary of the key results follows:

Pavement Design

The absence of significant slab cracking and joint faulting suggests that the pavement design used has been adequate for the load and environmental conditions experienced thus far. However, the relatively long panel length is expected to result in the eventual development of transverse cracks, which may deteriorate rapidly (as did the lone crack in the RCA concrete section) due to the low steel content (0.054 percent longitudinal steel by area of concrete). The steel design used on this project would be considered inadequate by most pavement design engineers.

There was slight evidence of dowel-concrete bearing failure around the only dowel retrieved from the RCA pavement section. This section should be monitored for continued deterioration in the future.

Material Properties

Backcalculated elastic modulus values averaged 24 percent lower in the RCA concrete sections than in the control section. Average backcalculated moduli of subgrade support were the same for both sections (37 kPa/mm [135 $lbf/in^2/in$]).

In general, the laboratory tests of concrete compressive strength, static modulus of elasticity, and thermal coefficient of expansion showed little difference in the physical properties of the two concrete mix designs used on this project. It appears that the RCA concrete may have a slightly reduced dynamic elastic modulus (13 percent lower) and tensile strength (18 percent lower) when compared with the control section concrete. These reduced values may be attributable to the higher total mortar content of the RCA concrete (12 percent more than in the control concrete). However, most test results indicated that the properties of the concrete in both sections are in the range of values expected for typical paving concrete.

It was noted that the backcalculated dynamic modulus values were generally higher than those obtained through laboratory testing, which were, in turn, higher than the statistically-determined values. This trend is consistent with the results obtained on the other test sections.

The surface texture of the fractured faces of the cores pulled at the joints in the recycled section was comparable to that of the cores retrieved from joints in the control section in spite of the lower natural aggregate content of the RCA concrete. The VSTR obtained for the core pulled at the crack in the recycled section was significantly higher than the values obtained for joint cores on either section. This was attributed mainly to the meander of the crack; the actual texture of the fractured surface was not considered to be significantly different than that of the joint surfaces.

Uranyl acetate testing revealed minor amounts of gel deposits in the mortar and around some of the aggregate particles in both sections, possibly indicating the presence of ASR. ASR-related distress was not observed, however.

Pavement Performance

Both pavement sections appear to be performing well so far, as evidenced by the following performance measures:

- Only one transverse crack was observed in the RCA concrete section (medium severity) and none were found in the control section. Longitudinal cracking was not observed in either section.
- Some spalling at the transverse joints was noticed, but only at low severity levels. The number of spalled joints in the recycled section was slightly lower than in the control section.
- Neither section exhibited significant transverse joint faulting.
- The survey team estimated PSR values of 3.9 and 4.0 for the RCA and control sections, respectively, indicating good serviceability.

Very high average joint load transfer efficiencies (about 91 percent) were measured in both the recycled and control sections. These high values can be attributed to the presence and continued proper function of the dowel load transfer system.

The sole crack in the recycled section had a load transfer efficiency of 75 percent. This relatively low load transfer efficiency can be attributed to the crack width. The load transfer is low even though the crack face VSTR is high because the longitudinal steel has ruptured, allowing the crack to open.

Overall

The findings of this study suggest that the subject pavement sections are currently performing comparably and can be considered adequate in almost all respects. The apparent reduced tensile strength and elastic modulus of the RCA concrete does not seem to have adversely affected performance so far. However, these pavements were only about 6 years old at the time of survey, and very little of the expected transverse panel cracking has appeared. In addition, there may be evidence of dowel-concrete bearing failures in the recycled concrete section. Additional performance monitoring is necessary to determine the long-term performance records of each pavement section.

Minnesota 2, I-90 near Beaver Creek

This project is the second of three Minnesota JRCP sections included in this study that were constructed using RCA concrete. Recycled concrete was used in the construction of both the eastbound and westbound travel lane surfaces. A control section was not constructed at this project, so direct comparisons of performance cannot be made to identify the effects of using recycled PCC aggregate. However, general observations were made and performance hypotheses were formulated on the basis of available design, construction and performance data.

<u>Project Information</u>

This project consists of approximately 6.4 km (4.0 mi) of RCA concrete pavement in the eastbound and westbound lanes of I-90 in Rock County. The project extends from the Minnesota-South Dakota border to near Beaver Creek, Minnesota. The original pavement was constructed in 1964 and subsequently developed severe D-cracking. In 1984, the 20-year-old D-cracked pavement was removed and crushed for use as coarse aggregate in the portland cement concrete layer of the reconstructed pavement.

Design Information

A new 230-mm (9-in) JRCP pavement was constructed along the original project alignment in 1984. The concrete slab was placed on a 76-mm (3-in) aggregate base and a 150-mm (6-in) aggregate subbase, all built over an AASHTO class A-1-a subgrade. Skewed transverse joints were cut at 8.2-m (27-ft) intervals, and 25-mm (1-in) epoxy-coated dowel bars were included at these joints to provide load transfer. The joints were sealed with preformed joint seals.

The outer traffic lane (driving lane) was constructed 4.3 m (14 ft) wide with rumble strips included within the outer 0.6 m (2 ft) to alert traffic that unintentionally wanders from the main 3.7-m (12-ft) travel lane. The inner lane was constructed 3.7 m (12 ft) wide. The outer shoulder extends 2.4 m (8 ft) further and consists of a 50-mm (2-in) AC surface layer over an aggregate base that was stabilized using fines from concrete recycling operation. Longitudinal edge drains are also present, with outlets spaced at 120-m (400-ft) intervals.

Slab reinforcement consists of an uncoated deformed welded wire fabric with 8 mm (0.30 in) diameter longitudinal wires spaced at 310 mm (12 in) center-to-center, resulting in a longitudinal steel content of 0.065 percent of the slab cross-sectional area. Transverse wires are 6 mm (0.23 in) in diameter and are also spaced at 310 mm (12 in) center-to-center. The longitudinal centerline joint is equipped with 760-mm (30-in) long, 13-mm (No. 4) epoxy-coated deformed tie bars spaced 760 mm (30 in) on center.

Mix Design

The mix design used in the construction of these sections is provided in table 28. The aggregate was made up of 58 percent by weight recycled concrete aggregate with a top size of 19 mm (0.75 in) and 42 percent natural sand (60 percent RCA by total aggregate volume). A Type I cement and Class C fly ash were used with sufficient water to produce a water-cementitious (cement plus fly ash) ratio of 0.46 and a water-cement ratio of 0.57. An air-entraining agent was also used to ensure the freeze-thaw durability of the mortar portion of the new pavement surface. The resulting plastic mix had an average air content of 5.5 percent and an average slump of 38 mm (1.5 in).

Construction Information

The original concrete pavement was removed and crushed in 1984 to produce coarse aggregate for use in the new concrete pavement surface. The crushed concrete fines were used as a stabilizing agent for the aggregate base under the shoulders. The paving process was followed by surface texturing, which included an astroturf drag and transverse tining. A curing compound was also applied to the pavement surface.

Table 28. Mix designs for MN 2.

Material	Recycled
Coarse Aggregate	979 kg/m^3
Fine Aggregate	701 kg/m^3
Cement	282 kg/m^3
Fly Ash	66 kg/m^3
Water	160 kg/m^3
w/(c+p) ratio	0.46

Climatic Conditions

This project is situated in the transition zone between the dry-freeze and wet-freeze environmental regions. The minimum and maximum average monthly temperatures are –9 and 23 °C (15 and 74 °F). The area experiences about 96 freeze-thaw cycles annually, and the freezing index is 720 °C-days (1300 °F-days). The Thornthwaite moisture index is 5, which reflects an average of 100 days of precipitation per year totaling 610 mm (24 in).

Traffic Loadings

The reconstructed pavement was opened to traffic in 1984 and, through the survey date in 1994, had been exposed to an estimated 7.8 million ESAL applications. In 1984, the two-way ADT was estimated at 16,800 vehicles per day. Using an estimated growth rate of 2.5 percent, the two-way ADT in 1994 would be 21,500 vehicles per day. An estimated 22 percent of this traffic are heavy trucks. The corresponding ESAL applications in the opening year (1984) and survey year (1994) are estimated at 573,000 and 872,000, respectively.

Selection of Distress Survey Section

The approved research work plan for this study called for the evaluation of only one recycled concrete pavement section at this site, since no control section was known to exist. However, initial condition surveys performed at the project site found that the eastbound lanes appeared to be exhibiting significantly more deteriorated transverse cracking than the westbound lanes. As a result, condition and distress surveys were conducted on two sections, one in each direction. However, coring and FWD testing were performed only in the eastbound lanes.

The eastbound and westbound sections were located adjacent to each other. The section in the eastbound lanes began at station 90+00 (near milepost 1.7) and extended approximately 305 m (1000 ft) eastward. The section in the westbound lanes began at station 100+05 (near milepost 1.9) and extended 305 m (1000 ft) to the west.

Drainage Survey

Both surveyed sections were cut into the existing terrain. The amount of cut (vertical distance from the top of the surrounding terrain to the pavement surface) varied from 1.8 to 6.1 m (6 to 20 ft). The ditches were approximately 1.2 m (4 ft) deep. Longitudinal edge drains, with outlets spaced at 120-m (400-ft) intervals, were provided and appeared to be functioning properly. Pumping of fines or moisture were not observed on either section. Both sections employ a crowned cross section, with transverse slopes varying from 1 percent within the traffic lanes to 4 percent on the shoulders.

Pavement Distress Survey

The pavement condition survey was conducted over the selected sections in both the eastbound and westbound lanes. Tables 84 and 85 in appendix A provide detailed summaries of the results of those surveys. Table 29 provides a summary of the average results for several key performance measurements. These tables indicate that both sections are exhibiting good performance and serviceability. However, many transverse cracks on both sections are beginning to breakdown and deteriorate. Faulting at the transverse cracks was also observed.

Table 29. Summary of performance data (average values) for MN 2.

Performance Measurement	Eastbound	Westbound
Corner Faulting, mm (Manual)	1.0	0.5
Wheel Path Faulting, mm (Manual)	0.8	0.3
Wheel Path Faulting, mm (Digital)	0.8	0.5
Deteriorated Transverse Cracks/km	60.6	41.6
Cracks/km	115.0	102.4
Longitudinal Cracking, m/km	0	0
Transverse Joint Spalling, % Joints	21	16
Joint Width, mm	11	11
PSR	4.1	4.3

Transverse Joint Faulting

Measurements of faulting obtained using either the manual or digital faultmeters suggest higher levels of faulting in the eastbound section, although few of the measured faults were large enough to cause a significant decrease in ride quality. For example, eastbound wheel path faults measured with the manual faultmeter averaged 0.8 mm [0.03 in], whereas the westbound faults in the same location averaged 0.3 mm [0.01 in]. There are no readily-apparent explanations for these directional differences in faulting since all known design, traffic, environmental, construction, and material variables are constant between the two sections, although it appears that surface water may flow across the road toward the eastbound lanes on this project, which might help to account for the increased faulting there.

Current faulting levels do not present a problem at this time, as evidenced by the high estimates of serviceability provided by the condition survey team (PSR = 4.1 and 4.3 for the eastbound and westbound sections, respectively) after 10 years of service and exposure to 7.8 million ESAL applications.

Transverse Cracking

Widespread transverse cracking was observed within both survey sections as approximately 84 percent of the panels examined in each section exhibited at least one crack. Low-severity cracking is generally expected in JRCP, and these sections are no exception (L/ℓ ratio is 8.2, using the average laboratory-determined value of the concrete elastic modulus and the average backcalculated subgrade modulus). However, many of these cracks had deteriorated to medium- or high-severity through faulting and spalling.

The deterioration of these cracks can probably be attributed, at least in part, to the extremely low longitudinal steel content (0.06 percent) and the harsh environmental conditions (in terms of both extreme temperatures and liberal use of deicing salts). Some cores taken through deteriorated transverse cracks revealed severe corrosion and rupture of the steel.

The preliminary assertion that the westbound lanes had more deteriorated cracks than the eastbound lanes proved to be untrue within the survey sample sections, which included more deteriorated cracks in the eastbound lanes (61 per km), including several high-severity cracks. The westbound lanes contained many medium-severity transverse cracks (42 per km), but no high-severity cracks were observed. There is no apparent reason for the difference in cracking between the two sections.

Nearly all medium- and high-severity transverse cracks on these sections were faulted. On the eastbound lanes, the average faulting at medium-severity cracks was 2.0 mm (0.08 in), compared to 4.1 mm (0.16 in) at high-severity cracks. The medium-severity cracks on the westbound lanes were faulted an average of 2.5 mm (0.10 in).

The greater crack faulting in the eastbound section probably contributes to the lower ride quality ratings for that lane.

Longitudinal Cracking

No longitudinal cracks were observed within either section.

Transverse Joint Spalling

Spalling of the transverse joints did not present a problem on either section. Spalling was observed at 21 and 16 percent of the transverse joints on the eastbound and westbound sections, respectively. Nearly all spalls were low severity, with 0 and 3 percent medium-severity spalls in the eastbound and westbound sections, respectively.

Present Serviceability Rating (PSR)

As described previously, the eastbound section has slightly higher faulting levels and more deteriorated cracks than the westbound section. Consequently, it also has a lower PSR than the westbound section (4.1 vs. 4.3). These values indicate good performance of the sections in terms of ride quality. However, performance trends to date suggest that existing deteriorated cracks will continue to deteriorate through additional faulting and spalling, and that existing low-severity cracks will begin to deteriorate. The result will probably be a dramatic reduction in serviceability in the near future.

It is also worth noting that, while these pavement sections are currently functionally adequate from a serviceability standpoint, they are approaching structural failure due to the rapid deterioration of the transverse cracks.

<u>FWD Testing</u>

Deflection testing on this section was conducted during early September 1994. The backcalculated values obtained from these deflection data are representative of the material properties that existed at that time under the prevailing environmental conditions. Tests performed on this section at other times under different conditions might be expected to vary significantly.

FWD testing was performed only on the eastbound evaluation section. The testing pattern included 5 slab centers, 10 transverse joints (approach and leave side), 10 transverse cracks (approach and leave side), and 10 panel edges. FWD testing was used to estimate pavement layer material properties (PCC elastic modulus and modulus of subgrade support or "k-value"), load transfer efficiencies across joints and cracks, and loss of support. Table 30 presents a summary of the average values for these parameters, as computed using the deflection testing data. Additional discussion of each parameter is provided below.

Table 30. Deflection testing results for MN 2.

Property	Eastbound
Elastic Modulus, GPa	47.7
k-value, kPa/mm	34.2
Joint Load Transfer, %	80
Crack Load Transfer, %	67
Average Midslab Deflection, μm	131
Average Edge Deflection, μm	128
Corners With Voids, %	0
Maximum Air Temperature During Testing, °C	22

PCC Elastic Modulus

Midpanel deflection measurements were used to backcalculate the elastic modulus (E) of the recycled concrete slab. Deflection testing for backcalculation was generally conducted on slabs without transverse cracks. However, some cracks, although not visible at the surface, may have initiated at the bottom of the slab. The presence of these cracks is a possible cause of large variations in test results.

The average backcalculated elastic modulus is 47.7 GPa (6,920,000 lbf/in^2), although considerable variation was observed for various test loads and locations (ranging from 32 to 64 GPa [4,660,000 to 9,370,000 lbf/in^2]), as illustrated in figure 40. It seems unlikely that the PCC modulus of elasticity varies so widely over such a short distance. It is also worth noting that the backcalculated subgrade modulus varies inversely with the backcalculated PCC modulus, as can be seen by comparing figures 40 and 41. Laboratory tests of cores from the eastbound lanes exhibited much less variability and produced an average concrete dynamic elastic modulus of 34.8 GPa (5,050,000 lbf/in^2).

Modulus of Subgrade Reaction (k-value)

A profile plot of the backcalculated k-values is illustrated in figure 41. The average backcalculated value is 34.2 kPa/mm (126 lbf/in^2/in). With the exception of one location (station 127), the backcalculated moduli exhibit little variability. Station 127 is an area of greater cut than the other test locations, which may account for the difference in test results if a different thickness or type of subgrade material is present. As discussed previously, the unusually high subgrade modulus at this station corresponds to an unusually low concrete elastic modulus value, which suggests that the overall

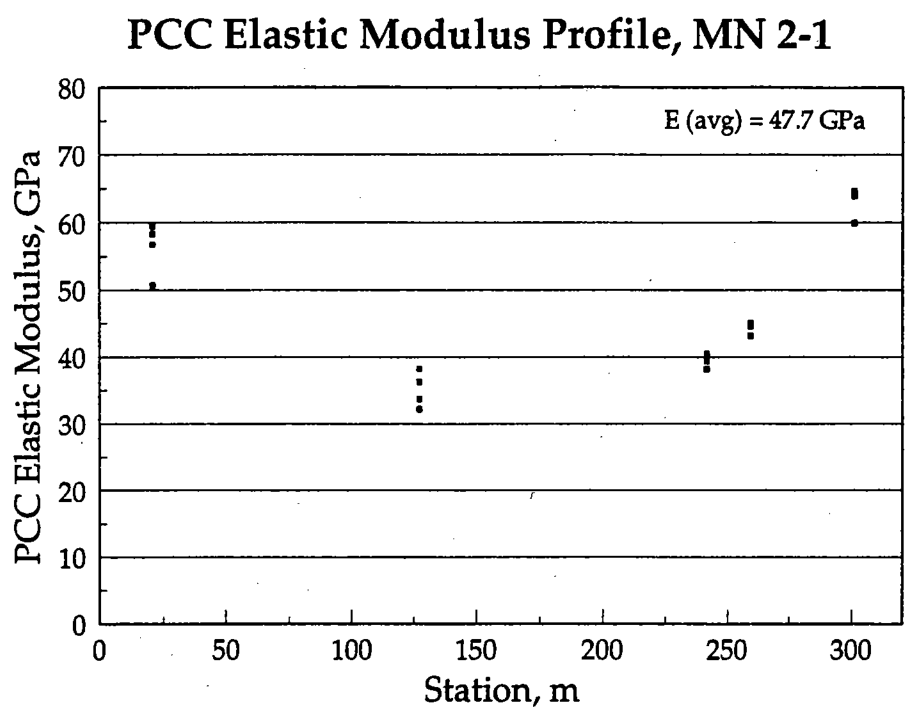


Figure 40. PCC elastic modulus profile for MN 2-1.

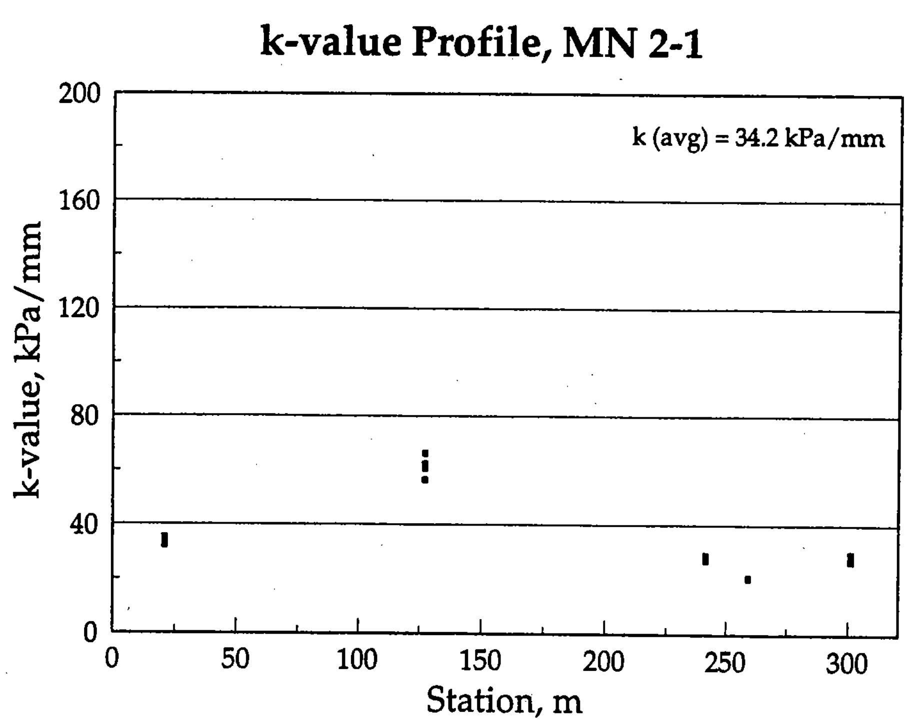


Figure 41. K-value profile for MN 2-1.

pavement structural capacity at this location may not be much different than elsewhere on the project and that the anomaly is a result of an unseen crack in the pavement structure or a flaw in the backcalculation algorithm.

Joint Load Transfer

Figure 42 shows a plot of the transverse joint load transfer efficiencies measured with the load plate placed on both the approach and leave sides of the joints of the eastbound section (MN 2-1). This figure shows that the transverse joints are generally exhibiting good load transfer, with an overall average load transfer efficiency of 80 percent and nearly every joint exhibiting load transfer efficiencies between 70 and 90 percent. Most joints also show close agreement between values obtained with the load placed on either side of the joint, with an average approach side load transfer efficiency of 80 percent and an average leave side load transfer efficiency of 79 percent.

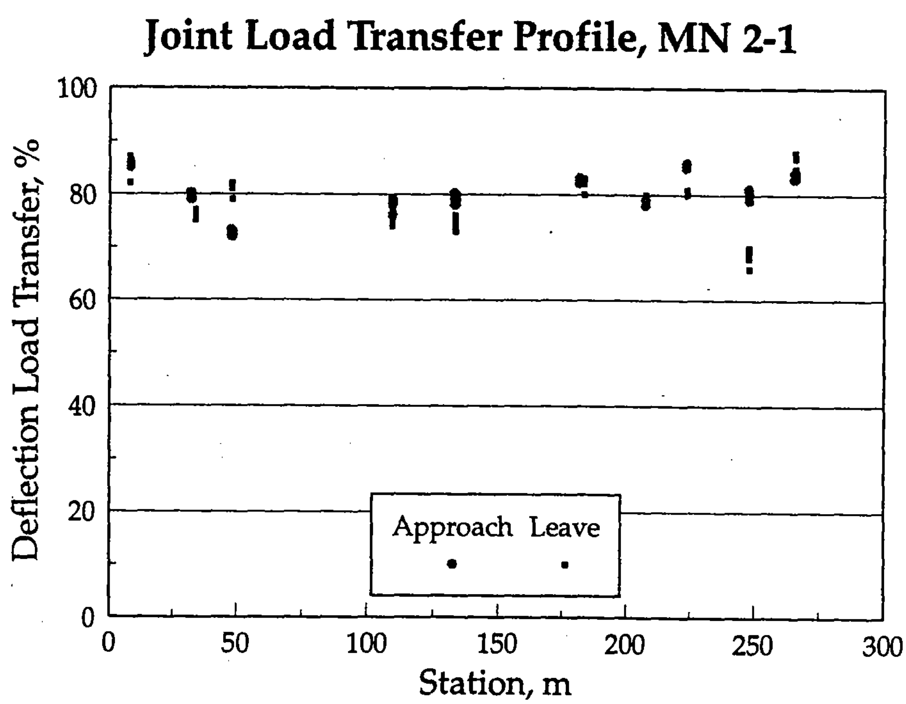


Figure 42. Joint load transfer profile for MN 2-1.

As described previously, these sections include 25-mm (1-in) epoxy-coated dowel bars. Although larger-diameter bars are generally recommended on such high-volume roadways, these dowels appear to have performed effectively to date, as evidenced by the high levels of load transfer and low levels of joint faulting and spalling observed. However, the one dowel that was retrieved in a core exhibited corrosion in the vicinity of the joint. There was also a build-up of an unknown black material between the epoxy coating and the joint, possibly the remnants of some sort of release agent applied to the dowels during construction.

Crack Load Transfer

A profile plot of the deflection load transfer efficiencies at the approach and leave cracks is illustrated in figure 43. Approach side measurements ranged from 41 to 84 percent, and were consistently lower than leave side measurement, which ranged from 60 to 92 percent. Overall average load transfer efficiencies averaged 67 percent, significantly lower than the 80 percent average joint load transfer efficiency.

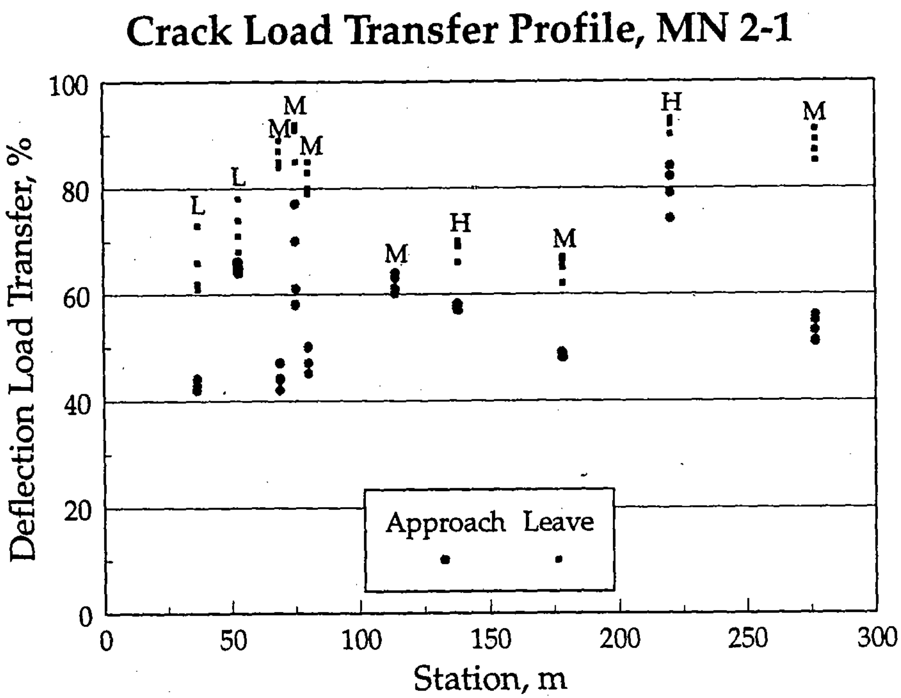


Figure 43. Crack load transfer profile for MN 2-1.

A possible explanation for the difference in load transfer between approach and leave sides of the cracks was found in the one core that contained a dowel. In this case, there were hairline cracks in the concrete along the upper portion of the dowel on the approach side and along the lower portion of the dowel on the leave side. The crack between the two slab faces was relatively tight above and below the dowel, but there was approximately a 9.5 mm (0.375 in) gap at the dowel. Since there was only one core that contained a dowel, it was difficult to ascertain how representative this occurrence may have been for the rest of the section. Refer to figure 44 for an illustration.

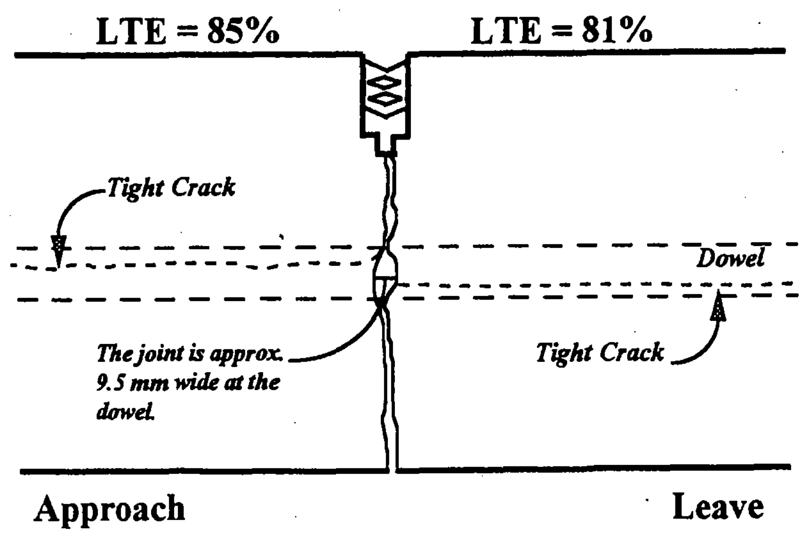


Figure 44. Cracking found along the dowel in the core retrieved at the joint.

It was also noted that all cracks in the cores propagated through the slab on an angle. Cracks which propagated on an incline directed away from the approach traffic tended to show a higher leave LTE than approach LTE. The largest incline observed was approximately 15 degrees, and resulted in an LTE increase of 12 percent for the leave side over the approach side. In addition, spalling at the bottom of the cores extended as much as 95 mm (3.75 in) under the leave side of the crack.

Crack severity is noted in figure 43, and it is apparent that there is no clear correlation between crack severity and load transfer efficiency on this project, as might be expected. For example, the cracks included in three of the retrieved cores had approximately the same crack opening at the top of the pavement. However, the crack width at mid-depth was wider for the cracks where the steel had ruptured. Therefore, even though a visual distress survey would indicate that the cracks were performing equally, the FWD testing would show the cracks with the unruptured steel (and therefore tighter cracks) to have higher LTE.

Loss of Support

Void detection on the leave side of the transverse joints and cracks was performed using the techniques described in the final report for NCHRP Project 1-21 and corner deflection data from this project. Figure 45 shows the loss of support profile plot for MN 2-1, which fails to indicate the presence of any voids under the slab corners. Pumping of moisture or fines at joints and cracks was not evident on this section, which is consistent with the results of the loss of support analysis.

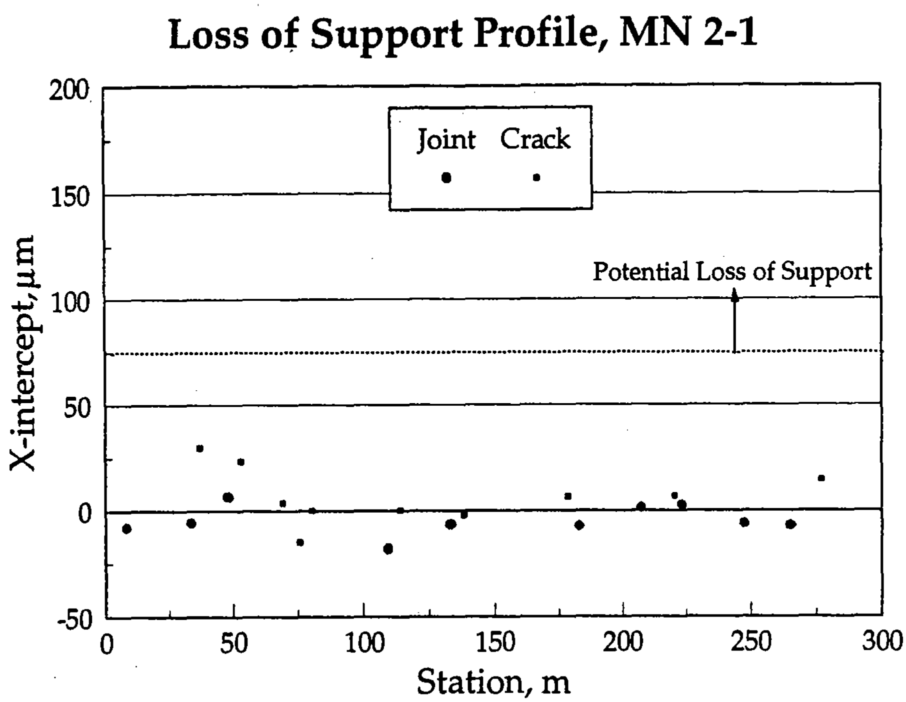


Figure 45. Loss of support profile for MN 2-1.

Core Testing

The number of cores for each laboratory test is indicated in table 31. A summary of the average values that were obtained during the laboratory testing of the field cores is presented below in table 32 (and table 83 in appendix A). Observations made during the testing, and relative comparisons are also provided below.

Table 31. Number of cores for each laboratory test in MN 2.

Laboratory Tests	Recycled Section	Control Section
Thermal Coefficient	3	n/a
Split Tensile Strength	2	n/a
Dynamic Modulus of Elasticity	3	n/a
Static Modulus of Elasticity	2	n/a
Compressive Strength	3	n/a
Volumetric Surface Texture	6	n/a

Table 32. Core testing results for MN 2.

Property	Recycled
Compressive Strength, MPa	39.2
Split Tensile Strength, MPa	4.1
Dynamic Elastic Modulus, GPa	34.8
Static Elastic Modulus, GPa	29.2
Thermal Coefficient, $(1x10^{-6})$/ °C	11.1
VSTR (for Failed Split Tensile Core), cm^3/cm^2	0.2775
VSTR (for Slab Faces at the Joints), cm^3/cm^2	0.2913
VSTR (for Slab Faces at the Cracks), cm^3/cm^2	0.3426

Petrographic Examination Summary

The recycled coarse aggregate used in this project contained rounded to angular gravel rock particles that were observed to be evenly distributed throughout the cement paste. The gravel rock is further characterized as original coarse aggregate containing igneous and sedimentary particles. The mortar and aggregate contents observed in this project were comparable to those observed in samples obtained from other recycled concrete projects (see table 33).

Table 33. Coarse aggregate and mortar contents for MN 2.

	Recycled
Coarse Aggregate, %	20.7
New Mortar, %	75.1
Recycled Mortar, %	4.2

Uranyl acetate testing of cores obtained from the eastbound lanes indicate the presence of minor amounts of silica gel in the mortar and around some of the aggregate particles. While this may indicate the presence of alkali-silica reactivity, no such distress was identified during the field surveys.

Mid-Panel Cores

The compressive strengths ranged from 37.6 to 41.0 MPa (5,450 to 5,950 lbf/in^2), with an average of 39.2 MPa (5,690 lbf/in^2), which is at the low end of the range of test results observed in this study. The split tensile strengths ranged between 3.3 and 4.8 MPa (480 and 700 lbf/in^2), with an average of 4.1 MPa (590 lbf/in^2), which is slightly higher than the average tensile strength observed for other RCA concrete specimens in this study.

The dynamic elastic modulus ranged from 33.7 to 35.6 GPa (4,890,000 to 5,160,000 lbf/in^2), with an average of 34.8 GPa (5,050,000 lbf/in^2), which is near the average for the range of test results observed in this study. As discussed previously, these test results are significantly lower and less variable than the results obtained through backcalculation of FWD data. The static elastic modulus ranged from 27.4 to 30.8 GPa (3,970,000 to 4,470,000 lbf/in^2), with an average of 29.2 GPa (4,230,000 lbf/in^2), which is at the low end of the range of test results observed in this study.

The thermal coefficients ranged from $10.4 \times 10^{-6}/$ °C to $11.7 \times 10^{-6}/$ °C ($5.8 \times 10^{-6}/$ °F to $6.5 \times 10^{-6}/$ °F) with an average of $11.1 \times 10^{-6}/$ °C ($6.2 \times 10^{-6}/$ °F). This average is comparable with values obtained for other RCA concrete specimens included in this study.

In summary, the tensile strength, dynamic modulus of elasticity and thermal coefficient of expansion/contraction of the RCA concrete cores obtained at this field site were typical for the RCA concrete specimens considered in this study. The compressive strength and static modulus of elasticity values obtained were lower than average, however. The lack of a control section at this site makes it difficult to evaluate the effects of these properties on pavement performance at this site.

Joint and Crack Cores

The surface texture of the transverse crack faces was generally greater than that of transverse joints (VSTR of 0.3426 cm^3/cm^2 vs. 0.2913 cm^3/cm^2). Transverse cracks are typically assumed to form or develop some time after construction, when the concrete is relatively strong. The resulting fracture plane often passes through aggregate particles, resulting in reduced surface texture. Conversely, transverse joints are typically formed or cut soon after the concrete is placed, and the resulting fracture plane often passes around aggregate particles, resulting in greater surface textures. In this case, however, the increased surface texture of a typical crack face might be expected to be less than that of a typical joint face because the increased bond strength between aggregate and paste would result in more particle fractures and fewer pullouts, as evidenced by the reduced surface texture of the split tensile specimens (0.2775 cm^3/cm^2). The increased surface texture of the crack faces is probably attributable to the slight meander of the transverse cracks.

The aggregate itself provided little surface texture in any of the cores because the top size is so small (19 mm [0.75 in]), and the cracks (at joints and cracks) tended to propagate through the aggregate particles. However, dowels are present at the joints so adequate load transfer can be maintained regardless of the surface texture present. The one dowel retrieved in a core exhibited a tight longitudinal crack in the concrete along the center of the dowel. In addition, the dowel exhibited some corrosion at the joint face. The crack may have been due to some slight misalignment of the dowel (although none was apparent) or the expansive action of the corrosion. No longitudinal cracks were observed at the surface near any joint or crack.

The longitudinal steel found in one of the crack cores had failed. The wire was severely corroded so it was not possible to determine if the corrosion caused failure or if the corrosion occurred after the failure. The failed reinforcement would allow the crack to open when temperatures drop, reducing load transfer and allowing the development of faulting and spalling at the crack. This is probably representative of the condition at all of the deteriorated cracks.

Project Summary

Recycled concrete pavement sections were constructed using identical designs in 1984 in both the eastbound and westbound lanes of I-90 near the Minnesota-South Dakota border. The design consists of 230-mm (9-in) JRCP (0.065 percent steel) on an aggregate base and subbase. The transverse joints contain 25-mm (1-in) dowel bars and are spaced 8.2 m (27 ft) apart. The outer lanes are widened by 0.6-m (2-ft) and asphalt concrete shoulders are present. Longitudinal edge drains are also provided.

Condition surveys were conducted over 305-m (1,000-ft) sections in each direction; coring and FWD testing were performed only within the eastbound section. Results of these surveys and tests suggest the following:

Pavement Design

- The longitudinal steel in the cores pulled at the transverse cracks was observed to be corroded and ruptured. This can be attributed to the small amount of steel present (0.065 percent), the harsh environment (with respect to both temperature extremes and the use of deicing chemicals), the use of plain (uncoated) steel reinforcing, and the high volume of heavy traffic present.

Material Properties

- Laboratory tests of concrete strength, elasticity and thermal properties produced values that were generally typical of those found for other recycled concrete projects, although the compressive strength and static modulus of elasticity were at the low end of the range of values observed on other projects. Laboratory-determined values of concrete elastic modulus were much lower and less variable than those determined through backcalculation using FWD test results.

- The backcalculated subgrade modulus values were fairly constant, averaging 34.2 kPa/mm (126 lbf/in^2/in). Higher values were obtained in an area of deeper foundation cut; backcalculated concrete modulus was unusually low at this same location.
- The surface texture of the transverse cracks is somewhat greater than that of the joints because the cracks tended to meander more. The surface texture associated with the RCA concrete was generally minimal due to the small size of the coarse aggregate and the reduced quantity of natural aggregates present in most RCA concrete mixtures.
- Uranyl acetate tests indicated the presence of small deposits of silica gel in the mortar and around some of the recycled concrete aggregate particles, indicating the possible presence of alkali-silica reaction activity.

Pavement Performance

- Both pavement sections had experienced a lot of transverse crack deterioration (42 to 61 deteriorated cracks per km). These cracks are generally spalled and faulted. Failure of the longitudinal steel and the relatively smooth transverse crack texture allows differential vertical movement at these cracks, thereby causing spalling of the RCA concrete, especially when incompressibles are entrapped within the cracks.
- Low-severity transverse joint spalling was observed at 21 and 13 percent of the transverse joints in the eastbound and westbound sections, respectively.
- The average load transfer efficiency at the transverse joints was about 80 percent, with little change observed when the load was moved from the approach side to the leave side of a given joint. The load transfer efficiency at the transverse cracks was also fairly high considering the deterioration, averaging 67 percent, although these values ranged from 57 to 77 percent. High air and pavement temperatures may have contributed to the unusually high load transfer measurements on this project.
- The low load transfer efficiencies obtained at the joint may be caused by the poor joint construction techniques previously described above or due to bearing failures around the dowels since the dowel diameter is only 25 mm (1 in).
- Other types of distress, such as transverse joint faulting and longitudinal cracking, were not observed in significant quantities in either section.
- Pavement ride quality was good at the time of survey (serviceability ratings averaged 4.1 and 4.3 for the eastbound and westbound sections, respectively). However, these are expected to decrease rapidly in the near future as the transverse crack deterioration (faulting and spalling) continues.
- A dowel was contained in one of the cores retrieved from a transverse joint. Some dowel corrosion was observed in the vicinity of the joint. A tight crack was observed in the concrete along the center of the dowel. This crack may have been caused by stresses resulting from improper dowel alignment or the expansive forces associated with the corrosion.

- Midpanel deflections on this project were among the highest measured in the course of this research project. These deflections were generally associated with soft foundation conditions, although unseen slab cracking may have contributed to the high deflections.

Overall

These findings suggest that the subject pavement sections were adequate in most respects. The major performance deficiency (transverse crack deterioration) appears to be attributable mainly to inadequate longitudinal reinforcement design, although the relatively poor crack surface texture and low strength of the recycled concrete may have contributed to the incidence of crack spalling after the longitudinal steel failed. Improved reinforcement designs or the use of short, unreinforced panels may have resulted in more acceptable performance.

Minnesota 3, U.S. 59 near Worthington

Increased demand for concrete construction aggregate and the depletion of existing accepted aggregate sources in the 1970's led the Minnesota Department of Transportation (Mn/DOT) to initiate a search to identify candidate projects for the State's first attempt to recycle an existing concrete pavement surface into coarse aggregate for a new concrete pavement. Candidate projects were identified in 1976 using a strict set of guidelines, and the project selected for recycling was a 26-km (16-mi) segment of U.S. 59 between Worthington and Fulda in southwestern Minnesota.[18] This project, completed in 1980, was the first major concrete recycling project in the United States in which a D-cracked concrete pavement was used to furnish coarse aggregate for new pavement.

<u>Project Information</u>

The original roadway was constructed in 1955 and consisted of a "thickened edge" PCC pavement with thickness varying from 180 mm (7 in) at the center to 230 mm (9 in) at the edges. The pavement was 7.3 m (24 ft) wide with transverse joints spaced at 6.1-m (20-ft) intervals. The pavement was placed over a minimum 75-mm (3-in) aggregate base, which was originally placed over a pre-existing bituminous surface. At the time of recycling, the existing concrete pavement was showing signs of D-cracking, which developed slowly over 29 years.

The original concrete pavement mix design involved two coarse aggregate sources: the northern half contained coarse aggregate from Edgerton, and the southern half contained coarse aggregate from Luverne. Sources familiar with the project indicate that the northern half was predominately made using Edgerton aggregate particles, and that the southern half was predominately made using Luverne aggregate particles. The cause of the D-cracking was attributed to the 55 to 60 percent concentration of limestone-dolomite in the coarse aggregate materials.

Halverson reported that RCA sources were depleted before the job was complete; thus, about 3 percent of the total project involved the use of virgin coarse aggregates.[19] These materials were used primarily in mailbox turnouts and other paving features to the mainline pavement structure. The source or sources of the virgin aggregate is not documented. However, it was noted that the virgin coarse aggregate(s) had the same maximum top size as that of the recycled coarse aggregates.

Ten alternative rehabilitation strategies were considered initially, and other strategies were added later. These candidate rehabilitation programs included: fracturing the existing pavement and placing either a PCC or AC overlay, placing an unbonded overlay, constructing a new PCC or AC pavement, and recycling the existing pavement into either a PCC or AC pavement. The strategy selected was to recycle the existing concrete pavement to provide coarse aggregate for a new 200-mm (8-in) recycled JPCP.

Design Information

Constructed in 1980, the experimental RCA concrete pavement section is a 200-mm (8-in) JPCP placed over a 25 to 38-mm (1 to 1.5-in) aggregate base (stabilized using fines from the concrete recycling process) and an aggregate subbase of varying thickness (the aggregate base for the previous pavement). The subgrade is reported to be an AASHTO A-1-a material. The RCA concrete pavement surface is 7.3 m (24 ft) wide, with one travel lane in each direction and 2.4-m (8-ft) wide bituminous-surfaced shoulders are present on each side. These shoulders consist of a 50-mm (2-in) bituminous surface layer, which is placed over an untreated aggregate layer that extends to the depth of the aggregate subbase beneath the mainline pavement. The skewed transverse joints are spaced at 4.0-4.9-4.3-5.8-m (13-16-14-19-ft) intervals, are sealed with a silicone sealant material, and do not contain dowel bars. The longitudinal centerline joint contains 760-mm (30-in) long, 16-mm (No. 5) epoxy-coated tie bars spaced 760 mm (30 in) apart. Longitudinal edge drains are also provided throughout the project.

Available literature indicates that conventional natural aggregates were used as coarse aggregate in the concrete used to construct a control section.[19] However, several cores were taken throughout the project in an attempt to locate this control section and these efforts proved futile (recycled concrete aggregate was observed in all cores). Furthermore, interviews with State and contractor workers that were present during construction indicated that a control section may never have been built on this project.

Mix Design

A section of the original pavement was removed and crushed for the purpose of developing trial mix designs. Five alternative mix designs were investigated, including mixes which included recycled coarse and fine aggregates, recycled coarse aggregate with natural sand, and partial substitution of cement with fly ash.

The fraction of the crushed concrete that passed the 75-mm (No. 200) sieve was tested for deleterious content and was found to require no washing. However, the angularity and highly absorptive nature of the crushed concrete fraction that passed the 4.75-mm (No. 4) sieve made it most suitable for use in shoulder construction and foundation stabilization and it was, therefore, eliminated from consideration for use in the paving mix design. In addition, the maximum particle size for the RCA coarse aggregate was limited to 19-mm (0.75-in) to reduce the potential for recurrent D-cracking in the new concrete pavement.

The compressive strengths of the RCA concrete mixtures exceeded those of the conventional mixtures at similar water-cement ratios. Freeze-thaw testing of the recycled mixtures indicated that the inclusion of fly ash would reduce the potential for D-cracking.

The test results described above were used to develop the project concrete mix design, which is shown in table 34. One hundred percent RCA was selected for use as coarse aggregate (bulk specific gravity = 2.41 and absorption capacity = 4.4 percent) and the selected fine aggregate was 100 percent natural sand (bulk specific gravity = 2.62). This mixture also included the replacement of 15 percent cement with 20 percent Class C fly ash.

Table 34. Mix design for MN 3-1.[20]

Material	Recycled
Coarse Aggregate	981 kg/m^3
Fine Aggregate	710 kg/m^3
Cement	276 kg/m^3
Fly Ash	65 kg/m^3
Water	151 kg/m^3
w/c+p Ratio	0.44

Construction Information

The first step in the recycling of the existing pavement was the removal of the bituminous overlays, asphalt concrete patches, and joint material. Breaking of the concrete pavement was accomplished using a trailer-mounted Link Belt 440 diesel pile hammer. The pavement fragments were loaded onto trucks and hauled to the crushing plant where a primary jaw crusher reduced the pieces to less than 305 mm (12 in) in size. In fact, the primary crusher reduced the concrete pieces to about a 75 mm (3 in) top size. This material was then transferred to a secondary cone crusher, where it was crushed and stockpiled in two sizes: coarse material passing 19-mm (3/4-in) sieve and retained on the 4.75-mm (No. 4) sieve; and fine material passing the 4.75-mm (No. 4) sieve.

Concrete paving operations began on August 1, 1980, and continued until September 10. The RCA fines were placed in a layer 25 mm (1 in) thick to serve as a firm platform for the paving equipment. Paving was performed using a traditional slip-form paver. The surface was textured using an astroturf drag followed by transverse tining. A curing compound was then applied to the surface. The transverse and longitudinal joints were sawed within 24 h of placing the concrete.

Tests of the plastic concrete at the job site yielded an average slump of 38 mm (1.5 in) and an air content of 5.5 percent. Concrete specimens were also cast for laboratory testing. Compressive strengths averaged 31.6 MPa (4580 lbf/in^2) after 60 days and center-point flexural strengths at 14 days averaged 4.5 MPa (650 lbf/in^2).

<u>Climatic Conditions</u>

This project is situated in the transition zone between the dry-freeze and wet-freeze environmental regions. The minimum and maximum average monthly temperatures are –10 and 23 °C (14 and 74 °F). The area experiences about 92 freeze-thaw cycles annually, and the freezing index is 830 °C-days (1500 °F-days). The Thornthwaite moisture index is 8, which reflects an average of 102 days of precipitation per year totaling 640 mm (25 in).

<u>Traffic Loadings</u>

This section was constructed and opened to traffic in 1980, at which time the two-way ADT was about 2,150 vehicles per day. The two-way ADT increased at a rate of about 1 percent per year to 2,470 vehicles per day in 1994, the time of the survey. At this time, the traffic stream included about 12 percent heavy trucks. Based on these traffic estimates, the recycled concrete pavement has been exposed to approximately 950,000 ESAL applications through 1994.

<u>Selection of Distress Survey Section</u>

The 26-km (16-mi) experimental project consisted of the same structural design throughout the entire length, although the RCA concrete material varied, as noted previously. The control section described in some literature sources could not be located.

The section selected for evaluation is located in the southbound lane of U.S. 59 beginning at milepoint 27.0 and extending southward approximately 305 m (1,000 ft). The natural aggregate contained in this section was from the Hallet-Luverne source. This section was constructed nearly at grade (no areas of significant cut or fill) and was considered representative of the entire project.

Drainage Survey

The section contains longitudinal edge drains. The drainage outlets were free of debris and vegetation, although their function was not observed directly (i.e., no rainfall events occurred during the survey). Some signs of low-severity pumping were observed on the section, and significant faulting was observed (as described below). Surface drainage was achieved through the use of 1 percent cross-slope on the travel lanes (center crowned) and 3 percent slope on the shoulders.

Pavement Condition Survey

The pavement condition survey was conducted over the 305-m (1,000-ft) pavement section described previously. A summary of the average results for some of the key distress measurements is presented table 35. A more complete summary of the condition survey results is provided in appendix A.

Table 35. Summary of performance data for MN 3-1.

Performance Measurement	Recycled
Corner Faulting, mm (Manual)	7.4
Wheelpath Faulting, mm (Manual)	n/a
Wheelpath Faulting, mm (Digital)	6.1
Percent Slabs Cracked	2
Longitudinal Cracking, m/km	19
Transverse Joint Spalling, % Joints	71
PSR	3.0

Transverse Joint Faulting

Severe transverse joint faulting was observed. Measurements taken at the slab corners using a manually-operated faultmeter averaged 7.4 mm (0.29 in) with some faults as large as 13.5 mm (0.53 in). Measurements taken in the outer wheel path using a digital faultmeter averaged 6.1 mm (0.24 in). Faults were generally greatest at the pavement edge and typically decreased to zero near the pavement centerline before increasing in the opposite direction in the opposing traffic lane. These faulting levels are well above the thresholds typically considered critical for pavements with short joint spacings.

The development of faulting on this project is not surprising given the lack of dowels or other mechanical load transfer devices at the transverse joints. However, the large amount of faulting that was observed as a result of a relatively small amount of traffic suggests that the pumping/faulting mechanism was especially active at this site. One possible reason for the rapid development of pumping/faulting is inadequate drainage (i.e., base material not sufficiently permeable to transport water to the collector pipes). Another likely candidate is the reduced potential for aggregate interlock due to the minimal joint face texture and reduced abrasion resistance often associated with RCA concrete, along with exceedingly large joint openings that accompany drying shrinkage and the large temperature drops that are common in this area. Other factors, such as the use of unstabilized materials in some foundation layers, may have contributed to the faulting problem as well.

Table 36 presents a summary of average transverse joint faulting measurements for each slab length combination observed on this project. The data seem to indicate greater faulting at transverse joints where the approach slab is longer than the leave slab. However, vertical slab displacements due to curling and warping vary with panel length, which can influence apparent faulting measurements. For example, differences in curling deformations at the joint between 4.0-m and 5.8-m (13-ft and 19-ft) slabs were found to be as high as 30 percent (using the ILLISLAB computer program). Deflections due to load, however, were nearly identical for the two slabs. Therefore, differences in slab elevation at these joints (i.e., faulting) are apparently explained by the differences in curling between panels of different lengths and are probably not due to actual differences in faulting at joints with different slab length combinations.

Table 36. Faulting measurements for each slab length combination.

Slab Length, m (Approach-Leave)	Corner Faulting, mm (Manual)	Wheelpath Faulting, mm (Digital)
5.8 - 4.0	8.4	7.4
4.0 - 4.9	5.8	4.8
4.9 - 4.3	8.4	7.1
4.3 - 5.8	6.6	5.3

Transverse Cracking

Only one transverse crack was observed on the recycled survey section. The crack was a medium-severity crack (faulting of 1.5 mm [0.06 in]) of a 4.9-m (16-ft) panel, and it extended across both lanes. There was no evidence of a culvert or other underground structure in the area of the crack. Very little transverse panel cracking was observed on this project, presumably due to the relatively short joint spacings, which resulted in L/ℓ ratios of 4.2, 4.5, 5.2 and 6.2 for the 4.0-, 4.3-, 4.9- and 5.8-m (13-, 14-, 16-, and 19-ft) panels (L/ℓ computed using the average laboratory-determined value of the concrete elastic modulus and the average backcalculated subgrade modulus).

Longitudinal Cracking

One longitudinal crack was observed on the survey section. This medium-severity, sealed crack was contained within a single panel, extending from one transverse joint to meet the centerline joint about three-fourths of a panel length away. There was no apparent explanation for the development of this crack and longitudinal cracking was not widely observed on the project.

Transverse Joint Spalling

Transverse joint spalling was observed at 71 percent of the surveyed transverse joints. However, 68 percent of the spalls were of low severity and only 3 percent were medium severity. These numbers correspond with joint seal damage, which was observed at 76 percent of the transverse joints (61 percent low severity and 15 percent medium severity). It should be noted that a silicone joint sealant was used on this job, which contained a large quantity of limestone coarse aggregate. This combination has recently been implicated in many joint sealant failures.

Other Distresses

Recurrent D-cracking was not observed anywhere on this project. In addition, corner breaks were not found on this project, in spite of the large faulting measurements, which probably indicate locally-poor slab support.

Present Serviceability Rating (PSR)

The average project survey team estimate of PSR for this section was 3.0, indicating that the ride quality has deteriorated to a point that will soon require rehabilitation. Given the limited amount of cracking and joint spalling on this project, the low serviceability ratings can be attributed almost entirely to the high levels of transverse joint faulting.

<u>FWD Testing</u>

Deflection testing on this section was conducted during September 7, 1994. The backcalculated values obtained from these deflection data are representative of the material properties that existed at that time under the prevailing environmental conditions. Tests performed on this section at other times under different conditions might be expected to vary significantly.

Pavement deflection testing was performed using a Dynatest model 8081 FWD. The testing pattern included 5 slab centers, 10 transverse joints (load placement on both the approach and leave sides of the joint), and 10 panel edges. A summary of the results of the deflection testing is provided in table 37. FWD testing was used to estimate pavement layer material properties (PCC elastic modulus and modulus of subgrade support or "k-value"), load transfer efficiencies across joints, and loss of support. Table 37 presents a summary of the average values for these parameters, as computed using the deflection testing data. Additional discussion of each parameter is provided below.

Table 37. Deflection testing results for MN 3-1.

Property	Recycled
Elastic Modulus, GPa	62.3
k-value, kPa/mm	28.5
Joint Load Transfer, %	37
Average Midslab Deflection, µm	142
Average Edge Deflection, µm	303
Corners With Voids, %	10
Maximum Air Temperature During Testing, °C	20

PCC Elastic Modulus

Midpanel deflection measurements were used to backcalculate the elastic modulus (E) of the recycled concrete slab. The average backcalculated elastic modulus is 62.3 GPa (9,000,000 lbf/in^2), although considerable variation (ranging from 42 to 88 GPa [6,100,000 to 12,800,000 lbf/in^2]) was observed for various test loads and locations, as illustrated in figure 46. The results obtained at any particular location differ by as much as 26.2 GPa (3,800,000 lbf/in^2), depending on the load level. In addition, the elastic modulus values generally seem exceptionally high when compared to typical values for conventional and recycled concrete materials. Laboratory tests of cores retrieved from the same areas as the test locations exhibited much less variability and produced an average concrete dynamic elastic modulus of 34.2 GPa (4,960,000 lbf/in^2).

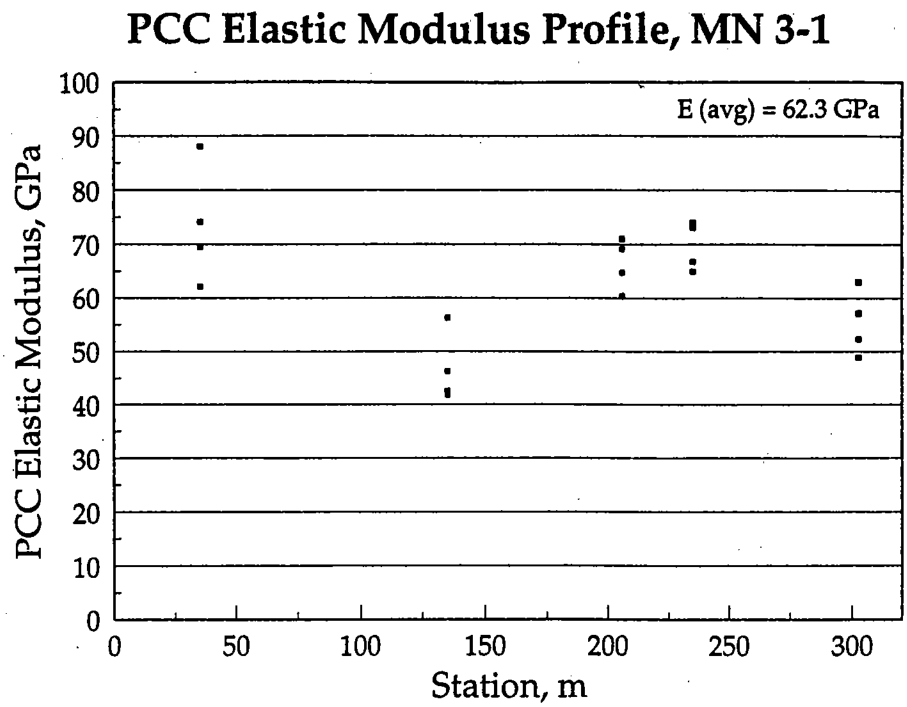


Figure 46. PCC elastic modulus profile for MN 3-1.

Modulus of Subgrade Reaction (k-value)

A profile plot of the backcalculated k-values is illustrated in figure 47. The backcalculated moduli range between 21 and 34 kPa/mm (77 to 125 $lbf/in^2/in$), with an average value of 28.5 kPa/mm (105 $lbf/in^2/in$). The results of these tests were quite consistent, exhibiting little variation between drops or location. While these effective moduli of foundation support seem somewhat low, it must be remembered that they represent the effects of the temperature and moisture conditions that existed at the time of testing on September 7, 1994.

Investigation of Backcalculation Results

The backcalculation results for both the PCC elastic modulus do not appear to be reasonable in comparison to similar pavement sections. One likely explanation for the variability between the different drops at the same station may be the effects of curling of the PCC slab. FWD testing was performed in the early afternoon of a sunny day, and it is likely that the slab was curled downward.

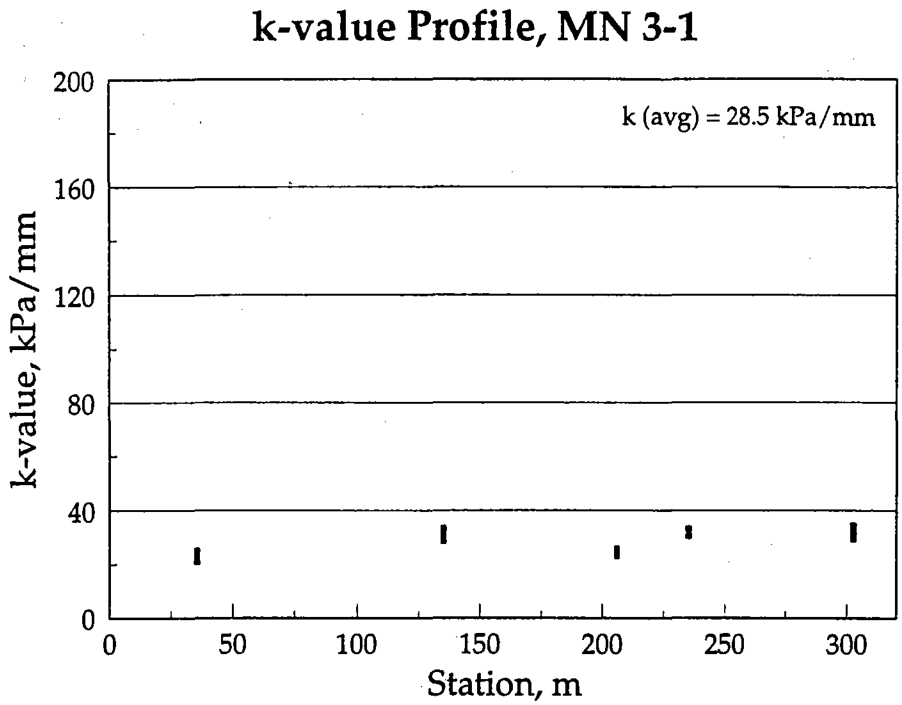


Figure 47. K-value profile for MN 3-1.

Downward curling can cause loss of contact between the center portion of the slab and the underlying layer and therefore loss of friction between the two layers. Once the load is applied in the basin testing, the contact between the slab and the underlying layer is reestablished, and the two layers begin to work as a single bonded layer. The effect of the initial loss of contact varies depending on the load level, with the effect being less significant at higher load levels. To help eliminate this variability, the results were again backcalculated ignoring the sensor directly under the load. These results are illustrated in figures 48 and 49. In comparison to figures 46 and 47, one can easily notice a considerable reduction in the variability of the results between different load levels.

The variability between the results at different stations are thought to be attributed to the variation in the underlying structure along the section. To investigate this effect, several deflection basins were generated using the computer program DIPLOMAT. The pavement was modeled as a 200-mm (8-in) PCC slab, a 150-mm (6-in) granular base, and a bituminous layer supported by a dense liquid foundation. The thickness of the underlying bituminous layer was varied from 100 to 150 mm (4 to 6 in). The obtained theoretical deflection basins were used as input parameters for backcalculation, in which the pavement structure is treated as a single layer resting on a dense liquid foundation. The resulting backcalculated values were then compared with backcalculated parameters from the field data.

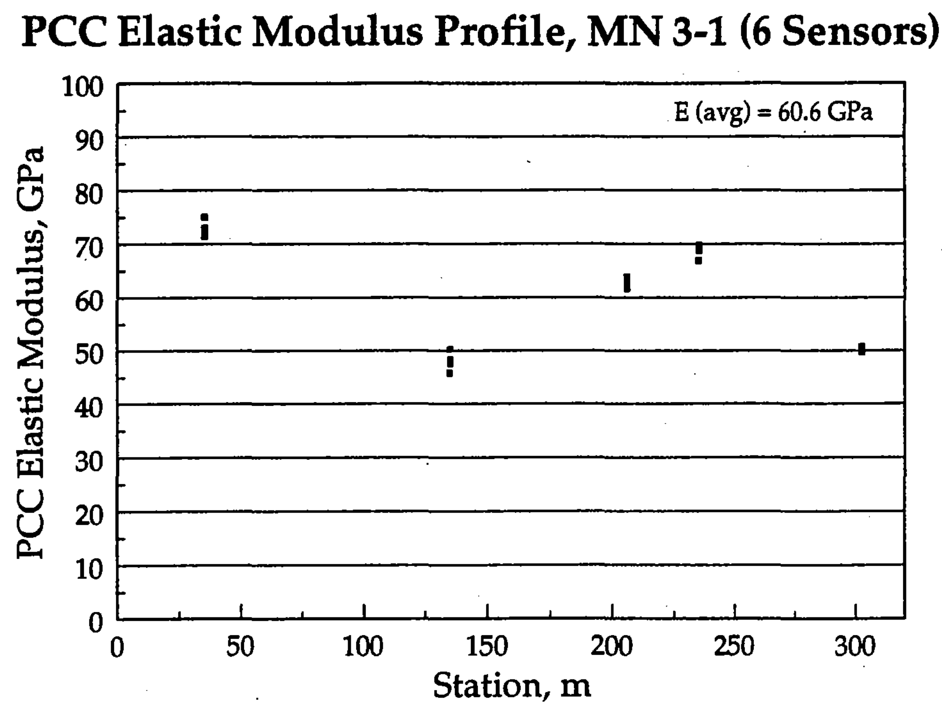


Figure 48. PCC elastic modulus profile for MN 3-1 without d_0 sensor.

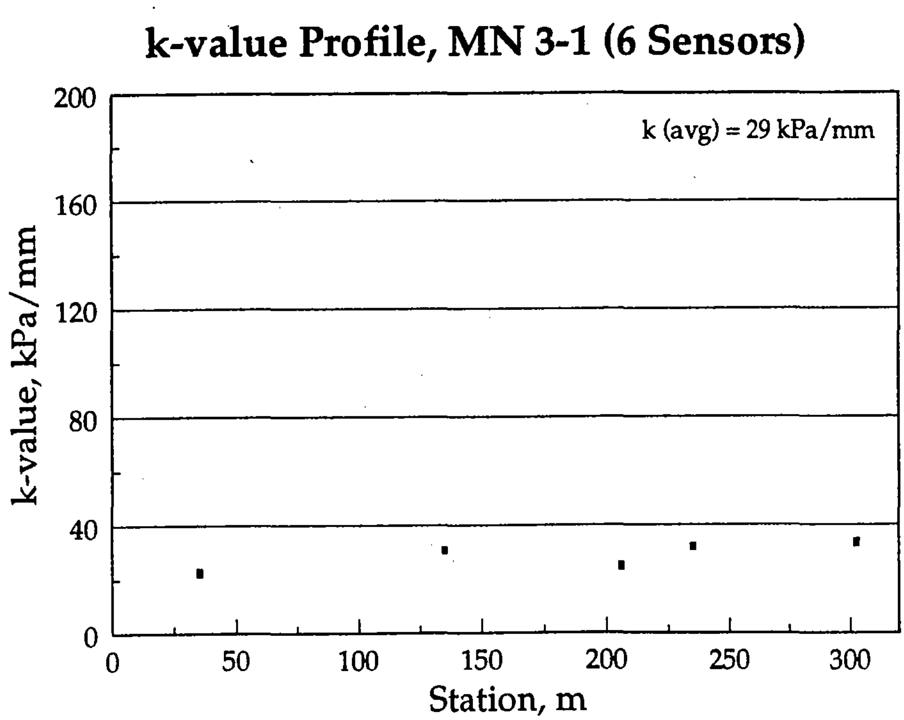


Figure 49. K-value profile for MN 3-1 without d_0 sensor.

Two cases were run to examine the effect of the underlying bituminous layer. First, using a PCC elastic modulus of 48.3 GPa (7,000,000 lbf/in^2), a base modulus of 0.34 GPa (50,000 lbf/in^2), a 150-mm (6-in) bituminous layer with a modulus of 4.1 GPa (600,000 lbf/in^2), and a k-value of 27 kPa/mm (100 lbf/in^2/in), DIPLOMAT produces the deflection profile found at station 235 (backcalculated PCC elastic modulus of 69.3 GPa [10,100,000 lbf/in^2] and backcalculated k-value of 33 kPa/mm [123 lbf/in^2/in]). For the second case, the thickness of the bituminous layer was changed to 100 mm (4 in), and the k-value was changed to 22 kPa/mm (80 lbf/in^2/in). The deflection profile from this structure matched that at station 135, with a backcalculated modulus and k-value of 50.3 GPa (7,300,000 lbf/in^2) and 29 kPa/mm (105 lbf/in^2/in), respectively. Therefore, the variability in the backcalculated elastic modulus values at different stations is likely the result of variability in the thickness of the bituminous layer.

Joint Load Transfer

Figure 50 shows a plot of the transverse joint load transfer efficiencies measured with the load plate placed on both the approach and leave sides of the joints in the southbound lanes of the pavement section. At each location, the average load transfer efficiency is significantly greater when the load is placed on the leave side of the joint; the average load transfer efficiencies are 28 and 46 percent when the load is placed on the approach and leave sides of the joint, respectively. One possible explanation for this consistent variation as a function of load placement is that the fracture plane at the joints angles toward the leave side of the joint through the pavement thickness. This would enable the leave slab to bear on the approach slab without the reverse being true. Cores taken through the joints do not support this theory, however.

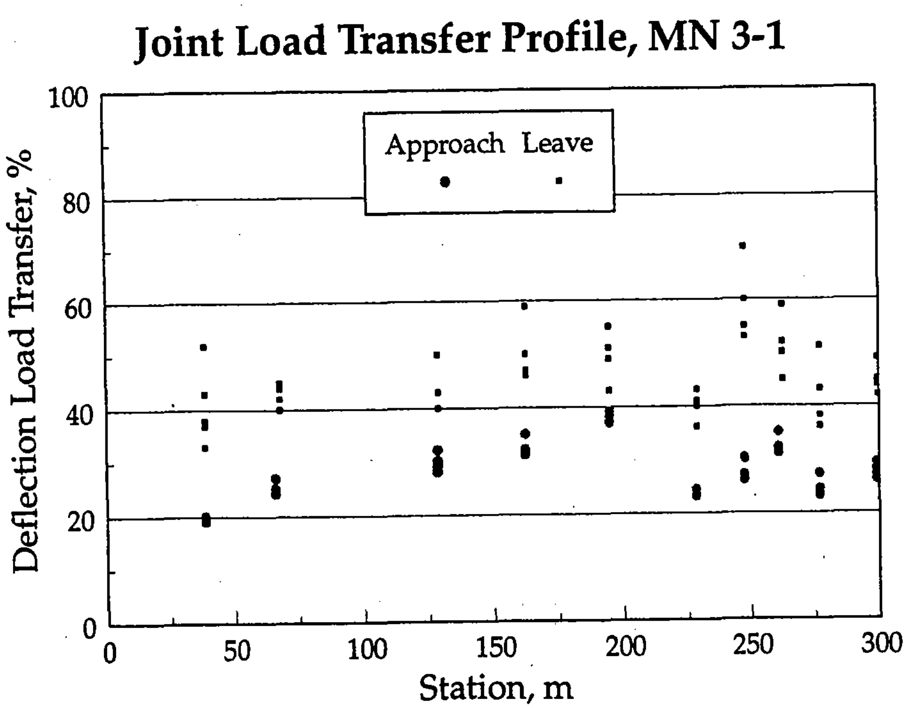


Figure 50. Joint load transfer profile for MN 3-1.

From a pavement performance standpoint, the more important observation is that all of the load transfer measurements were quite low, with only one test value exceeding 60 percent and many values approaching 20 percent. This poor load transfer efficiency may be attributable to the absence of dowels or other mechanical load transfer devices, and the poor potential for grain interlock when the surface texture is low and joint movements are large.

Loss of Support

Void detection on the leave side of the transverse joints and cracks was performed using the techniques described in the final report for NCHRP Project 1-21 and corner deflection data from this project. Figure 51 shows the loss of support profile plot, which indicates significant potential for loss of support under only 1 of the 10 joints tested, although several other joints exhibited strongly positive x-intercept values. These results suggest that the observed high faulting levels have developed without the accompanying development of large areas of deep voids. In other words, the amounts of fines being removed from beneath the leave side of the joints and deposited on the stabilized foundation under the approach sides of the joints may be transported from areas that are either small and deep or large and very shallow; either type of foundation erosion would be difficult to detect with the FWD. These loss of support values are also supported by the lack of corner cracking on this project.

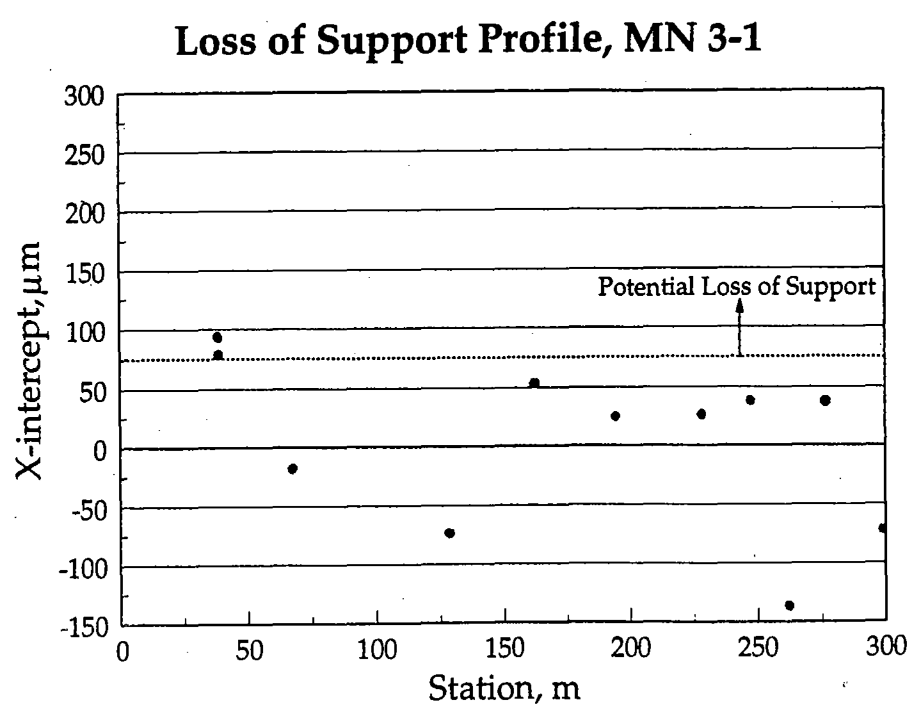


Figure 51. Loss of support profile for MN 3-1.

Coring

Ten cores were retrieved from MN 3-1. Four 100-mm (4-in) diameter cores were taken at midpanel locations for compressive strength testing and linear traverse analysis. Another 150-mm (6-in) diameter core was taken at midslab for split tensile strength testing. Three 150-mm (6-in) cores were also taken at transverse joints to document the presence of any recurrent D-cracking and to measure the texture of the fractured concrete surface (an indication of aggregate interlock load transfer potential). Two additional 100-mm (4-in) diameter cores were taken 0.3 and 0.6 m (1 and 2 ft) from a transverse joint for use in determining the lateral extent of any recurrent D-cracking. All cores were taken through the thickness of the concrete slab but did not extend into the base layer. The average thickness of the cores was 200 mm (7.9 in).

Core Testing

The number of cores for each laboratory test is indicated in table 38. A summary of the average values that were obtained during the laboratory testing of the field cores is presented below in table 39 (and table 83 in appendix A). Observations made during the testing, and relative comparisons are also provided below.

Table 38. Number of cores for each laboratory test in MN 3-1.

Laboratory Tests	Recycled Section	Control Section
Thermal Coefficient	2	n/a
Split Tensile Strength	1	n/a
Dynamic Modulus of Elasticity	2	n/a
Static Modulus of Elasticity	1	n/a
Compressive Strength	2	n/a
Volumetric Surface Texture	3	n/a

Table 39. Core testing results for MN 3-1.

Property	Recycled
Compressive Strength, MPa	44.1
Split Tensile Strength, MPa	4.1
Dynamic Elastic Modulus, GPa	34.2
Static Elastic Modulus, GPa	31.2
Thermal Coefficient, (1×10^{-6})/ °C	8.9
VSTR (for Failed Split Tensile Core), cm^3/cm^2	0.1603
VSTR (for Slab Faces at the Joints), cm^3/cm^2	0.2475
VSTR (for Slab Faces at the Cracks), cm^3/cm^2	n/a

Petrographic Examination Summary

The recycled coarse aggregate used in this project contained rounded to angular gravel rock particles that were observed to be evenly distributed throughout the cement paste. The gravel rock is further characterized as original coarse aggregate containing igneous, metamorphic, and sedimentary particles. The mortar and aggregate contents observed in this project were comparable to those observed in samples obtained from other recycled concrete projects (see table 40), except that a Class C fly ash was also observed in this recycled concrete mixture.

Table 40. Coarse aggregate and mortar contents for MN 3-1.

	Recycled
Coarse Aggregate, %	18.0
New Mortar, %	67.5
Recycled Mortar, %	14.5

Uranyl acetate testing of cores obtained from the southbound lane indicate the presence of minor amounts of silica gel in the mortar and around some of the aggregate particles. While this may indicate the presence of alkali-silica reactivity, no such distress was identified during the field surveys.

Mid-Panel Cores

The compressive strengths ranged from 40.5 to 47.8 MPa (5,870 to 6,930 lbf/in^2), with an average of 44.1 MPa (6,390 lbf/in^2), which was near the average for the recycled concrete projects included in this study. Only one test of split tensile strengths was performed, with a result of 4.1 MPa (590 lbf/in^2), which is typical for the other RCA concrete specimens tested in this study.

The dynamic elastic modulus ranged from 32.4 to 36.0 GPa (4,700,000 to 5,220,000 lbf/in^2), with an average of 34.2 GPa (4,960,000 lbf/in^2), which is near the average for the range of test results observed in this study. As discussed previously, these test results are significantly lower and less variable than the results obtained through backcalculation of FWD data. The static elastic modulus is a result of only one test at 31.2 GPa (4,520,000 lbf/in^2), which is also typical of the range of test results observed in this study.

The thermal coefficients ranged from 8.4 x 10^{-6}/ °C to 9.4 x 10^{-6}/ °C (4.7 x 10^{-6}/ °F to 5.2 x 10^{-6}/ °F) with an average of 8.9 x 10^{-6}/ °C (5.0 x 10^{-6}/ °F). These values were the lowest observed for any RCA concrete tested in this study, well below the average of 11.2 x 10^{-6}/ °C (6.2 x 10^{-6}/ °F).

Extra cores were taken from this project to help determine if recycling the aggregate helped to mitigate D-cracking present in the original pavement. Upon initial examination, the cores did not appear to exhibit any signs of recurrent D-cracking. A closer examination, by means of freeze-thaw (ASTM C 666) and linear traverse testing (ASTM C 457), indicated that these initial indications were false.

Freeze-thaw testing was performed on four cores taken from mid-panel. All durability factors were well below 60 (see table 41), which is often considered the minimum level for acceptable performance. Table 41 also shows that the specimens failed relatively quickly (88 cycles). This contradicts the performance observed in the field since, as previously indicated, D-cracking was not observed.

Table 41. Average durability factors obtained from freeze-thaw testing (ASTM C 666)

Durability Factor		Cycles to Failure
RDM-based	Dilation-based	
21	19	88

Slices were taken from the bottoms of several of the midpanel cores, and were polished and examined at the microscopic level. Very few aggregate particles exhibited cracking; those that did contained only tight cracks which appeared to have occurred during the aggregate formation or the crushing processes. It was determined that there are microcracks in the old mortar, some of which have propagated into the new mortar. It is difficult to determine if these microcracks are a direct result of freeze-thaw cycling or if they were initiated during the crushing process. Regardless of the origin, these microcracks make the concrete more susceptible to freeze-thaw damage and thereby provide one explanation for the low durability factors.

Linear traverse testing was performed on these same slices to determine whether the concrete air void system may have contributed to the observed poor durability. The linear traverse results given in table 42 show that a high air content (8.3 percent) was measured. This is to be expected since the total mortar content (old plus new) is increased for RCA concrete. The low specific surface of 203.6 cm^{-1} can be explained by the large entrapped air voids, which were present in all of the cores. In general, it appears that the air void system is marginally acceptable (using typically accepted criteria for spacing factor, specific surface, and total air content). This would suggest that most freeze-thaw damage in this concrete is mainly due to aggregate durability problems. It should be noted, however, that an examination of a core subjected to freeze-thaw testing revealed failures in the vicinity of large entrapped air voids which were close to the surface of the core.

Table 42. Averages of the linear traverse calculations.

Linear Traverse Results	
Average Chord Intercept, mm	0.20
Voids per centimeter	27.15
Specific Surface, cm^{-1}	203.6
Paste to Air Ratio	3.66
Air Content, %	8.3
Spacing Factor, mm	0.18

The pavement is currently 15 years old. The freeze-thaw testing indicated that the concrete is not durable. This may mean that the pavement could begin to deteriorate substantially in the near future. It is also possible that D-cracking will never cause any substantial problems to the performance of this pavement if the concrete is not often critically saturated in the field.

Further research should be conducted to determine how the cracks were initiated in the old mortar and if they alone (without the large amounts of entrapped air being present) would be sufficient to decrease the durability of the concrete to an unacceptable level. The large amount of entrapped air present also decrease the durability of the concrete. It is difficult to determine if the entrapped air voids are the result of a harsh mix, which is commonly associated with the use of recycled aggregate, or the use of improper construction techniques. Additional work should be performed to help determine the cause of these entrapped air voids.

In summary, all laboratory testing indicated typical RCA concrete properties except for the unusually low coefficient of thermal expansion values. Most measures of concrete strength and elasticity were lower than the typical values obtained for paving concrete used on the other study sections. However, the lack of a control section on this project makes direct comparisons impossible.

Joint and Crack Cores

The coarse aggregate particles are round to angular and uniformly distributed. A moderate amount of large voids are present in the cement paste. Cracks at the joints tended to propagate around the aggregate particles but along a relatively straight plane. The small aggregate top size (19 mm [0.75 in]) also contributed to the low VSTR (0.2475 cm^3/cm^2). Low joint load transfer efficiencies are a direct result of the low VSTR's since no mechanical devices were used.

A modified linear traverse examination of polished sections of concrete from this project suggested the material is comprised of as much as 82 percent mortar. While a higher mortar content is expected in RCA concrete (due to the inclusion both new and recycled mortar), this unusually large value may also be due to random variations in aggregate content through the selected slice.

Several cores taken at the joint were also trimmed and polished for microscopic examination. As with the mid-panel cores, small cracks were found in the aggregate but they were not considered detrimental. Micro-cracking was also found in the old mortar, some of which propagated into the new mortar. More cracking was observed at the bottom portion of the pavement, which is consistent with theories of increased freeze-thaw in regions subject to more frequent saturation.

Project Summary

In 1980, the 25-year-old D-cracked concrete pavement on U.S. 59 was removed and crushed to produce coarse aggregate for a new concrete pavement surface. This project represented the first major concrete recycling project in the United States to use a D-cracked concrete pavement as a source of coarse aggregate for a new concrete paving mixture. The recycled concrete aggregate was used as the coarse aggregate portion of the mix and was supplemented with a natural sand. The recycled pavement was a 200-mm (8-in) JPCP with a "random" joint spacing and no dowels.

A condition survey was conducted over a 305-m (1,000-ft) sections in the southbound direction; coring and FWD testing were also performed. Results of these surveys and tests suggest the following:

Pavement Design

The use of properly designed joint load transfer devices (i.e., epoxy-coated dowel bars) probably would have reduced faulting to acceptable levels. While other factors may certainly have contributed to the faulting problems (e.g., possible lack of adequate subsurface drainage, poor aggregate interlock load transfer), it is likely that this pavement would still be considered serviceable had dowels been used.

In addition, it is likely that the use of a drainable base layer would have improved pavement performance by more rapidly removing water from the pavement structure, although there the actual function of the existing drainage system could not be evaluated in the absence of a rainfall event during the survey.

Material Properties

The laboratory-determined values of strength and elasticity for the RCA concrete used on this project were typical of that observed for other RCA concrete projects. It was also slightly lower than that of conventional concrete paving mixtures, but was certainly within a range that would be considered acceptable for concrete paving. Concrete modulus values determined through backcalculation of FWD test results were both variable and unreasonably high; alternative backcalculation procedures are now being considered to verify the first analyses.

The thermal coefficient of expansion of the RCA concrete was significantly lower than expected, and was even lower than typically found for conventional concrete. There is no apparent explanation for this phenomenon, especially in consideration of the relatively high mortar content and low natural aggregate content of the RCA concrete.

Backcalculated subgrade modulus values (k) showed little variation, but averaged only 28.5 kPa/mm (105 $lbf/in^2/in$)—much lower than would have been expected for an effective value of an AASHTO A-1-a roadbed soil. These results are also being re-examined using alternative backcalculation procedures.

Transverse joint load transfer values were quite low, averaging less than 40 percent. This can be attributed to the lack of dowel load transfer devices, as mentioned previously. In addition, the small top size of the coarse aggregate (19 mm [3/4 in]) and many years of abrasion under service loads contributed to poor aggregate interlock potential. It was also noted that the load transfer efficiency was consistently much higher on the leave side of each joint, although a reason for this trend is not readily apparent.

Freeze-thaw testing indicated that the concrete is not durable since all of the durability factors were well below 60. The linear traverse results were marginally acceptable. A visual examination of a core subjected to freeze-thaw testing revealed failures in the vicinity of large entrapped air voids which were close to the surface of the core. Also, small microcracks were found in the old mortar of cores not subjected to freeze-thaw testing. Some of these microcracks propagated into the new mortar. Both of these factors could contribute to the poor durability of the concrete.

The results of uranyl acetate testing indicate the presence of moderate amounts of silica gel deposits in the RCA concrete mortar and around some of the aggregate particles. This may indicate the presence of ASR, although no such distress was observed in the field or in the petrographic examination.

Pavement Performance

This pavement section exhibited little pavement distress other than low-severity joint spalling and severe joint faulting (an average of 7.4 mm [0.29 in] at the panel corners). Cracking was virtually nonexistent and recurrent D-cracking was not observed.

The joint faulting is considered the primary source of serviceability loss on this project (PSR estimated at 3.0). This faulting is considered excessive, even for a undoweled pavement, in the context of the pavement service life (14 years) and vehicle loading (950,000 ESAL's) to date. Potential load transfer- and drainage-related solutions to this faulting problem were described previously. It is unlikely that any undoweled concrete pavement would have resisted faulting for long in this environment and under these loads. However, the reduced surface texture of the RCA concrete joint faces and the probable reduced abrasion resistance of the material may have accelerated the development of faulting on this project.

Overall

This project was one of the oldest RCA concrete projects considered in this study. It appears that the concrete mixture used has performed well and that the primary performance-related problems have been related to inadequate structural or drainage design issues. It is especially important to note that there was no evidence of recurrent D-cracking on this project, indicating that at least some pavements with a history of durability problems can be successfully recycled into new concrete paving mixtures.

Minnesota 4, U.S. 52 near Zumbrota

This project is the third of three Minnesota JRCP sections included in this study that were constructed using RCA concrete. The project includes both recycled concrete and control sections with the same structural designs and traffic.

<u>Project Information</u>

The project is located in the northbound lanes of U.S. 52 near Zumbrota, Minnesota. It is a four-lane divided highway. The recycled concrete section incorporated aggregate produced by crushing the original pavement, which was 53 years old in 1984 when the new pavement was constructed. The RCA concrete pavement section extends from station 898+00 to 998+50, and the control section extends from station 998+50 to 1048+81.

Design Information

The recycled and control sections employ the same basic design, consisting of 230-mm (9-in) JRCP over a 130-mm (5-in) aggregate base, 1,070-mm (42-in) granular subbase and an AASHTO A-7-5 roadbed soil. The transverse joints are skewed, spaced at 8.2-m (27-ft) intervals, and contain 25-mm (1-in) epoxy-coated dowel bars. The transverse joints are sealed with a preformed joint sealant.

The outer traffic lane (driving lane) was paved 3.7 m (12 ft) wide with asphalt concrete shoulders and no additional edge support. The outer shoulder extends 2.4 m (8 ft) from the outer traffic lane and consists of a 50-mm (2-in) AC surface layer over an aggregate base. The inside shoulder has the same structural design as the outer shoulder and extends 0.6 m (2 ft) from the edge of the 3.7-m (12-ft) inner travel lane. Longitudinal edge drains have been added along the outside lane since the pavement sections were constructed. The drain outlets are located approximately every 110 m (350 ft).

Slab reinforcement consists of an uncoated deformed welded wire fabric with 7.6-mm (0.30-in) diameter longitudinal wires spaced at 310 mm (12 in) center-to-center, resulting in a longitudinal steel content of 0.065 percent of the slab cross-sectional area. Transverse wires are 5.8 mm (0.23 in) in diameter and are also spaced at 310 mm (12 in) center-to-center. The longitudinal centerline joint is equipped with 760-mm (30-in) long, 13-mm (No. 4) epoxy-coated deformed tie bars spaced 760 mm (30 in) on center.

Mix Design

The recycled section concrete contains recycled concrete coarse aggregate and natural sand fine aggregate. The control section concrete contains the same natural sand fine aggregate and virgin gravel coarse aggregate (fine grained dolomite). Table 43 presents the gradations of the aggregate used in both mixtures and indicates that the maximum coarse aggregate sizes in the RCA and control section concrete mixtures were 25 and 38 mm (1.0 and 1.5 in), respectively. The recycled concrete coarse aggregate had a bulk specific gravity of 2.42, compared to a value of 2.68 for the virgin coarse aggregate. The natural sand used in both mixes had a bulk specific gravity of 2.63.

Table 43. Aggregate gradations (percent passing each sieve) of recycled and control sections.

Sieve	Recycled		Control	
	Coarse	Fine	Coarse	Fine
38 mm (1.5 in)	100		100	
25 mm (1.0 in)	100		n/a	
19 mm (3/4 in)	99		46	
12.7 mm (1/2 in)	n/a		n/a	
9.53 mm (3/8 in)	n/a		n/a	
4.75 mm (No. 4)	11	99	15	99
2.36 mm (No. 8)		84		86
1.18 mm (No. 16)		66		66
0.600 mm (No. 30)		44		44
0.300 mm (No. 50)		17		16
0.150 mm (No. 100)		2		2

The concrete mix designs used in the two sections are provided in tables 44 and 79. The recycled section contains somewhat less coarse aggregate than the control section (about 15 percent less by weight, 7 percent less by volume), but contains about 9 percent more fine aggregate and 33 percent more fly ash, presumably to improve the workability of the plastic concrete mixture. The slump of the RCA mixture was reported as 38 mm (1.5 in); the slump of the control mixture was not reported. Both mixtures included about the same amount of water and cement, resulting in a water-cement ratio of about 0.55; however, the increased amount of fly ash in the RCA mixture produced a lower water-cementitious (fly ash plus cement) ratio than the control mixture (0.44 vs. 0.47). Both mixes also contained an air-entraining agent (Protex) in quantities selected to produce 5.5 percent air content.

Table 44. Mix designs for MN 4.

Material	Recycled	Control
Coarse Aggregate	983 kg/m^3	1166 kg/m^3
Fine Aggregate	713 kg/m^3	653 kg/m^3
Cement	276 kg/m^3	278 kg/m^3
Fly Ash	65 kg/m^3	49 kg/m^3
Water	151 kg/m^3	153 kg/m^3
w/c ratio	0.55	0.55
w/c+p Ratio	0.44	0.47

Construction Information

The 53-year old concrete slab was removed and crushed to provide aggregate for the recycled concrete pavement. The control section used a virgin coarse aggregate material. The recycled and control sections were both placed using the same construction techniques. Surface texture was provided using an astroturf drag followed by transverse tining. A curing compound was also applied to the surface.

Climatic Conditions

The MN 4 test sections are located in the wet-freeze environmental region. The minimum and maximum average monthly temperatures are -11 and 23 °C (13 and 73 °F). The area experiences about 95 freeze-thaw cycles annually, and the freezing index is 720 °C-days (1,300 °F-days). The Thornthwaite moisture index is 20, which reflects an average of 110 days of precipitation per year totaling an average of 740 mm (29 in).

Traffic Loadings

The reconstructed pavement sections were opened to traffic in 1984 and, through the survey date in 1994, had been exposed to an estimated 3.2 million ESAL applications in the driving lane. In 1984, the two-way ADT was estimated at 7,820 vehicles per day, including about 15 percent heavy trucks. As of 1994, the two-way ADT had increased to about 10,010 vehicles per day with the same relative proportion of heavy trucks. The corresponding ESAL applications in the opening year (1984) and the survey year (1994) are estimated at 235,000 and 359,000, respectively.

Selection of Distress Survey Section

With the exception of the differences in aggregate source and mix design, the RCA and control paving sections are considered to be identical with respect to structural design, traffic, environment, etc. Thus, one major criterion for the selection of project survey sections was to find representative sections constructed over similar grades. However, the project contains many horizontal and vertical curves and several bridges and it was difficult to meet this criterion.

The control section that was selected began at station 1035+01 and extended 305 m (1,000 ft) northward. This section was constructed at grade and did not include any significant horizontal or vertical curves. A representative RCA concrete paving section with these characteristics could not be located, and the selected section that was selected is located partially in a fill section and on a horizontal curve. This section begins at station 983+88, about 150 m (500 ft) north of the bridge over the Zumbro River.

Drainage Survey

Both sections contain longitudinal edge drains with outlets spaced every 110 m (350 ft); the drain systems appear to be functioning properly. Signs of pumping, such as accumulations of water or fines along the joints, were not observed on either section.

The surface drainage varies somewhat between the two survey sections. The longitudinal grade of the recycled concrete section varies between about 0.5 and 1.0 percent in the direction of travel, and transverse slopes vary from 1 percent within the normally-crowned sections to 4.5 percent within the superelevated cross sections. The shoulder slopes range between 1.5 and 6.5 percent. The control section is also located on a 1 percent longitudinal grade, but the cross-sectional crown and transverse slope are constant at 1 percent within the traffic lanes and 4 percent on the shoulders.

Pavement Distress Survey

The pavement condition survey was performed over the recycled and control survey sections. A complete summary of the survey results is provided in tables 84 and 85 in appendix A, and a summary of the average results for key distress and performance variables is presented in table 45. The results of the distress survey indicate that the major difference in the performances of the two sections is in the amount of deteriorated transverse cracking. The recycled section shows extensive deterioration at the transverse cracks, whereas no deteriorated cracks are present on the control section. All other performance indicators reveal similar performance levels.

Table 45. Summary of performance data (average values) for MN 4.

Performance Measurement	Recycled	Control
Corner Faulting, mm (Manual)	0.3	0.5
Wheelpath Faulting, mm (Manual)	0.5	0.5
Wheelpath Faulting, mm (Digital)	1.0	0.8
Percent Slabs Cracked	88	22
Deteriorated Transverse Cracks/km	79.8	0.0
Cracks/km	115.0	26.3
Longitudinal Cracking, m/km	0	0
Transverse Joint Spalling, % Joints	76	92
Joint Width, mm	11	11
PSR	4.0	4.2

Transverse Joint Faulting

Transverse joint faulting was measured in the driving lane outer wheel path and at the panel corners closest to the outer shoulder of the recycled and control sections. No significant difference in faulting was measured between the two sections, with average faulting levels of about 0.5 mm (0.02 in) measured in the outer wheel path using a manual faultmeter and slightly higher levels measured using the digital faultmeter. These low faulting levels indicate that good transverse joint load transfer exists through a combination of dowel bars and aggregate interlock, and/or foundation drainage is adequate.

Transverse Cracking

There was a significant difference in the amount and severity of transverse panel cracking between the RCA and control sections. Eighty-eight percent of the RCA panels exhibited transverse cracking, and 63 percent of the panels contained deteriorated (medium- or high-severity) cracks. Only 22 percent of the control section panels were cracked and none of the cracks observed were deteriorated. Transverse panel cracking was expected on this JRCP project because of the large panel lengths (L/ℓ ratios of 7.8 and 8.2 for the recycled and control sections, respectively, where L/ℓ was computed using the average laboratory-determined value of the concrete elastic modulus and the average backcalculated subgrade modulus), and it might be considered surprising that there was not more cracking on the control section. However, these cracks would be of low severity in a properly designed and reinforced

JRCP structure. Thus, the poor performance of the RCA section bears closer examination.

Possible explanations for the relatively poor performance of the RCA pavement section include differences in the physical and mechanical properties of the concrete (i.e., differences in concrete strength, thermal coefficient and drying shrinkage, as well as differences in the surface texture and abrasion resistance of the fractured surfaces at the transverse cracks). These parameters are discussed in detail below. There may also be random variations in foundation support or other unknown sources of bias that may have affected the study results, although every effort was made to minimize the potential for such problems during construction and in the selection of study sections.

Longitudinal Cracking

Longitudinal cracking was not observed within either survey section.

Transverse Joint Spalling

More transverse joints exhibited spalling on the control section than in the recycled section. Joint spalling was observed on 76 and 92 percent of the transverse joints on the recycled and control sections, respectively. However, nearly all of the observed spalls were low severity. Only 3 percent of the spalls on the control section were medium severity. Thus, the differences in joint spalling between the two sections are relatively insignificant.

Present Serviceability Rating (PSR)

The average PSR values of the recycled and control sections are 4.0 and 4.2, respectively. The difference is likely due to the increased number of deteriorated transverse cracks on the recycled section. These cracks have developed some faulting, which adversely affects the ride quality.

<u>FWD Testing</u>

Pavement deflection testing was performed using a Dynatest model 8081 FWD. The testing pattern typically included 5 slab centers, 10 transverse joints (approach and leave), 10 transverse cracks (approach and leave), and 10 edges. However, only 8 transverse cracks were present on the control section, so only 8 were available for testing. FWD testing was used to determine material properties (PCC elastic modulus and subgrade k-value), load transfer efficiencies across joints and cracks, and loss of support. A summary of the average values obtained from the deflection data is provided in table 46.

Table 46. Deflection testing results for MN 4.

Property	Recycled	Control
Elastic Modulus, GPa	30.3	44.6
k-value, kPa/mm	24.4	33.1
Joint Load Transfer, %	78	86
Crack Load Transfer, %	74	94
Average Midslab Deflection, μm	186	138
Average Edge Deflection, μm	237	185
Corners With Voids, %	0	0
Maximum Air Temperature During Testing, °C	28	33

PCC Elastic Modulus

The elastic modulus (E) of the concrete slab was backcalculated using the center-of-slab deflection measurements. Figure 52 shows a profile plot of the elastic modulus for the recycled section using four drops at five different locations. The average elastic modulus is 30.3 GPa (4,390,000 lbf/in^2), although the values range from 22 to 38 GPa (3,190,000 to 5,510,000 lbf/in^2). At each particular location, the elastic modulus values from the four drops are within a small range, indicating good repeatability between the drops. However, the values obtained at different locations vary considerably. There seems to be a trend of increasing elastic modulus toward the north end of the test section. The superelevation for the horizontal curve begins about 120 m (400 ft) into the test section.

Figure 52 shows a similar profile plot for the control section. The average backcalculated elastic modulus of the concrete slab is 44.6 GPa (6,470,000 lbf/in^2), with values ranging from 40 to 49 GPa (5,800,000 to 7,110,000 lbf/in^2). Again, the variability between the estimates obtained using any of the four drops at any particular location was minimal. Furthermore, the estimates obtained at different locations do not vary as widely as for the recycled section. The control section is located on a straight section at grade, which probably provides more consistent results.

The average backcalculated elastic modulus on the control section is about 50 percent higher than the average value obtained on the recycled section. Previous studies have found the elastic modulus of conventional concrete to be typically 20 to 40 percent higher than the elastic modulus of recycled concrete.[15] The unusually large difference in this case may be partially attributable to differences in the quantities of coarse aggregate in the two mixtures (the control section contains about 19 percent more by weight, and about 10 percent more by volume) or differences in the particle

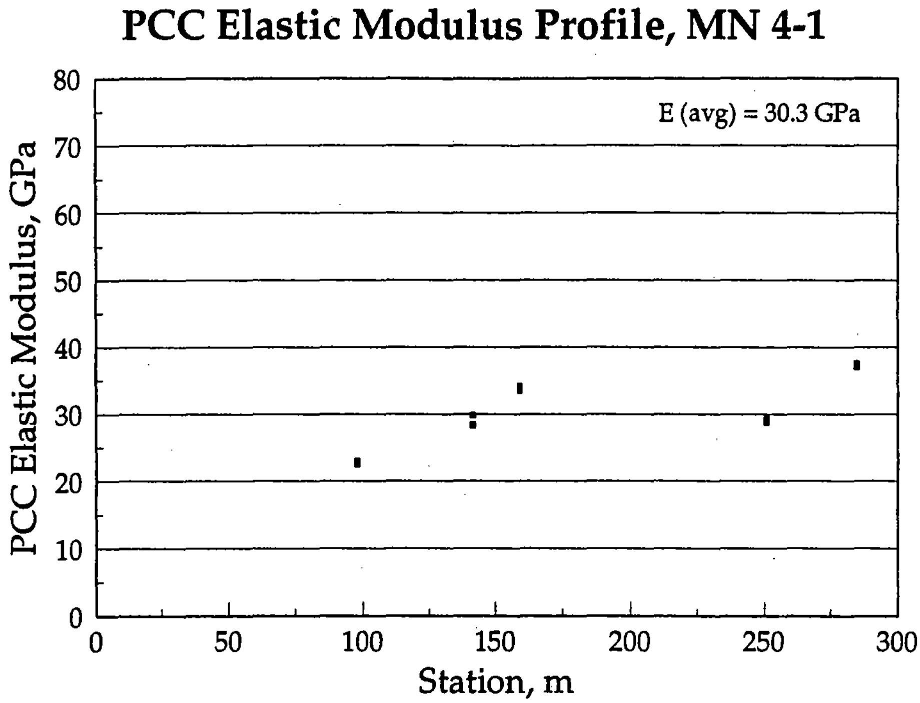


Figure 52. PCC elastic modulus profile for MN 4-1 (recycled section).

size distributions in the two mixtures (the control section contained larger particles than the recycled mixture). Since the elastic modulus of rock is typically much higher than that of cement mortar or paste, the use of greater proportions of aggregate might be expected to produce higher concrete elastic modulus values.

Modulus of Subgrade Reaction (k-value)

A profile plot of the backcalculated k-values for the recycled and control sections are illustrated in figures 53 and 54, respectively. The average backcalculated k-value for the recycled section is 24.4 kPa/mm (90 lbf/in^2/in), with values ranging from 17 to 34 kPa/mm (63 to 125 lbf/in^2/in). On the control section, the average of all tests was 33.1 kPa/mm (122 lbf/in^2/in), and values ranged from 16 to 51 kPa/mm (59 to 188 lbf/in^2/in), with the generally higher values measured at the south end of the section. The control section foundation support generally appeared to be stronger than that of the recycled pavement section.

Both sets of backcalculated k-values appear to be somewhat lower than might be expected. One possible explanation for this is that the area received heavy rainfall throughout most of the day preceding testing, and the stiffness of a fine-grained soil, such as this AASHTO A-7-5 soil, is generally sensitive to changes in moisture conditions. Therefore, tests conducted under drier conditions would probably produce higher k-values.

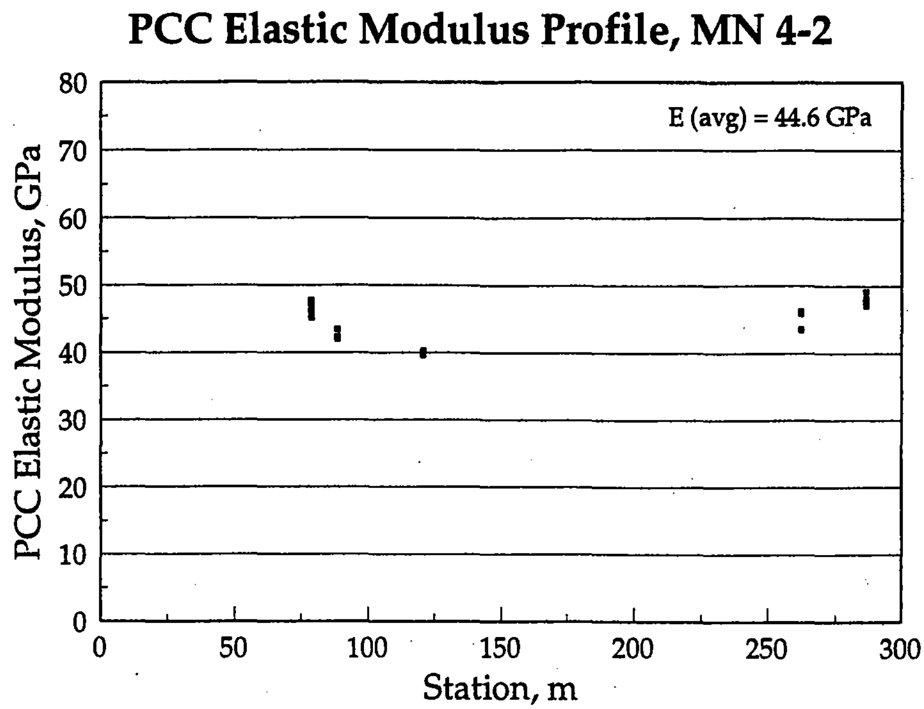


Figure 53. PCC elastic modulus profile for MN 4-2 (control section).

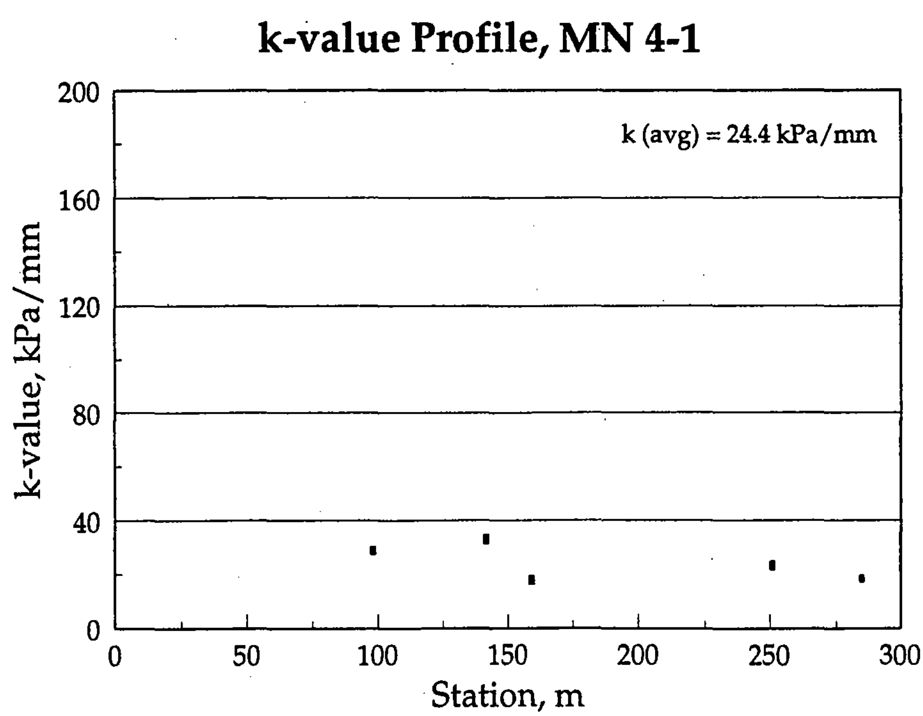


Figure 54. K-value profile for MN 4-1 (recycled section).

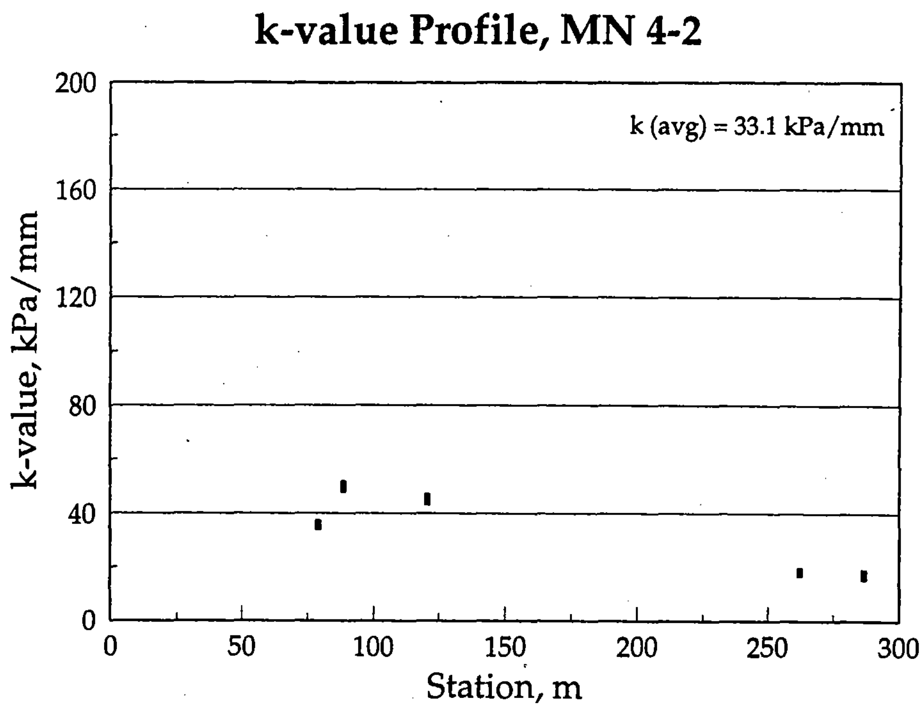


Figure 55. K-value profile for MN 4-2 (control section).

Joint Load Transfer

The load transfer efficiencies measured with the load placed on both the approach and leave sides of the recycled section transverse joints are shown in figure 56. These values are computed as the ratio of the deflection on the unloaded side of the joint to the deflection on the loaded side of the joint. The average deflection load transfer efficiency for this section is 78 percent, with little difference observed between the average load transfer efficiencies measured with the load placed on the approach or leave side of the joints (78 and 77 percent, respectively). There was some variation in load transfer efficiency between joints (averages ranged from 56 to 89 percent), indicating some joints may be more deteriorated than other joints. For example, a large entrapped air void was found under the dowel in one of the cores pulled from the joints. The crack was greatly deteriorated around the dowel and a crack which extended transversely along the slab face on top of the dowel was also present. All of these factors would contribute to a low LTE.

The Minnesota Department of Transportation also conducted FWD testing on the same day and provided the results of their tests to the project research team. A plot of the transverse joint load transfer efficiencies in the RCA pavement section is shown in figure 57. Each point represents the average of tests at three different load levels—40, 53 and 71 kN (9, 12, and 16 kips). This plot closely resembles the one presented in figure 56. The load transfer efficiency varies from 63 to 93 percent, with an average of 79 percent.

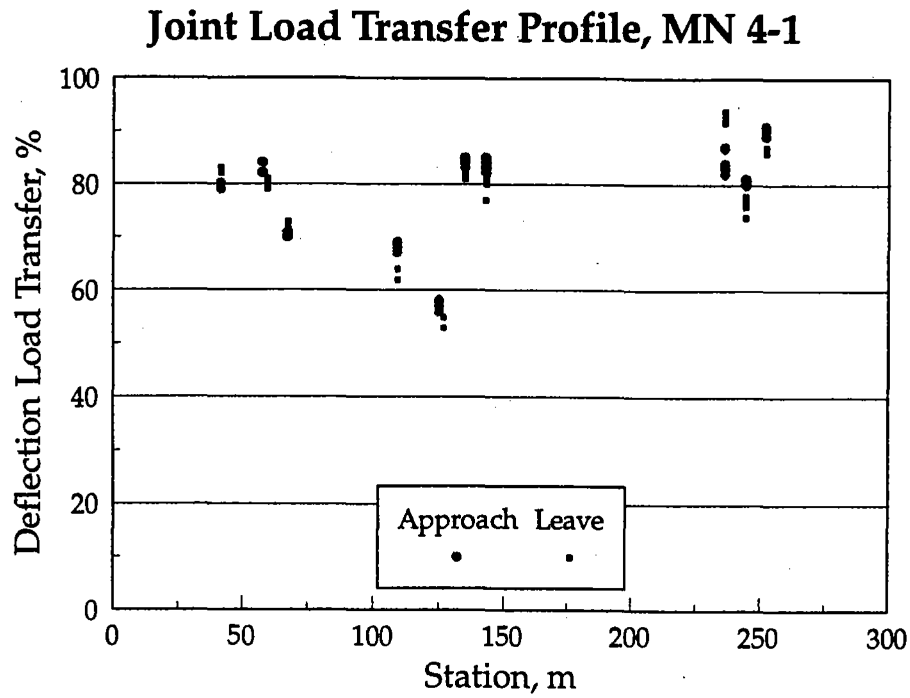


Figure 56. Joint load transfer profile for MN 4-1 (recycled section).

Figure 58 illustrates the joint load transverse efficiencies for the control section. The average load transfer efficiency is 86 percent, with values measured on the approach and leave sides of the joints averaging 87 and 85 percent, respectively. The variation in load transfer efficiencies between test locations was generally not large and only a few tests produced values lower than 80 percent.

Although the faulting levels on the recycled and control sections are about the same, the deflection results indicate that the control section is providing slightly better load transfer across transverse joints. It may be that the transverse joints within the control section are just beginning to break down but have not yet shown major signs of deterioration. A visual examination of the cores pulled at the joints support this hypothesis since cores pulled from both sections contained break-offs at the bottom of the core and both had deteriorated crack faces. The difference was that the cores from the control section did not contain cracks which propagated transversely along the dowel. At the time of the survey, the sections were only 10 years old and had been subjected to approximately 3.2 million ESAL's. These sections should be monitored to see if performance differences become more apparent as the they grow older and are exposed to more ESAL's. Given the current load transfer efficiency trends, it might be expected that deterioration and faulting at the transverse joints will begin to occur on the recycled section sooner than on the control section.

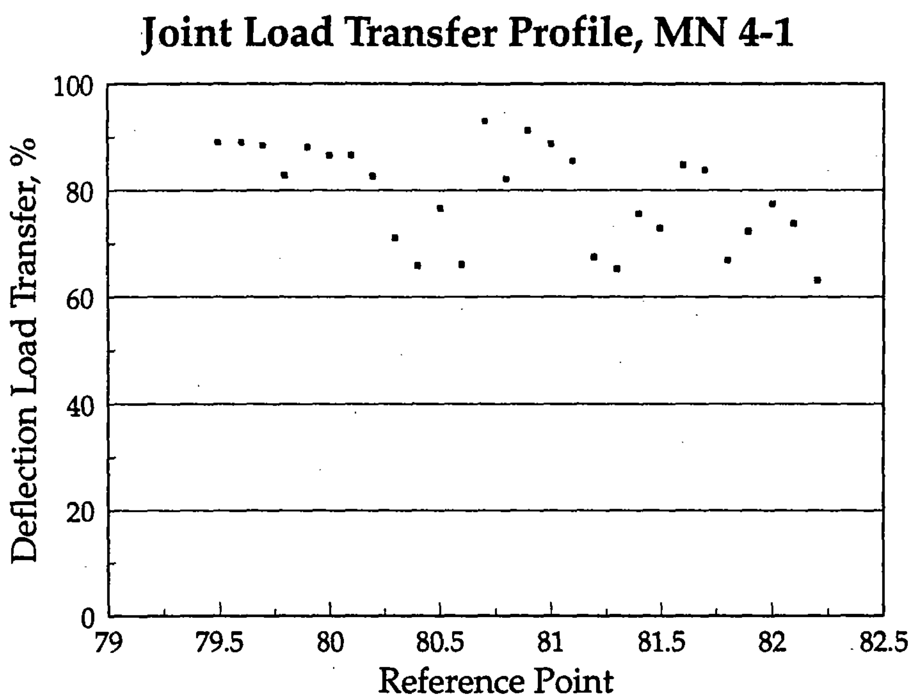


Figure 57. Joint load transfer profile for MN 4-1 from MnDOT data.

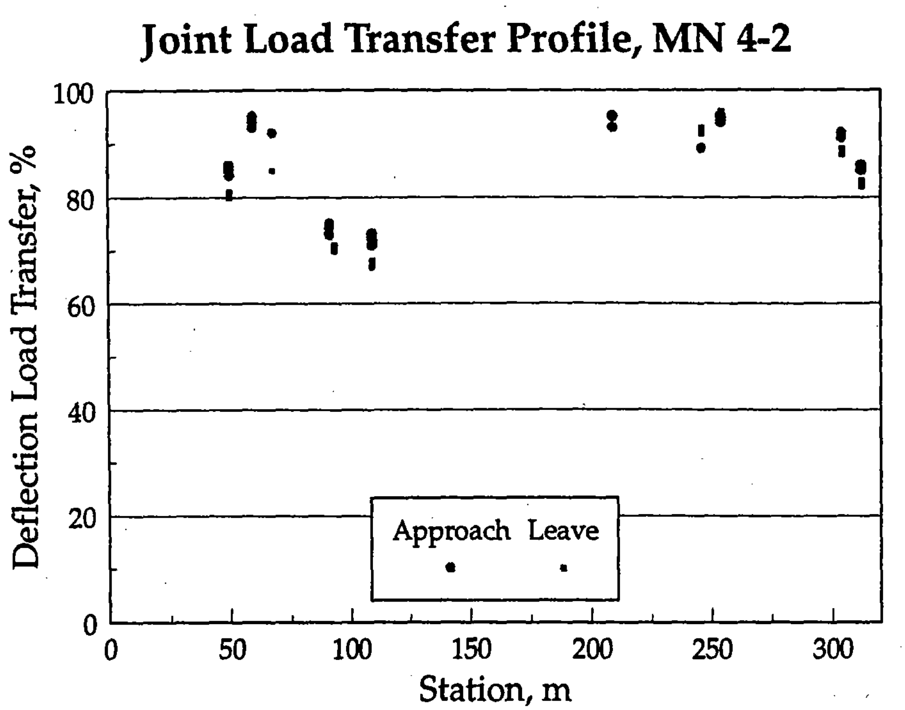


Figure 58. Joint load transfer profile for MN 4-2 (control section).

In summary, the recycled and control sections are currently exhibiting about the same level of load transfer and are performing well. Both sections are equipped with 25-mm (1-in) dowel bars, which might, therefore, be considered adequate for these sections on the basis of their performance to date. However, the sections are only 10 years old and have been subjected to only about 3.2 million ESAL applications. It also appears that the RCA section may be on the verge of a major loss of transverse joint load transfer efficiency. A re-evaluation in another 5 years may show some degradation in the overall performance and a more significant difference in performance between the sections.

Crack Load Transfer

The load transfer efficiencies at approach and leave cracks of the recycled section are illustrated in figure 59. The letter above or below each group of data points indicates the severity of the crack. The average load transfer efficiency is 74 percent, although test results varied greatly with load plate location and crack severity. The low-severity cracks generally exhibited the higher load transfer efficiencies, averaging over 90 percent. The medium-severity cracks, however, generally exhibited load transfer efficiencies below 70 percent, with some values below 40 percent. The medium-severity crack load transfer efficiencies also showed large variations with load plate placement, with much lower values typically resulting when the load plate was located on the approach side of the cracks.

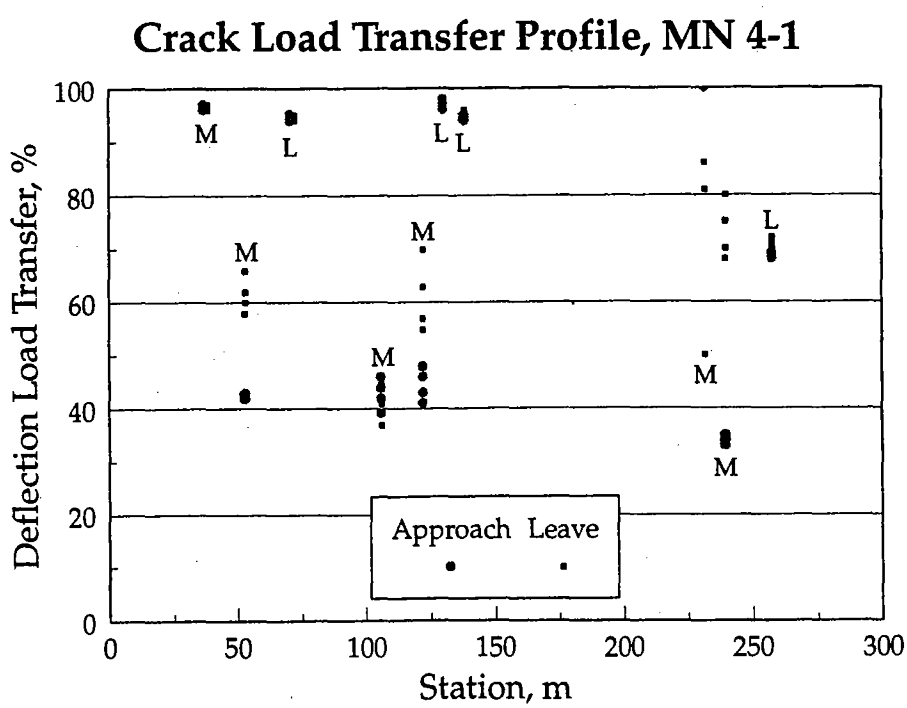


Figure 59. Crack load transfer profile for MN 4-1 (recycled section).

The load transfer efficiencies for the control section cracks are shown in figure 60. The average load transfer efficiency for the control section is 94 percent, about 20 percent higher than that of the RCA pavement section. The results of these tests showed little variability between tests and at different locations. This is presumably because all of the transverse cracks on the control section are of low severity and have remained tight, thereby providing a high degree of load transfer.

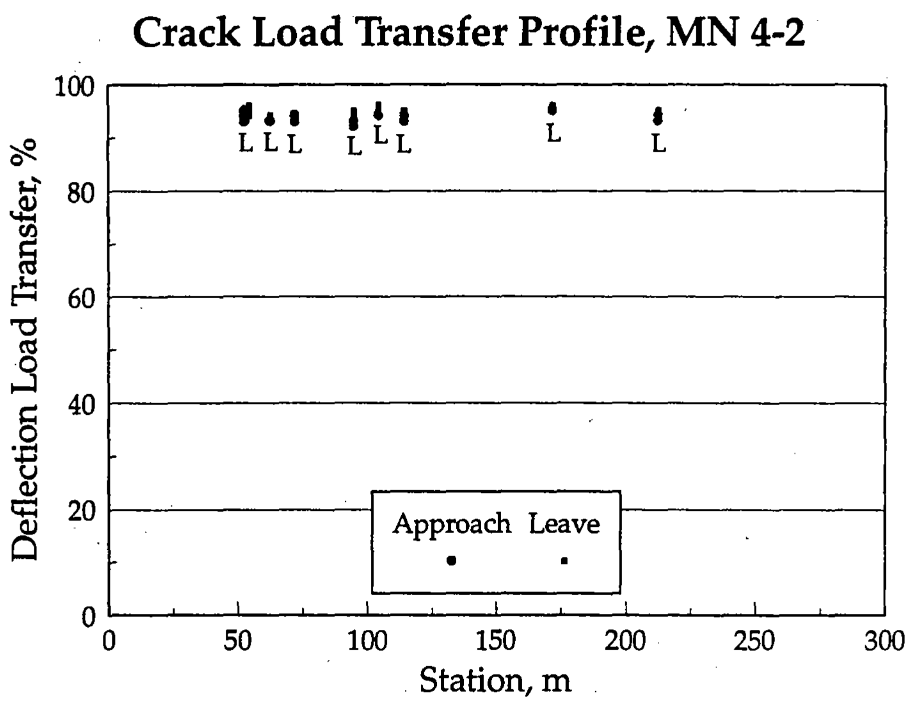


Figure 60. Crack load transfer profile for MN 4-2 (control section).

Loss of Support

The detection of voids was performed using the corner deflections on the leave side of transverse joints and cracks and procedures described in the final report for NCHRP 1-21. Figures 61 and 62 illustrate the potential for loss of support along the recycled and control sections, respectively. Neither sections shows significant potential for loss of support at transverse joints or cracks. These results are consistent with the lack of observed pumping and significant faulting throughout the sections.

Coring

Eleven cores were taken on both the recycled and control sections: five at midpanel, three at transverse joints, and three at transverse cracks. All cores were 150-mm (6-in) in diameter and extended through the thickness of the concrete slab. No cores were taken through the aggregate base course. The average thicknesses of the cores on the

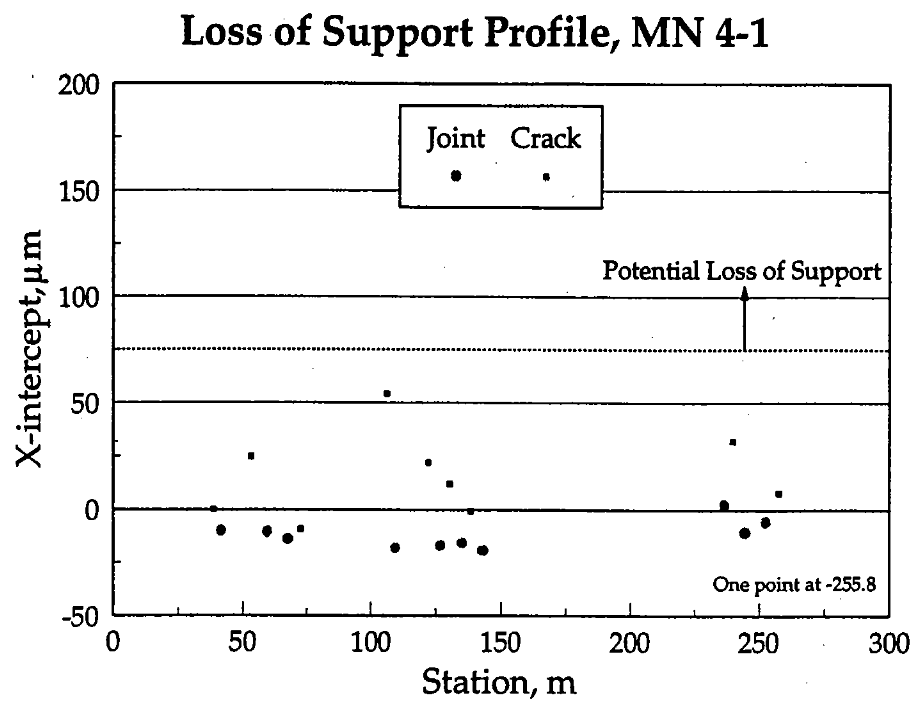


Figure 61. Loss of support profile for MN 4-1 (recycled section).

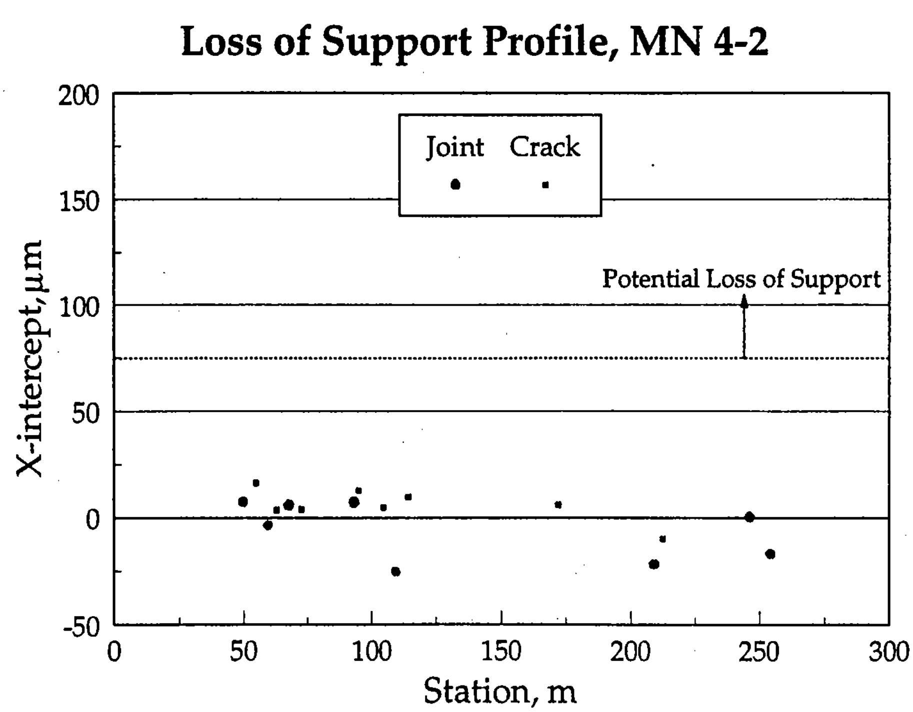


Figure 62. Loss of support profile for MN 4-2 (control section).

the recycled and control sections were 231 and 226 mm (9.1 and 8.9 in). These cores were tested in the laboratory to determine the physical and mechanical properties of the two concrete mixtures used on this project, as described in more detail below.

Core Testing

The number of cores for each laboratory test is indicated in table 47. A summary of the average values that were obtained during the laboratory testing of the field cores is presented below in table 48 and in table 83 in appendix A. Observations made during the testing and comparisons between the performance of the control and recycled sections are also provided below.

Table 47. Number of cores for each laboratory test in MN 4.

Laboratory Tests	Recycled Section	Control Section
Thermal Coefficient	3	3
Split Tensile Strength	1	1
Dynamic Modulus of Elasticity	3	3
Static Modulus of Elasticity	1	1
Compressive Strength	3	3
Volumetric Surface Texture	6	5

Table 48. Core testing results for MN 4.

Property	Recycled	Control
Compressive Strength, MPa	42.8	47.6
Split Tensile Strength, MPa	4.3	4.3
Dynamic Elastic Modulus, GPa	35.4	41.8
Static Elastic Modulus, GPa	30.1	33.3
Thermal Coefficient, $(1 \times 10^{-6})/{}^{\circ}C$	11.6	11.2
VSTR (for Failed Split Tensile Core), cm^3/cm^2	0.1398	n/a
VSTR (for Slab Faces at the Joints), cm^3/cm^2	0.2372	0.2807
VSTR (for Slab Faces at the Cracks), cm^3/cm^2	0.3362	0.2508

Petrographic Examination Summary

The coarse aggregate for the recycled section contain angular-to-rounded gravel rock particles that were observed to be evenly distributed throughout the cement paste. The gravel rock is further characterized as original coarse aggregate containing predominately igneous and metamorphic particles. The coarse aggregate for the control section contains angular gravel rock particles that were also observed to be evenly distributed throughout the cement paste. The gravel rock is further characterized as very fine grained dolomite. A Class C fly ash was included in both the recycled and control concrete mixtures.

The new mortar contents of both the recycled and control materials were estimated using linear traverse techniques. The RCA concrete specimen was found to contain significantly more new mortar than the control concrete (69 percent vs. 51 percent, see table 49). A slight increase in new mortar would be expected, given the higher fly ash content of the recycled mixture. The remainder of the apparent increased new mortar content in the RCA specimen may be due to random variations in aggregate distribution. The RCA concrete specimen also contained an additional 14 percent of old mortar. The net result is an extremely high estimate of mortar content (and low natural coarse aggregate content) for the RCA concrete material, as indicated in table 49. If true, this would be reflected in differences in other properties of the concrete (i.e., strength, elasticity and thermal coefficient of expansion).

Table 49. Coarse aggregate and mortar contents for MN 4.

	Recycled	Control
Coarse Aggregate, %	16.5	48.5
New Mortar, %	69.7	51.5
Recycled Mortar, %	13.9	n/a

Uranyl acetate testing of cores obtained both sections indicated the presence of minor amounts of silica gel in the mortar and around some of the aggregate particles. While this may indicate the presence of alkali-silica reactivity, no such distress was identified during the field surveys.

Mid-Panel Cores

The compressive strengths of the RCA concrete cores ranged between 38.9 and 47.2 MPa (5,640 and 6,840 lbf/in^2), with an average of 42.8 MPa (6,210 lbf/in^2). Compressive strengths for the control section cores ranged between 43.7 and 50.1 MPa (6,340 and 7,270 lbf/in^2), averaging 47.6 MPa (6,900 lbf/in^2). Diametral or split cylinder tensile testing was performed on only one core from each section; strengths of 4.3 MPa (630 lbf/in^2) were obtained for both sections.

The dynamic elastic modulus for the RCA concrete cores ranged from 35.0 to 35.8 GPa (5,080,000 to 5,190,000 lbf/in^2), with an average of 35.4 GPa (5,130,000 lbf/in^2). Control section values ranged from 40.8 to 43.3 GPa (5,920,000 to 6,280,000 lbf/in^2), with an average of 41.8 GPa (6,060,000 lbf/in^2). The static elastic moduli for these sections were estimated using one core from each section; the elastic moduli of RCA concrete and control concrete cores were 30.1 and 33.3 GPa (4,370,000 and 4,830,000 lbf/in^2), respectively. Thus, the results of the dynamic testing suggest that the use of the RCA aggregate resulted in the production of a lower modulus concrete than was obtained using concrete that included only natural coarse aggregate. This conclusion is supported by the results of the static elastic modulus tests, although this data is quite limited.

The thermal coefficient of expansion ranged from 10.7 x 10^{-6}/ °C to 12.4 x 10^{-6}/ °C (5.9 x 10^{-6}/ °F to 6.9 x 10^{-6}/ °F) for the recycled section, with an average of 11.6 x 10^{-6}/ °C (6.5 x 10^{-6}/ °F). The control section thermal coefficients ranged from 10.7 x 10^{-6}/ °C to 12.1 x 10^{-6}/ °C (6.0 x 10^{-6}/ °F to 6.7 x 10^{-6}/ °F) for the control section, with an average of 11.2 x 10^{-6}/ °C (6.2 x 10^{-6}/ °F). The higher total mortar content of RCA concrete would have been expected to produce significantly higher thermal expansion coefficients; this trend was not observed for the samples that were obtained from this project, since the average values obtained cannot be considered significantly different. It is possible that the effects of mortar content were offset by differences in the thermal expansion coefficients and restraining effects of the natural aggregate included in the RCA and control sections. It is also possible that the thermal characteristics of the mortar were not sufficiently different from those of the coarse aggregate particles to produce significant changes in thermal expansion for the two concrete samples. Additional testing would be required to investigate these issues more completely.

In general, the laboratory tests of concrete strength, elasticity, and thermal coefficient of expansion indicate that the control section concrete was significantly stronger and stiffer than the RCA concrete, but that the two materials have comparable coefficients of thermal expansion. The properties of the concrete in both sections are in the range of values expected for typical paving concrete.

Joint and Crack Cores

Visual observations of the RCA concrete cores provided independent verification of several of the conclusions drawn during the petrographic study. For example, it was noted that the coarse aggregate particles are predominately angular but that there are some round particles present. It was also observed that each section contains a uniformly distributed aggregate blend, but that the recycled section had an extremely low coarse aggregate content and high mortar content, with slightly less than half of the mortar coming from recycled particles. A moderate amount of large voids were also present in the paste.

Volumetric surface texture ratios (VSTR's) obtained for cores retrieved from joints in the RCA and control sections are approximately the same, with slightly lower values for the RCA concrete (0.2372 cm^3/cm^2 vs. 0.2807 cm^3/cm^2). VSTR's obtained for cores retrieved from cracks in the RCA and control sections exhibited a different trend (0.3362 cm^3/cm^2 vs. 0.2508 cm^3/cm^2). These values are slightly deceiving. The crack propagated through the aggregate particles in both the control and recycled section. Therefore, the surface texture obtained was due primarily to the path along which the crack propagated. One of the cores pulled from the recycled section contained a crack which propagated in a "C" pattern. This resulted in an extremely high VSTR. The average VSTR for the cracks in the recycled section when this core is not included is 0.2264 cm^3/cm^2. This value is closer to what would be expected for the RCA section because the higher mortar content of the RCA concrete is associated with fewer coarse aggregate particles, thereby resulting in a lower VSTR. In any event, all joints on these sections included steel dowels that appear to be functioning adequately (see previous discussion of FWD test results), so the contribution of fracture plane surface texture is minimal to joint load transfer on this project.

VSTR's for the cracks are higher than for the joints in both sections and it was noted that the slab faces at the joints were much more deteriorated than those at the cracks, which probably developed well after construction and were not subject to as much abrasion due to the passage of heavy traffic loads.

A large entrapped air void was found under one of the two dowels contained within cores retrieved from the recycled section. Dowel bar corrosion was observed in the vicinity of this void. Both cores containing dowels exhibited cracks along the slab face at the depth of the dowel. The dowel bar in one core pulled from the control section was also severely corroded and the concrete had failed in several areas around this dowel. This core also contained a 6.4-mm (0.25-in) thick layer of compacted fines within the joint, which had apparently been pumped up from the base layer.

Almost all of the cores retrieved from transverse cracks were held tightly together by the longitudinal reinforcement. The one exception to this was a core from the RCA section that contained severely corroded steel, which had allowed the crack to open wide. The longitudinal steel had not completely failed in any of the transverse crack cores in either section. One core pulled from the control section contained a crack that was not continuous through the core, which prevented the core from being split for VST testing. In general, cores pulled at transverse cracks in the control section tended to have tighter cracks than those pulled in the recycled section, which helps to explain the relatively high LTE's (average of 94 percent) obtained at the transverse cracks in the control section.

In summary, no significant mechanical differences were observed between the cores retrieved from the RCA concrete and control sections. Both sections should be monitored for future evidence of dowel corrosion and dowel-concrete bearing failure.

Project Summary

This project provides a direct comparison of the performances of recycled concrete and traditional concrete pavement sections constructed in 1984 using identical structural designs (230-mm [9-in] JRCP with an effective steel content of 0.065 percent; 3.7-m [12-ft] outer lanes, 8.2-m [27-ft] transverse joint spacing; 25-mm [1-in] epoxy-coated dowel bars; longitudinal edge drains) and subjected to identical traffic (3.2 million ESAL through 1994) and environmental conditions. The only known differences between the two sections are in the type of coarse aggregate and the mix proportions used in the two sections. The recycled section contains 25-mm (1.0-in) top size, recycled concrete aggregate produced from the pre-existing 53-year-old concrete pavement. The control section contains only 38-mm (1.5-in) top size natural aggregate. In addition, the RCA section contains significantly less coarse aggregate, more fine aggregate and more fly ash than the control section.

The results of a condition survey, deflection testing, and laboratory tests on retrieved cores indicate that these material and mix design differences resulted in stronger, stiffer concrete in the control section, which may be partially responsible for the decreased severity of transverse cracking in that section. A summary of the key points of the evaluation follows:

Pavement Design

The absence of significant joint faulting suggest that the load transfer and drainage designs used have been adequate for the load and environmental conditions experienced thus far. It is worth noting, however, that there were cracks present in the plane of the dowel bars within the two RCA concrete section cores that contained dowels. These may suggest an imminent failure of the joint load transfer systems on this section. Primary candidate reasons for this cracking include corrosion (which seems quite possible) and high bearing stresses in dowels with inadequate concrete cover (which seems less likely).

Problems with joint spalling and deteriorated transverse cracking (RCA section only) were of greater concern on this project. While concrete panels of this length (8.2 m [27 ft]) are expected to crack, the longitudinal reinforcement is intended to hold these cracks tight and provide good load transfer capacity while preventing crack deterioration. The relatively low longitudinal steel content (0.065 percent) of these sections is at least partially responsible for the deterioration of transverse cracks in the RCA section, although all cracks in the control section, which was comparably reinforced, are still tight. Since the thermal coefficients of the RCA and control concrete are comparable, there are no readily-apparent reasons for the difference in performance between the cracks in these two sections. It is possible that the RCA sections cracked earlier in the pavement's life than did the control section (since the strength of the RCA concrete is significantly lower than that of the control section concrete), which would mean that the RCA section cracks have been subjected to more heavy vehicle action since forming. If this is the reason for the difference in performance between the two sections, then the control section cracks can be expected to deteriorate as heavy vehicle loadings accumulate.

The difference in the number of deteriorated transverse cracks between the recycled and control section may be due to several causes. First, the recycled section has many more transverse cracks (including low-severity cracks) than the control section, and thus a greater potential for deterioration. Second, the recycled concrete aggregate consists of original aggregate and cement paste, which is typically softer and provides less resistance to abrasion at the crack face. Another possible reason is the smaller aggregate top size of the recycled concrete aggregate, which provides less interaction between aggregate particles. More than likely, the increased number of deteriorated transverse cracks is a result of the combination of all these factors.

One possible reason for earlier cracking of the RCA section is the difference in the supporting foundation stiffness (backcalculated k-values for the RCA section averaged 30 percent lower than for the control section and midpanel deflection measurements were higher in the RCA section). It is also possible that the drying shrinkage of the RCA concrete was much higher than that of the control section concrete, which would have produced larger "subgrade drag-related" tensile forces in that section, causing higher steel strains and greater crack widths.

Both sections exhibited significant amounts of low-severity joint spalling, which may be due to localized failures of the silicone joint sealant.

Materials Properties

As noted previously, the RCA mixture contained less coarse aggregate and more fly ash than the control mixture. In addition, the recycled coarse aggregate is composed of both natural aggregate particles and old mortar, resulting in a much higher total mortar content than for the control section concrete. These mix characteristics are probably most responsible for the reductions in strength and elasticity observed for the RCA concrete (10 percent lower compressive strength, 10 to 15 percent lower modulus of elasticity). They are probably also responsible for the slightly lower VSTR measurements obtained for the RCA concrete cores. However, the expected increase in thermal coefficient was not observed, possibly because of offsetting thermal coefficients for the natural aggregates in each material or because the thermal coefficients of the mortars are not much different than those of the aggregates. This is an area that bears additional study.

Control section VSTR's for both the cracks and joints were slightly higher than for the RCA concrete, although the difference was not considered significant. VSTR's for the cracks in each section were significantly higher than those obtained at the joints.

Fractures at the joints tended to propagate around the aggregate particles, while those at the transverse cracks tended to go through more aggregate particles.

In spite of the property differences noted above, the concrete in both sections have physical and mechanical properties that are in the range of values expected for typical paving concrete.

Uranyl acetate testing for both sections suggest that minor amounts of silica gel deposits are present in the mortar and around some of the aggregate particles, indicating the possible presence of ASR activity.

Pavement Performance

The recycled concrete section exhibited significantly more transverse cracking than did the control section. Possible reasons for this are cited in the pavement design summary section, above. The higher number of transverse cracks (medium- and high-severity) in the recycled concrete section (there were none in the control section) is probably a primary reason for the lower PSR in that section.

Other performance measurements, such as faulting and spalling, were about the same on both sections. Neither section had faulted significantly, but there is a lot of low severity joint spalling in both sections, with slightly more observed in the control section. The amount of joint spalling was directly related to the amount of joint seal damage observed in each section. As mentioned previously, it is possible that the rate of joint sealant failure in each section was influenced by the amount of limestone present in each mixture. Recent studies have shown that silicone sealants are more prone to failure when the concrete contains significant quantities of limestone coarse aggregate.

The dowel bar contained within the core pulled from the control section was severely corroded and the concrete had failed in several areas around the dowel. In addition, the joint was wide and packed with fines that had apparently pumped up from the base layer. If this condition is widespread, the control section can be expected to deteriorate within the next few years.

Overall

The findings of this project study suggest that the recycled concrete section is deteriorating more rapidly than the control section, although the present serviceability of both sections is still quite good after 10 years of moderate traffic in a harsh environment. Differences in cracking may be caused by one or more of the following factors: earlier crack formation in the RCA concrete due to its suspected (but not proven) relatively higher shrinkage; earlier crack development in the RCA concrete section due to the lower strength of the RCA concrete; or reduced support under the RCA concrete section. The reason for these differences in cracking should be investigated further. Corrosion of the steel dowels and mesh reinforcing was also noted, in spite of the use of epoxy-coated dowels.

Although the strength and elasticity of the RCA concrete were generally lower than for the control concrete, the physical properties of both materials appear to have been well within accepted norms for paving concrete. Thus, it would seem that the two sections should perform comparably in the long term. Continued monitoring of these sections is suggested, since the RCA section is clearly starting to deteriorate and the control section is exhibiting signs of dowel and steel corrosion that could lead to rapid development of distress in the near future.

Wisconsin 1, I-94 near Menomonie

This project included two sections of pavement, both constructed using recycled concrete aggregate. The chief difference between the sections is that one includes dowel bars at the transverse joints and the other does not contain any mechanical load transfer devices.

<u>Project Information</u>

This project is a 21.7-km (13.5-mi) recycled concrete pavement section located in the eastbound lane of I-94 near Menomonie, Wisconsin. The original pavement was a JRCP constructed in 1959; it was reconstructed in 1984. The 25-year-old concrete from the original pavement was crushed and used as coarse aggregate in the concrete surface of the new pavement.

<u>Design Information</u>

The recycled concrete aggregate was used to construct a 280-mm (11-in) JPCP. The pavement is constructed on a 150-mm (6-in) aggregate base and a 230-mm (9-in) granular subbase, both composed of natural aggregate. The transverse joints are skewed and spaced at intervals of 3.7-4.0-5.8-5.5 m (12-13-19-18 ft). WI 1-2 contains 35-mm (1.38-in) epoxy-coated dowel bars at the transverse joints. WI 1-1, on the other hand, contains no load transfer devices. Both sections have a 150-mm (6-in) thick tied PCC shoulder on a 280-mm (11-in) aggregate base course. The shoulder and centerline joints contain 610-mm (24-in) long, 13-mm (No. 4) epoxy-coated tie bars spaced 1200 mm (48 in) apart. No provisions for drainage are incorporated into the design.

<u>Mix Design</u>

Detailed information concerning the concrete and base aggregates is not available for the Wisconsin sections. However, the concrete pavement in both sections is known to consist of recycled concrete coarse aggregate and natural sand fine aggregate. The gradations of the aggregate are believed to conform with the limits presented in table 50. Precise concrete mixture proportions are also unavailable.

Table 50. Probable concrete aggregate gradations for WI 1.

Sieve	Recycled Mix	
	Coarse	Fine
51 mm (2.0 in)	100	
38 mm (1.5 in)	90-100	
25 mm (1.0 in)	20-55	
19 mm (3/4 in)	0-15	
9.53 mm (3/8 in)	0-5	100
4.75 mm (No. 4)		90-100
1.18 mm (No. 16)		45-80
0.300 mm (No. 50)		10-30
0.150 mm (No. 100)		2-10

Construction Information

The 25-year-old JRCP was removed and crushed to provide aggregate for construction of the new pavement. Recycled concrete aggregate was used only for the coarse aggregate portion of the reconstructed pavement sections. The concrete was placed using construction techniques consistent with those used for conventional concrete pavements in Wisconsin. After placing the concrete, the pavement surface was tined in the transverse direction and a liquid membrane curing compound was applied.

Climatic Conditions

The WI 1 test sections are located in the wet-freeze environmental region. The area experiences about 115 days of precipitation per year, totaling an average of 760 mm (30 in) of precipitation annually. The Thornthwaite moisture index at this site is about 30. The freezing index averages about 1,050 °C-days (1,900 °F-days), and the sections are exposed to about 102 freeze-thaw cycles per year. The minimum and maximum average monthly temperatures are -10 and 22 °C (14 and 71 °F), respectively.

Traffic Loadings

These pavement sections were opened to traffic in 1984, at which time the two-way ADT was about 12,400 vehicles per day. As of 1994, the two-way ADT had increased to approximately 16,700 vehicles per day, including about 20 percent truck traffic. Based on this information, the number of ESAL applications from the time the pavement was opened to traffic through 1994 is approximately 7.0 million.

Selection of Distress Survey Section

The sections selected for detailed survey were both approximately 305 m (1,000 ft) long. The section with undoweled joints began at milepost 39.6 and extended eastward to a point near milepost 39.8. The section with doweled joints started at milepost 40.1 and extended eastward to a point near milepost 40.3. Both sections were constructed at grade (no significant cut or fill) with a crowned cross-section. A few weeks before the detailed survey was performed, the entire undoweled portion of the project was texture planed (diamond ground). The project team had no advance notice of this operation, which forced some modifications to the distress survey procedures for measuring joint and crack faulting, as described below.

Drainage Survey

The undoweled section was constructed on a 0.5 percent longitudinal grade (elevation decreasing to the east). Transverse slopes varied from 2.5 percent on the outer traffic lane to 4.0 percent on the outer shoulder. The doweled section, on the other hand, was constructed on a level grade with transverse slopes of 1.5 and 3.0 percent on the outer traffic lane and outer shoulder, respectively. Neither section contains any elements for controlling subsurface drainage (i.e., longitudinal edge drains or a permeable base layer). Signs of low- to medium-severity pumping were observed within the undoweled section on the inside lane. However, no such signs were observed within the doweled section.

Pavement Distress Survey

The pavement condition survey was conducted over 305-m (1,000-ft) sections of the doweled and undoweled portions of the project, as described previously. A complete summary of the results of the survey is provided in appendix A. A summary of the average results for key distress and performance measures is presented in table 51. It should be noted that the measurement of transverse joint faulting along section WI 1-1 was accomplished across the joints of the tied concrete shoulder (which was not texture planed) adjacent to the outer traffic lane. This was done because the traffic lanes had been texture planed, removing all joint faulting.

Overall, the doweled section has performed better than the undoweled section, especially in terms of faulting and spalling at the transverse joints. Neither section was seriously distressed and both exhibited good ride quality, although it must be remembered that the undoweled section had recently been texture planed to eliminate faulting and ride quality problems.

Table 51. Summary of performance and distress data (average values) for WI 1.

Performance Measurement	Undoweled*	Doweled
Corner Faulting, mm (Manual)	0.0	0.3
Wheelpath Faulting, mm (Manual)	2.0	0.3
Wheelpath Faulting, mm (Digital)	2.8	0.5
Transverse Cracking, % Slabs	8	2
Longitudinal Cracking, m/km	0	0
Transverse Joint Spalling, % Joints	97	23
Joint Width, mm	10	11
PSR	4.1	3.8

* Note: Measurement of transverse joint faulting was performed across the joints of the tied concrete shoulder (which was not texture planed) directly adjacent to the outer traffic lane.

Transverse Joint Faulting

As previously mentioned, section WI 1-1 contained no transverse joint load transfer devices, while section WI 1-2 contained 35-mm (1.38-in) dowel bars. Consequently, the undoweled section developed severe faulting (as evidenced by the need for texture planing) while faulting on the doweled section was minimal. This performance illustrates the importance of load transfer devices on pavements that are subjected to high volumes of heavy traffic in climates that produce large temperature variations and large amounts of precipitation. Furthermore, the need for mechanical joint load transfer devices may be even more acute on pavements that are constructed using RCA concrete pavement; the smaller particle sizes that are often used to prevent recurrent D-cracking, the possible decreased abrasion resistance of these particles and the typically larger coefficient of thermal expansion of the concrete probably decrease the reliability of aggregate interlock load transfer in these pavements.

Cracking

Approximately 8 percent of the slabs in the undoweled section exhibited transverse cracking, compared to 2 percent within the doweled section. All transverse cracks observed in either section were rated as low severity. On the undoweled section, transverse cracking occurred predominately in the 5.8-m (19-ft) slabs, with only one crack forming in the 5.5-m (18-ft) slabs and no cracking in the shortest panel sizes. This suggests that increased curling and warping stresses (associated with longer panel lengths and losses of foundation support) may have combined with traffic load-related stresses to produce these cracks. It is commonly assumed that the ratio of slab length to its radius of relative stiffness must be less than 5.5 when the pavement is on a softer foundation or excessive cracking will occur. The values of this ratio for the 5.5- and 5.8-m (18- and 19-ft) slabs are 5.00 and 5.29, respectively, for the undoweled section and 5.31 and 5.60, respectively, for the doweled section. Therefore, it is not surprising that some of these longer slabs have cracked. The only crack observed within the doweled section was located directly over a concrete culvert, presumably due to a localized problem with foundation consolidation.

Longitudinal cracks were not observed on either section.

Transverse Joint Spalling

Transverse joint spalling was quite extensive on the undoweled section, occurring at 97 percent of the transverse joints, including 79 percent low-severity spalls and 18 percent medium-severity spalls. Spalling on the doweled section was observed at 23 percent of the transverse joints, with only 13 percent being medium- or high-severity spalls. The higher incidence of transverse joint spalling on the undoweled section can probably be attributed to increased differential movements in the absence of good load transfer, together with the intrusion of incompressibles into these poorly sealed joints.

Most of the spalling is only low severity and seems to have had little effect on the overall performance of the pavement sections to date.

Present Serviceability Rating (PSR)

The average PSR of the undoweled section was 4.1, slightly higher than the 3.8 PSR of the doweled section. This phenomenon is attributed to the recent texture planing of the undoweled section, which virtually eliminated all faults and surface irregularities. Prior to grinding, it is likely that the PSR of the undoweled section was substantially lower.

<u>FWD Testing</u>

Pavement deflection testing was performed at 5 slab center locations, the approach and leave sides of 10 transverse joints, at 10 locations along the lane/shoulder joint, and at the approach and leave sides of the four cracks in the undoweled section and the lone crack in the doweled section. The results of this testing were used to determine the PCC elastic modulus, effective modulus of subgrade reaction, load transfer efficiencies across joints and cracks, and to identify loss of support under slab corners. A summary of the average values for these parameters, as computed from the deflection data, is provided in table 52.

Table 52. Deflection testing results for WI 1.

Property	Undoweled	Doweled
Elastic Modulus, GPa	46.3	29.0
k-value, kPa/mm	36.4	45.6
Joint Load Transfer, %	32	74
Crack Load Transfer, %	48	59
Shoulder Load Transfer, %	94	98
Average Midslab Deflection, µm	96	105
Average Edge Deflection, µm	116	120
Corners With Voids, %	10	0
Maximum Air Temperature During Testing, °C	16	16

PCC Elastic Modulus

The elastic modulus (E) of the concrete slab was backcalculated using the center-of-slab deflection measurements. Figure 63 shows the variation in backcalculated elastic modulus along the undoweled recycled section. The backcalculated elastic modulus values ranged from 41 to 78 GPa (5,950,000 to 11,300,000 lbf/in^2), with an average value of 46.3 GPa (6,710,000 lbf/in^2). Elastic modulus values obtained from dynamic lab tests of drilled cores averaged 32.3 GPa (4,680,000 lbf/in^2) and exhibited much less variability than backcalculated values.

Figure 64 presents similar data for the doweled section. The average elastic modulus of the concrete slab is 29.0 GPa (4,210,000 lbf/in^2), with values ranging from 19 to 44 GPa (2,800,000 to 6,400,000 lbf/in^2). The two test locations at the east end of the project yielded much higher elastic modulus values than the other three locations. The reason for these differences is unknown and little variability was observed between values backcalculated using different FWD loads at a given test location. Elastic modulus values obtained from dynamic lab tests of drilled cores averaged 32.1 GPa (4,660,000 lbf/in^2) and exhibited much less variability than backcalculated values.

The average backcalculated elastic modulus value of the undoweled section is more than 50 percent higher than that of the doweled section; there is much better agreement between the values measured during laboratory testing of drilled cores. The lab test results appear to offer the more reasonable values, since both sections are believed to be constructed using identical materials.

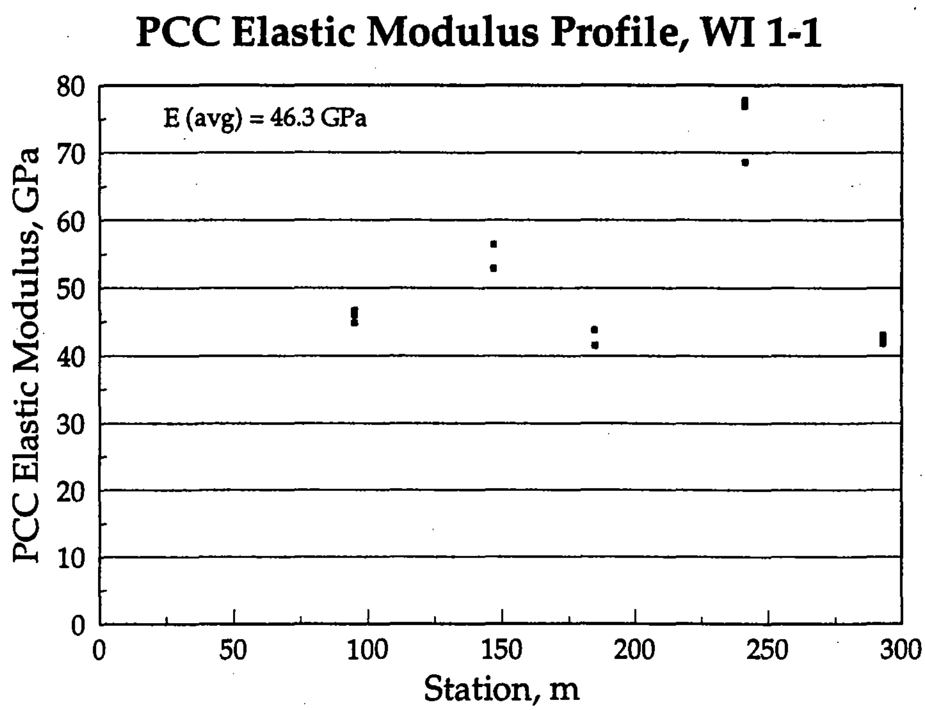


Figure 63. PCC elastic modulus profile for WI 1-1 (undoweled section).

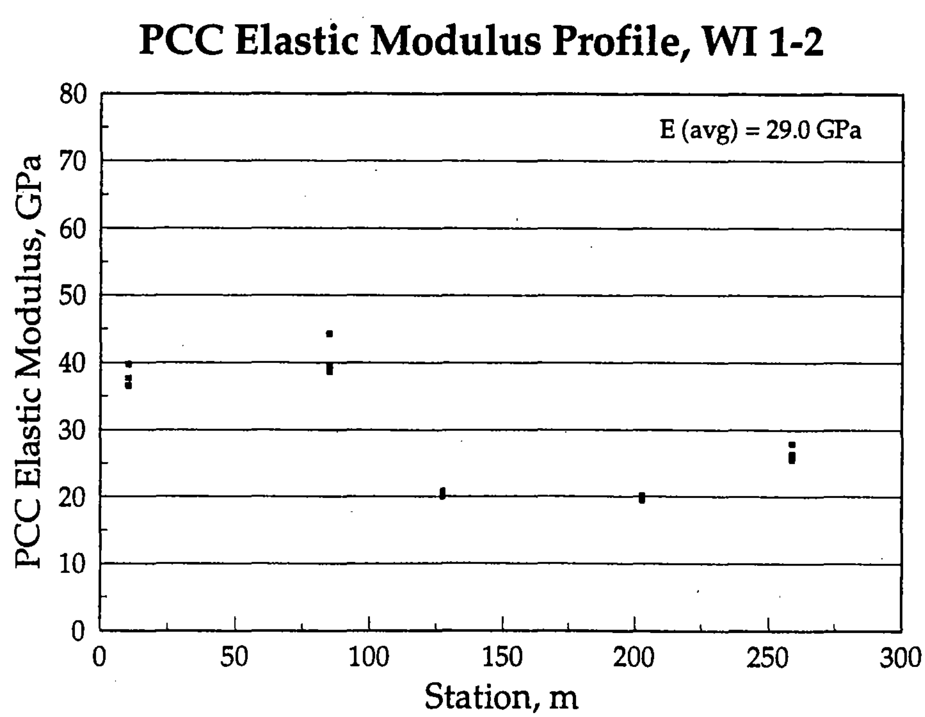


Figure 64. PCC elastic modulus profile for WI 1-2 (doweled section).

Effective Modulus of Subgrade Reaction (k-value)

Deflection testing was performed during October 1994; thus, the backcalculated subgrade modulus values obtained are representative only of the conditions that were present at that time. Variations in backcalculated subgrade modulus along the undoweled section are illustrated in figure 65. These values range from 16 to 47 kPa/mm (58 to 172 lbf/in2/in), with an average of 36.4 kPa/mm (134 lbf/in^2/in). Similarly, figure 66 presents a plot of effective modulus values along the length of the doweled pavement sample, which ranged from 28 to 56 kPa/mm (104 to 206 lbf/in^2/in), and averaged 45.6 kPa/mm (168 lbf/in^2/in).

As with the backcalculated PCC elastic modulus values, there are some unexplained differences between the effective subgrade modulus values computed for the doweled and undoweled sections. It is interesting to note that the unusually low backcalculated subgrade values correspond to unusually high backcalculated PCC elastic modulus values, while the peak deflection values and laboratory test values exhibit little variability. This suggests that the backcalculation results may not be as reliable or accurate as the lab test results.

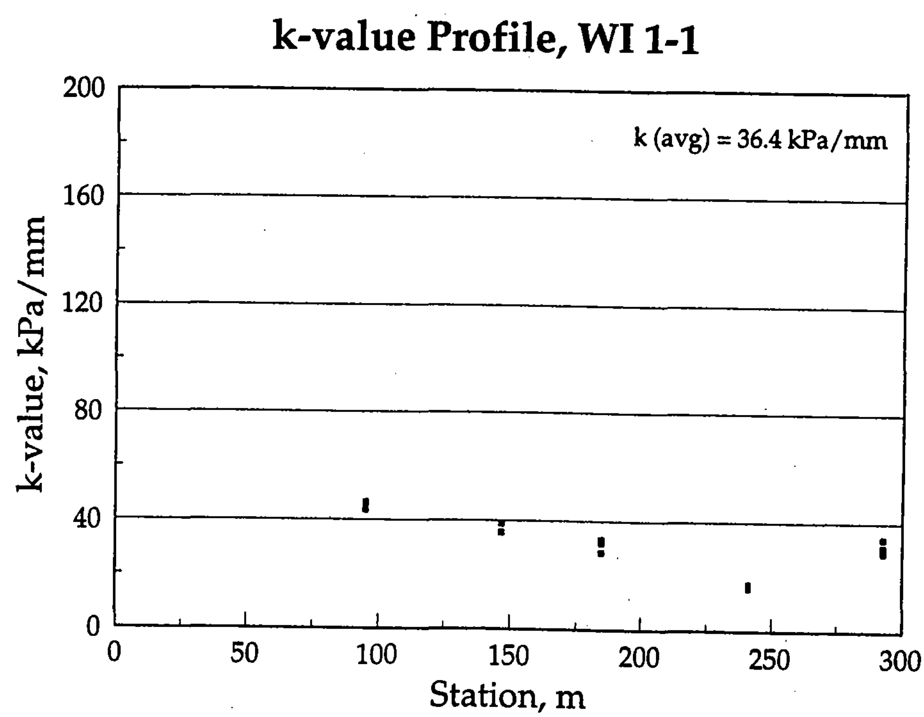


Figure 65. K-value profile for WI 1-1 (undoweled section).

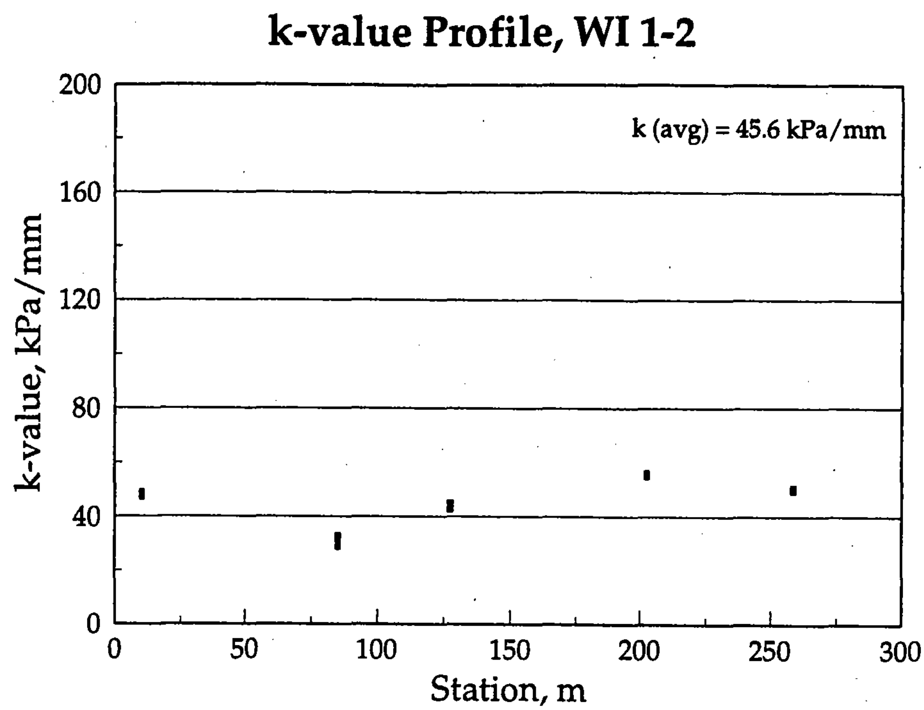


Figure 66. K-value profile for WI 1-2 (doweled section).

Joint Load Transfer

The load transfer efficiencies at both the approach and leave joints of the undoweled section are shown in figure 67. The load transfer efficiencies represent the ratio of the deflection on the loaded side of the joint to the deflection on the unloaded side of the joint. Little variation was observed between values computed from the results of tests performed at a single location using loads of different magnitude or between tests performed on either side of the joints. The average load transfer efficiencies at the approach and leave sides of the joints were 31 and 32 percent, respectively. Figure 68 illustrates the joint load transverse efficiencies for the doweled section. Again, the average load transfer efficiencies at the approach and leave joints are nearly the same, with values of 75 and 74 percent, respectively. As expected, these values are significantly higher than the load transfer efficiencies on the undoweled section, indicating the effectiveness of the dowel bars at transferring load and stress.

Crack Load Transfer

The load transfer efficiencies at the approach and leave cracks of the undoweled section are illustrated in figure 69. The average load transfer efficiency is 48 percent, with efficiencies for approach and leave side loadings averaging 53 percent and 43 percent, respectively. For this section, significantly better load transfer is exhibited at

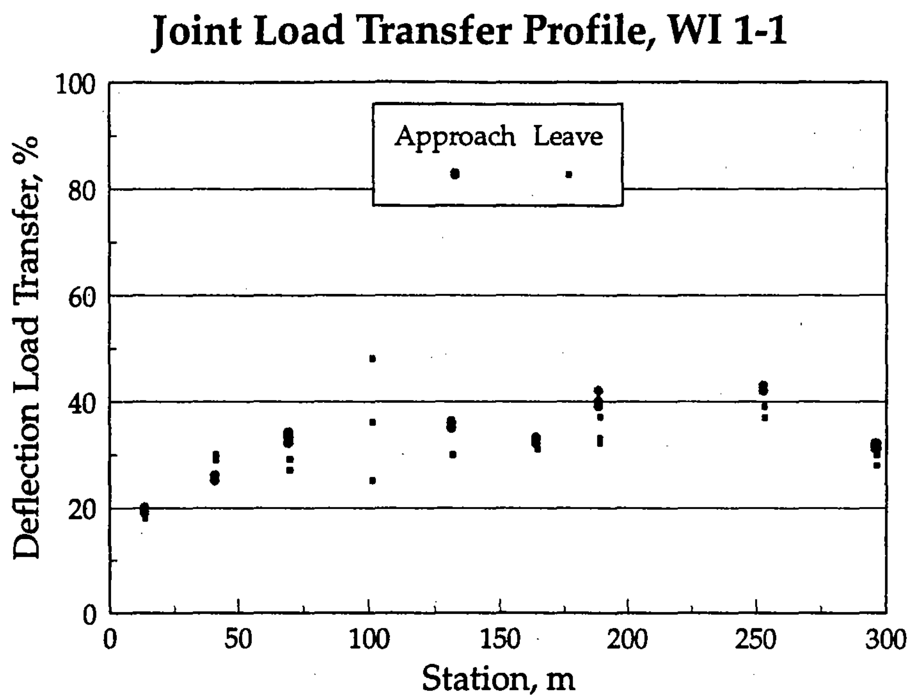


Figure 67. Joint load transfer profile for WI 1-1 (undoweled section).

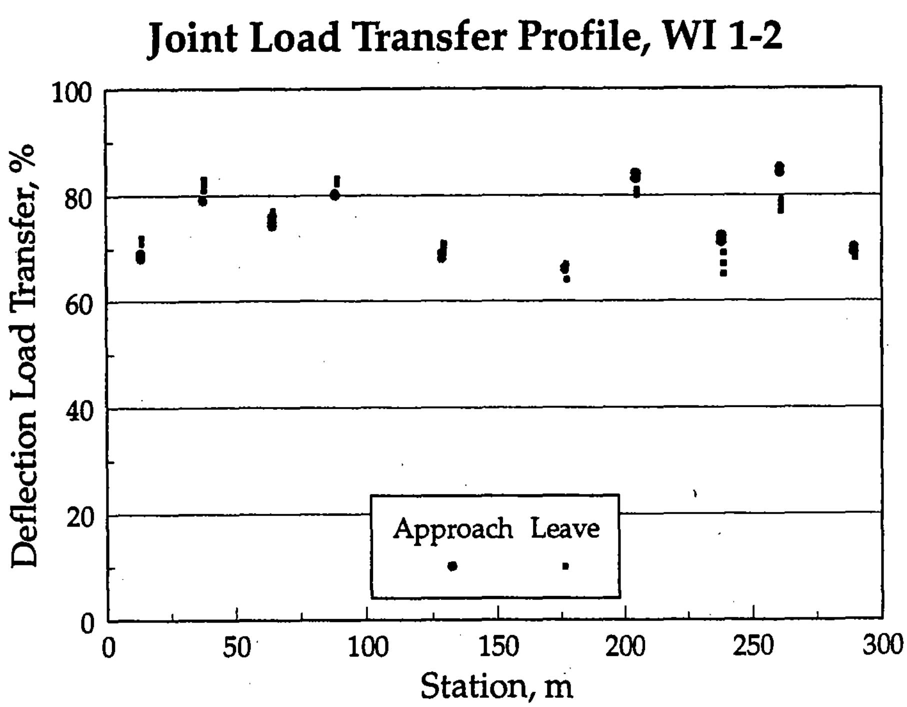


Figure 68. Joint load transfer profile for WI 1-2 (doweled section).

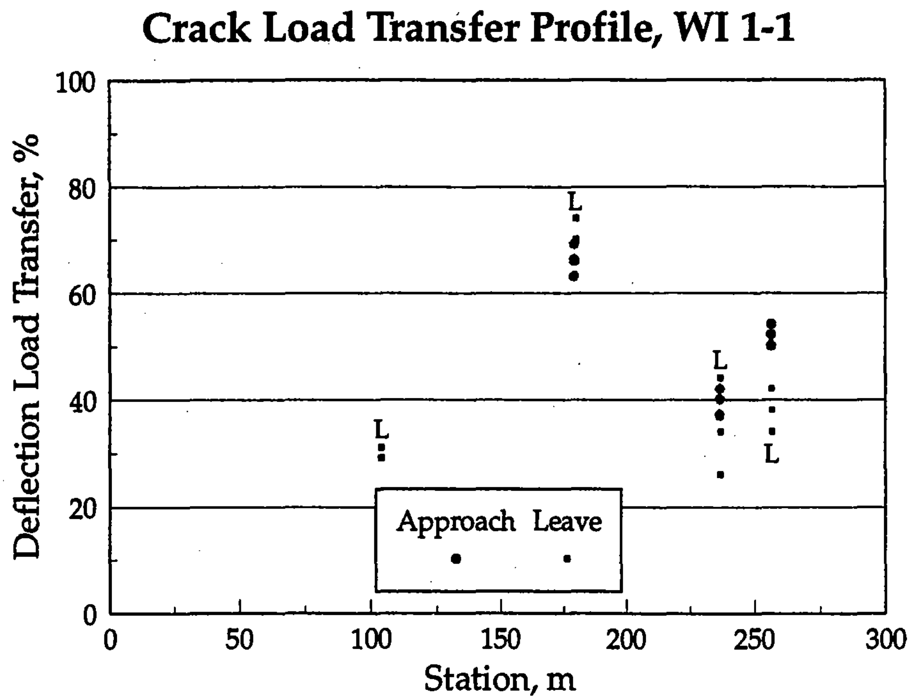


Figure 69. Crack load transfer profile for WI 1-1 (undoweled section).

the transverse cracks than at the transverse joints (48 percent vs. 32 percent), even though both rely solely on aggregate interlock for load transfer. One possible explanation for this phenomenon is that the effective panel length is reduced when the unreinforced slab cracks so the crack widths will most likely be less than or equal to the joint widths. For example, if a 3.7-m (12-ft) and 5.5-m (18-ft) panel lie next to each other and the 5.5-m (18-ft) panel has a midpanel crack making the effective panel length 2.7 m (9 ft), then the joint and crack widths would be 4.2 mm (0.164 in) and 2.9 mm (0.113 in), respectively. These joint/crack widths assume the temperature of the pavement at the time the concrete set was 29 °C (85 °F) and the temperature of the pavement at the time the crack width was measured was 16 °C (60 °F). This assumption can be made for both sections since all cracks are in either an 5.5- or 5.8-m (18- or 19-ft) panel and they are all midpanel cracks. In addition, all of the observed cracks are of low severity.

The load transfer efficiencies at the approach and leave side of the crack found in the doweled section are illustrated in figure 70. This single crack developed directly over a concrete culvert and was probably associated with construction-related difficulties associated with the placement of the culvert. The average load transfer efficiency at the crack is 59 percent (significantly less than that observed at the doweled joints, but comparable to the crack transfer efficiencies measured in the undoweled section), with nearly identical values obtained at the approach and leave sides of the crack.

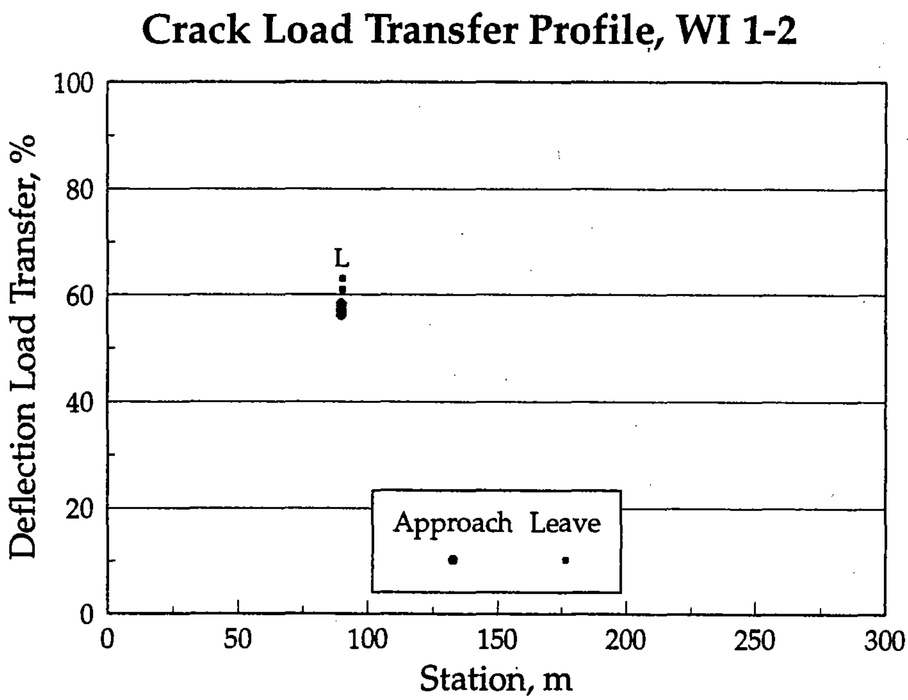


Figure 70. Crack load transfer profile for WI 1-2 (doweled section).

In past studies of recycled concrete pavement, deterioration of the transverse crack faces has been a primary concern. The transverse cracks in these recycled concrete sections have not yet deteriorated significantly, but may have formed only recently. They should be monitored for possible deterioration in the near future.

Shoulder Load Transfer

The load transfer efficiencies across the tied PCC shoulder are illustrated in figures 71 and 72 for the undoweled and doweled sections, respectively. The tied PCC shoulders on the undoweled and doweled sections were paved separately from the mainline pavement. The average load transfer efficiency is 94 percent on the undoweled sections and 98 percent on the doweled section. With one exception, the load transfer efficiencies are greater than 90 percent at each location. The one exception occurs on the undoweled section, where a value of 52 percent was obtained. This low value may be the result of improper construction or failure of the tie bar system within this localized area.

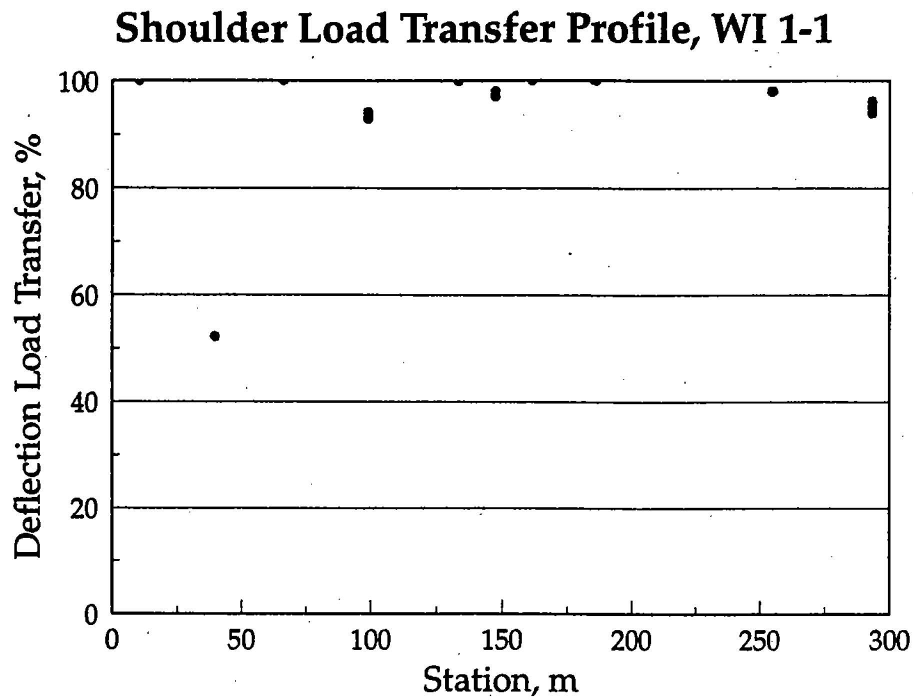


Figure 71. Shoulder load transfer profile for WI 1-1 (undoweled section).

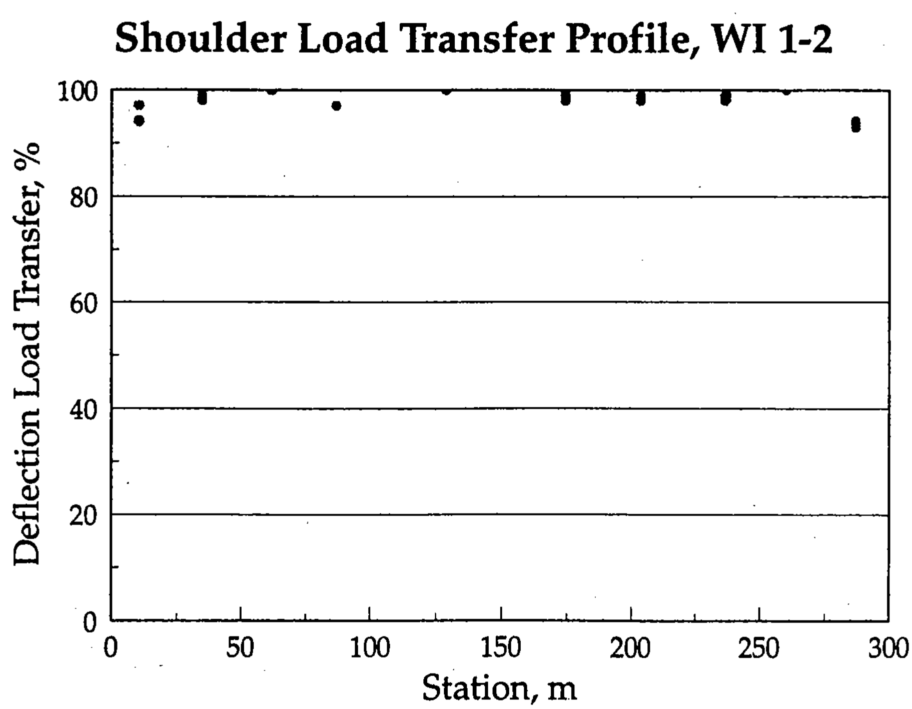


Figure 72. Shoulder load transfer profile for WI 1-2 (doweled section).

Loss of Support

The detection of voids was performed using the corner deflections on the leave side of transverse joints and cracks. Figures 73 and 74 show the loss of support profile for the undoweled and doweled sections, respectively. The undoweled section shows a potential for loss of support at one joint and at one crack. The doweled section, on the other hand, does not show any potential loss of support. The increased vertical differential movement at the undoweled transverse joints results in more pumping, which creates voids under the leave slab.

Coring

Eleven cores were retrieved from the undoweled section, and nine cores were taken from the doweled section. For each section, five cores were drilled from the midpanel regions and three were taken through transverse joints (in the outer lane and outer wheel path). In addition, three cores were taken through transverse cracks in the undoweled section (again in the outer wheel path) and one core was taken through the only transverse crack found in the doweled section. All cores were 150 mm (6 in) in diameter and were an average of 277 and 279 mm (10.9 and 11.0 in) thick in the undoweled and doweled sections, respectively. These cores were tested in the laboratory to determine the properties of the recycled aggregate concrete.

No cores or samples were obtained from the aggregate base course.

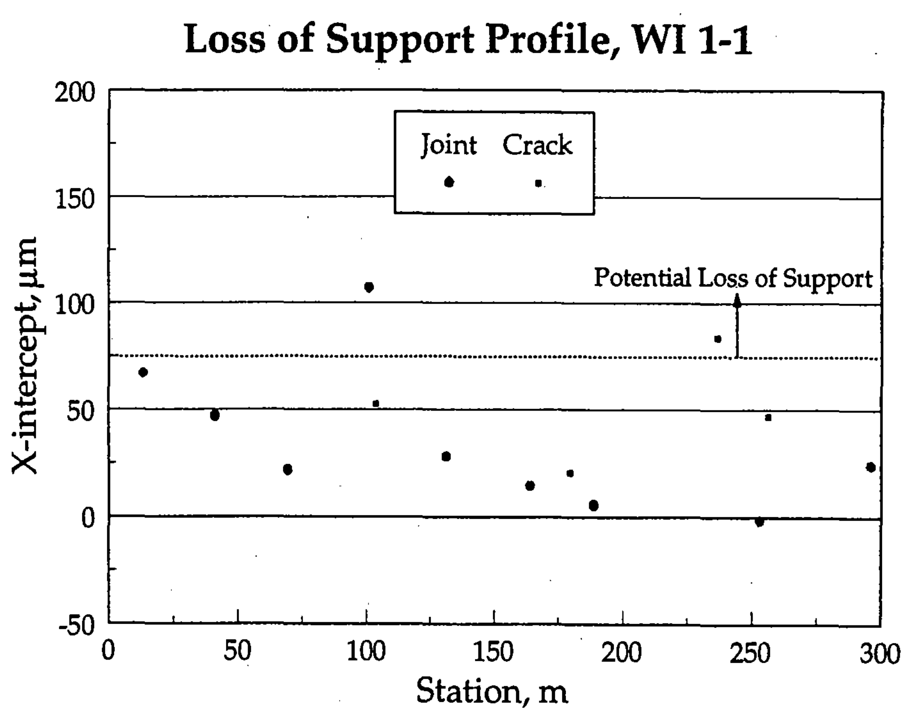


Figure 73. Loss of support profile for WI 1-1 (undoweled section).

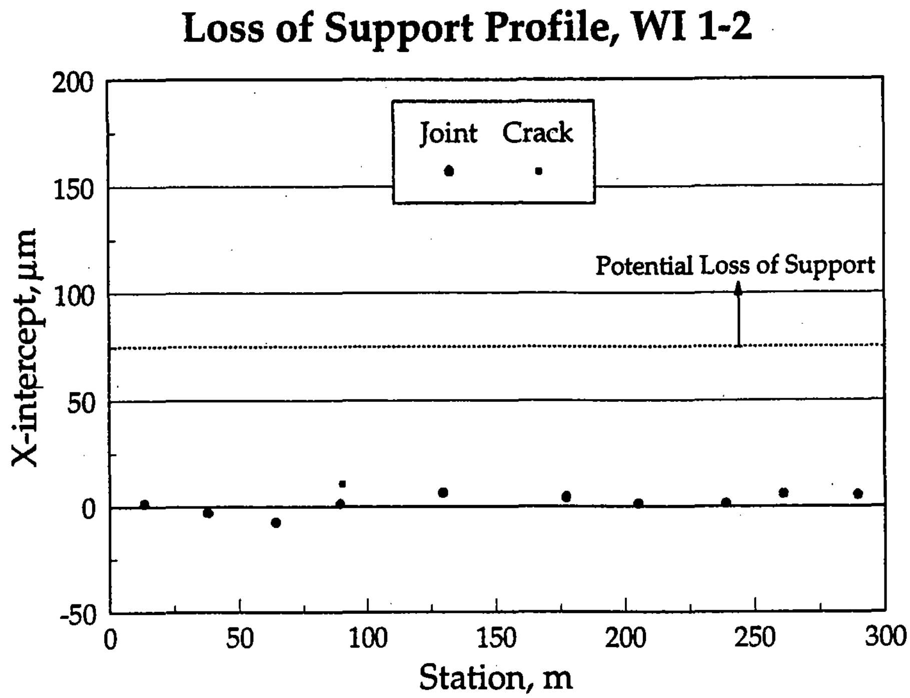


Figure 74. Loss of support profile for WI 1-2 (doweled section).

Core Testing

The number of cores for each laboratory test is indicated in table 53. A summary of the average values obtained during the laboratory testing of the field cores is presented in table 54. Comparisons and observations made during the testing are provided below.

Table 53. Number of cores for each laboratory test in WI 1.

Laboratory Tests	Undoweled Section	Doweled Section
Thermal Coefficient	3	3
Split Tensile Strength	1	1
Dynamic Modulus of Elasticity	3	3
Static Modulus of Elasticity	1	1
Compressive Strength	4	4
Volumetric Surface Texture	6	4

Table 54. Core testing results for WI 1.

Property	Undoweled	Doweled
Compressive Strength, MPa	34.2	35.1
Split Tensile Strength, MPa	3.0	3.0
Dynamic Elastic Modulus, GPa	32.3	32.1
Static Elastic Modulus, GPa	29.0	28.0
Thermal Coefficient, $(1 \times 10^{-6})/\ ^{\circ}C$	11.3	12.5
VSTR (for Failed Split Tensile Core), cm^3/cm^2	0.4223	0.4167
VSTR (for Slab Faces at the Joints), cm^3/cm^2	0.3682	0.3980
VSTR (for Slab Faces at the Cracks), cm^3/cm^2	0.5833	0.3852

Petrographic Examination

The cores obtained from the doweled and undoweled sections contain rounded to angular coarse aggregate particles that originated in gravel rock deposits. The gravel rock is further characterized as original coarse aggregate containing igneous and metamorphic particles. The aggregate particles in the undoweled section cores were evenly distributed throughout the cement paste while the aggregate particles in the doweled section cores were distributed somewhat unevenly. Both sections have approximately the same high mortar content (see table 55).

Table 55. Coarse aggregate and mortar contents for WI 1.

	Undoweled	Doweled
Coarse Aggregate, %	17.0	15.6
New Mortar, %	66.0	69.8
Recycled Mortar, %	17.0	14.7

Mid-Panel Cores

Compression and split tensile strengths for both recycled projects are low relative to the other projects, but still well within typical specifications. The compressive strengths ranged from 31.8 to 37.1 MPa (4,610 to 5,380 lbf/in^2) for the undoweled section, with an average of 34.2 MPa (4,960 lbf/in^2). The compressive strengths ranged from 28.9 to 37.6 MPa (4,190 to 5,450 lbf/in^2) for the doweled section, with an average of 35.1 MPa (5,090 lbf/in^2). The split tensile values obtained for both sections are the result of only one test per section. A split tensile strength of 3.0 MPa (440 lbf/in^2) was obtained for each section.

The dynamic elastic modulus for the undoweled section ranged from 31.0 to 33.9 GPa (4,500,000 to 4,920,000 lbf/in^2), with an average of 32.3 GPa (4,680,000 lbf/in^2). The dynamic elastic modulus for the doweled section ranged from 31.7 to 32.7 GPa (4,600,000 to 4,740,000 lbf/in^2), with the average of 32.1 GPa (4,660,000 lbf/in^2). Static elastic moduli for both sections are based on only one test each, with the undoweled section being 29.0 GPa (4,210,000 lbf/in^2) and the doweled section being 28.0 GPa (4,060,000 lbf/in^2). It should be noted that these static elastic modulus values were the lowest of the field projects being considered herein.

Thermal coefficients ranged from 10.7 x 10^{-6}/ °C to 12.2 x 10^{-6}/ °C (6.0 x 10^{-6}/ °F to 6.8 x 10^{-6}/ °F) for the undoweled section, with an average of 11.3 x 10^{-6}/ °C (6.3 x 10^{-6}/ °F). Thermal coefficients ranged from 12.3 x 10^{-6}/ °C to 12.8 x 10^{-6}/ °C (6.8 x 10^{-6}/ °F to 7.1 x 10^{-6}/ °F) for the doweled section, with an average of 12.5 x 10^{-6}/ °C (6.9 x 10^{-6}/ °F).

With the exception of the thermal coefficient for the doweled section being higher than that of the undoweled section, all laboratory tests showed the properties of the two pavement sections were comparable. It was noted that the compression, split tensile, and static elastic modulus test values were sufficient for pavement construction, but were low compared to the values associated with other recycled pavement sections.

Joint Cores

Cores pulled from the joints in the undoweled section exhibited severe deterioration and large crack widths. A significant amount of fines was observed on the crack faces. This could be from pumping of the base layer or from the entry of fine deicing materials entering through the unsealed joints at the pavement surface. Cracks tended to propagate around the aggregate particles at the joints, but on a relatively straight plane. The lower 38 mm (1.5 in) of the two cores that were pulled from the doweled joints (but did not contain dowels) were observed to be spalled. This could be related to the presence of ASR, which was indicated in moderate amounts by uranyl acetate tests.

The core containing the dowel exhibited cracks extending along the dowel. The crack propagated along the bottom of the dowel on the approach side and along the center portion of the dowel on the leave side for the entire width of the core. The bottom portion of the dowel is severely corroded. All of these factors may contribute to the low LTE (59 percent) obtained for the doweled joints.

The VSTR's obtained for this project are high, indicating excellent potential for load transfer via aggregate interlock provided that the undoweled joint widths are adequately small. However, the smallest computed joint width is 4.77 mm (0.19 in), which is significantly greater than the 0.76 mm (0.03 in) typically assumed necessary for good aggregate interlock load transfer. VSTR's for the doweled section are slightly higher than the undoweled section (0.3980 cm^3/cm^2 compared to 0.3682 cm^3/cm^2). This difference was determined not to be statistically significant with 99.5 percent confidence.

Transverse Crack Cores

The transverse cracks generally appeared to propagate around the aggregate and meander through the depth of the slab, which provided good surface texture. Only one core was pulled at a crack in the doweled section. The bottom 100 mm (4 in) and the top 25 mm (1 in) or more of the core were spalled, thereby reducing the effective depth of contact at the crack face to less than 127 mm (5 in). There was also a significant amount of fines present on the fractured face. This core was taken at a crack which occurred directly over a concrete culvert and is not considered to be representative of the section.

The average VSTR at the cracks in the undoweled section was higher than that of the crack in the doweled section (0.5833 cm^3/cm^2 vs. 0.3852 cm^3/cm^2). This is because the crack in the doweled section exhibited a greater amount of deterioration than those in the undoweled. All cracks in the undoweled section were of low severity, and the VSTR's were sufficiently high to maintain good aggregate interlock load transfer across sufficiently tight cracks (i.e., crack widths less than 0.76 mm [0.03 in]). Since all cracks occurred in either the 5.5- or 5.8-m (18- to 19-ft) panels, the potential crack width is as great as 3.76 mm (0.15 in), well above this value.

The load transfer efficiencies, VSTR's and VST's per lineal cm at the cracks are higher than those at the undoweled joints (see table 56). Load transfer increases with increasing VSTR's and VST's per lineal cm provided the crack widths and radii of relative stiffness are constant. Since the width of the crack in the doweled section is smaller than those in the undoweled section, the LTE is larger even though the VSTR and VST per lineal cm are smaller.

Table 56. Surface texture and load transfer efficiencies for WI 1 cracks and undoweled joints.

Test Method	WI 1-1 Crack	WI 1-2 Crack	Undoweled Joint
VSTR, cm^3/cm^2	0.5833	0.3852	0.3682
VST, cm^3/cm	16.15	10.76	7.86
LTE, %	48	59	32

Project Summary

Both WI 1 sections contain recycled concrete aggregate. The only difference in the design of the sections is WI 1-1 does not contain any load transfer devices at the transverse joints, while WI 1-2 contains 35-mm (1.38-in) dowel bars. Both sections are 280-mm (11-in) JPCP with a 150-mm (6-in) aggregate base and a 230-mm (9-in) granular subbase. The transverse joints are skewed and sawed at random intervals. A tied PCC shoulder is also provided. Both pavement sections have been exposed to approximately 7 million ESAL applications over their 11-year service life.

The field survey and testing results of these two field test sections provided the following findings and possible conclusions:

- As expected, the major difference between these two sections is in the performance of the joints and cracks. In the undoweled section, faulting decreased the serviceability to the point that diamond grinding had to be performed after 10 years of service. The doweled section had developed very little faulting at the transverse joints.
- The undoweled section has a lower joint load transfer efficiency than the doweled section. In spite of apparently high potential for aggregate interlock (as indicated by the excellent surface texture measurements), the undoweled joints are generally too wide to provide adequate load transfer without a mechanical load transfer device.
- The undoweled section exhibited greater faulting and more low-severity joint spalling than the doweled section. This is probably due to the poor load transfer capacity in the undoweled section, which facilitates pumping and faulting and may produce more surface and subsurface spalling in the presence of entrapped incompressibles.
- The one dowel present in a core pulled from a doweled joint was severely corroded at the joint face, presumably due to the entry of water and deicing salts through the unsealed joint.

- The use of mechanical load transfer devices and a joint sealing program would have probably improved the performance of this pavement.
- Very few cracks were observed and all were low-severity cracks found in the longest panels, suggesting that they may have formed relatively recently. These faces of these cracks exhibited excellent surface texture characteristics, but typically provided low load transfer efficiencies because the crack widths are too large for effective load transfer through aggregate interlock. These cracks should be monitored for deterioration similar to that observed in the undoweled joints (i.e., faulting and spalling). In addition, the section should be monitored for the development of additional cracks in the longest panels.
- The longest panels may have been too long to prevent midpanel cracking in the conditions present at the project site. Alternatively, steel reinforcement could have been provided to hold the midpanel cracks tight.
- The thermal coefficient for the doweled section is slightly higher than that for the undoweled section (12.5×10^{-6}/ °C vs. 11.3×10^{-6}/ °C [6.9×10^{-6}/ °F vs. 6.3×10^{-6}/ °F]). The difference between these two average values is statistically significant at the 90 percent level. All other properties of the concrete in the two sections were found to be comparable.
- Uranyl acetate testing indicate minor amounts of silica gel deposits in the mortar and around the aggregate particles in the undoweled section; moderate amounts of silica gel were indicated in the mortar and around some of the aggregate particles in the doweled section. These deposits may indicate the presence of alkali-silica reaction activity.

Wisconsin 2, I-90 near Beloit

As with the WI 1 sections, both WI 2 sections were constructed using recycled concrete aggregate. The entire project was constructed as a continuously-reinforced concrete pavement (CRCP) with a constant nominal structural section throughout the project length. However, some portions of the pavement had developed more deterioration (punchouts and deteriorated transverse cracks) than others. As a result, two sections were selected for evaluation, with one in each of the different performance areas.

<u>Project Information</u>

This construction project involved the reconstruction of 7.6 km (4.7 mi) of I-90 between mileposts 173.0 and 177.7 in Rock County, Wisconsin, between Beloit and Janesville. The existing pavement was a JRCP that was originally constructed in 1957. After 29 years of service, this pavement was removed, crushed, and used as aggregate for a new CRCP.

Design Information

The reconstructed RCA concrete pavement section was built in 1986. The pavement consisted of a 250-mm (10-in) CRCP over a 150-mm (6-in) aggregate base and a 230-mm (9-in) granular subbase. The design steel content was 0.67 percent of the cross-sectional area. Concrete shoulders (JPCP with a 4.6-m [15-ft]) joint spacing) were constructed 150 mm (6 in) thick over a 250-mm (10-in) aggregate base and were tied to the mainline pavement using 610-mm (24-in) long, 13-mm (No. 4), epoxy-coated tie bars spaced 1,200 mm (48 in) apart. The same tie bar design was used at the longitudinal centerline joint. No provisions were made for removal of water from within or beneath the pavement system.

Mix Design

Very little aggregate gradation and mix design information is available for these sections. However, the aggregate gradations are believed to conform to the specifications given in table 50. In addition, both sections contained RCA as the coarse aggregate and natural sand as the fine aggregate.

Construction Information

The original 29-year-old concrete pavement was removed and crushed. This crushed aggregate was then used as the coarse aggregate portion of the recycled concrete. Normal construction and paving techniques were used to place the recycled concrete. The pavement surface was tined transversely and liquid membrane curing compound was applied.

Climatic Conditions

These sections are located in the wet-freeze environmental region. The minimum and maximum average monthly temperatures are -6 and 23 °C (22 and 73 °F). The sections are exposed to about 90 freeze-thaw cycles per year, and the freezing index in the area is 430 °C-days (780 °F-days). The area also experiences about 118 days of precipitation each year for a total annual precipitation of 790 mm (31 in). The resulting Thornthwaite moisture index is 25.

Traffic Loadings

Both sections were constructed and opened to traffic in 1986 and have been subjected to same traffic loadings over the 8 years of service that preceded the field testing and evaluation. When opened to traffic in 1986, the two-way ADT was about 22,622 vehicles per day; traffic had increased to nearly 29,000 vehicles per day by 1994. About 20 percent of the traffic stream consists of heavy trucks, resulting in the application of approximately 7.9 million ESAL's to the design lanes of these pavement sections through 1994.

Selection of Distress Survey Section

As noted previously, some portions of the project exhibited significantly more deterioration (punchouts and deteriorated transverse cracks) than others. Pavement sections were selected to be representative of each of these levels of performance, and a complete survey and evaluation were performed over each section. Both sections were located in the northbound (westbound) lanes of I-90. The section with more distress, designated WI 2-1, began at milepost 176.8 (station 313+65) and extended northward for 305 m (1,000 ft). The section with generally good performance, designated WI 2-2, began at milepost 176.2 (station 281+99) and also extended northward for 305 m (1000 ft). Both sections employed crowned cross-sections and were constructed nearly at grade, although WI 2-2 was partially located in a slight cut section.

Drainage Survey

WI 2-1 rests on a level grade with no significant areas of cut or fill. The cross-section is crowned, with a 1 percent slope on the traffic lanes and a 3 percent slope on the shoulders. The depth of the ditch line from the pavement surface was approximately 1.8 m (6 ft). No drainage features (e.g., longitudinal edge drains or permeable base elements) were provided. Evidence of low-to-medium severity pumping of moisture and fines was observed along the lane-shoulder joint.

Approximately the first 150 m (500 ft) of WI 2-2 was also constructed on a level grade, while the remaining portion was constructed in a cut section (average cut of about 3 m [10 ft]). The longitudinal slopes varied from level to 1 percent (downward in the direction of traffic). A crowned cross-section was also provided, with a 1-percent cross-slope on the traffic lanes and a 3-percent cross-slope on the shoulders. As with WI 2-1, no drainage features were incorporated in this section, although no signs of pumping of moisture or fines were observed in WI 2-2.

Pavement Distress Survey

The pavement condition survey was conducted over the entire length of both sections. A complete summary of the results of the survey is provided in tables 84 and 85 in appendix A. A summary of the average results for key distress and performance measures is presented in table 57. The average transverse crack spacing was determined by dividing the section length (305 m [1,000 ft]) by the total number of cracks within the section. The average transverse crack width was calculated by averaging the crack widths within a representative 30-m (100-ft) subsection.

Table 57. Summary of performance data (average values) for WI 2.

Performance Measurement	WI 2-1	WI 2-2
Punchouts/km	6.8	0.0
Deteriorated Transverse Cracks/km	134.0	29.8
Cracks/km	1292.0	1427.0
Average Crack Spacing, mm	774	701
Average Crack Width, mm	1.1	0.7
Longitudinal Cracking, m/km	0	0
PSR	3.9	4.0

Transverse Crack Spacing and Width

WI 2-1 had an average crack spacing of 770 mm (2.54 ft), and an average crack width of 1.1 mm (0.044 in). WI 2-2 exhibited a narrower spacing between transverse cracks (700 mm [2.30 ft]), and a narrower average crack width (0.7 mm [0.028 in]). One study has shown that, after a CRCP crack opens more than 0.76 mm (0.03 in), aggregate interlock is virtually nonexistent.[21] Another study found similar results for joints in JPCP.[22] Although CRCP cracks and JPCP joints are conceptually different, both discontinuities rely on the same mechanism (aggregate interlock) to provide load transfer. Sixty-two percent of the cracks on WI 2-1 are open wider than 0.76 mm (0.03 in), compared to only 36 percent on WI 2-2. Similarly, 26 and 3 percent of the cracks on WI 2-1 and 2-2 are open wider than 1.3 mm (0.05 in). Once aggregate interlock diminishes, load transfer is accomplished solely through the reinforcing steel, resulting in larger slab deflections and the development of pumping and localized failures (punchouts).

Deteriorated Transverse Cracks

During the distress survey, a crack was labeled deteriorated if it was "working" (as evidenced by faulting) or had spalled over more than 10 percent of the crack length. Section WI 2-1 had 134 deteriorated transverse cracks per km (216/mi), compared to only 30 deteriorated transverse cracks per km (48/mi) on WI 2-2. The greater degree of crack deterioration on WI 2-1 may be directly related to the wider average crack openings on that section, which allow greater horizontal and vertical movement at the crack and accelerate the deterioration process.

Punchouts

Punchouts can be considered a result of extreme crack deterioration in CRCP. They are, therefore, also related to crack width and movements. As expected, WI 2-1, with wider and more deteriorated cracks, also has more punchouts. WI 2-1 exhibited 6.8 punchouts per km (11/mi), whereas no punchouts were observed on WI 2-2. The wide cracks lead to punchouts as the reinforcing steel ruptures due to corrosion and/or heavy traffic loads. A longitudinal crack then forms between the two transverse cracks between the two wheel paths, forming a punchout.

Present Serviceability Rating (PSR)

Although WI 2-1 shows more deterioration, the difference in estimated PSR is negligible. WI 2-1 and 2-2 have average PSR values of 3.9 and 4.0, respectively. However, as WI 2-1 continues to deteriorate, the PSR is expected to drop at an increasing rate. On the other hand, WI 2-2 shows only minimal deterioration and should continue to provide good service.

<u>FWD Testing</u>

Pavement deflection testing was performed using a Dynatest model 8081 FWD. The testing pattern included 5 slab centers, 10 transverse cracks (with load placement on both the approach and leave sides of the cracks) and 10 slab edges near mid-panel at the lane-shoulder joint. The results of these tests were used to determine material properties (PCC elastic modulus and subgrade k-value), load transfer efficiencies across cracks, and loss of support. A summary of the average values obtained from the deflection data is provided in table 58.

Table 58. Deflection testing results for WI 2.

Property	WI 2-1	WI 2-2
Elastic Modulus, GPa	40.3	40.9
k-value, kPa/mm	95.0	104.0
Crack Load Transfer, %	93	93
Shoulder Load Transfer, %	56	59
Average Midslab Deflection, µm	70	66
Average Edge Deflection, µm	136	125
Corners With Voids, %	0	0
Maximum Air Temperature During Testing, °C	7	9

PCC Elastic Modulus

The elastic modulus (E) of the concrete slab was backcalculated using center-of-slab deflection measurements. Considerable effort was taken to ensure the testing was conducted at least 1 m (3 ft) away from any transverse cracks. Testing too close to transverse cracks could invalidate any backcalculation results. Figure 75 presents a plot of the concrete elastic modulus at five different locations along WI 2-1 (4 load tests per location). Backcalculated elastic modulus values range from 36 to 46 GPa (5,200,000 to 6,700,000 lbf/in^2) with an average of 40.3 GPa (5,840,000 lbf/in^2). These values were generally slightly higher than the results of dynamic tests of elastic modulus performed on cores, which averaged 37.2 GPa (5,390,000 lbf/in^2), as discussed below. The results of multiple tests at each location are consistent, and there was little variation in results obtained at the different locations.

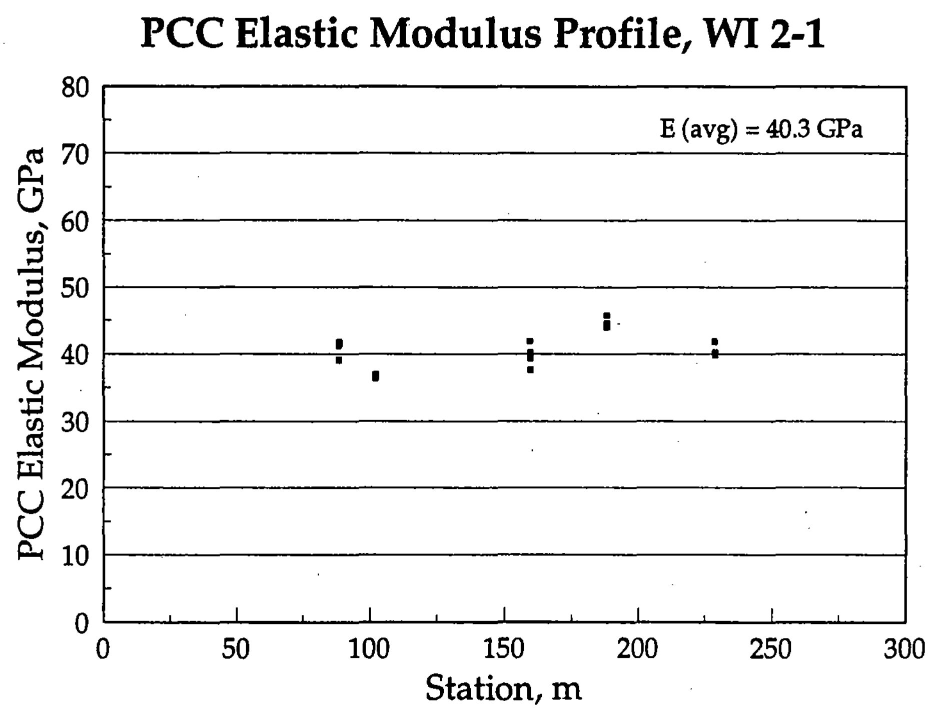


Figure 75. PCC elastic modulus profile for WI 2-1.

A similar plot of backcalculated concrete modulus values for WI 2-2 is presented in figure 76. These values range from 30 to 49 GPa (4,350,000 to 7,110,000 lbf/in^2) and average 40.9 GPa (5,930,000 lbf/in^2). These values were also generally higher than the results of laboratory-based dynamic tests of cores from the section, which averaged 39.0 GPa (5,660,000 lbf/in^2). The FWD test results were fairly consistent at all locations except for one (station 187), which may have been located too closely to a transverse crack, which would have resulted in a lower backcalculated elastic modulus value. As expected, the backcalculated elastic modulus values for the two sections are comparable, as both are constructed using the same materials and structural design.

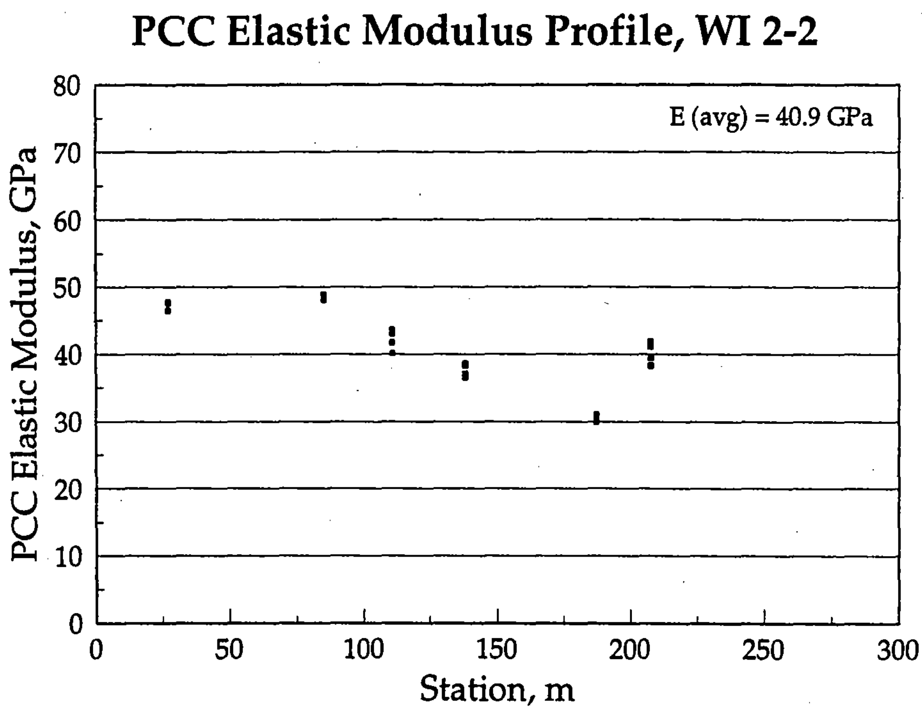


Figure 76. PCC elastic modulus profile for WI 2-2.

Modulus of Subgrade Reaction (k-value)

Profile plots of the backcalculated k-values for WI 2-1 and 2-2 are illustrated in figures 77 and 78, respectively. As with the elastic modulus values, similar results were obtained for both sections. The average k-value for WI 2-1 was 95.0 kPa/mm (350 lbf/in^2/in), with values ranging from 75 to 111 kPa/mm (276 to 409 lbf/in^2/in). On section WI 2-2, the k-values ranged from 77 to 130 kPa/mm (284 to 478 lbf/in^2/in), and the average of all tests was 104.0 kPa/mm (383 lbf/in^2/in). The two sections are located about 0.8 km (0.5 mi) apart and were constructed on the same subgrade, so similar k-values were expected.

Crack Load Transfer

For CRCP, aggregate interlock at transverse cracks must be maintained, or the reinforcing steel will rupture and punchouts will develop. Thus, the load transfer efficiency at transverse cracks is a critical parameter for monitoring the performance of CRCP. Deflection testing was conducted at the approach and leave side of 10 transverse cracks in each section.

For the WI 2 sections, good load transfer across transverse cracks was measured. Figures 79 and 80 show profile plots of the load transfer across transverse cracks on

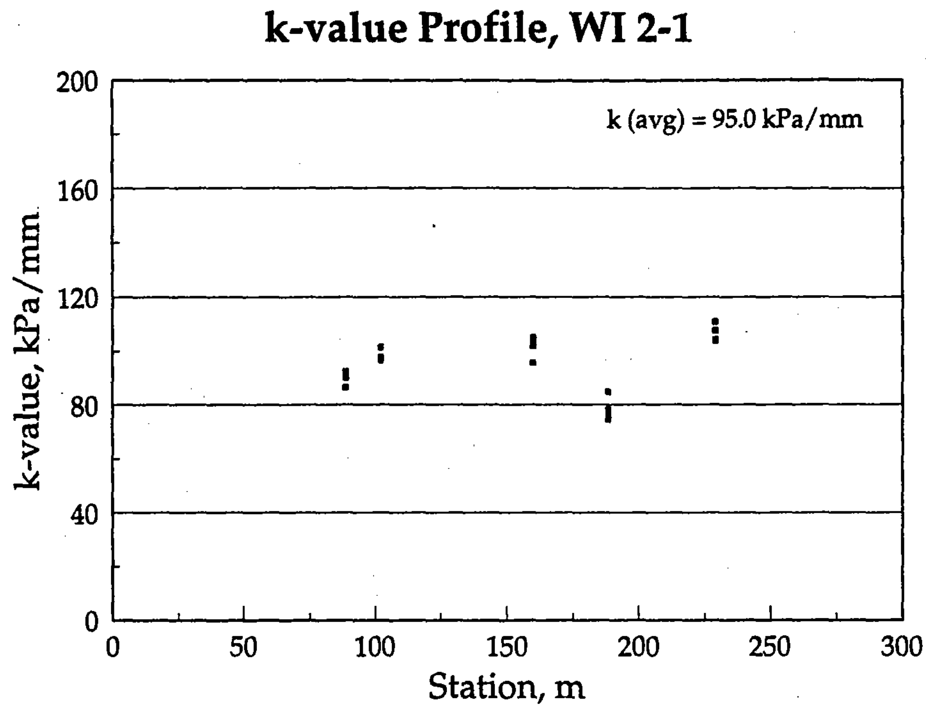


Figure 77. K-value profile for WI 2-1.

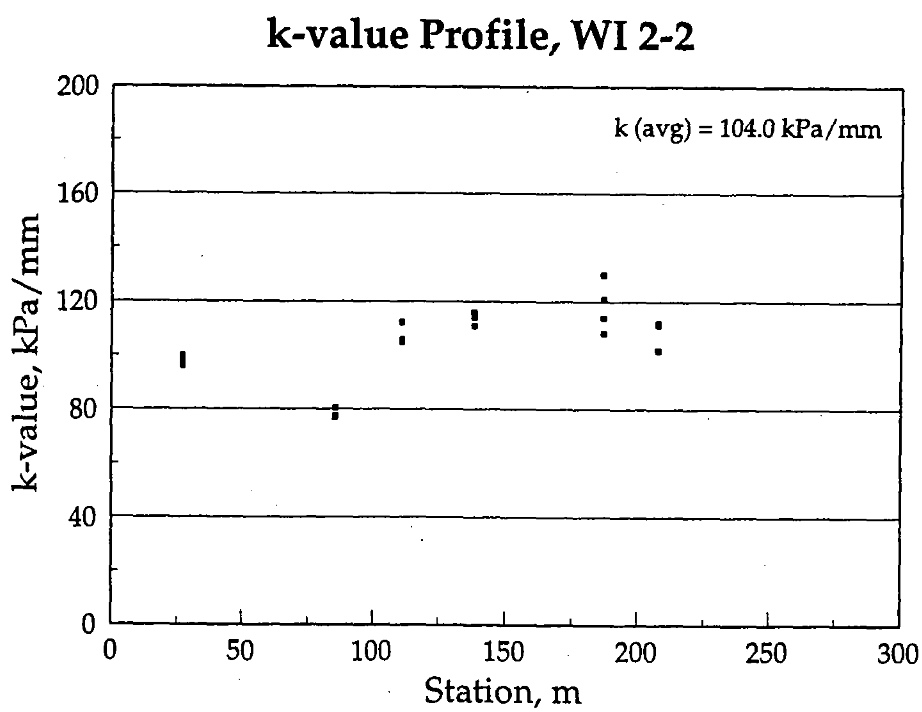


Figure 78. K-value profile for WI 2-2.

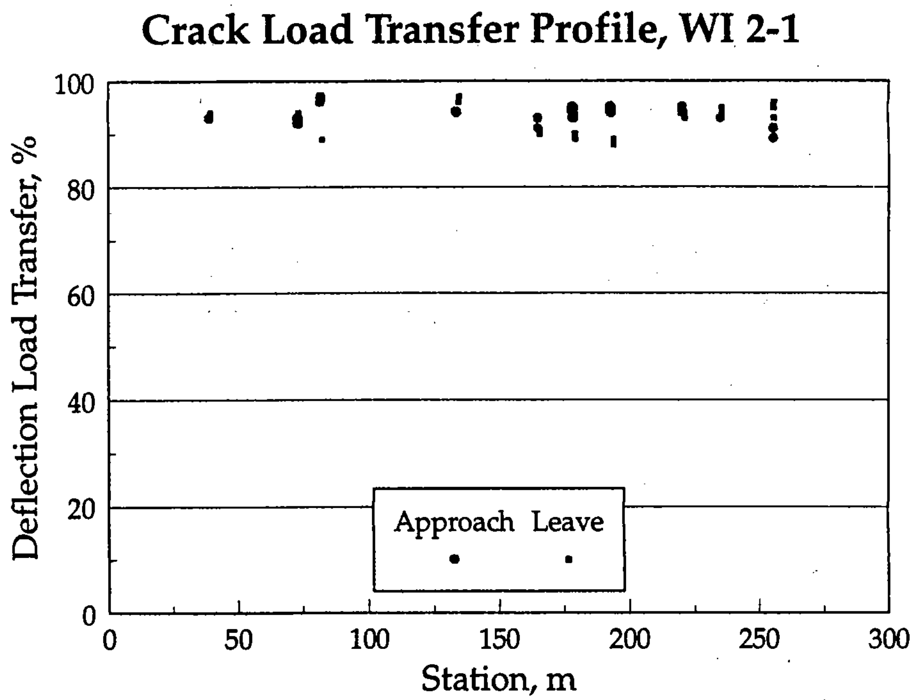


Figure 79. Crack load transfer profile for WI 2-1.

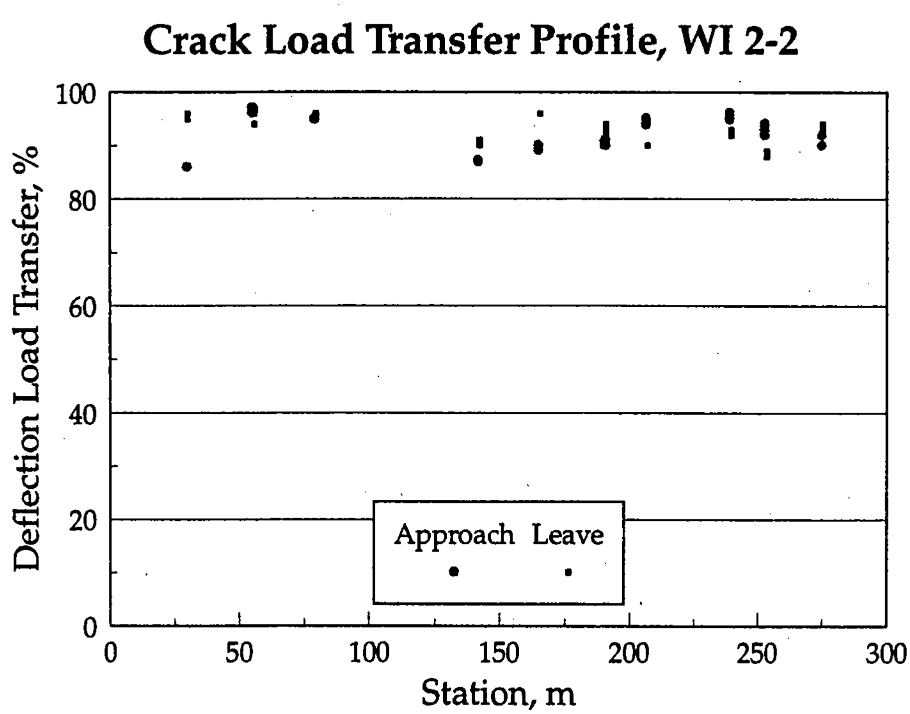


Figure 80. Crack load transfer profile for WI 2-2.

WI 2-1 and 2-2, respectively. The average load transfer on each section is 93 percent. These values indicate the strong interlock of the aggregate particles at the crack face and the effectiveness of the reinforcing steel at keeping the cracks tight. The wider cracks on WI 2-2 did not show reduced load transfer efficiencies.

Shoulder Load Transfer

The load transfer efficiencies across the tied PCC shoulder are shown in figures 81 and 82 for WI 2-1 and WI 2-2, respectively. The tied PCC shoulders on both sections were pavement separately from the mainline pavement. The average load transfer efficiency for WI 2-1 is 56 percent, with values ranging from 34 to 100 percent. For WI 2-2, the values range from 35 to 97 percent with an average value of 59 percent.

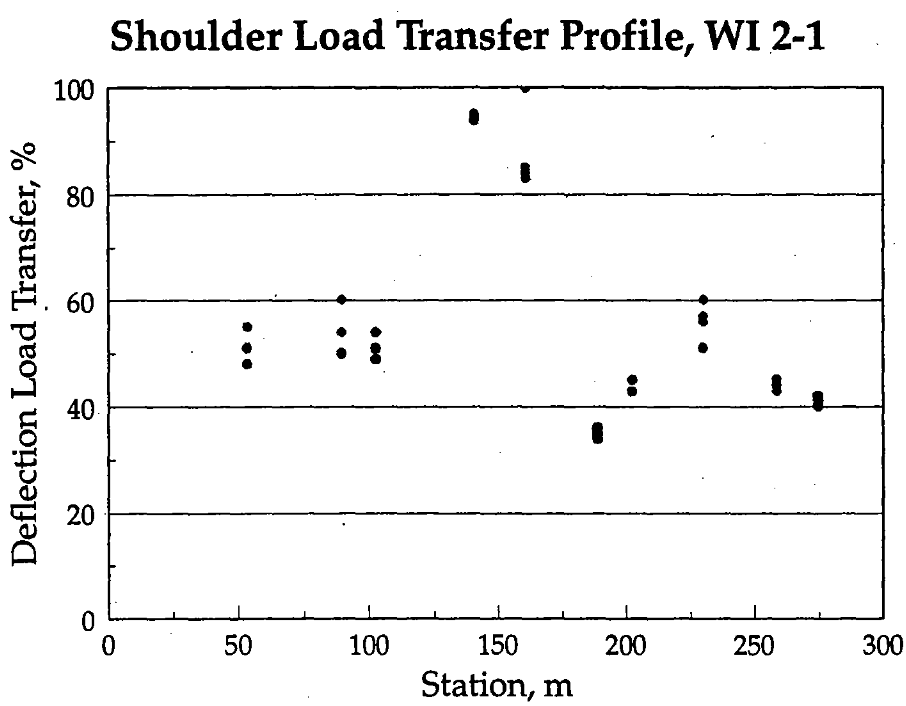


Figure 81. Shoulder load transfer profile for WI 2-1.

The deflection load transfer efficiencies measured across the tied PCC shoulder (56 and 59 percent) represent fairly low values. These low deflection load transfer efficiencies correspond to stress load transfer efficiencies of about 12 to 15 percent, assuming a slab thickness of 250 mm (10 in), a PCC elastic modulus of 38.1 GPa (5,500,000 lbf/in^2) and a k-value of 99.5 kPa/mm (366 lbf/in^2/in). The low deflection load transfer may be due to an inadequate tie bar design (small 13-mm [0.5-in] tie bars spaced at 1,200-mm [48-in] intervals).

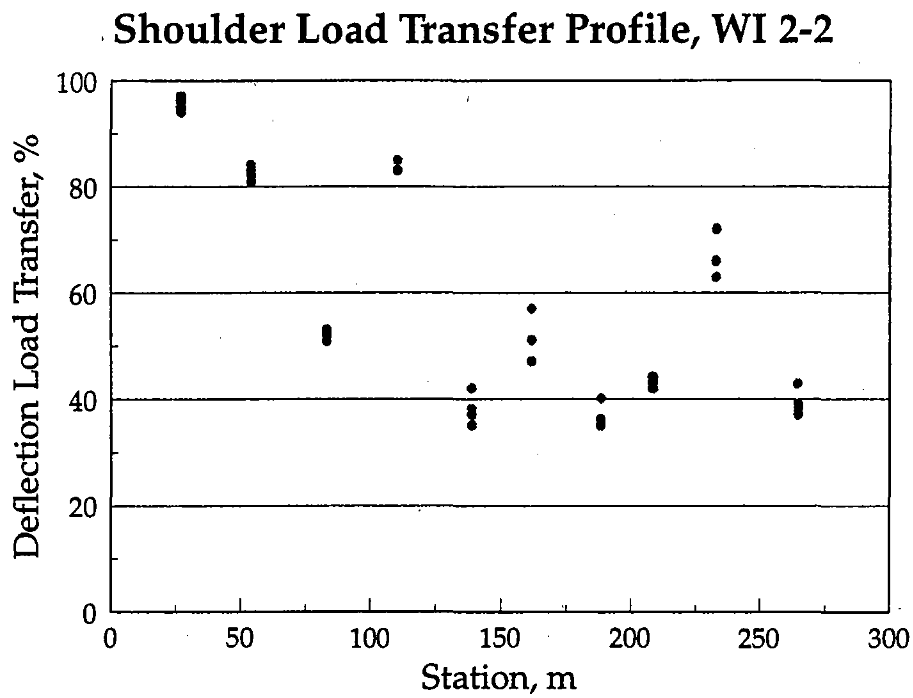


Figure 82. Shoulder load transfer profile for WI 2-2.

Other studies have also found poor load transfer across tied shoulders, bringing into question the effectiveness of tied shoulders in improving the performance of the mainline pavement. For example, a recent FHWA study on CRCP performance found that tied concrete shoulders had no apparent contribution to reducing edge deflections.[23]

Loss of Support

The detection of voids was performed using the corner deflections on the leave side of transverse cracks and procedures described in the final report for NCHRP 1-21. Figures 83 and 84 illustrate the potential for loss of support along the recycled and control sections, respectively. Neither section shows strong potential for loss of support at the transverse cracks, although there appears to be marginal potential for loss of support in the southern half of WI 2-1, which is consistent with the evidence of low-to-medium severity pumping of moisture and fines that was observed along the lane-shoulder joint in this section.

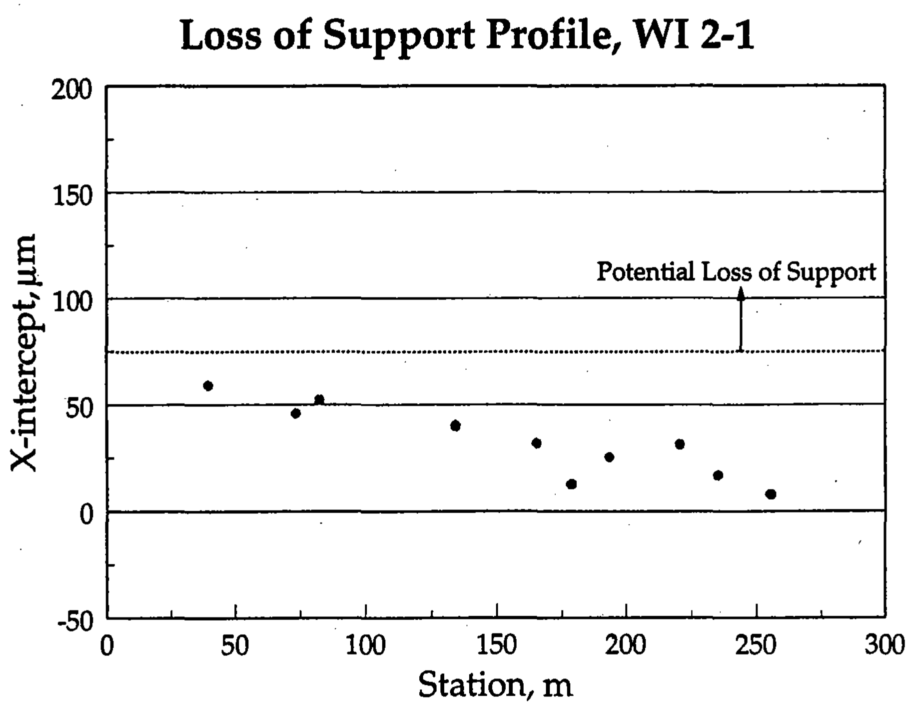


Figure 83. Loss of support profile for WI 2-1.

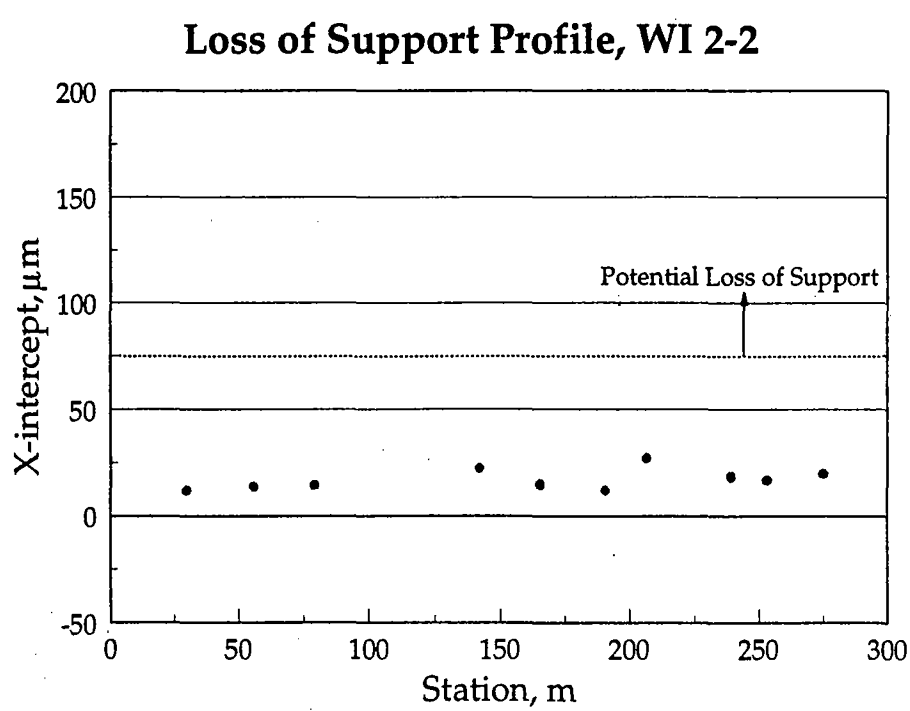


Figure 84. Loss of support profile for WI 2-2.

Coring

Seven cores were retrieved from WI 2-1, including four from midpanel locations and three from transverse cracks. On WI 2-2, five cores were taken from midpanel locations and four were taken from transverse cracks. The midpanel cores were taken from the center of the lane, approximately 2 m (6 ft) from the lane-shoulder joint. The cores taken from transverse cracks were retrieved from the outer wheel path, approximately 0.6 m (2 ft) from the lane-shoulder joint. All cores were 150-mm (6-in) in diameter and extended through the thickness of the concrete slab. No cores were taken through the aggregate base course. These cores were tested in the laboratory to determine the physical and mechanical properties of the concrete mixture used on this project and to determine whether there were any significant material differences between the two evaluation sections, as described in more detail below.

Core Testing

The number of cores for each laboratory test is indicated in table 59. A summary of the average values that were obtained during the laboratory testing of the field cores is presented below in table 60 and in table 83 in appendix A. Observations made during the testing, and comparisons between the performance of the control and recycled sections, are also provided below.

Table 59. Number of cores for each laboratory test in WI 2.

Laboratory Tests	WI 2-1	WI 2-2
Thermal Coefficient	3	3
Split Tensile Strength	3	2
Dynamic Modulus of Elasticity	3	3
Static Modulus of Elasticity	0	0
Compressive Strength	1	2
Volumetric Surface Texture	1	1

Table 60. Core testing results for WI 2.

Property	WI 2-1	WI 2-2
Compressive Strength, MPa	55.5	44.3
Split Tensile Strength, MPa	3.5	4.1
Dynamic Elastic Modulus, GPa	37.2	39.0
Static Elastic Modulus, GPa	n/a	n/a
Thermal Coefficient, $(1x10^{-6})/°C$	10.6	13.5
VSTR (for Failed Split Tensile Core), cm^3/cm^2	0.3359	0.3107
VSTR (for Slab Faces at the Joints), cm^3/cm^2	n/a	n/a
VSTR (for Slab Faces at the Cracks), cm^3/cm^2	0.2385	0.3726

Petrographic Examination Summary

The coarse aggregate for both recycled sections is comprised of rounded-to-angular particles that were classified as gravel rock deposits and were observed to be evenly distributed throughout the cement paste. The gravel rock is further characterized as original coarse aggregate containing sedimentary, igneous, and metamorphic particles. A Class C fly ash as per ASTM C 618 may also have been introduced into both recycled concrete mixtures, although this cannot be verified with construction records, which were not provided by the Wisconsin DOT.

The relative proportions of new mortar, recycled mortar and natural aggregate particles in specimens from the two sections were estimated using linear traverse techniques, as shown in table 61. Total mortar contents for the two sections were found to be comparable (79 percent vs. 86 percent), although WI 2-1 appeared to contain more recycled mortar and less new mortar than WI 2-2. WI 2-1 was also found to contain about 50 percent more natural aggregate particle content than WI 2-2. These values are derived from measurements made of a single slice of one core from each section, so it is impossible to say whether the differences observed are significant or due to variability in particle and mortar distributions of the concrete. The project team suspects that the differences are not significant and that pavement performance differences are attributable to other issues, such as drainage and localized variations in foundation support.

Table 61. Coarse aggregate and mortar contents for WI 2.

	WI 2-1	WI 2-2
Coarse Aggregate, %	21.0	13.7
New Mortar, %	69.3	79.6
Recycled Mortar, %	9.7	6.7

Uranyl acetate testing of cores obtained from both sections indicates the presence of significant amounts of silica gel in the mortar and around some of the aggregate particles. While this may indicate the presence of alkali-silica reactivity, no such distress was identified during the field surveys.

Mid-Panel Cores

Only one measurement of the compressive strength of the WI 2-1 concrete was possible because other midpanel cores contained reinforcing steel that could not be trimmed from the specimen in a manner that would allow compression testing in accordance with ASTM procedures. The compressive strength of the lone WI 2-1 specimen was 55.5 MPa (8,050 lbf/in^2), which was the highest RCA concrete strength found in this study. Compressive strengths for the WI 2-2 section cores ranged between 40.4 and 48.2 MPa (5,860 and 6,990 lbf/in^2), averaging 44.3 MPa (6,420 lbf/in^2). Diametral or split cylinder tensile strengths ranged from 3.1 to 4.0 MPa (450 to 580 lbf/in^2) for WI 2-1, with an average of 3.5 MPa (508 lbf/in^2). The split tensile strengths ranged from 4.0 to 4.2 MPa (580 to 610 lbf/in^2) for WI 2-2, with an average of 4.1 MPa (590 lbf/in^2).

The dynamic elastic modulus of the WI 2-1 cores ranged from 35.4 to 39.6 GPa (5,130,000 to 5,740,000 lbf/in^2), with an average of 37.2 GPa (5,390,000 lbf/in^2). The dynamic elastic modulus ranged from 36.1 to 40.7 GPa (5,230,000 to 5,900,000 lbf/in^2) for WI 2-2, with an average of 39.0 GPa (5,660,000 lbf/in^2). These values compare favorably with those obtained through backcalculation of FWD data, as described previously. Static modulus tests were not performed for this project because suitable test specimens could not be produced from the highly reinforced cores.

The thermal coefficient of expansion ranged from 10.2 x 10^{-6}/ °C to 10.8 x 10^{-6}/ °C (5.7 x 10^{-6}/ °F to 6.0 x 10^{-6}/ °F) for WI 2-1, with an average of 10.6 x 10^{-6}/ °C (5.9 x 10^{-6}/ °F). The WI 2-2 section thermal coefficients ranged from 12.6 x 10^{-6}/ °C to 14.0 x 10^{-6}/ °C (7.0 x 10^{-6}/ °F to 7.8 x 10^{-6}/ °F), with an average of 13.5 x 10^{-6}/ °C (7.5 x 10^{-6}/ °F), which was the highest thermal coefficient measured among all of the field sites studied. The higher thermal coefficient of the WI 2-2 section corresponds with an apparent increased total mortar content in that same section. It is possible that this apparent difference in thermal coefficients is responsible for the performance differences noted between the two sections. Additional research would be necessary to further investigate these possible links.

The laboratory results showed that WI 2-1 consistently had lower values for all testing except for compression, when compared to WI 2-1. Despite the lower values of WI 2-1, it was still believed that the laboratory tests produced typical results for a recycled concrete. Laboratory results showed WI 2-2 exceeding typical expectations on the testing mentioned above.

In general, the laboratory tests of concrete strength and elasticity provided no clear trends of material differences and both sections appear to have acceptable strength and elasticity properties for concrete paving applications. Thermal coefficient testing does suggest a large difference in the volumetric stability of the two materials and it is possible that this difference is due to the relative composition of the two materials. However, further testing should be performed to verify this link and to determine whether the different thermal properties are responsible for any of the observed differences in field performance that were observed.

Crack Cores

Many of the cracks that appeared to be of medium severity at the pavement surface were very tight below the surface and often did not propagate the entire depth of the core. Five of the seven crack cores pulled from these two sections contained cracks which did not propagate through the entire depth of the core. The two cores that were cracked completed through the core also exhibited cracks which had propagated into the slab along the reinforcing bars on both sides of the crack.

The two cores upon which VST testing was performed appeared to have different surface textures above and below the reinforcing bars, with the areas above the longitudinal steel having higher VSTR values. This phenomenon could be explained by the hypothesis that the top portion of the crack formed due to shrinkage at a relatively early age, propagating around the weakly-bonded aggregate particles. As the strength of the mortar and bond increased with time, any further propagation of the crack took place through many of the particles, resulting in reduced surface texture.

The VSTR for WI 2-2 appears higher than that for section 1 (0.3726 vs. 0.2385 cm^3/cm^2). Each of these values represents the results of only one test because it was difficult to retrieve specimens of sufficient size for testing without including a deformed bar that made it impossible to separate the core halves. With the large variability typically associated with concrete fracture paths, it is difficult to determine the significance of this difference based on only one measurement.

It was noted that none of the longitudinal steel included in the cores exhibited signs of corrosion.

Performance of Comparable CRCP in Wisconsin

One reason that WI 2 was included in this research study is that it was believed that performance data from a comparable "control" section (i.e., a similarly-designed Wisconsin CRCP constructed using traditional concrete aggregate instead of RCA) could be obtained from a study of CRCP being conducted concurrently for FHWA. The second volume of the project report is entitled "Field Investigation of CRC Pavements," and describes the results of field and laboratory tests similar to those performed by the project team on this study.[24]

Five of the 25 projects evaluated were located in Wisconsin and 3 are comparable to the WI 2 project described in this report.[24] Unfortunately, all three are also constructed using RCA concrete (a point mentioned nowhere in the report), which makes them useful only to validate the findings of the WI 2 project described herein; none can be considered a "control" section for this study. A brief summary of the findings for these projects is provided below.

Section WI-2

This section is probably the closest in all respects to the WI 2-1 and WI 2-2 study sections. It is located about 3.2 km (2 mi) further north on the same highway (between mileposts 180 and 181) and was constructed in 1985, so environmental and traffic conditions are practically identical. All aspects of the structural and geometric design are identical between the two projects.

Performance can be summarized as follows: 1991 PSI = 4.8; average crack spacing = 0.88 m (2.90 ft); 90 percent of transverse cracks were rated as "medium severity;" with remaining 10 percent "low severity;" no other distress present; average crack width = 0.58 mm (0.02 in) in the morning, 0.27 mm (0.01 in) in the afternoon; and load transfer efficiency at cracks and lane-shoulder joint average 93 percent.

Material properties can be summarized as follows: PCC modulus of elasticity (lab testing) = 31.0 GPa (4,500,000 lbf/in^2); average split tensile strength = 3.4 MPa (490 lbf/in^2); and coefficient of thermal expansion = 10.2/°C (5.67/°F).

The performance of this project is considered comparable to that of the WI 2 project currently under study. The only notable differences are the PSI (which is much higher for the previously-studied project), the large number of cracks rated "medium-severity" on the previously-studied project (presumably due to the use of different classification criteria, given the overall high performance rating given this project), and the much lower coefficient of thermal expansion for the previously-studied project (presumably a result of differences in measuring equipment).

Section WI-3

This section is also quite similar to the WI 2-1 and WI 2-2 study sections. It is located about 80 km (50 mi) further north on the same highway (between mileposts 136 and 135) and was constructed in 1984. The total traffic on this section is higher (1991 ADT estimated at 35,100 vehicles per day, and nearly 4.0 million ESAL's estimated through 1991), although environmental conditions are comparable. All aspects of the structural design are identical between the two projects, although the previously-studied section WI-3 is a six-lane pavement.

Performance can be summarized as follows: 1991 PSI = 3.9; average crack spacing = 1.06 m (3.48 ft); 56 percent of transverse cracks were rated as "medium severity;" with remaining 44 percent "low severity;" average crack width = 0.54 mm (0.021 in) in the morning, 0.41 mm (0.016 in) in the afternoon; and load transfer efficiency at cracks average 92 percent. It was also noted during a windshield survey of the entire 8-km (5-mi) project that there were 4 patches and about 56 m (190 ft) of longitudinal cracking.

Material properties can be summarized as follows: PCC modulus of elasticity (lab testing) = 26.9 GPa (3,900,000 lbf/in^2); average split tensile strength = 3.2 MPa (470 lbf/in^2); and coefficient of thermal expansion = 9.45/°C (5.25/°F).

The performance of this project is considerably poorer than that of the WI 2 project currently under study. However, it is carrying much higher volumes of heavy traffic and has been in service longer. Other notable differences are large number of cracks rated "medium-severity" on the previously-studied project (presumably due to the use of different classification criteria, as noted previously), the much lower coefficient of thermal expansion for the previously-studied project (presumably a result of differences in measuring equipment, as noted previously), and the presence of some longitudinal cracking and patching. It does not appear that this project can be appropriately considered comparable to the WI 2 project being considered under this study.

Section WI-4

This section is also structurally similar to the WI 2-1 and WI 2-2 study sections. It is located about 113 km (70 mi) further north on the same highway (near milepost 111) and was constructed in 1984. The total traffic on this section is much higher (1991 ADT estimated at 42,550 vehicles per day, and more than 4.0 million ESAL's estimated through 1991), although environmental conditions are comparable. All aspects of the structural design are identical between the two projects, although the previously-studied section WI-3 is a six-lane pavement with the reinforcing steel protected cathodically instead of through epoxy-coating.

Performance can be summarized as follows: 1991 PSI = 3.4; average crack spacing = 1.40 m (4.59 ft); 85 percent of transverse cracks were rated as "medium severity;" with remaining 15 percent "low severity;" average crack width = 0.63 mm (0.025 in) in the morning, 0.45 mm (0.018 in) in the afternoon; and load transfer efficiency at cracks average 91 percent. It was also noted during a windshield survey of the entire 8-km (5-mi) project that there were seven patches.

Material properties can be summarized as follows: PCC modulus of elasticity (lab testing) = 35.2 GPa (5,100,000 lbf/in^2); average split tensile strength = 4.3 MPa (620 lbf/in^2); and coefficient of thermal expansion = 9.05/°C (5.03/°F).

The performance of this project is considerably poorer than that of the WI 2 project currently under study. However, it is carrying much higher volumes of heavy traffic and has been in service longer. Other notable differences are large number of cracks rated "medium-severity" on the previously-studied project (presumably due to the use of different classification criteria, as noted previously), the much lower coefficient of thermal expansion for the previously-studied project (presumably a result of differences in measuring equipment, as noted previously), and the presence of some patching. It does not appear that this project can be appropriately considered comparable to the WI 2 project being considered under this study.

Project Summary

This project provides an example of the performance potential for RCA concrete in a heavily-traveled CRC pavement. The project was constructed in 1986 using recycled concrete as the coarse aggregate and natural sand as the fine aggregate in a new, 250-mm (10-in) CRCP with tied concrete shoulders, placed over a 150-mm (6-in) base and a 230-mm (9-in) granular subbase. However, different levels of performance were observed between two sections with no discernible differences in materials, structural design, construction, traffic or environmental exposure. A summary of the results of the project evaluation is provided below.

Pavement Design

While the structural design of the pavement generally appears to have been adequate thus far, there are some signs of moderate pumping in the more deteriorated of the two pavement sections (WI 2-1), particularly at the south end of the section where deflections are somewhat higher than elsewhere. The inclusion of a pavement drainage system might have eliminated what may become a more urgent pumping problem in the future. This would probably be considered especially beneficial for this CRC pavement, since the performance of this type of pavement is particularly sensitive to losses of foundation support.

Material Properties

The average backcalculated k-values for the two sections are within 10 percent of each other, although it is possible that there is some loss of support at the south end of the WI 2-1 section that exhibited more deterioration.

The strength and elasticity properties of the concrete included in both evaluation sections appear comparable, although the thermal coefficient of the more deteriorated section was much lower than for the other section (10.6×10^{-6}/ °C vs. 13.5×10^{-6}/ °C [5.9×10^{-6}/ °F vs. 7.5×10^{-6}/ °F]). The difference between these two average values was determined to be statistically significant at the 99.5 percent level. The higher thermal coefficient of the WI 2-2 section corresponds with an apparent increased total mortar content in that same section. It is possible that this apparent difference in thermal coefficients is at least partially responsible for the performance differences noted between the two sections. Additional research would be necessary to further investigate these possibilities.

Uranyl acetate tests of concrete specimens obtained from both sections found evidence of considerable amounts of silica gel deposits in the mortar and around some of the aggregate particles, indicating the possible presence of ASR.

Pavement Performance

WI 2-1 showed a larger spacing between transverse cracks and the cracks were open wider. This section also exhibited four times as many deteriorated transverse cracks than WI 2-2, and 6.8 punchouts per km (11/mi), compared to no punchouts in WI 2-2. Most of this additional distress was concentrated near the south end of the WI 2-1 section, which is where evidence of pumping was observed and where deflection testing failed to provide evidence of strong, uniform support.

The load transfer efficiencies at the transverse cracks in each section were similar, averaging 93 percent on both sections.

The estimated PSR values for the two sections were comparable, indicating that the increased distress in WI 2-1 hasn't resulted in a significant loss of ride quality yet.

Wyoming 1, I-80 near Pine Bluffs

During the early 1980's, the pavement section of I-80 west of Pine Bluffs was suffering from extensive map cracking due to an alkali-aggregate reaction, with resulting potholes, spalling, and joint failures. Several rehabilitation alternatives were considered. Restoration and overlay options were considered infeasible due to the extent of the deterioration. Reconstruction (with either AC or PCC) was also considered, but rejected for the following reasons:

- High cost of producing and hauling new construction materials to the site.
- Cost of disposing of the existing concrete upon removal.
- No available source of quality aggregate near the site.

PCC recycling (existing PCC into a new concrete pavement surface) was selected as the most feasible and economical rehabilitation alternative for this section.

Project Information

The project selected for recycling was about 11 km (7 mi) long, and was located between mileposts 393.4 and 400.5 on I-80 between Cheyenne and Pine Bluffs. The original pavement was constructed essentially parallel to U.S. 30 in 1965. This pavement was a 200-mm (8-in) thick PCC pavement over a 150-mm (6-in) crushed stone base and a silty-loam subgrade. The highway consisted of two 3.7-m (12-ft) wide lanes in each direction with a 3.2-m (10.5-ft) wide outside shoulder and a 0.7-m (2.5-ft) wide inside shoulder. The shoulders consisted of 64-mm (2.5-in) AC over a 290-mm (11.5-in) crushed stone base.

To improve ride quality, isolated areas of the highway were overlaid with AC in the traffic lanes, and the potholes were patched with asphalt material. These restoration measures soon failed due to reflective cracking and delamination.[25] Roughness testing with a Mays Ride Meter indicated roughness levels of 164 and 180 mm/km (10.3 and 11.3 in/mi) in the eastbound and westbound lanes, respectively. A K.J. Law Lock-Wheel Friction Tester measured average friction numbers in the eastbound and westbound lanes of 30 and 33, somewhat below the commonly accepted critical friction number of 35. These measurements indicated an immediate need to rehabilitate the pavement.

Design Information

In 1985, the original PCC pavement for the recycled section was removed, along with 50 mm (2 in) of the underlying crushed stone base and the AC shoulders. The recycled section consisted of a 250-mm (10-in) JPCP on the remaining 100-mm (4-in) crushed stone base. The transverse joints were skewed and placed at "random" intervals of 4.3-4.9-4.0-3.7 m (14-16-13-12 ft). No load transfer devices were installed at the transverse joints. The recycled pavement was constructed 11.6 m (38 ft) wide (full width paving) with two 3.7-m (12-ft) wide lanes, a 3.0-m (10-ft) wide outside shoulder, and a 1.2-m (4-ft) wide inside shoulder. The centerline and shoulder joints are equipped with 610-mm (24-in) long, 13-mm (No. 4) tie bars spaced 610 mm (24 in) apart. The transverse and longitudinal joints were sealed with a silicone sealant.

Mix Design

In order to ensure the feasibility of the recycled concrete mix, several tests were conducted. The first series of tests (ASTM C 227, *Standard Test Method for Potential Alkali Reactivity of Cement-Aggregate Combinations [Mortar-Bar Method]*) were performed by the Portland Cement Association. These tests indicated that a blend of the recycled concrete aggregate and virgin limestone aggregate would have limited potential for alkali-aggregate reaction, and that the addition of sufficient quantities of a pozzolan would control any remaining recycled concrete aggregate reactivity.[26] A series of in-house tests was then conducted in accordance with ASTM C 289-81, *Standard Test Method for Potential Reactivity of Aggregates (Chemical Method)*, to identify an acceptable source of limestone for blending with the recycled concrete aggregate. An acceptable limestone source was located at the State Granite Quarry about 48 km (30 mi) west of Cheyenne. Tests were also performed in accordance with ASTM C 441-81, *Effectiveness of Mineral Admixtures in Preventing Excessive Expansion of Concrete due to Alkali-Aggregate Reaction*, to explore possible pozzolan sources. A Class F fly ash was found to provide optimum results for reduction of mortar expansion and average mixture expansion.

The presence of the reactive aggregate in the existing pavement provided considerable concern.. However, the results of extensive testing indicated that the reaction could be controlled by using the following techniques:[26]

- Use a low alkali (less than 0.60 percent Na2O) Type II cement.
- Blend the recycled concrete aggregate with a quality virgin material to reduce the amount of constituents.
- Use a fly ash meeting the requirements of ASTM C 618 (Table 2A) for reduction of expansion.

It was also expected that the use of the virgin coarse aggregate would increase flexural strengths and promote aggregate interlock at the joints and that the use of fly ash would improve the workability and durability properties of the recycled mix.[26]

The final recycled concrete mix design included a 60/40 split of coarse and fine aggregate, a 65/35 split of recycled and virgin coarse aggregate, and a 22/78 split of recycled and virgin fine aggregate. The virgin coarse aggregate was graded according to AASHTO Grading Band No. 57, while the recycled concrete aggregate was crushed to 25-mm (1-in) maximum size and was split on the 4.75-mm (No. 4) sieve for use in either the coarse or fine aggregate blends. The resulting gradations for the coarse and fine aggregate *portions* of both the recycled and control sections are provided in table 62. This table indicates that the coarse aggregates are graded almost identically, but that the recycled section contains a less coarse grading of fine aggregate (fineness modulus for recycled = 2.88, fineness modulus for control = 3.21). The smaller particles in the recycled fine aggregate section may have contributed to the fairly rapid (but localized) reoccurrence of ASR.

A summary of some of the physical properties of the aggregates used in both sections of this project are shown in table 63. The specific gravity of the coarse recycled concrete aggregate was about 8 percent lower for the coarse virgin aggregate, and the specific gravity of the fine recycled concrete aggregate was about 10 percent lower than for the fine virgin aggregate. The absorption capacities of the recycled concrete aggregate products were considerably higher than that of their virgin aggregate counterparts, and the Los Angeles abrasion test resulted in a 34 percent greater mass loss for the recycled concrete aggregate than for the natural aggregate.

Table 62. Aggregate gradations (percent passing each sieve) of recycled and control sections.

Sieve	Recycled		Control	
	Coarse	Fine	Coarse	Fine
51 mm (2.0 in)	100		100	
38 mm (1.5 in)	100		100	
25 mm (1.0 in)	100		98	
19 mm (3/4 in)	71		73	
12.7 mm (1/2 in)	35		34	
9.53 mm (3/8 in)	19		18	
4.75 mm (No. 4)	1.2	100	3.3	95
2.36 mm (No. 8)		86		75
1.18 mm (No. 16)		63		57
0.600 mm (No. 30)		38		35
0.300 mm (No. 50)		17		13
0.150 mm (No. 100)		8		4
0.075 mm (No. 200)		2.9		1.5

Table 63. Aggregate properties for RCA and control mixes.

Concrete Mix	Aggregate Type	Specific Gravity	Absorption Capacity, %	L.A. Abrasion, %	Sodium Sulfate, %
Recycled	Recycled Coarse	2.45	3.31	39.7	
	Virgin Coarse	2.65	0.83	29.4	0.79
	Recycled Fine	2.36	6.45		
	Virgin Fine	2.63	0.75		0.64
Control	Virgin Coarse	2.65	1.07	24.3	3.55
	Virgin Fine	2.61	0.60		0.80

A preliminary mix design was developed but had to be modified during the construction phase of the project to ensure that the supply of virgin coarse aggregate would not be depleted. The amount of recycled coarse aggregate in the mix was increased and was balanced by also increasing the amount of virgin fine aggregate. The amount of water in the mix was also adjusted to reflect these changes. An air-entraining admixture was also added to both mixes in quantities to produce average total air contents of 5.5 percent. The resulting batch proportions for the recycled and control concrete mixes are shown in table 64. The RCA concrete mixture contains a slightly lower volume of coarse aggregate, a slightly higher volume of fine aggregate and a higher volume of cementitious materials. The use of fly ash in the RCA concrete mixture resulted in a higher water-cement ratio for the recycled concrete mix, but a lower water-cementitious material (cement plus fly ash) ratio. The average slump of the RCA mixture was 32 mm (1.25 in), while it was 44 mm (1.75 in) for the control mixture.

Table 64. Concrete mix proportions for RCA and control mixes.

Material	Recycled	Control
Recycled Coarse Aggregate	669 kg/m^3	
Recycled Fine Aggregate	150 kg/m^3	
Virgin Coarse Aggregate	357 kg/m^3	1108 kg/m^3
Virgin Fine Aggregate	535 kg/m^3	686 kg/m^3
Cement	290 kg/m^3	349 kg/m^3
Fly Ash	79 kg/m^3	0 kg/m^3
Water	141 kg/m^3	153 kg/m^3
w/(c+p) Ratio	0.38	0.44

Construction Information

Construction of the recycled section began in August of 1984. The project was constructed in four phases with crossovers at both ends and at an intermediate location within the project. The first primary step was the removal of the existing PCC pavement and AC shoulders. A Whip Hammer pavement breaker was used to break up the existing pavement. The material was then loaded onto trucks and carried to a crushing plant. The recycled concrete and the virgin aggregate were crushed at the same plant using a 107-cm (42-in) primary jaw crusher.

The new pavement was placed using one 11.6-m (38-ft) pass of a Caterpillar SF-550 slip-form paver. Tie bars were placed in the pavement at the longitudinal shoulder joint and at the centerline using manually-triggered hydraulic inserters. A tube float and astroturf drag followed the paving operation. Finally, the pavement surface was tined in the transverse direction and sprayed with a curing compound.

Concrete Properties

Beams cast during construction of the recycled concrete section were tested at 28 days, resulting in an average flexural strength of approximately 4.8 MPa (700 lbf/in^2). The strengths of concrete specimens cast from the control section concrete are not available. However, testing during the mix design phase gave average flexural strengths of 5.0 and 6.1 MPa (730 and 890 lbf/in^2) at 7 and 28 days, respectively, and 28-day compressive strengths averaging 29.8 MPa (4,320 lbf/in^2).

Climatic Conditions

The WY 1 test sections are located in the dry-freeze environmental region. The minimum and maximum average monthly temperatures are -3 and 21 °C (26 and 70 °F). The freezing index is 270 °C-days (480 °F-days), and the sections are exposed to about 140 freeze-thaw cycles per year. The area experiences about 90 days of precipitation per year for a total annual precipitation of 360 mm (14 in), resulting in a Thornthwaite moisture index of -10.

Traffic Loadings

The recycled section was opened to traffic in 1985. The two-way ADT has increased from about 4,410 vehicles per day in 1985 to approximately 6,700 vehicles per day in 1994. The percentage of trucks on the highway has varied from around 35 to 45 percent. The cumulative ESAL applications in the design lane (through 1994) is estimated at 3.6 million.

The control section was opened to traffic in 1984. The two-way ADT has increased from about 4,280 vehicles per day in 1984 to approximately 6,700 vehicles per day in 1994. The percentage of trucks on the highway has varied from around 35 to 45 percent. The cumulative ESAL applications in the design lane (through 1994) is estimated at 3.8 million.

Selection of Distress Survey Sections

Several different criteria were considered when selecting the sections to be surveyed. Both sections were constructed on level grade without any significant cut or fill. Sections without any horizontal curves, bridges, or other discontinuities were preferred; however, the only viable location for the control section was on a horizontal curve. Finally, the sections were both located in the eastbound lanes and exposed to similar traffic loadings.

Drainage Survey

The sections did not contain any positive provisions for drainage. However, no signs of moisture-related problems, such as pumping of fines or cattails in the ditches were evident. The transverse slopes on the recycled section were around 1 percent on both the traffic lane and shoulder. On the control section, the transverse slopes ranged from 2 to 4 percent on the traffic lane and were generally 4 percent on the outside shoulder. On both sections, the ditch line was located about 1.2 m (4 ft) below the pavement surface.

Pavement Distress Survey

The pavement condition survey was conducted over sections approximately 305 m (1,000 ft) in length. A complete summary of the results of the survey are provided in appendix A. A summary of the average results for the key variables is shown in table 65. Overall, the recycled and control sections are exhibiting about the same levels of performance in terms of distress and serviceability.

Table 65. Summary of performance data (average values) for WY 1.

Performance Measurement	Recycled	Control
Corner Faulting, mm (Manual)	2.3	2.0
Wheel Path Faulting, mm (Manual)	2.3	2.0
Wheel Path Faulting, mm (Digital)	2.0	2.0
Transverse Cracking, % Slabs	0	0
Longitudinal Cracking, m/km	55	14
Transverse Joint Spalling, % Joints	25	16
Joint Width, mm	10	11
PSR	3.6	3.6

Transverse Joint Faulting

Average faulting levels at the transverse joints are approximately the same for both the recycled and control sections. Very little difference in faulting measurements was noted regardless of measurement location (corner vs. wheel path) or fault measuring equipment used (manual vs. digital). The average faulting in both sections averaged about 2.0 mm (0.08 in), ranging from -0.5 to 7.9 mm (-0.02 to 0.31 in) in the recycled section and from 0.2 to 5.3 mm (0.01 to 0.21 in) in the control section. This represents a slight increase from the average value of 1.3 mm (0.05 in) measured in the eastbound lanes by the Wyoming Department of Highway in 1989, as described under "Other Performance Test Results" below.[27]

This region receives very little precipitation each year, and no signs of pumping were evident on either section. However, the little rainfall that does occur can be trapped within the pavement and, without dowel bars to provide load transfer, has apparently lead to the observed transverse joint faulting.

Transverse Cracking

No transverse cracking was present in either recycled or control sections. Both sections employ a short, "random" 4.3-4.9-4.0-3.7-m (14-16-13-12-ft) joint spacing. This short joint spacing pattern has apparently been very effective at limiting thermal curling stresses and the initiation of transverse cracks. JPCP are designed to eliminate uncontrolled transverse cracking, and these pavement sections are no exception (L/ℓ ratio ranges from 3.7 to 5.0 for the various panels lengths using the average laboratory-determined value of the concrete elastic modulus and the average backcalculated subgrade modulus for each section).

Longitudinal Cracking

Some longitudinal cracking was observed in each of the survey sections. The recycled section contained one medium-severity longitudinal crack that extended through four slabs, which amounts to 55 m/km (288 ft/mi). The control section contained one medium-severity crack extending across a 4.3-m (14-ft) slab, resulting in 14 m/km (74 ft/mi) of longitudinal cracking. One study noted the existence of this cracking at that time and attributed it to a combination of shrinkage and late sawing.[27]

Transverse Joint Spalling

Some spalling of the transverse joints was apparent on both sections, with slightly more spalling occurring on the recycled section. Spalling occurred at 25 and 16 percent of the joints for the recycled and control sections, respectively, with all of the spalling being of low severity except for one high-severity joint spall in each section. The slight difference in transverse joint spalling between the recycled and control sections was considered insignificant since almost all of the spalling was localized low severity fraying of the joints. One study also noted this spalling and attributed it to localized lodging of incompressible materials at the joint surface.[27] Other possible mechanisms include snowplow damage and localized weakening of the concrete during the sawing process.

Recurrent Alkali-Silica Reactivity

Several very small (0.1 - 0.3 m^2 [1 - 3 ft^2]), localized areas of the RCA concrete section exhibited map cracking, presumably due to recurrent alkali-silica reactivity (ASR).

Only one of these areas was observed within the distress survey section. Cores from the RCA concrete section were observed to fluoresce brightly when subjected to the uranyl acetate test, as described below under "Petrographic Examination," which tends to support the hypothesis that ASR was present. There was no spalling or scaling associated with these small areas of deterioration, and the performance of the RCA concrete section was not adversely affected at the time of survey. There was nothing that resembled map cracking or alkali-silica reactivity in the control section.

Present Serviceability Rating (PSR)

The average PSR values of the recycled and control sections are 3.6. These values are consistent with the other distress measurements, which are also similar for both sections. The roughness associated with these sections is due almost entirely to faulting at the transverse joints, as little cracking or other deterioration exists.

Other Performance Test Results

The pavement condition has been monitored by the Wyoming Department of Highways since it was constructed. A 1989 report by Oyler reflected the results of a condition survey, and tests skid resistance, International Roughness Index (IRI), and Rut Depth Rating.[27] The average results of these tests are highlighted in table 66. The performance data indicate that the pavement was in good condition at the time of the testing in 1989, with very little cracking or spalling, and only slight faulting.

Table 66. 1989 performance data for RCA concrete pavement.[27]

Measurement	Lane	Average Value
IRI, m/km	EB	1.5
	WB	1.6
Rut Depth Rating, mm	EB	0.3
	WB	0.5
Skid Resistance	EB	52
	WB	49
Transverse Joint Faulting, mm	EB	1.3
	WB	1.5

FWD Testing

Pavement deflection testing was performed using a Dynatest model 8081 FWD. The typical project test pattern included 5 slab centers, 10 transverse joints (with load placement on both the approach and leave sides of the joints), 10 transverse cracks (with load placement on both sides of the cracks), and 10 slab edges at the lane-shoulder joint near midpanel. However, no transverse cracks were found in either section, so these tests could not be performed. FWD testing was used to determine material properties (PCC elastic modulus and subgrade k-value), joint load transfer efficiencies, and loss of support. Table 67 provides a summary of the results obtained from the deflection testing data.

Table 67. Deflection testing results for WY 1.

Property	Recycled	Control
PCC Elastic Modulus, GPa	32.1	50.5
k-value, kPa/mm	52.7	42.9
Joint Load Transfer, %	19	55
Crack Load Transfer, %	n/a	n/a
Shoulder Load Transfer, %	87	53
Average Midslab Deflection, µm	106	87
Average Edge Deflection, µm	153	139
Corners With Voids, %	80	10
Maximum Air Temperature During Testing, °C	16	27

Elastic Modulus

The elastic modulus values of the PCC slab were backcalculated using the center-of-slab deflection measurements. Figure 85 shows a profile of the elastic modulus values for the recycled section obtained using four drops at each of five different locations. The average backcalculated elastic modulus is 32.1 GPa (4,660,000 lbf/in^2), with values ranging from 23 to 41 GPa (3,340,000 to 5,950,000 lbf/in^2). Figure 85 indicates good repeatability in elastic modulus estimates obtained using different drops at the same location, but a fairly large variation between values obtained at different locations.

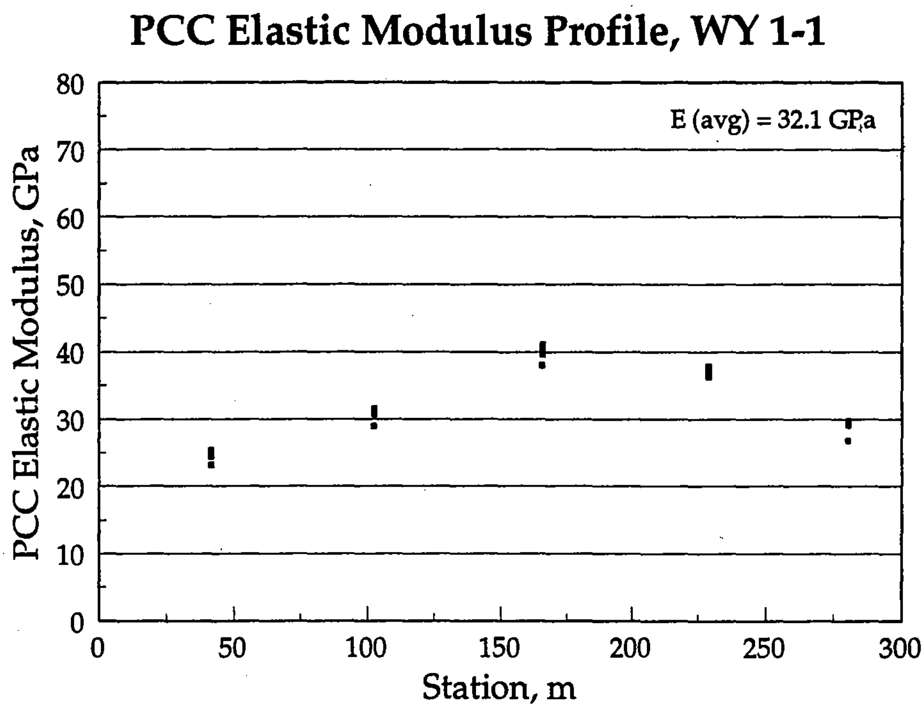


Figure 85. PCC elastic modulus profile for WY 1-1 (recycled section).

A similar profile plot for the control section is shown in figure 86. The average elastic modulus value is 50.5 GPa (7,320,000 lbf/in^2), with values ranging from 40 to 61 GPa (5,800,000 to 8,850,000 lbf/in^2). With the exception of one location, the backcalculated values are about the same for the four different drops. Variation between the different locations is noticed, with values toward the west end of the section being lower.

The backcalculated elastic modulus values for the control section are more than 50 percent greater than those of the recycled section. Laboratory testing of cores obtained from the project sections (discussed in the coring section, below) does not support this finding, however. While it is possible that there are significant differences in the elastic modulus of the concrete contained within the two survey sections, no apparent performance differences have been observed and it seems more likely that variations in the pavement layers and construction (different contractors used on recycled and control sections) are responsible for the apparently inflated and highly variable backcalculated modulus values obtained at different project locations.

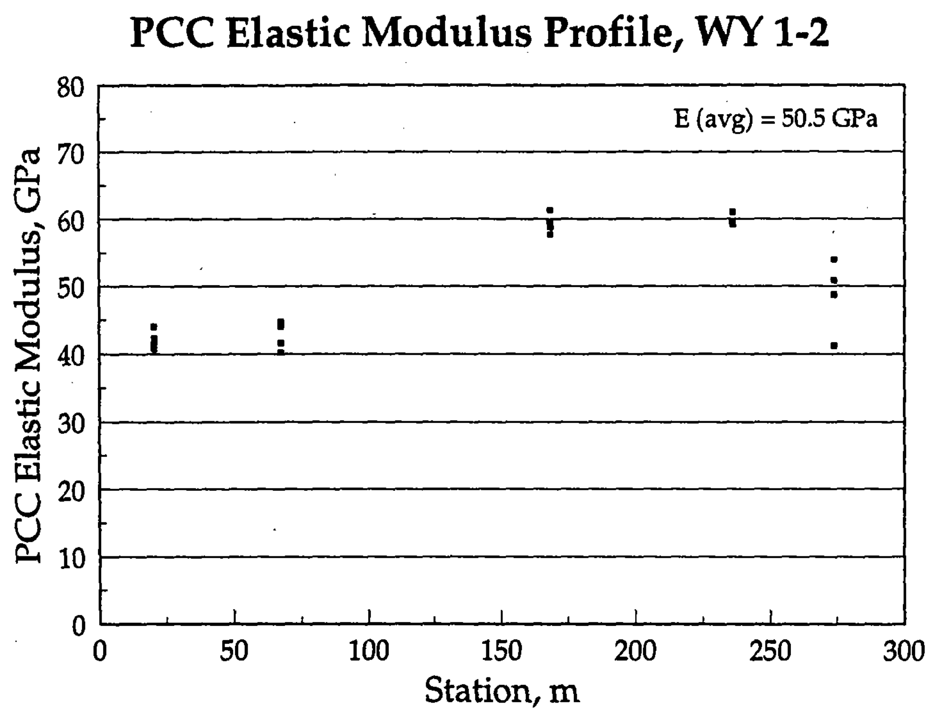


Figure 86. PCC elastic modulus profile for WY 1-2 (control section).

Modulus of Subgrade Reaction (k-value)

Figure 87 illustrates a profile plot of the backcalculated k-values for the recycled section. The average k-value is 52.7 kPa/mm (194 lbf/in^2/in), with values ranging from 40 to 64 kPa/mm (149 to 236 lbf/in^2/in). A similar profile plot for the control section is shown in figure 88. These k-values range from 32 to 70 kPa/mm (118 to 258 lbf/in^2/in), and the average of all tests is 42.9 kPa/mm (158 lbf/in^2/in). There are no easily observed reasons for the differences between the backcalculated k-values obtained for these two sections. Possible explanations include variations in foundation stiffness, variations within the pavement layers, or differences in temperature gradients at the time of testing. It is also possible that the observed differences reflect the effects of recent precipitation events and variations in soil drainage along the project, although it seems more likely that the stiffness of the foundation varies along the project length. In any case, the values obtained are representative only of the conditions that existed at the time of testing in October 1994.

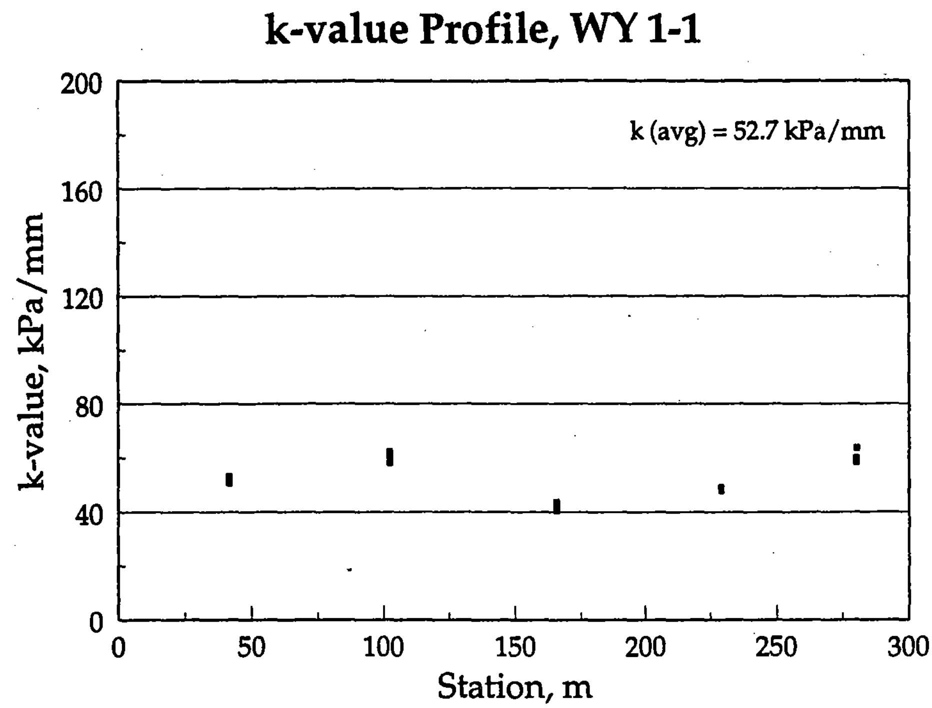


Figure 87. K-value profile for WY 1-1 (recycled section).

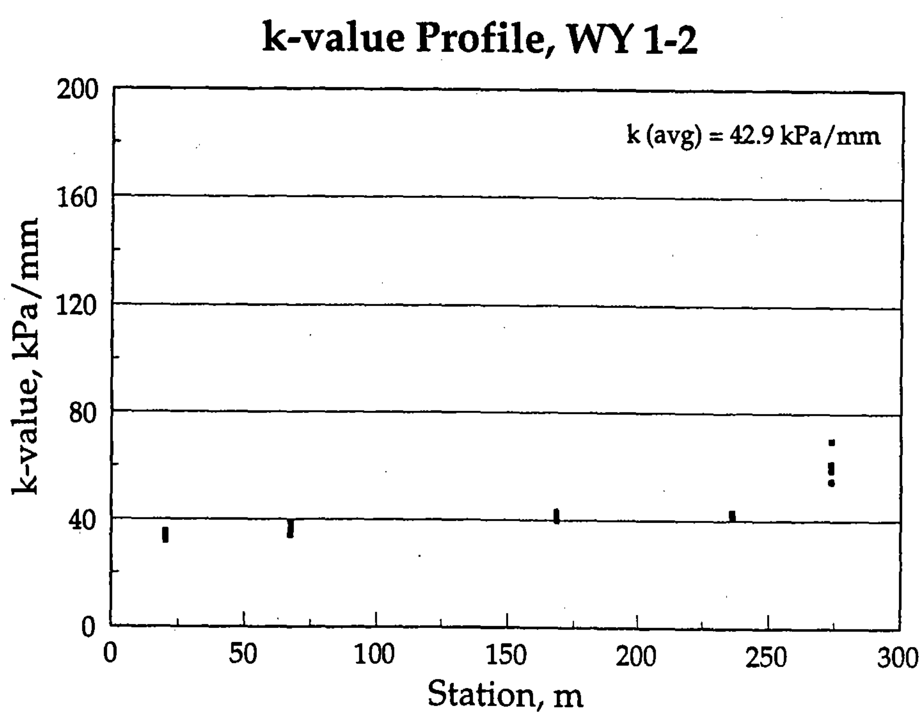


Figure 88. K-value profile for WY 1-2 (control section).

Joint Load Transfer

The load transfer efficiencies measured with the load placed on both the approach and leave sides of the recycled section transverse joints are shown in figure 89. These values are computed as the ratio of the deflection on the unloaded side of the joint to the deflection on the loaded side of the joint. The average deflection load transfer efficiency for this section is only 19 percent, which indicates a low degree of aggregate interlock due to some combination of poor joint face texture, wide joint openings and/or joint face abrasion. Large differences in the load transfer efficiencies were observed with load placement on either side of the joints. The average load transfer efficiencies for load placement on the approach and leave sides of the joint are 14 and 23 percent, respectively. The air temperature at the time of testing was about 16 °C (61 °F) and that pavement surface temperatures ranged from about 11 to 15 °C (52 to 59 °F).

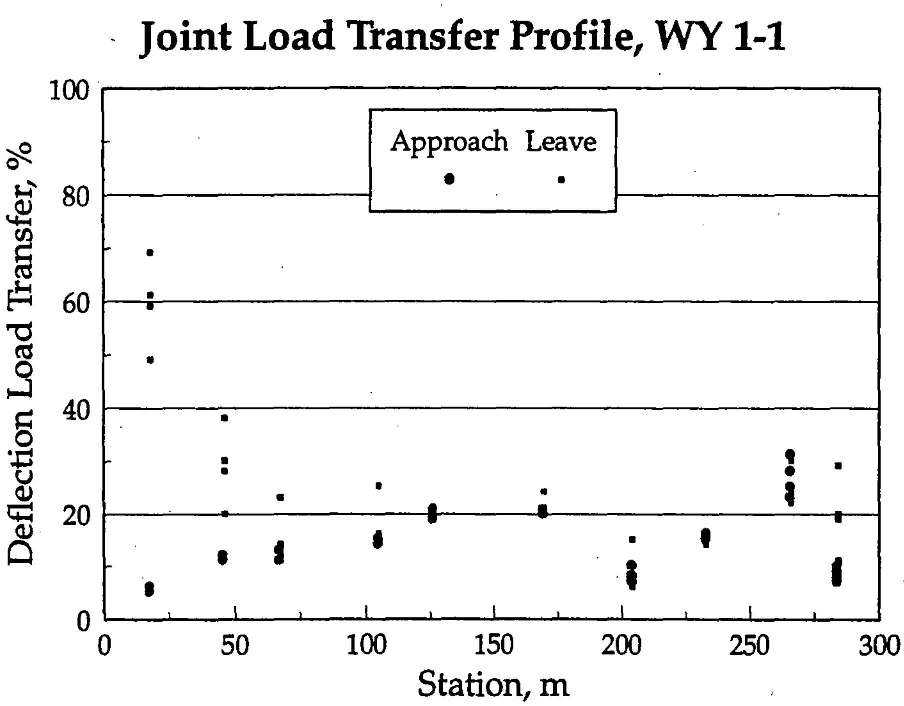


Figure 89. Joint load transfer profile for WY 1-1 (recycled section).

Figure 90 illustrates the joint load transverse efficiencies for the control section. The average load transfer efficiency is 55 percent (much higher than on the RCA concrete section), with values measured on the approach and leave sides of the joints averaging 39 and 71 percent, respectively. The air temperature at the time of testing was about 27 °C (81 °F) and pavement surface temperatures ranged from about 24 to 27 °C (75 to 81 °F). These differences in air and pavement temperature at the time of testing provide at least a partial explanation for the much better load transfer observed in the control section than in the RCA section. In addition, almost all of the joints in this section exhibited large differences in load transfer when the load plate was moved from the approach side of the joint to the leave side. The reason for this phenomenon may be due to the inclination of the cracks under the transverse joint. The joints may be cracking toward the leave side of the joint, resulting in a higher load transfer efficiency on the leave side. However, the cores do not support this hypothesis.

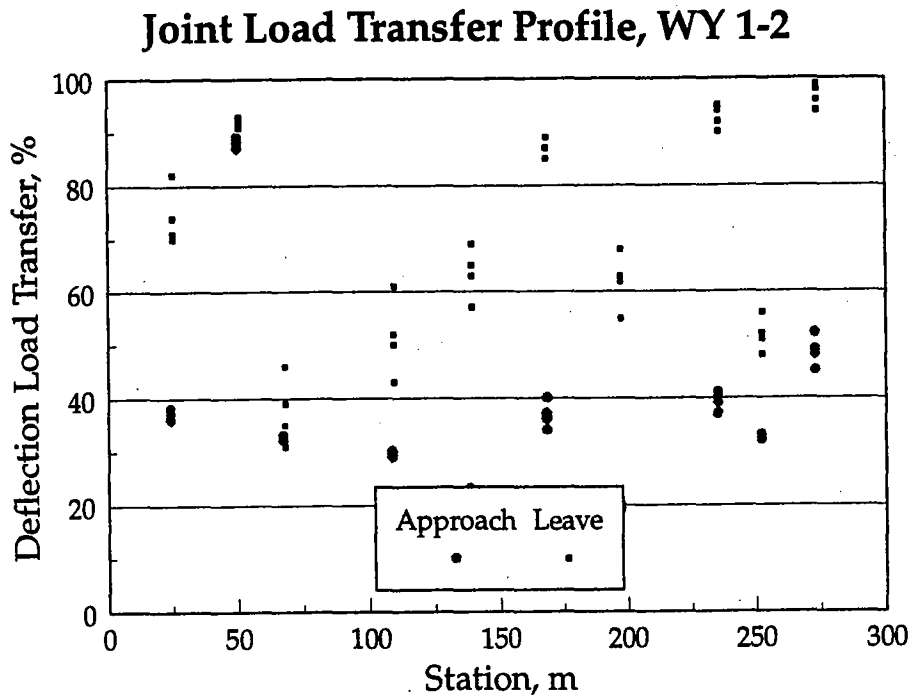


Figure 90. Joint load transfer profile for WY 1-2 (control section).

In summary, the recycled and control sections are both currently exhibiting levels of load transfer efficiency that are probably unacceptably low for a pavement carrying so many heavy vehicle loads. While the test results seem to indicate that the control section has better load transfer capacity than the RCA section, this conclusion should probably not be drawn because of the large difference in ambient (and pavement) temperatures that existed when these sections were tested.

Shoulder Load Transfer

Figures 91 and 92 illustrate the load transfer efficiencies across the tied PCC shoulder for the recycled and control sections, respectively. The entire pavement section, including the traffic lanes and the inner and outer shoulders, were paved monolithically on both the recycled and control sections. The average load transfer efficiency for the recycled section is 87 percent, compared to only 53 percent on the control section. This trend is opposite to that observed at the transverse joints. One reason noted for the difference at the transverse joints was the higher temperature during testing on the control section. Although the temperature was also higher when testing at the shoulder, temperature variations have less effect on load transfer efficiency across longitudinal joints due to shorter slab length and less associated movements. The sections were constructed using different contractors, which may have contributed to the observed differences.

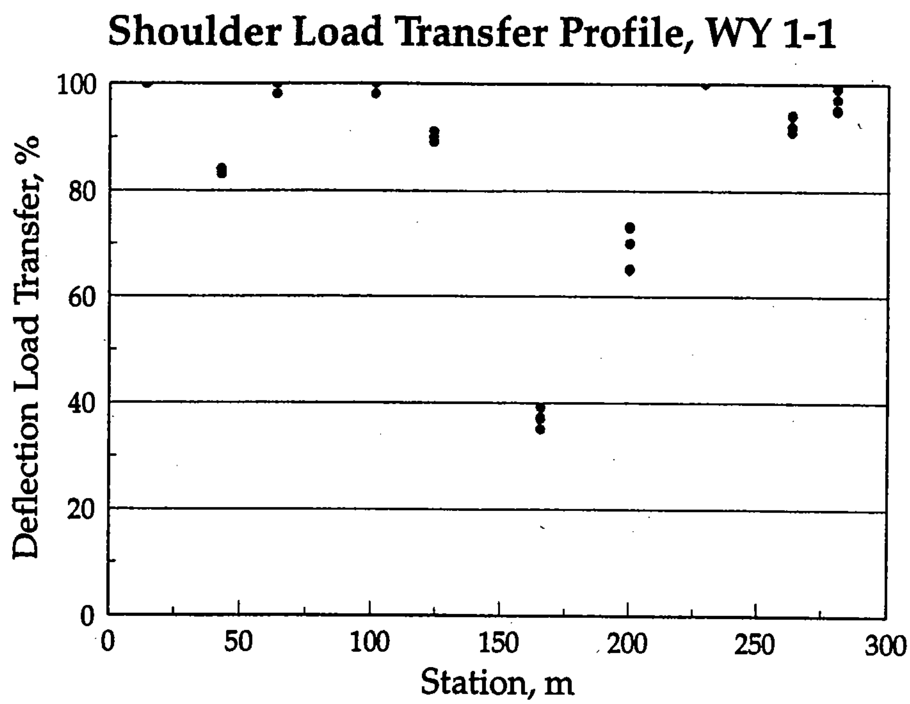


Figure 91. Shoulder load transfer profile for WY 1-1 (recycled section).

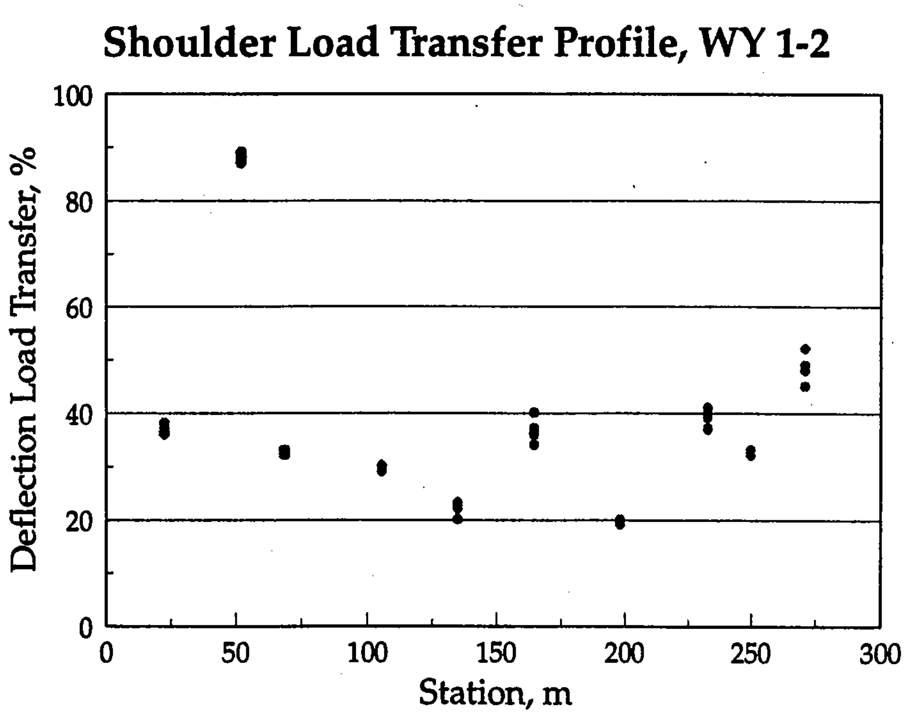


Figure 92. Shoulder load transfer profile for WY 1-2 (control section).

Loss of Support

The detection of voids was performed data using the corner deflections on the leave side of transverse joints and cracks and procedures described in the final report for NCHRP 1-21. Figures 93 and 94 illustrate the potential for loss of support along the recycled and control sections, respectively. These figures show that the RCA concrete section has developed a loss of support under almost all of the joints tested; only a few joints in the control section seem to have developed voids beneath the slab corners.

However, these results must be considered in the context of the temperature gradients that existed when the testing was performed. Temperature measurements were taken at various depths at representative locations in both pavement sections during the time when FWD testing was being performed. Table 68 presents a summary of this information and indicates that temperature gradients ranged from -0.33 to -0.05 °C/cm (-1.51 to -0.25 °F/in) when the RCA pavement section was being tested, while they ranged from +0.26 to +0.40 °C/cm (+1.19 to +1.84 °F/in) when the control section was being tested. Thus, the corners of the RCA pavement section were probably curled upward, while those in the control section were curled downward. The most reliable measurements would be those taken near the end of the RCA section, when the temperature gradient was closest to zero. At this time, voids are still indicated for the RCA pavement section. However, given the comparable degrees of faulting that have developed within each section, it is very possible that the same voids exist beneath the control section as well, even though FWD tests failed to reveal them because of the strong positive temperature gradients that existed at the time of testing.

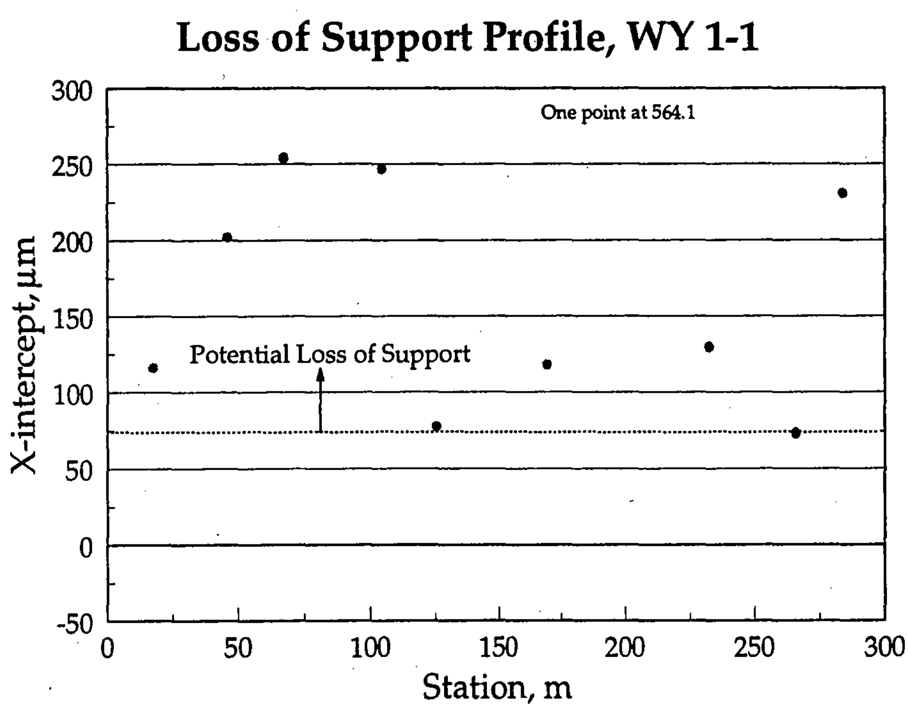


Figure 93. Loss of support profile for WY 1-1 (recycled section).

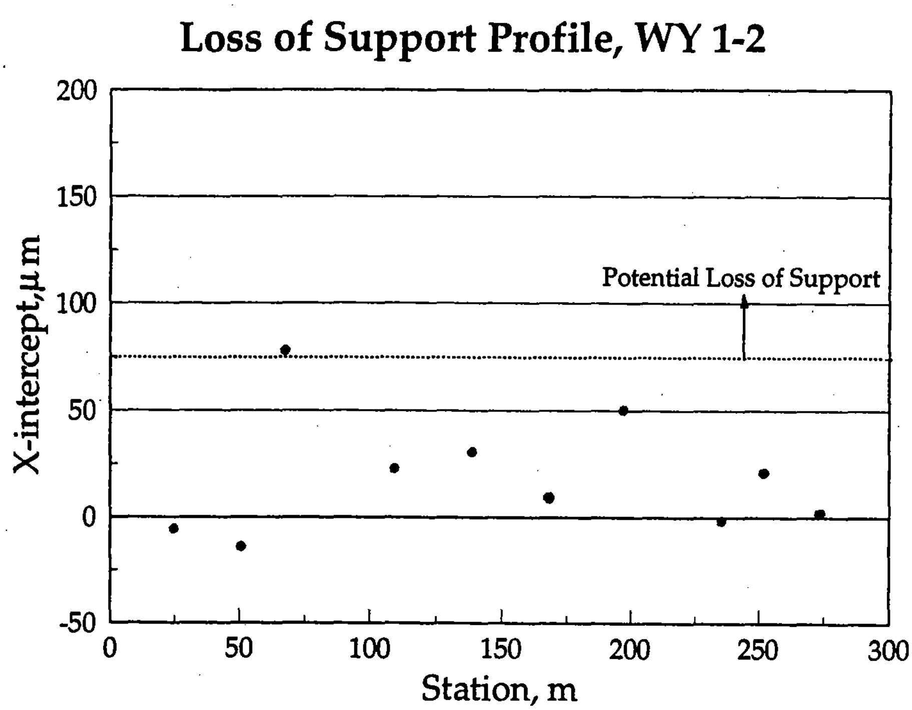


Figure 94. Loss of support profile for WY 1-2 (control section).

Table 68. Temperature gradients during FWD testing for WY 1.

Project	Time	H1 (22.9 cm)	H2 (12.7 cm)	H3 (2.5 cm)	Average Temp. Gradient (H3-H1) / 20.34
WY 1-1	7:30 am	17.6 °C	14.7 °C	10.8 °C	-0.33
	8:10 am	16.7 °C	14.4 °C	12.4 °C	-0.21
	8:40 am	16.8 °C	14.7 °C	13.4 °C	-0.17
	9:10 am	16.4 °C	14.5 °C	15.3 °C	-0.05
WY 1-2	10:30 am	18.7 °C	18.9 °C	24.0 °C	+0.26
	11:00 am	18.2 °C	20.0 °C	25.2 °C	+0.34
	11:30 am	18.4 °C	20.1 °C	26.2 °C	+0.38
	Noon	18.8 °C	20.7 °C	26.9 °C	+0.40

Coring

The coring plan called for 11 cores to be taken from both the recycled and control sections: 5 at midpanel, 3 at transverse joints, and 3 at transverse cracks. However, since no transverse cracks were observed within either survey section, only eight cores were retrieved from each section. On both sections, the four midpanel cores intended for strength and elastic modulus tests were 100 mm (4 in) in diameter; all other cores were 150-mm (6-in) in diameter. The average thickness of the cores retrieved from the recycled and control sections were 251 and 267 mm (9.9 and 10.5 in), respectively, which compares favorably with the nominal design thickness of 254 mm (10.0 in). These cores were tested in the laboratory to determine the physical and mechanical properties of the two concrete mixtures used on this project, as described in more detail below.

Core Testing

The number of cores for each laboratory test is indicated in table 69. A summary of the average values that were obtained during the laboratory testing of the field cores is presented below in table 70 and in table 83 in appendix A. Observations made during the testing, and comparisons between the performance of the control and recycled sections are also provided below.

Table 69. Number of cores for each laboratory test in WY 1.

Laboratory Tests	Recycled Section	Control Section
Thermal Coefficient	3	3
Split Tensile Strength	1	1
Dynamic Modulus of Elasticity	3	3
Static Modulus of Elasticity	1	1
Compressive Strength	3	3
Volumetric Surface Texture	3	3

Table 70. Core testing results for WY 1.

Property	Recycled	Control
Compressive Strength, MPa	48.7	44.7
Split Tensile Strength, MPa	3.7	3.2
Dynamic Elastic Modulus, GPa	35.0	36.7
Static Elastic Modulus, GPa	33.2	29.1
Thermal Coefficient, $(1x10^{-6})/$ °C	13.3	10.8
VSTR (for Failed Split Tensile Core), cm^3/cm^2	0.1711	0.3019
VSTR (for Slab Faces at the Joints), cm^3/cm^2	0.2927	0.5043
VSTR (for Slab Faces at the Cracks), cm^3/cm^2	n/a	n/a

Petrographic Examination Summary

The coarse aggregate for the recycled section was found to consist of a very fine-grained dolomitic limestone blended with gravel rock deposits that were observed to be rounded-to-angular. The gravel rock is further characterized as original coarse aggregate containing pink to white coarse-or medium-grained igneous particles. This aggregate was evenly distributed through the mortar. The coarse aggregate for the control section was found to consist of rounded-to-angular gravel rock particles that were also evenly distributed through the mortar. The gravel rock is further characterized as pink to white fine-or very fine-grained igneous particles. An ASTM C 618 Class F fly ash was included in the recycled concrete mixture.

The mortar and coarse aggregate contents of both the recycled and control materials were estimated using linear traverse techniques (see table 71). The RCA concrete was found to contain more new mortar than the control section concrete (about 68 percent vs. 57 percent), which was expected, given the higher total cementitious material content in the recycled mixture. In addition, the RCA concrete contained an additional 8 percent old mortar. As a result, the RCA concrete contained less natural coarse aggregate than the control concrete (23 percent vs. 43 percent).

Table 71. Coarse aggregate and mortar contents for WY 1.

	Recycled	Control
Coarse Aggregate, %	23.8	43.1
New Mortar, %	68.4	56.9
Recycled Mortar, %	7.8	n/a

Uranyl acetate testing indicated the presence of moderate amounts of silica gel in the mortar and around some of the aggregate particles in the RCA concrete, possibly indicating the presence of ASR activity. Only minor amounts of silica gel were indicated in similar tests performed on the control section samples.

Mid-Panel Cores

The compressive strengths of the RCA concrete cores ranged between 45.0 and 53.5 MPa (6,530 and 7,760 lbf/in^2), with an average of 48.7 MPa (7,060 lbf/in^2). Compressive strengths for the control section cores ranged between 42.5 and 46.4 MPa (6,160 and 6,730 lbf/in^2), averaging 44.7 MPa (6,480 lbf/in^2). Diametral or split cylinder tensile testing was performed on only one core from each section; strengths of 3.7 and 3.2 MPa (540 and 460 lbf/in^2) were obtained for the recycled and control sections respectively. It should be noted that the average compressive and split tensile strengths were unexpectedly higher for the RCA concrete specimens than for the control concrete specimens. Factors that may have contributed to the increased strength of the RCA mixture include its lower water-cementitious ratio (0.38 vs. 0.44), its use of fly ash as a partial replacement for cement (which has been shown to produce higher long-term strength through pore-refinement and production of additional cementitious products), and its use of some recycled concrete fines, which has been shown to produce concrete with higher strength.[17]

The dynamic elastic modulus for the RCA concrete cores ranged from 34.1 to 35.5 GPa (4,950,000 to 5,150,000 lbf/in^2), with an average of 35.0 GPa (5,080,000 lbf/in^2). Control section values ranged from 35.7 to 38.3 GPa (5,180,000 to 5,550,000 lbf/in^2), with an average of 36.7 GPa (5,320,000 lbf/in^2). The static elastic moduli for these sections were estimated using one core from each section; the elastic moduli of RCA concrete and control concrete cores were 33.2 and 29.1 GPa (4,810,000 and 4,220,000 lbf/in^2), respectively. Previous studies have suggested that the modulus of elasticity of RCA concrete is typically 15 to 30 percent lower than that of conventional concrete.[15] In this case, however, laboratory tests suggest that the elasticity of the RCA mixture was comparable to that of the control material. The most probable reasons for this are the same as those presented for the higher RCA strengths, above.

It is worth noting that the backcalculated dynamic modulus values agreed reasonable well with the laboratory values for the RCA concrete, but not at all for the control section concrete. This might suggest that the backcalculated k-values in the RCA section are also reasonable, while those obtained in the control section should be used with more caution.

The thermal coefficient of expansion ranged from 11.2×10^{-6} / °C to 17.3×10^{-6} / °C (6.2×10^{-6} / °F to 9.6×10^{-6} / °F) for the recycled section, with an average of 13.3×10^{-6} / °C (7.4×10^{-6} / °F). The control section thermal coefficients ranged from 10.2×10^{-6} / °C to 11.3×10^{-6} / °C (5.7×10^{-6} / °F to 6.3×10^{-6} / °F) for the control section, with an average of 10.8×10^{-6} / °C (6.0×10^{-6} / °F). The higher total mortar content of RCA concrete would have been expected to produce significantly higher thermal expansion coefficients, and this was the case for the samples that were obtained from this project.

In summary, the laboratory tests of cores obtained from the RCA and control sections suggest that the two materials have comparable strength and elasticity characteristics, in spite of reduced coarse aggregate quantities in the RCA mixture. The higher-than-expected strength and elastic modulus of the RCA mixture is attributed to the higher cementitious content, lower water-cementitious ratio, and use of recycled fines in that mixture. However, the reduced content of natural coarse aggregate is also probably the source of the higher thermal coefficient of the RCA concrete.

Joint Cores

The VSTR obtained for the control section joints (0.5043 cm^3/cm^2) is significantly higher than that obtained for the recycled section joints (0.2927 cm^3/cm^2), although both values would probably be considered representative of adequate surface texture for aggregate interlock across a reasonably tight joint. A similar trend in surface textures was noted when the surfaces of the split tensile test specimens were measured (0.3019 cm^3/cm^2 for the control vs. 0.1711 cm^3/cm^2 for the RCA sample). The exceptionally good texture of the control mixture is probably due to a combination of the high strength of the aggregate particles, the relatively large amount of aggregate used (one of the highest proportions of those considered in this study), and the large aggregate top size.

In spite of the relatively good surface texture of the joint faces in both sections, load transfer efficiencies for both sections were extremely low, presumably because there were not dowels used and because the joint widths were much greater than the 0.76 mm (0.03 in) typically deemed necessary for adequate aggregate interlock load transfer.

Project Summary

This project provides a direct comparison of the performances of recycled and traditional concrete pavement sections constructed in 1985 and 1984 respectively using identical structural designs (250-mm [10-in] JPCP constructed without mesh reinforcing, mechanical load transfer devices or pavement drainage, and with an average transverse joint spacing of 4.2 m [14 ft]) subjected to identical traffic (3.6 and 3.8 million ESAL's through 1994, respectively) and environmental conditions. The major difference between the sections, other than aggregate type, is the mix design. The RCA concrete mixture contained comparable volumes of coarse and fine aggregate (both were blends of RCA and natural materials), but contained significantly more cementitious material, which resulted in a lower water-cementitious ratio. In addition, the RCA was produced from pavement that had suffered from a severe ASR reaction.

The results of a condition survey, deflection testing and laboratory tests on retrieved cores indicate the recycled and control sections are constructed of generally similar materials and are exhibiting generally similar performances, with a few important exceptions. A summary of the key findings follows.

Material Properties

The laboratory tests of cores obtained from the RCA and control sections suggest that the two materials have comparable strength and elasticity characteristics (the strength of the RCA concrete was actually significantly higher than that of the control concrete), in spite of reduced coarse aggregate quantities in the RCA mixture. The higher-than-expected strength and elastic modulus of the RCA mixture is attributed to the higher cementitious content, lower water-cementitious ratio, and use of recycled fines in that mixture.

The thermal coefficient of expansion of the RCA concrete was about 19 percent higher than that of the control concrete, a difference that was determined to be statistically significant at the 80 percent level. This difference is probably attributable to the higher total mortar content (new mortar plus recycled mortar) and recycled concrete fines used in the RCA concrete mixture.

The texture of the control section concrete joint faces and fractured lab test specimens was much higher than for the RCA concrete specimens. This was probably due to a combination of the high strength of the virgin aggregate particles, the relatively large amount of aggregate used, and the large aggregate top size.

Pavement Performance

Transverse cracks were not observed within either survey section, and the effects of joint spalling and longitudinal cracking were minimal.

The average faulting at the transverse joints was approximately 2.0 mm (0.08 in) in both sections in 1994. This is an increase of 50 percent above the 1.3 mm (0.05 in) reported by the Wyoming Department of Highways in 1989. In addition, the recycled section seems to show a strong potential for loss of support on the leave side of the transverse joints, and it is considered likely that a similar situation exists in the control section. This suggests that dowels or other mechanical load transfer devices should have been included at the joints, especially given the high volume of heavy truck traffic that travels this route. The use of pavement drains might have also retarded the development of faulting.

The average transverse joint load transfer efficiency on the recycled section was only 19 percent; the control section joint load transfer efficiency averaged 55 percent. Given the relatively high VSTR values obtained for either section, these load transfer efficiencies seem low (although the higher value for the control section is consistent with the much higher surface texture ratios obtained for that material). Higher values would have probably been measured if the joints had been tighter or with the use of dowels.

Small, localized areas of recurrent ASR were scattered along the RCA portion of the construction project, although only one such area was identified in the survey section. Uranyl acetate testing of cores obtained from the recycled specimen indicated the possible presence of moderate amounts of ASR products. Similar tests on control section specimens found indications of only minor amounts of ASR products.

4. SUMMARY

The prime focus of this interim report was to investigate inservice concrete pavements that were constructed using recycled concrete aggregates (RCA) in the portland cement concrete surface. This interim report builds upon the comprehensive literature review performed in the Task A (and documented in the Task A Interim Report) with a summary of the results of evaluations of selected recycled concrete pavements.

The selection process concentrated on the following three pavement categories in order to develop a good understanding of the success and failure conditions associated with recycled concrete pavement performance:

1. JRCP with nonworking transverse cracks and little or no distress, or JPCP without transverse cracks and exhibiting little or no distress.
2. JRCP with deteriorated transverse cracks or JPCP with any transverse cracks.
3. JRCP and JPCP exhibiting other distresses that might be related to the use of recycled concrete aggregate.

Based on the selection criteria, nine pavement projects in the United States were chosen. These projects were located in Connecticut, Kansas, Minnesota, Wisconsin, and Wyoming.

Analyses of the project origins, pavement designs, mix designs, construction records, material properties, climatic conditions, traffic loadings, drainage surveys, pavement distress surveys, falling weight deflectometer (FWD) deflection testing, petrographic core testing, and laboratory testing of cores for each project were used to develop the following preliminary findings.

Material Properties - Aggregate

- The Connecticut, Minnesota 2, Wisconsin 2-1, Wisconsin 2-2, and Wyoming recycled pavements exhibited low recycled mortar contents (less than 10 percent), which suggests that the concrete crushing operations were effective in removing most of the old mortar from the original aggregate in these cases.

  Of these five projects, only the Connecticut and Wyoming projects featured control sections constructed using natural coarse aggregate. In both cases, the performances of the RCA and control sections were similar. It is believed that these similarities probably stem from the fact that both sections included comparable amounts of natural aggregate (since the RCA particles contained little old mortar).

- The Connecticut, Kansas, and Wyoming RCA gradations approximately complied with the guidelines provided in ASTM C 33, "Standard Specification for Concrete Aggregates." Verification of compliance with ASTM C 33 for the other projects was not possible due to lack of information. The results of slump and strength tests of these three projects suggest that the plastic and hardened properties of the RCA concrete mixtures would be considered acceptable for conventional concrete materials.

- The fineness modulus for the Connecticut, Kansas, Minnesota 4, and Wyoming recycled pavements were in accordance with the guidelines provided in ASTM C 33, "Standard Specification for Concrete Aggregates." Again, verification of compliance to ASTM C 33 for the other projects was not possible due to lack of information.

  ASTM C 33 specifies that the fineness modulus be between 2.3 and 3.1. The Kansas and Wyoming recycled pavements included RCA fines and used a fine aggregate gradation that was closer to the middle of the specified range (2.75 and 2.88, respectively) than that of their corresponding control pavements (2.93 and 3.21, respectively). The Connecticut and MN 4 projects had all natural fine aggregate with essentially constant fineness modulus values for the RCA and control sections (2.66 and 2.88 for Connecticut and MN 4, respectively). Any effect of the fineness modulus on the strength and workability of the concrete mixtures was not apparent in this study, although research by Fergus and Yrjanson (presented in the Task A Interim Report) found that the inclusion of approximately 25 percent RCA fines would enhance the strength of the resulting concrete mixture.[17,28]

- The specific gravities of the recycled concrete coarse aggregates considered in this study were typically 0.2 - 0.3 lower than the values of their control section coarse aggregate counterparts. These specific gravities were generally near the lower end of the range typically considered "normal" for conventional aggregates (between 2.4 and 2.9).

Material Properties - Plastic Concrete

- The reported average air contents appeared to meet their respective mix design specifications. It was not clear what type of air content measuring device was used to produce these measurements, so it is difficult to comment on the influence of air entrainment (records indicated that a "Roll-O-Meter" was used on the Kansas project). The porous nature of recycled concrete aggregate particles makes the "Roll-O-Meter" the commonly-preferred air test apparatus over the "Press-R-Meter" typically used for conventional concrete construction testing.

- The few available construction records indicate that the recycled concrete mixtures provided reduced workability (as expected) due to the inherent angularity, rough surface texture and high absorption characteristics of the RCA.

Material Properties - Hardened Concrete

- The average coefficient of thermal expansion for RCA concrete samples was generally higher than for the control section concrete samples (MN 1 was the lone exception where the RCA and control values were equal). The increase coefficient of the RCA sections may be due to the lower natural aggregate contents of these materials, which affords less restraint to volumetric expansion in response to temperature and moisture fluctuations.

- The laboratory-determined dynamic elastic modulus values for the recycled pavements were always lower than that of their corresponding control pavements, although none of the measured values would be considered unusually high or low for concrete pavement materials. The RCA values were between 1 and 18 percent lower than for the control concrete; previous studies suggested that a difference of 15 to 50 percent would be more common.[1]

  Dynamic elastic modulus values obtained by backcalculation from nondestructive deflection test data exhibited the same general trends, although the differences between the RCA and control section test values were closer to those suggested in the literature.

- Static elastic modulus values were also lower for the recycled pavements than for the corresponding control pavements, except for the Wyoming project.

- It was noted for many projects that the backcalculated PCC modulus resulted in unrealistically high values when compared to test cores. It is assumed that directly measured values of PCC moduli are more accurate than backcalculated values and should be used when available. This is not to say that current backcalculation procedures are grossly inadequate, or that accurate values of E cannot be obtained by backcalculation. Probable reasons for the differences between backcalculated and measured PCC moduli values include:

  1. Limitation of current backcalculation procedures (e.g., the assumption that all pavement layers are adequately represented by a two-layer system, the effective confinement on the response of granular base layers, etc.).
  2. Variability of pavement layer thicknesses and properties.
  3. Differences in the nature of the applied load for each test (i.e., quasi-static for ASTM C 469, dynamic with small strains for ASTM C 215, and dynamic with large strains for FWD testing and backcalculation procedures).

There is clearly a need for improved backcalculation techniques that will address the issues mentioned above and produce results that are more consistent with directly measured test results.

- While most previous studies have indicated lower average compressive strengths for RCA concrete, presumably due to the use of weaker composite particles, the opposite trend was observed in the study.[1] In all cases except for the Minnesota 4 project, the average compressive strengths of the cores obtained from the RCA sections was higher than the average strength of cores obtained from the control sections. These results can be attributed to one or both of the following in each case where the RCA concrete was stronger than the control: 1) the RCA concrete mixture used a lower w/c or w/c+p ratio; and 2) the use of about 25 percent RCA fines (as was done in the Kansas and Wyoming projects) has been associated with higher compressive strengths.[17] The different trend in the Minnesota 4 project is probably due, at least in part, to differences in the natural aggregate component of each mixture: the gravel in the RCA pavement had a compressive strength of approximately 40 MPa (5,800 psi) while the dolomite in the control section had a compressive strength exceeding 100 MPa (14,500 psi).

- No clear trend of average split tensile strength between RCA and control sections was observed. This may be due to the lack of test results (often only one per section). The Kansas and Minnesota 1 projects yielded expected results with the recycled concrete strengths being lower than that of their control. The Connecticut, Minnesota 4, and Wyoming projects yielded unexpected results with the recycled concrete strengths being equal or greater than their control concrete strengths.

- Fergus found that the use of limited quantities of RCA fines produced increased concrete tensile strength, with an optimum RCA fines content of about 25 percent.[17] The Kansas and Wyoming projects included 25 percent and 22 percent RCA fines, respectively. However, these projects cannot be used to support or disprove the Fergus study findings because of differences in the aggregate top size (which also affects concrete strength) between the recycled and control sections (i.e., 38 mm vs. 19 mm [1.5 in vs. 0.75 in] for Kansas and 25 mm vs. 38 mm [1.0 in vs. 1.5 in] for Wyoming).

- In all but two of the recycled doweled pavements (Connecticut and Wisconsin 1-2), the VSTR's for the cracks are greater than those at the joints. It is hypothesized that this is because the fracture plane tended to meander more since they were formed later than the joints and had to propagate a greater distance through the slab.

- The lower the VSTR, the tighter the crack must be to maintain aggregate interlock load transfer. The CRCP section in the Wisconsin 2-1 project had a lower VSTR than any other crack, but still maintained the second highest load transfer efficiency. This was attained because there was sufficient steel present to keep the crack tight. It must also be remembered that smaller VSTR's can result in a greater portion of the load being transferred across the crack by the longitudinal steel when the crack is sufficiently wide, possibly causing premature failure in the steel.

- VSTR's obtained with recycled aggregates will equal that of virgin aggregates provided the mortar content (old plus new) is approximately equivalent to the mortar content in the virgin mix. This keeps the total amount of natural coarse aggregate particles available for aggregate interlock equivalent.

- Aggregates used in the Connecticut project have a very high shear strength (approximately 59 MPa [8,500 psi]). As a result, the surface texture was very high even though many of the cracks present were of high and medium severity and the fact that this project had endured more ESAL's than any of the other projects.

- Reducing the nominal top size of the coarse aggregate typically results in lower VSTR's for both recycled and natural aggregates. However, even large aggregate top sizes can produce fracture planes with poor surface textures when the aggregate is weak in tension and fractures as the pavement cracks. Aggregate particles that have low shear strength will also abrade rather easily during load transfer, further reducing the effective aggregate top size and the available surface texture for grain interlock.

Structural Details

- All of the jointed concrete pavements included in this study either did or would have benefited from the inclusion of mechanical load transfer devices at the transverse joints, regardless of traffic level or environment.

- All of the undoweled joints exhibited poor load transfer regardless of the foundation stiffness or surface texture present at the slab face. Rapid loss of serviceability was noted due to the effects of poor load transfer efficiency, even in sections with short slab lengths and no cracking. This is because the computed potential joint openings all exceeded 0.76 mm (0.03 in), which is typically considered the maximum allowable for adequate aggregate interlock load transfer.

- The comparison of joint load transfer and faulting measurements on the Wisconsin 1-2 project (doweled joints) and the Wisconsin 1-1 project (undoweled joints) exemplifies the benefits of using load transfer devices in JPCP. The same benefits of using load transfer devices in JRCP were seen in the Connecticut, Minnesota 1, Minnesota 2, and Minnesota 4 projects.

Unacceptable faulting levels were found only on the Minnesota 3 and Kansas projects, both of which are undoweled. Again, this stresses the need for dowels. There was no apparent correlation between the development of faulting and the type of concrete used (RCA or conventional).

Six epoxy-coated dowel bars were included in the field-drilled cores. These dowels were observed to be corroded in the vicinity of the joint face. In many cases, it appeared that corrosion was sufficient to cause joint lock-up and other pavement distresses. Although the number of cores containing epoxy-coated dowel bars was relatively low, all dowels exhibited corrosion, with some dowels being more severely corroded than others. This phenomenon is beyond the scope of this project and may indicate a need for further study.

- Recycled or conventional JPCP should have panel lengths which are sufficiently short (L/l < 4.0 for stabilized base, 6.0 for granular base) to avoid panel cracking, since no reinforcing steel is available to hold the cracks tight. For example, acceptably low L/l ratios and minimal cracking was observed on the Kansas, Minnesota 3, Wisconsin 1 and Wyoming projects.

- All of the jointed concrete pavements evaluated, except for the Connecticut project, included skewed joints. There was no evidence that the use of skewed joints either improved or degraded performance on these projects.

Pavement Performance

- The Minnesota 4 project was the only project evaluated that displayed significantly more transverse cracking in the RCA concrete section than in the control section (88 percent cracked slabs vs. 22 percent). The undoweled Wisconsin 1-1 project exhibited *slightly more cracking* than the doweled Wisconsin 1-2 section (8 percent cracked slabs vs. 2 percent), and the outer lane of the Connecticut RCA section exhibited *much less cracking* than did the outer lane of the control (66 percent crack slabs vs. 93 percent). The Kansas, Minnesota 1 and Wyoming projects all exhibited little or no cracking in the RCA or control sections. In each case where there was a difference in the observed cracking, the section with the greater amount of cracking had a lower compressive strength and lower backcalculated modulus of subgrade support.

- Although the recycled pavements typically contain higher mortar contents, there was no direct correlation between mortar content and cracking distresses. However, the Minnesota 4 recycled pavement exhibited a significantly higher percentage of slabs cracked when compared to its control pavement (88 percent vs. 22 percent). This wide range of variability might be partly attributed to the recycled pavement exhibiting 83.6 percent mortar content and the control pavement exhibiting only 51.5 percent mortar content. The other projects that have recycled to control comparisons revealed a narrower range of variability between their mortar contents.

- Joint spalling was only present to a significant extent on the Minnesota 3, Minnesota 4, Wisconsin 1 and Connecticut projects. All of these sections also exhibited a large amount of joint sealant damage. There did not appear to be any relationship between spalling and the type of pavement (RCA or conventional).

- Uranyl acetate testing indicated a moderate amount of silica gel in the mortar and around the aggregate particles for the recycled Wyoming pavement section (which was produced from a pavement previously damaged by ASR), and indicated only minor amounts of silica gel in the control section. Although the Wyoming pavements are still fairly young, the possible recurrence of ASR activity in the RCA section is evident. Whether this will eventually develop into widespread distress remains to be seen; only a few localized areas of possible recurrent ASR activity were visible during the condition survey. Therefore, the benefits of the ASR mitigation techniques used in this recycling project (i.e., using low alkali cement [less than 0.6 percent Na_2O], blending RCA with quality virgin aggregates, and using Class F fly ash as a means to lessen the potential of a reoccurrence) will be better measured as this pavement ages, since it is currently only 10 years old.

- Uranyl acetate testing indicated *considerable* amounts of silica gel deposits in the mortar and around the aggregate particles in the Wisconsin 2 recycled concrete pavements. These deposits may indicate the presence of alkali-silica reaction, although ASR distresses were not identified during the condition survey. The pavement is only 10 years old, so it is possible that ASR distresses will begin to appear in the near future.

- The Kansas, Minnesota 2, and Minnesota 3 recycled pavements were similar in that their original pavements exhibited some magnitude of D-cracking. It appears that the introduction of fly ash and the reduction of aggregate top size helped improve the durability of the Minnesota 2 and 3 projects. Additionally, the Kansas project is displaying good durability without fly ash introduction, in spite of the use of a larger top size recycled aggregate gradation than is being used in the control section.

  The Minnesota 3 pavement is currently 15 years old. Freeze-thaw testing of cores retrieved from this pavement indicate that the concrete is not durable. The large entrapped air voids and the microcracks found in the old mortar are two factors which appear to contribute to the poor durability of the RCA concrete. This may mean that the pavement could begin to deteriorate substantially in the near future. It is also possible that D-cracking will never cause any substantial problems to the performance of this pavement if the concrete is not often critically saturated in the field.

It is important to note that the survey records did not indicate a D-cracking reoccurrence in any of these projects. Again, due to the young age of these pavements it is not certain whether or not reoccurrence will eventually emerge.

- FHWA's recently released technical advisory T 5080.17, Portland Cement Concrete Mix Design and Field Control, recommends a minimum cement content of 342 kg/m^3 (574 lb/yd^3) for durability.[29] The Connecticut, Kansas, and Wyoming 1-2 pavement sections all exceeded this minimum cement content. When the included fly ash is considered to contribute toward the cementitious content, the Minnesota 2, Wisconsin 2, and Wyoming 1-1 pavement sections also meet this criterion. The Minnesota 1-1, Minnesota 3, and Minnesota 4 pavement sections did not contain the recommended amounts of cement or cementitious material. In spite of the fact that three of these sections did not conform to the recommendation of the technical advisory, there was no visible evidence of freeze-thaw damage on any of the field sections included in this study (although the cores retrieved from the MN 3 project performed poorly in laboratory freeze-thaw testing). In addition, petrographic examinations of project cores did not reveal any incipient cracks or other characteristics that would indicate poor frost resistance. As a result, it appears that project compliance with the recommended minimum cement content of 342 kg/m^3 (574 lb/yd^3) was not an issue in this study.

- The Connecticut and Kansas recycled pavements contained an aggregate top size that was larger than that of their corresponding control pavements.

  This was especially odd for the Kansas project since the original pavement had encountered D-cracking; Kansas DOT tests of the recycled aggregates indicated that the larger top size would be as frost resistant as the material used in the control section (durability factor = 99 and dilation = 0.005 for the recycled material versus 98 and 0.008 for the control). In contrast, the Wyoming recycled pavement (originally suffering from ASR distresses) contained an aggregate top size that was smaller than that of its corresponding control pavement. A recycled concrete, like that of the Wyoming project, will typically have a lower aggregate top size due to the reclamation process and desired upgrade of aggregate durability.

Other Conclusions and Recommendations

- As noted previously, there were very few cases where values of PCC modulus of elasticity obtained through backcalculation using FWD data compared favorably with those measured directly in the laboratory using field-drilled cores. In addition, it was often observed that the backcalculated values of PCC moduli (E) and foundation stiffness (k-value) varied inversely among the field study sites;

this relationship was fairly consistent and appears to be a product of the backcalculation process.

The scope and goals of this project did not include an investigation of the accuracy of techniques and algorithms for backcalculating the properties of concrete pavement layers. However, the results of the tests performed under this study suggest that such an investigation is needed.

In conclusion, the field and laboratory studies associated with this interim report have provided results that were somewhat representative of the literature review contained within the Task A Interim Report. A separate document is being prepared to synthesize the results of the work performed under Tasks A and B (Literature Review and Evaluation of Field Study Sites), hypotheses concerning causative relationships between RCA concrete properties and pavement performance and propose a work plan for laboratory testing to validate these relationships.

APPENDIX A. SUMMARY OF FIELD AND LABORATORY DATA

A complete summary of all data elements collected for each section can be found within appendix A. The following tables are located in this appendix:

1. Table 72. General and climatic data.
2. Table 73. Structural design data.
3. Table 74. Joint design data.
4. Table 75. Reinforcement design and construction data.
5. Table 76. Outer shoulder and drainage design data.
6. Table 77. Aggregate data.
7. Table 78. Aggregate gradation data.
8. Table 79. PCC mixture design data.
9. Table 80. PCC strength data (from construction records).
10. Table 81. Traffic data.
11. Table 82. Deflection data.
12. Table 83. Laboratory testing results.
13. Table 84. Primary performance data.
14. Table 85. Secondary performance data.

Table 72. General and climatic data.

Project Sec. ID	Const. Date	Pvt. Type	No. of Lanes	Milepost Location	Station	Section Length, m	Testing Date	Climatic Region	Moisture Index	Freezing Index	No. of FT Cycles/yr	Max. Ave., °C Monthly Temp.	Min. Ave., °C Monthly Temp.	No. of Days Precip/yr	Ann. Ave. Precip, mm
WB I-84 Waterbury, CT (Good)															
CT 1-1	1980	JRCP	3	33.71	7.00	299	10/18/94	WF	70	140	90	22	-2	138	1190
CT 1-2	1980	JRCP	3	33.94	0.00	183	10/18/94	WF	70	140	90	22	-2	138	1190
NB K-7 Johnson County, KS (Good)															
KS 1-1	1985	JPCP	2		184.90	317	11/9/94	WF	22	56	80	27	-2	100	860
SB K-7 Johnson County, KS (Good)															
KS 1-2	1985	JPCP	2		255.07	458	11/9/94	WF	22	56	80	27	-2	100	860
WB I-94 Brandon, MN (Good)															
MN 1-1	1988	JRCP	2	90.90	2924.35	313	9/14/94	DF	5	1170	105	22	-12	106	610
MN 1-2	1988	JRCP	2	87.00	2760.88	313	9/14/94	DF	5	1170	105	22	-12	106	610
EB I-90 Beaver Creek, MN (Cracked)															
MN 2-1	1984	JRCP	2	1.70	90.00	313	9/8/94	DF	5	720	96	23	-9	100	610
WB I-90 Beaver Creek, MN (Cracked)															
MN 2-2	1984	JRCP	2	1.90	100.05	313	9/8/94	DF	5	720	96	23	-9	100	610
SB US 59 Worthington, MN (Other)															
MN 3-1	1980	JPCP	1	27.00	859.94	306	9/7/94	DF	8	830	92	23	-10	102	640
NB US 52 Zumbrota, MN (Cracked)															
MN 4-1	1984	JRCP	2		983.88	305	9/12/94	WF	20	720	95	23	-11	110	740
MN 4-2	1984	JRCP	2		1035.01	304	9/12/94	WF	20	720	95	23	-11	110	740
EB I-94 Menomonie, WI (Cracked)															
WI 1-1	1984	JPCP	2	39.60	849.70	306	10/27/94	WF	30	1050	102	22	-10	115	760
WI 1-2	1984	JPCP	2	40.10		310	10/26/94	WF	30	1050	102	22	-10	115	760
WB I-90 Beloit, WI (Other)															
WI 2-1	1986	CRCP	2	176.80	313.65	305	11/17/94	WF	25	430	90	23	-6	118	790
WI 2-2	1986	CRCP	2	176.20	281.99	305	11/17/94	WF	25	430	90	23	-6	118	790
EB I-80 Pine Bluffs, WY (Other)															
WY 1-1	1985	JPCP	2	400.02		307	9/27/94	DF	-10	270	140	21	-3	90	360
WY1-2	1984	JPCP	2	400.20		305	9/27/94	DF	-10	270	140	21	-3	90	360

Table 73. Structural design data.

Project Sec. ID	Const. Date	Pvt. Type	Joint Spacing, m	Section Length, m	PCC Surface: Driving Lane Width, m	PCC Surface: Slab Thickness, mm – Design	PCC Surface: Slab Thickness, mm – Core	PCC Surface: Percent Steel	PCC Surface: E dyn., GPa	Base: Type	Base: Percent Stabilizer	Base: Design Thick, mm	Base: k eff., kPa/mm	Subbase: Type	Subbase: Design Thick, mm	Subgrade: AASHTO Soil Type	Edge Support
WB I-84 Waterbury, CT (Good)																	
CT 1-1	1980	JRCP	12	299	3.7	230	230	0.09	37.0	AGG	n/a	250	105.1	none	n/a	n/a	none
CT 1-2	1980	JRCP	12	183	3.7	230	230	0.09	44.9	AGG	n/a	460	68.4	none	n/a		none
NB K-7 Johnson County, KS (Good)																	
KS 1-1	1985	JPCP	4.7	317	3.7	230	240	n/a	38.6	CTB	6.0	100	67.6	Lime-stab	150		none
SB K-7 Johnson County, KS (Good)																	
KS 1-2	1985	JPCP	4.7	458	3.7	230	250	n/a	40.6	CTB	6.0	100	69.0	Lime-stab	150		none
WB I-94 Brandon, MN (Good)																	
MN 1-1	1988	JRCP	8.2	313	4.3	280	290	0.06	42.1	AGG	n/a	150	36.7	Stab Sub	150-760	A-6	0.6 m widened
MN 1-2	1988	JRCP	8.2	313	4.3	280	280	0.06	52.2	AGG	n/a	150	36.7	Stab Sub	150-760	A-6	0.6 m widened
EB I-90 Beaver Creek, MN (Cracked)																	
MN 2-1	1984	JRCP	8.2	313	4.3	230	230	0.06	47.7	AGG	n/a	80	34.2	AGG	150	A-1-a	0.6 m widened
WB I-90 Beaver Creek, MN (Cracked)																	
MN 2-2	1984	JRCP	8.2	313	4.3	230	n/a	0.06	n/a	AGG	n/a	80	n/a	AGG	150	A-1-a	0.6 m widened
SB US 59 Worthington, MN (Other)																	
MN 3-1	1980	JPCP	4.0-4.9-4.3-5.8	306	3.7	200	200	n/a	62.3	Stab AGG	n/a	30	28.5	AGG	150	A-1-a	none
NB US 52 Zumbrota, MN (Cracked)																	
MN 4-1	1984	JRCP	8.2	305	3.7	230	230	0.06	30.3	AGG	n/a	130	24.4	Gran	1070	A-7-5	none
MN 4-2	1984	JRCP	8.2	304	3.7	230	230	0.06	44.6	AGG	n/a	130	33.1	Gran	1070	A-7-5	none
EB I-94 Menomonie, WI (Cracked)																	
WI 1-1	1984	JPCP	3.7-4.0-5.8-5.5	306	3.7	280	280	n/a	46.3	AGG	n/a	150	36.4	Gran	230		Tied PCC shoulder
WI 1-2	1984	JPCP	3.7-4.0-5.8-5.5	310	3.7	280	280	n/a	29.0	AGG	n/a	150	45.6	Gran	230		Tied PCC shoulder
WB I-90 Beloit, WI (Other)																	
WI 2-1	1986	CRCP	n/a	305	3.7	250	250	0.67	40.3	AGG	n/a	150	95.0	Gran	230		Tied PCC shoulder
WI 2-2	1986	CRCP	n/a	305	3.7	250	250	0.67	40.9	AGG	n/a	150	104.0	Gran	230		Tied PCC shoulder
EB I-80 Pine Bluffs, WY (Other)																	
WY 1-1	1985	JPCP	4.3-4.9-4.0-3.7	307	3.7	250	250	n/a	32.1	AGG	n/a	100	52.7	none	n/a		Tied PCC shoulder
WY 1-2	1984	JPCP	4.3-4.9-4.0-3.7	305	3.7	250	270	n/a	50.5	AGG	n/a	100	42.9	none	n/a		Tied PCC shoulder

Table 74. Joint design data.

		PCC Slab		Transverse Joint						Centerline Joint						Lane Shoulder Joint					
Project	Pvt.	Thickness,	Percent	Joint	Joint	Dowel			Sealant	Joint	Depth,	Tie Bar				Joint	Depth,	Tie Bar			
Sec. ID	Type	mm	Steel	Spacing, m	Skew	Dia.,mm	Len.,mm	Coating	Type	Type	mm	Dia.,mm	Len.,mm	Spc.,m	Coating	Type	mm	Dia.,mm	Len.,mm	Coating	Spc.,m
WB I-84 Waterbury, CT (Good)																					
CT 1-1	JRCP	230	0.09	12	n	38*	410	none	Hot Pour	Butt Joint	100	14	180	1.5	none	n/a	n/a	n/a	n/a	n/a	n/a
CT 1-2	JRCP	230	0.09	12	n	38*	410	none	Hot Pour	Butt Joint	100	14	180	1.5	none	n/a	n/a	n/a	n/a	n/a	n/a
NB K-7 Johnson County, KS (Good)																					
KS 1-1	JPCP	230	n/a	4.7	y	none	n/a	n/a	Silicone	Plastic Insert	110	13	610	0.8	Epoxy	n/a	n/a	n/a	n/a	n/a	n/a
SB K-7 Johnson County, KS (Good)																					
KS 1-2	JPCP	230	n/a	4.7	y	none	n/a	n/a	Silicone	Plastic Insert	110	13	610	0.8	Epoxy	n/a	n/a	n/a	n/a	n/a	n/a
WB I-94 Brandon, MN (Good)																					
MN 1-1	JRCP	280	0.06	8.2	y	32	380	Epoxy	Preform	Saw Cut	70-110	16	910	0.9	Epoxy	n/a	n/a	n/a	n/a	n/a	n/a
MN 1-2	JRCP	280	0.06	8.2	y	32	380	Epoxy	Preform	Saw Cut	70-110	16	910	0.9	Epoxy	n/a	n/a	n/a	n/a	n/a	n/a
EB I-90 Beaver Creek, MN (Cracked)																					
MN 2-1	JRCP	230	0.06	8.2	y	25	380	Epoxy	Preform	Saw Cut	70-90	13	760	0.8	Epoxy	n/a	n/a	n/a	n/a	n/a	n/a
WB I-90 Beaver Creek, MN (Cracked)																					
MN 2-2	JRCP	230	0.06	8.2	y	25	380	Epoxy	Preform	Saw Cut	70-90	13	760	0.8	Epoxy	n/a	n/a	n/a	n/a	n/a	n/a
SB US 59 Worthington, MN (Other)																					
MN 3-1	JPCP	200	n/a	4.0-4.9-4.3-5.8	y	none	n/a	n/a	Silicone	Saw Cut	60-80	16	760	0.8	Epoxy	n/a	n/a	n/a	n/a	n/a	n/a
NB US 52 Zumbrota, MN (Cracked)																					
MN 4-1	JRCP	230	0.06	8.2	y	25	380	Epoxy	Preform	Saw Cut	70-90	13	760	0.8	Epoxy	n/a	n/a	n/a	n/a	n/a	n/a
MN 4-2	JRCP	230	0.06	8.2	y	25	380	Epoxy	Preform	Saw Cut	70-90	13	760	0.8	Epoxy	n/a	n/a	n/a	n/a	n/a	n/a
EB I-94 Menomonie, WI (Cracked)																					
WI 1-1	JPCP	280	n/a	3.7-4.0-5.8-5.5	y	none	n/a	n/a	Preform	Saw Cut	140	13	610	1.2	Epoxy		140	13	610	Epoxy	
WI 1-2	JPCP	280	n/a	3.7-4.0-5.8-5.5	y	35	460	Epoxy	Preform	Saw Cut	110	13	610	1.2	Epoxy		140	13	610	Epoxy	
WB I-90 Beloit, WI (Other)																					
WI 2-1	CRCP	250	0.67	n/a	n/a	n/a	n/a	n/a	n/a	Tape	90	13	610	1.2	Epoxy	Tape	90	13	610	Epoxy	1.2
WI 2-2	CRCP	250	0.67	n/a	n/a	n/a	n/a	n/a	n/a	Tape	90	13	610	1.2	Epoxy	Tape	90	13	610	Epoxy	1.2
EB I-80 Pine Bluffs, WY (Other).																					
WY 1-1	JPCP	250	n/a	4.3-4.9-4.0-3.7	y	none	n/a	n/a	Silicone	Saw Cut	125	13	610	0.6		Saw Cut	125	13	610		1
WY 1-2	JPCP	250	n/a	4.3-4.9-4.0-3.7	y	none	n/a	n/a	Silicone	Saw Cut	125	13	610	0.6		Saw Cut	125	13	610		1

*I-beam load transfer devices.

Table 75. Reinforcement design and construction data.

Project Sec. ID	Pvt. Type	Slab T, mm	Joint Spacing, m	Transverse Reinforcing Type[1]	Transverse Reinforcing Type[2]	Transverse Reinforcing Bar Dia., mm	Transverse Reinforcing Spacing, mm	Longitudinal Reinforcing Type[1]	Longitudinal Reinforcing Type[2]	Longitudinal Reinforcing Bar Dia., mm	Longitudinal Reinforcing Spacing, mm	% Long. Steel	Steel Coating	Depth to Steel, mm from surface	Air Temperature at Placement, °C Min.	Max.	Ave.	Curing Period, Days	Time to Sawing, hrs	Curing Method[3]	PCC Texture Method[4]
WB I-84 Waterbury, CT (Good)																					
CT 1-1	JRCP	230	12	WWF	S	13	310	WWF	S	6	150	0.09	none	60	21	24	22	3-14	15-20	1	2
CT 1-2	JRCP	230	12	WWF	S	13	310	WWF	S	6	150	0.09	none	60	21	24	22	3-11	15-20	1	2
NB K-7 Johnson County, KS (Good)																					
KS 1-1	JPCP	230	4.7	none	n/a	n/a	n/a	none	n/a	n/a	n/a	n/a	n/a	n/a	12	24	18	4	6-8	1	1
SB K-7 Johnson County, KS (Good)																					
KS 1-2	JPCP	230	4.7	none	n/a	n/a	n/a	none	n/a	n/a	n/a	n/a	n/a	n/a	12	24	18	4	6-8	1	1
WB I-94 Brandon, MN (Good)																					
MN 1-1	JRCP	280	8.2	WWF	D	6	310	WWF	D	8	310	0.054	none	70-90				12		1	1,5
MN 1-2	JRCP	280	8.2	WWF	D	6	310	WWF	D	8	310	0.054	none	70-90				12		1	1,5
EB I-90 Beaver Creek, MN (Cracked)																					
MN 2-1	JRCP	230	8.2	WWF	D	6	310	WWF	D	8	310	0.065	none	70-90				12		1	1,5
WB I-90 Beaver Creek, MN (Cracked)																					
MN 2-2	JRCP	230	8.2	WWF	D	6	310	WWF	D	8	310	0.065	none	70-90				12		1	1,5
SB US 59 Worthington, MN (Other)																					
MN 3-1	JPCP	200	4.0-4.9-4.3-5.8	none	n/a	n/a	n/a	none	n/a	n/a	n/a	n/a	n/a	n/a				12	<24	1	1,5
NB US 52 Zumbrota, MN (Cracked)																					
MN 4-1	JRCP	230	8.2	WWF	D	6	310	WWF	D	8	310	0.065	Epoxy	70-90				12		1	1,5
MN 4-2	JRCP	230	8.2	WWF	D	6	310	WWF	D	8	310	0.065	Epoxy	70-90				12		1	1,5
EB I-94 Menomonie, WI (Cracked)																					
WI 1-1	JPCP	280	3.7-4.0-5.8-5.5	none	n/a	n/a	n/a	none	n/a	n/a	n/a	n/a	n/a	n/a						1	1
WI 1-2	JPCP	280	3.7-4.0-5.8-5.5	none	n/a	n/a	n/a	none	n/a	n/a	n/a	n/a	n/a	n/a						1	1
WB I-90 Beloit, WI (Other)																					
WI 2-1	CRCP	250	n/a	DB				DB				0.67								1	1
WI 2-2	CRCP	250	n/a	DB				DB				0.67								1	1
EB I-80 Pine Bluffs, WY (Other)																					
WY 1-1	JPCP	250	4.3-4.9-4.0-3.7	none	n/a	n/a	n/a	none	n/a	n/a	n/a	n/a	n/a	n/a					< 8	1	1,5
WY 1-2	JPCP	250	4.3-4.9-4.0-3.7	none	n/a	n/a	n/a	none	n/a	n/a	n/a	n/a	n/a	n/a			18		< 8	1	1,5

[1]DB = deformed bars; WWF = welded wire fabric.

[2]S = smooth WWF; D = deformed WWF.

[3]1=curing compound, 2=burlap, 3=waterproof paper, 4=ployethylene sheeting, 5=burlap-ployethylene blanket, 6=cotton mat, 7=hay.

[4]1=tine, 2=broom, 3=burlap drag, 4=grooved float, 5=astroturf.

Table 76. Outer shoulder and drainage design data.

Project	Pvt.	Base		Subbase		AASHTO	Outer Shoulder				Drainage		Cut and Fill of Sample Unit		
							Surface		Base		Drainage	Depth to	Percent		
Sec. ID	Type	Type	T, mm	Type	T, mm	Soil Type	Type	T, mm	Type	T, mm	Type	Ditch, m	Cut	Fill	@Grade
WB I-84 Waterbury, CT (Good)															
CT 1-1	JRCP	AGG	250	none	n/a		AC	100	AGG	460	None	n/a	8%	0%	92%
CT 1-2	JRCP	AGG	460	none	n/a		AC	100	AGG	460	None	n/a	0%	100%	0%
NB K-7 Johnson County, KS (Good)															
KS 1-1	JPCP	CTB	100	Lime-stab	150		AC	150-230	Lime-stab	150	None	7.6	0%	100%	0%
SB K-7 Johnson County, KS (Good)															
KS 1-2	JPCP	CTB	100	Lime-stab	150		AC	150-230	Lime-stab	150	None	7.6	0%	100%	0%
WB I-94 Brandon, MN (Good)															
MN 1-1	JRCP	AGG	150	Stab Sub	150-760	A-6	AC	150	AGG	Varies	Edge	1.2	46%	0%	54%
MN 1-2	JRCP	AGG	150	Stab Sub	150-760	A-6	AC	150	AGG	Varies	Edge	0.9	23%	0%	77%
EB I-90 Beaver Creek, MN (Cracked)															
MN 2-1	JRCP	AGG	80	AGG	150	A-1-a	AC	50	Stab AGG	Varies	Edge	1.2	100%	0%	0%
WB I-90 Beaver Creek, MN (Cracked)															
MN 2-2	JRCP	AGG	80	AGG	150	A-1-a	AC	50	Stab AGG	Varies	Edge	1.2	100%	0%	0%
SB US 59 Worthington, MN (Other)															
MN 3-1	JPCP	Stab AGG	30	AGG	150	A-1-a	AC	50	AGG	80	Edge	1.2	0%	0%	100%
NB US 52 Zumbrota, MN (Cracked)															
MN 4-1	JRCP	AGG	130	Gran	1070	A-7-5	AC	50	AGG	Varies	Edge	1.2	0%	20%	80%
MN 4-2	JRCP	AGG	130	Gran	1070	A-7-5	AC	50	AGG	Varies	Edge	1.2	0%	0%	100%
EB I-94 Menomonie, WI (Cracked)															
WI 1-1	JPCP	AGG	150	Gran	230		PCC	150	AGG	280	None	0.9	0%	0%	100%
WI 1-2	JPCP	AGG	150	Gran	230		PCC	150	AGG	280	None	0.9	0%	0%	100%
WB I-90 Beloit, WI (Other)															
WI 2-1	CRCP	AGG	150	Gran	230		PCC	150	AGG	250	None	1.8	0%	0%	100%
WI 2-2	CRCP	AGG	150	Gran	230		PCC	150	AGG	250	None	1.2	50%	0%	50%
EB I-80 Pine Bluffs, WY (Other)															
WY 1-1	JPCP	AGG	100	none	n/a		PCC	250	AGG	100	None	1.2	0%	0%	100%
WY 1-2	JPCP	AGG	100	none	n/a		PCC	250	AGG	100	None	1.2	0%	0%	100%

Table 77. Aggregate data.

	Aggregate Composition										Amt. of Virgin Agg. Added, %		Insoluble Residue, %	Bulk Specific Gravity		Type of Crusher	Producer	Age of PCC at Recycling
	Coarse Aggregate, %[1]								Fine Agg., %[2]									
	LS	G	CD	CTR	RLS	RG	RTR	RDL	NS	RCS	Coarse	Fine		Coarse	Fine			
WB I-84 Waterbury, CT (Good)																		
CT 1-1	0	0	0	0	0	0	100	0	100	0	0	100		2.53	2.65	gyratory	O & G	23
CT 1-2	0	0	0	100	0	0	0	0	100	0	100	100		2.81	2.65		O & G	n/a
NB K-7 Johnson County, KS (Good)																		
KS 1-1	0	0	0	0	100	0	0	0	75	25	0	75		2.38	2.60		Martin Marietta	25
SB K-7 Johnson County, KS (Good)																		
KS 1-2	100	0	0	0	0	0	0	0	100	0	100	100	3.81	2.60	2.60		J.A. Tobin	n/a
WB I-94 Brandon, MN (Good)																		
MN 1-1	0	0	0	0	0	100	0	0	100	0	0	100						28
MN 1-2	0	0	100	0	0	0	0	0	100	0	100	100						n/a
EB I-90 Beaver Creek, MN (Cracked)																		
MN 2-1	0	0	0	0	0	100	0	0	100	0	0	100		2.44	2.62			20
WB I-90 Beaver Creek, MN (Cracked)																		
MN 2-2	0	0	0	0	0	100	0	0	100	0	0	100		2.44	2.62			n/a
SB US 59 Worthington, MN (Other)																		
MN 3-1	0	0	0	0	0	100	0	0	100	0	0	100		2.41	2.62	jaw		25
NB US 52 Zumbrota, MN (Cracked)																		
MN 4-1	0	0	0	0	0	100	0	0	100	0	0	100		2.42	2.63			53
MN 4-2	0	100	0	0	0	0	0	0	100	0	100	100		2.68	2.63			n/a
EB I-94 Menomonie, WI (Cracked)																		
WI 1-1	0	0	0	0	0	100	0	0	100	0	0	100						25
WI 1-2	0	0	0	0	0	100	0	0	100	0	0	100						25
WB I-90 Beloit, WI (Other)																		
WI 2-1	0	0	0	0	0	100	0	0	100	0	0	100						28
WI 2-2	0	0	0	0	0	100	0	0	100	0	0	100						28
EB I-80 Pine Bluffs, WY (Other)																		
WY 1-1	0	35	0	0	0	0	0	65	78	22	35	78		2.45	2.36	jaw		20
WY 1-2	0	100	0	0	0	0	0	0	100	0	100	100		2.65	2.61			n/a

[1]LS=limestone, G=gravel, CD=crushed diorite, CTR=crushed trap rock, RLS=recycled limestone, RG=recycled gravel, RTR=recycled trap rock, RDL=recycled dolomiti

[2]NS=natural sand, RCS=recycled PCC sand

Table 78. Aggregate gradation data.

	Coarse Aggregate, % Passing								Fine Aggregate, % Passing						
	51 mm	38 mm	25 mm	22 mm	19 mm	16 mm	13 mm	9.5 mm	4.75 mm	2.36 mm	1.18 mm	0.60 mm	0.30 mm	0.15 mm	0.075 mm
WB I-84 Waterbury, CT (Good)															
CT 1-1	100	98	86		66		37	25	100	93	75	51	11	4	0.8
CT 1-2	100	100	80		55		48	16	100	93	75	51	11	4	0.8
NB K-7 Johnson County, KS (Good)															
KS 1-1	100	100	81		62		42	30	98	91	75	47	12	2	1.0
SB K-7 Johnson County, KS (Good)															
KS 1-2	100	100	100	100	100		74	41	98	97	69	32	9	2	0.0
WB I-94 Brandon, MN (Good)															
MN 1-1					100										
MN 1-2															
EB I-90 Beaver Creek, MN (Cracked)															
MN 2-1															
WB I-90 Beaver Creek, MN (Cracked)															
MN 2-2															
SB US 59 Worthington, MN (Other)															
MN 3-1	100	100	100	100	100										
NB US 52 Zumbrota, MN (Cracked)															
MN 4-1	100	100	100		99				99	84	66	44	17	2	
MN 4-2	100	100			46			15	99	86	66	44	16	2	
EB I-94 Menomonie, WI (Cracked)															
WI 1-1															
WI 1-2															
WB I-90 Beloit, WI (Other)															
WI 2-1															
WI 2-2															
EB I-80 Pine Bluffs, WY (Other)															
WY 1-1	100	100	100		71		35	19	100	86	63	38	17	8	2.9
WY 1-2	100	100	98		73		34	18	95	75	57	35	13	4	1.5

Table 79. PCC mixture design data.

	Coarse Agg., kg/m³	Fine Agg., kg/m³	Cement, kg/m³	Flyash, kg/m³	Water, kg/m³	w/c Ratio	w/c+p Ratio	Cement Type	Flyash Type	Flyash Source	Alkali Content of Cement, %	Admixture 1		Avg. Air Content, %	Average Slump, mm
												Type	Amount		
WB I-84 Waterbury, CT (Good)															
CT 1-1	1,302	476	362	0	144	0.40	0.40	I	n/a	n/a		AEA		5.0	76
CT 1-2	1,225	641	362	0	163	0.45	0.45	I	n/a	n/a		AEA		4.0	64
NB K-7 Johnson County, KS (Good)															
KS 1-1	848	848	357	0	147	0.41	0.41	II	n/a	n/a	0.47	AEA		6.2*	38
SB K-7 Johnson County, KS (Good)															
KS 1-2	884	884	357	0	147	0.41	0.41	II	n/a	n/a	0.47	AEA		6.2*	64
WB I-94 Brandon, MN (Good)															
MN 1-1	976	712	288	51	160	0.56	0.47	I	C			AEA		5.5	
MN 1-2								I	C			AEA			
EB I-90 Beaver Creek, MN (Cracked)															
MN 2-1	979	701	282	66	160	0.57	0.46	I	C			AEA		5.5	38
WB I-90 Beaver Creek, MN (Cracked)															
MN 2-2	979	701	282	66	160	0.57	0.46	I	C			AEA		5.5	38
SB US 59 Worthington, MN (Other)															
MN 3-1	981	710	276	65	151	0.55	0.44	I	C			AEA		5.5	38
NB US 52 Zumbrota, MN (Cracked)															
MN 4-1	983	713	276	65	151	0.55	0.44	I	C			Protex	4 oz	5.5	38
MN 4-2	1,166	653	278	49	153	0.55	0.47	I	C			Protex	4 oz	5.5	
EB I-94 Menomonie, WI (Cracked)															
WI 1-1								I							
WI 1-2								I							
WB I-90 Beloit, WI (Other)															
WI 2-1			285	65				I	C						
WI 2-2			285	65				I	C						
EB I-80 Pine Bluffs, WY (Other)															
WY 1-1	1,026	685	290	79	141	0.49	0.38	II	F			Protex N	5 oz	5.5	32
WY 1-2	1,108	686	349	0	153	0.44	0.44	II	n/a	n/a				5.5	44

*"Roll-O-Meter" was used to measure air content.

Table 80. PCC strength data (from construction records).

	Flexural Strength							Compressive Strength (ASTM C39)						Splitting Tensile Strength (ASTM C496)						Elastic Modulus						
	Test Type[1]	Age, Days	Strength, MPa					Age, Days	Strength, MPa					Age, Days	Strength, MPa					Test Method[2]	Age, Days	Modulus, MPa				
			Min.	Max.	Ave.	n	Std. Dev.		Min.	Max.	Ave.	n	Std. Dev.		Min.	Max.	Ave.	n	Std. Dev.			Min.	Max.	Ave.	n	Std. Dev.
WB I-84 Waterbury, CT (Good)																										
CT 1-1		7	3.3	3.5	3.4	2	0.146																			
CT 1-2		7	2.9	3.2	3.0	2	0.195																			
NB K-7 Johnson County, KS (Good)																										
KS 1-1		6			3.9																					
SB K-7 Johnson County, KS (Good)																										
KS 1-2		6			4.2																					
WB I-94 Brandon, MN (Good)																										
MN 1-1																										
MN 1-2																										
EB I-90 Beaver Creek, MN (Cracked)																										
MN 2-1																										
WB I-90 Beaver Creek, MN (Cracked)																										
MN 2-2																										
SB US 59 Worthington, MN (Other)																										
MN 3-1	2	14			4.5			60			31.6															
NB US 52 Zumbrota, MN (Cracked)																										
MN 4-1																										
MN 4-2																										
EB I-94 Menomonie, WI (Cracked)																										
WI 1-1																										
WI 1-2																										
WB I-90 Beloit, WI (Other)																										
WI 2-1																										
WI 2-2																										
EB I-80 Pine Bluffs, WY (Other)																										
WY 1-1		28			4.8																					
WY 1-2		28			6.1			28			29.8															

[1] 1=third-point (ASTM C78); 2=center-point (ASTM C293). [2] 1=test on cores (ASTM C469); 2= test on cylinders; 3=ACI correlation.

Table 81. Traffic data.

Project Sec. ID	Const. Date	Survey Date	Opening Year Traffic: 2-Way ADT, vehicles/day	Opening Year Traffic: Percent Trucks	Opening Year Traffic: ESAL's, thousands	1994 Traffic: 2-Way ADT, vehicles/day	1994 Traffic: Percent Trucks	1994 Traffic: ESAL's, thousands	Cumulative Outer Lane ESAL's Thru 1994, millions
WB I-84 Waterbury, CT (Good)									
CT 1-1	1980	1994	56,000	10.1	728	75,000	10.3	1,258	15.9
CT 1-2	1980	1994	56,000	10.1	728	75,000	10.3	1,258	15.9
NB K-7 Johnson County, KS (Good)									
KS 1-1	1985	1994	7,310	11.0	188	12,095	7.0	189	2.2
SB K-7 Johnson County, KS (Good)									
KS 1-2	1985	1994	7,310	11.0	188	12,095	7.0	189	2.2
WB I-94 Brandon, MN (Good)									
MN 1-1	1988	1994	8,170	32.0	462	9,475	32.0	595	3.7
MN 1-2	1988	1994	8,170	32.0	462	9,475	32.0	595	3.7
EB I-90 Beaver Creek, MN (Cracked)									
MN 2-1	1984	1994	16,780	22.0	573	21,480	22.0	872	7.8
WB I-90 Beaver Creek, MN (Cracked)									
MN 2-2	1984	1994	16,780	22.0	573	21,480	22.0	872	7.8
SB US 59 Worthington, MN (Other)									
MN 3-1	1980	1994	2,150	12.0	51	2,471	12.0	78	0.9
NB US 52 Zumbrota, MN (Cracked)									
MN 4-1	1984	1994	7,820	15.0	235	10,010	15.0	359	3.2
MN 4-2	1984	1994	7,820	15.0	235	10,010	15.0	359	3.2
EB I-94 Menomonie, WI (Cracked)									
WI 1-1	1984	1994	12,439	20.0	553	16,717	20.0	721	7.0
WI 1-2	1984	1994	12,439	20.0	553	16,717	20.0	721	7.0
WB I-90 Beloit, WI (Other)									
WI 2-1	1986	1994	22,622	20.0	789	28,657	20.0	975	7.9
WI 2-2	1986	1994	22,622	20.0	789	28,657	20.0	975	7.9
EB I-80 Pine Bluffs, WY (Other)									
WY 1-1	1985	1994	4,410	34.6	259	6,720	43.5	478	3.6
WY 1-2	1984	1994	4,280	34.6	252	6,720	43.5	478	3.8

Table 82. Deflection data.

Project Sec. ID	Const. Date	Pvt. Type	Joint Spacing, m	Dowel Dia, mm	Slab T, mm	Base Type	Midslab Deflection, µm High	Low	Ave.	Load Transfer at Joint, % App«	Leave»	Ave.	Load Transfer at Crack, % App«	Leave»	Ave.	Edge Deflection, µm High	Low	Ave.	Shoulder LTE, %	Test Temp (air), °C
WB I-84 Waterbury, CT (Good)																				
CT 1-1	1980	JRCP	12	38*	230	AGG	99	59	82	90.3	89.9	90.1	74.3	77.4	75.9	188	106	148	n/a	20
CT 1-2	1980	JRCP	12	38*	230	AGG	110	74	89	85.3	86.2	85.8	85.2	83.7	84.5	134	94	114	n/a	23
NB K-7 Johnson County, KS (Good)																				
KS 1-1	1985	JPCP	4.7	none	230	CTB	81	63	74	21.1	38.8	30.0	n/a	n/a	n/a	200	126	143	n/a	12
SB K-7 Johnson County, KS (Good)																				
KS 1-2	1985	JPCP	4.7	none	230	CTB	72	64	69	33.4	40.4	36.9	n/a	n/a	n/a	148	87	109	n/a	11
WB I-94 Brandon, MN (Good)																				
MN 1-1	1988	JRCP	8.2	32	280	AGG	91	81	87	89.9	92.2	91.1	67.5	82.8	75.2	174	113	142	n/a	23
MN 1-2	1988	JRCP	8.2	32	280	AGG	88	82	85	87.7	93.5	90.6	n/a	n/a	n/a	127	96	107	n/a	27
EB I-90 Beaver Creek, MN (Cracked)																				
MN 2-1	1984	JRCP	8.2	25	230	AGG	164	109	131	80.4	78.8	79.6	56.7	77.2	67.0	173	98	128	n/a	22
WB I-90 Beaver Creek, MN (Cracked)																				
MN 2-2	1984	JRCP	8.2	25	230	AGG	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
SB US 59 Worthington, MN (Other)																				
MN 3-1	1980	JPCP	4.0-4.9-4.3-5.8	none	200	Stab AGG	154	127	142	28.1	46.2	37.2	n/a	n/a	n/a	342	255	303	n/a	20
NB US 52 Zumbrota, MN (Cracked)																				
MN 4-1	1984	JRCP	8.2	25	230	AGG	200	160	186	78.0	77.0	77.5	71.4	76.8	74.1	299	194	237	n/a	28
MN 4-2	1984	JRCP	8.2	25	230	AGG	176	106	138	87.2	84.8	86.0	93.7	94.7	94.2	218	143	185	n/a	33
EB I-94 Menomonie, WI (Cracked)																				
W I 1-1	1984	JPCP	3.7-4.0-5.8-5.5	none	280	AGG	107	81	96	32.5	30.8	31.7	52.6	43.2	47.9	148	102	116	93.7	16
WI 1-2	1984	JPCP	3.7-4.0-5.8-5.5	35	280	AGG	123	85	105	74.6	74.1	74.4	57.0	60.7	58.9	128	113	120	98.1	16
WB I-90 Beloit, WI (Other)																				
WI 2-1	1986	CRCP	n/a	n/a	250	AGG	73	65	70	n/a	n/a	n/a	93.5	92.5	93.0	165	123	136	55.9	7
WI 2-2	1986	CRCP	n/a	n/a	250	AGG	74	63	66	n/a	n/a	n/a	91.9	93.0	92.5	149	107	125	58.9	9
EB I-80 Pine Bluffs, WY (Other)																				
WY 1-1	1985	JPCP	4.3-4.9-4.0-3.7	none	250	AGG	122	99	106	14.3	23.2	18.8	n/a	n/a	n/a	229	85	153	86.8	16
WY 1-2	1984	JPCP	4.3-4.9-4.0-3.7	none	250	AGG	104	74	87	38.6	70.9	54.8	n/a	n/a	n/a	160	112	139	52.8	27

*I-beam load transfer devices.

Table 83. Laboratory testing results.

Project Sec. ID	Const. Date	Pvt. Type	Slab T, mm	Thermal Coef., $1x10^{-6}/°C$	Split Tensile Strength, MPa	Dynamic E, GPa	Static E, GPa	Compressive Strength, MPa	Volumetric Surface Texture, cm^3/cm^2		
									Lab Fractured Surface	Joint	Crack
WB I-84 Waterbury, CT (Good)											
CT 1-1	1980	JRCP	230	11.6	3.8	31.7	n/a	39.2	0.4479	0.6016	0.3467
CT 1-2	1980	JRCP	230	10.6	3.3	32.8	n/a	35.4	0.3209	0.4933	0.5376
NB K-7 Johnson County, KS (Good)											
KS 1-1	1985	JPCP	230	10.5	3.2	35.3	n/a	47.9	0.2613	0.2678	n/a
SB K-7 Johnson County, KS (Good)											
KS 1-2	1985	JPCP	230	9.4	3.6	35.8	n/a	43.7	0.2595	0.3321	n/a
WB I-94 Brandon, MN (Good)											
MN 1-1	1988	JRCP	280	11.2	3.9	36.2	31.4	47.3	0.2487	0.2586	0.6043
MN 1-2	1988	JRCP	280	11.3	4.6	41.0	32.1	46.5	0.3805	0.2766	n/a
EB I-90 Beaver Creek, MN (Cracked)											
MN 2-1	1984	JRCP	230	11.1	4.1	34.8	29.2	39.2	0.2775	0.2913	0.3426
WB I-90 Beaver Creek, MN (Cracked)											
MN 2-2	1984	JRCP	230	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
SB US 59 Worthington, MN (Other)											
MN 3-1	1980	JPCP	200	8.9	4.1	34.2	31.2	44.1	0.1603	0.2475	n/a
NB US 52 Zumbrota, MN (Cracked)											
MN 4-1	1984	JRCP	230	11.6	4.3	35.4	30.1	42.8	0.1398	0.2372	0.3362
MN 4-2	1984	JRCP	230	11.2	4.3	41.8	33.3	47.6	n/a	0.2807	0.2508
EB I-94 Menomonie, WI (Cracked)											
WI 1-1	1984	JPCP	280	11.3	3.0	32.3	29.0	34.2	0.4223	0.3682	0.5833
WI 1-2	1984	JPCP	280	12.5	3.0	32.1	28.0	35.1	0.4167	0.3980	0.3852
WB I-90 Beloit, WI (Other)											
WI 2-1	1986	CRCP	250	10.6	3.5	37.2	n/a	55.5	0.3359	n/a	0.2385
WI 2-2	1986	CRCP	250	13.5	4.1	39.0	n/a	44.3	0.3107	n/a	0.3726
EB I-80 Pine Bluffs, WY (Other)											
WY 1-1	1985	JPCP	250	13.3	3.7	35.0	33.2	48.7	0.1711	0.2927	n/a
WY 1-2	1984	JPCP	250	10.8	3.2	36.7	29.1	44.7	0.3019	0.5043	n/a

Table 84. Primary performance data.

Project Sec. ID	Const. Date	Pvt. Type	Slab T,mm	Joint Spacing, m	Dowel Dia., mm	Base Type	PSR	Faulting, mm					Transverse Cracking								Trans. Joint Spalling				Long. Cracking, m/km
								Corner			Wheel Path		% Slabs Cracked				Cracks/km				% of Joints				
								Min.	Max.	Ave.	Manual	Digital	L	M	H	Total	L	M	H	Total	L	M	H	Total	
WB I-84 Waterbury, CT (Good)																									
CT 1-1	1980	JRCP	230	12	38*	AGG	3.4	-0.8	2.5	0.5	0.5	0.3	42	12	12	66	36.7	13.4	13.4	63.5	16	40	36	92	0
CT 1-1C	1980	JRCP	230	12	38*	AGG							38	4	8	50	60.2	3.4	6.7	70.3					
CT 1-1L	1980	JRCP	230	12	38*	AGG							67	12	17	96	96.9	13.4	13.4	123.7					
CT 1-2	1980	JRCP	230	12	38*	AGG	3.5	-0.5	4.1	0.5	0.3	0.3	53	13	27	93	82.0	10.9	21.9	114.8	12	25	0	37	0
CT 1-2C	1980	JRCP	230	12	38*	AGG							40	20	27	87	49.2	16.4	21.9	87.5					
CT 1-2L	1980	JRCP	230	12	38*	AGG							20	7	33	60	16.4	5.5	27.3	49.2					
NB K-7 Johnson County, KS (Good)																									
KS 1-1	1985	JPCP	230	4.7	none	CTB	3.8	-0.5	6.1	2.3	2.3	2.3	0	0	0	0	0.0	0.0	0.0	0.0	22	7	0	29	0
SB K-7 Johnson County, KS (Good)																									
KS 1-2	1985	JPCP	230	4.7	none	CTB	3.8	0.0	9.1	3.8	3.3	3.3	0	0	0	0	0.0	0.0	0.0	0.0	22	4	0	26	0
WB I-94 Brandon, MN (Good)																									
MN 1-1	1988	JRCP	280	8.2	32	AGG	3.9	0.0	1.8	0.5	0.5	0.5	0	1	0	1	0.0	3.2	0.0	3.2	49	0	0	49	0
MN 1-2	1988	JRCP	280	8.2	32	AGG	4.0	-0.5	1.3	0.5	0.3	0.5	0	0	0	0	0.0	0.0	0.0	0.0	41	0	0	41	0
EB I-90 Beaver Creek, MN (Cracked)																									
MN 2-1	1984	JRCP	230	8.2	25	AGG	4.1	0.0	2.5	1.0	0.8	0.8	39	34	11	84	54.4	44.7	15.9	115.0	21	0	0	21	0
WB I-90 Beaver Creek, MN (Cracked)																									
MN 2-2	1984	JRCP	230	8.2	25	AGG	4.3	-0.5	2.0	0.5	0.3	0.5	47	34	0	81	60.8	41.6	0.0	102.4	13	3	0	16	0
SB US 59 Worthington, MN (Other)																									
MN 3-1	1980	JPCP	200	4.0-4.9-4.3-5.8	none	St AGG	3.0	3.8	13.5	7.4	n/a	6.1	0	2	0	2	0.0	3.3	0.0	3.3	68	3	0	71	19
NB US 52 Zumbrota, MN (Cracked)																									
MN 4-1	1984	JRCP	230	8.2	25	AGG	4.0	-0.5	2.0	0.3	0.5	1.0	25	60	3	88	35.2	76.6	3.2	115.0	76	0	0	76	0
MN 4-2	1984	JRCP	230	8.2	25	AGG	4.2	0.0	1.3	0.5	0.5	0.8	22	0	0	22	26.3	0.0	0.0	26.3	89	3	0	92	0
EB I-94 Menomonie, WI (Cracked)																									
WI 1-1	1984	JPCP	280	3.7-4.0-5.8-5.5	none	AGG	4.1	-1.0	1.0	0.0	2.0	2.8	8	0	0	8	16.3	0.0	0.0	16.3	79	18	0	97	0
WI 1-2	1984	JPCP	280	3.7-4.0-5.8-5.5	35	AGG	3.8	0.0	0.8	0.3	0.3	0.5	2	0	0	2	3.2	0.0	0.0	3.2	10	6	7	23	0
WB I-90 Beloit, WI (Other)																									
WI 2-1	1986	CRCP	250	n/a	n/a	AGG	3.9	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	1158	121	13	1292	n/a	n/a	n/a	n/a	0
WI 2-2	1986	CRCP	250	n/a	n/a	AGG	4.0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	1398	29.8	0	1427	n/a	n/a	n/a	n/a	0
EB I-80 Pine Bluffs, WY (Other)																									
WY 1-1	1985	JPCP	250	4.3-4.9-4.0-3.7	none	AGG	3.6	-0.5	7.6	2.3	2.3	2.0	0	0	0	0	0.0	0.0	0.0	0.0	24	0	1	25	55
WY 1-2	1984	JPCP	250	4.3-4.9-4.0-3.7	none	AGG	3.6	0.3	5.3	2.0	2.0	2.0	0	0	0	0	0.0	0.0	0.0	0.0	15	0	1	16	14

*I-beam load transfer devices.

Table 85. Secondary performance data.

Project Sec. ID	Const. Date	Pvt. Type	Slab T, mm	Joint Spacing, m	Dowel Dia., mm	Base Type	Compression Cracks % slabs cracked L	M	H	All	Joint Seal Damage % of Joints L	M	H	Total	Pumping N/L/M/H	Durability Distresses D-Cracking	Reactive Aggregate	Outer Shoulder Condition Overall	Sealant
WB I-84 Waterbury, CT (Good)																			
CT 1-1	1980	JRCP	230	12	38*	AGG	0	0	0	0	52	28	8	88	N	N	N	Good	n/a
CT 1-2	1980	JRCP	230	12	38*	AGG	0	0	0	0	6	25	6	37	N	N	N	Good	n/a
NB K-7 Johnson County, KS (Good)																			
KS 1-1	1985	JPCP	230	4.7	none	CTB	0	0	0	0	0	0	0	0	L	N	N	Good	Good
SB K-7 Johnson County, KS (Good)																			
KS 1-2	1985	JPCP	230	4.7	none	CTB	0	0	0	0	8	3	0	11	M	N	N	Good	Good
WB I-94 Brandon, MN (Good)																			
MN 1-1	1988	JRCP	280	8.2	32	AGG	0	0	0	0	0	0	0	0	N	N	N	Good	Fair
MN 1-2	1988	JRCP	280	8.2	32	AGG	0	0	0	0	0	0	0	0	N	N	N	Good	Fair
EB I-90 Beaver Creek, MN (Cracked)																			
MN 2-1	1984	JRCP	230	8.2	25	AGG	0	0	0	0	0	0	0	0	N	N	N	Fair	n/a
WB I-90 Beaver Creek, MN (Cracked)																			
MN 2-2	1984	JRCP	230	8.2	25	AGG	0	0	0	0	0	0	0	0	N	N	N	Fair	n/a
SB US 59 Worthington, MN (Other)																			
MN 3-1	1980	JPCP	200	4.0-4.9-4.3-5.8	none	Stab AGG	0	0	0	0	61	15	0	76	L	N	N	Fair	n/a
NB US 52 Zumbrota, MN (Cracked)																			
MN 4-1	1984	JRCP	230	8.2	25	AGG	0	0	0	0	10	0	0	10	N	N	N	Fair	n/a
MN 4-2	1984	JRCP	230	8.2	25	AGG	0	0	0	0	84	0	0	84	N	N	N	Fair	n/a
EB I-94 Menomonie, WI (Cracked)																			
WI 1-1	1984	JPCP	280	3.7-4.0-5.8-5.5	none	AGG	0	0	0	0	0	0	95	95	M	N	N	Good	n/a
WI 1-2	1984	JPCP	280	3.7-4.0-5.8-5.5	35	AGG	0	0	0	0	0	0	97	97	N	N	N	Good	n/a
WB I-90 Beloit, WI (Other)																			
WI 2-1	1986	CRCP	250	n/a	n/a	AGG	0	0	0	0	n/a	n/a	n/a	n/a	N	N	N	Good	n/a
WI 2-2	1986	CRCP	250	n/a	n/a	AGG	0	0	0	0	n/a	n/a	n/a	n/a	N	N	N	Good	n/a
EB I-80 Pine Bluffs, WY (Other)																			
WY 1-1	1985	JPCP	250	4.3-4.9-4.0-3.7	none	AGG	0	0	0	0	38	43	16	97	N	N	N	Good	Poor
WY 1-2	1984	JPCP	250	4.3-4.9-4.0-3.7	none	AGG	0	0	0	0	38	58	0	96	N	N	N	Good	Good

*I-beam load transfer devices.

APPENDIX B. PETROGRAPHIC ANALYSIS OF FIELD CORES

The results of the petrographic analysis for each section can be found within appendix B. The following results are located within this appendix:

1. Petrographic examination of hardened concrete for each field core (twenty-four cores in all).

2. Summary report of the petrographic examinations for each project.

3. Shearing strength estimations related to the petrographic examinations.

4. Tensile strength estimations related to the petrographic examinations.

Compressive strength estimations related to the petrographic examinations.

PETROGRAPHIC EXAMINATION OF HARDENED CONCRETE

PROJECT: DTFH61-93-C-00133 "Physical and Mechanical Properties of Recycled PCC Aggregate Concrete"
SAMPLE: Concrete Core-half from Leave Side of Crack
SAMPLE NAME: CT 1-1C3
SOURCE PAVEMENT OF SAMPLE: CT 1, I-84 near Waterbury
SOURCE PAVEMENT TYPE: Recycled PCC
CORING LOCATION ON SOURCE PAVEMENT: Sta. 9+82, 0.6 m (2 ft) from EOP
CONDITION RATING OF SOURCE PAVEMENT: Good

SAMPLE:

Dimensions: Diameter, 150 mm (6 in); Length, 230 mm (9 in).
Top Surface: Faint transverse striations.
Crack Surface: High textural relief, many coarse aggregate particles, 19 mm (3/4 in) and 13 mm (1/2 in) exposed. Crack surface is coated with brown, sandy soil. Some areas of crack surface appear clean and eroded.
Bottom Surface: See remarks.
Reinforcement: Transverse 6 mm (1/4 in) diam. wire at 170 mm (6 1/2 in) depth, rusted through on crack surface.
ASR Deposits: Slight U-V green color in mortar on crack surface and on core side. Minor ASR indicated.
Other Deposits: None noted.

AGGREGATES:

Coarse: Recycled PCC containing dark gray, fine-grained crushed trap rock original coarse aggregate. Stated top size, 51 mm (2 in). Original coarse aggregate particles are highly angular and unevenly distributed. Many areas of concrete contain only cement paste and fine aggregate.
Fine: Quartzose natural sand.

MORTAR:

Color: Gray, original and new mortar.
Hardness: Moderately hard.
Luster: Dull.
Cracks/Large Voids: Few large voids.

REMARKS: Crack is angled from vertical, bottom surface of pavement not present.

PETROGRAPHIC EXAMINATION OF HARDENED CONCRETE

PROJECT: DTFH61-93-C-00133 "Physical and Mechanical Properties of Recycled PCC Aggregate Concrete"
SAMPLE: Concrete Core-half from Approach Side of Joint
SAMPLE NAME: CT 1-1J2
SOURCE PAVEMENT TYPE: Recycled PCC
SOURCE PAVEMENT OF SAMPLE: CT 1, I-84 near Waterbury
CORING LOCATION ON SOURCE PAVEMENT: Sta. 3+42, 0.6 m (2 ft) from EOP
CONDITION RATING OF SOURCE PAVEMENT: Good

SAMPLE:

Dimensions: Diameter, 150 mm (6 in); Length, 230 mm (8 7/8 in).
Top Surface: Faint to no transverse striations, smooth.
Crack Surface: High textural relief, few coarse aggregate particles, 19 mm (3/4 in) and 13 mm (1/2 in) exposed. Crack surface is coated with brown, sandy soil.
Bottom Surface: Gravel base particles attached.
Reinforcement: None present.
ASR Deposits: Slight U-V green color in mortar on crack surface and on core side. Minor ASR indicated.
Other Deposits: None noted.

AGGREGATES:

Coarse: Recycled PCC containing dark gray, fine-grained crushed trap rock original coarse aggregate. Stated top size, 51 mm (2 in). Original coarse aggregate particles are highly angular and unevenly distributed. Many areas of concrete contain only cement paste and fine aggregate.
Fine: Quartzose natural sand.

MORTAR:

Color: Gray, original and new mortar.
Hardness: Moderately hard.
Luster: Dull.
Cracks/Large Voids: Few large voids.

REMARKS: Joint saw-cut is approximately 60 mm (2 3/8 in) deep, with bituminous hot-poured joint sealant attached. Crack below saw-cut is roughly vertical.

PETROGRAPHIC EXAMINATION OF HARDENED CONCRETE

PROJECT: DTFH61-93-C-00133 "Physical and Mechanical Properties of Recycled PCC Aggregate Concrete"
SAMPLE: Concrete Core-half from Leave Side of Crack
SAMPLE NAME: CT 1-2C1
SOURCE PAVEMENT TYPE: PCC
SOURCE PAVEMENT OF SAMPLE: CT 1, I-84 near Waterbury
CORING LOCATION ON SOURCE PAVEMENT: Sta. 0+67, 0.6 m (2 ft) from EOP
CONDITION RATING OF SOURCE PAVEMENT: Good

SAMPLE:

Dimensions: Diameter, 150 mm (6 in); Length, 190 mm (7 1/2 in).
Top Surface: Smooth, few coarse aggregates exposed.
Crack Surface: High textural relief, many coarse aggregate particles, 19 mm (3/4 in) and 13 mm (1/2 in) exposed. Crack surface appears clean and etched.
Bottom Surface: See remarks.
Reinforcement: Transverse 6 mm (1/4 in) diam. wire at 150 mm (6 in) depth; small piece exposed on core-side.
ASR Deposits: Slight U-V green color in mortar on core side. Minor ASR indicated.
Other Deposits: None noted.

AGGREGATES:

Coarse: Dark gray, fine-grained crushed trap rock. Stated top size, 38 mm (1 1/2 in). Coarse aggregates are highly angular and evenly distributed.
Fine: Quartzose natural sand.

MORTAR:

Color: Gray.
Hardness: Moderately hard.
Luster: Dull.
Cracks/Large Voids: Few large voids.

REMARKS: Crack is angled from vertical, bottom surface of pavement not present.

PETROGRAPHIC EXAMINATION OF HARDENED CONCRETE

PROJECT: DTFH61-93-C-00133 "Physical and Mechanical Properties of Recycled PCC Aggregate Concrete"
SAMPLE: Concrete Core-half from Approach Side of Joint
SAMPLE NAME: CT 1-2J1
SOURCE PAVEMENT TYPE: PCC
SOURCE PAVEMENT OF SAMPLE: CT 1, I-84 near Waterbury
CORING LOCATION ON SOURCE PAVEMENT: Sta. 1+20, 0.6 m (2 ft) from EOP
CONDITION RATING OF SOURCE PAVEMENT: Good

SAMPLE:

Dimensions: Diameter, 150 mm (6 in); Length, 240 mm (9 1/4 in).
Top Surface: Smooth, few coarse aggregates exposed.
Crack Surface: High textural relief, many coarse aggregate particles, 19 mm (3/4 in) and 13 mm (1/2 in) exposed. Crack surface is coated with brown, sandy soil.
Bottom Surface: Rough with few coarse aggregates exposed.
Reinforcement: None present.
ASR Deposits: Slight U-V green color in mortar on core side. Minor ASR indicated.
Other Deposits: None noted.

AGGREGATES:

Coarse: Dark gray, fine-grained crushed trap rock. Stated top size, 38 mm (1 1/2 in). Coarse aggregate is highly angular and evenly distributed.
Fine: Quartzose natural sand.

MORTAR:

Color: Gray.
Hardness: Moderately hard.
Luster: Dull.
Cracks/Large Voids: Few large voids.

REMARKS: Joint saw-cut is approximately 60 mm (2 3/8 in) deep. Crack below saw-cut is roughly vertical.

PETROGRAPHIC EXAMINATION OF HARDENED CONCRETE

PROJECT: DTFH61-93-C-00133 "Physical and Mechanical Properties of Recycled PCC Aggregate Concrete"
SAMPLE: Concrete Core-half from Approach Side of Joint
SAMPLE NAME: KS 1-1J3
SOURCE PAVEMENT OF SAMPLE: KS 1, K-7 in Johnson County
SOURCE PAVEMENT TYPE: Recycled PCC
CORING LOCATION ON SOURCE PAVEMENT: Sta. 7+78, 0.6 m (2 ft) from EOP
CONDITION RATING OF SOURCE PAVEMENT: Good

SAMPLE:

Dimensions: Diameter, 150 mm (6 in); Length, 240 mm (9 1/4 in).
Top Surface: Smooth, no coarse aggregate exposed.
Crack Surface: Moderate textural relief, very few coarse aggregate particles, 13 mm (1/2 in) exposed. Crack surface is coated with tan clay soil.
Bottom Surface: Rough, with traces of bituminous material and yellowish clay attached.
Reinforcement: None present.
ASR Deposits: Many small U-V green patches in mortar. Moderate ASR indicated.
Other Deposits: None noted.

AGGREGATES:

Coarse: Recycled PCC with tan, very fine grained limestone original coarse aggregate particles. Stated top size, 38 mm (1 1/2 in). Coarse aggregate particles are angular and evenly distributed.
Fine: Quartzose natural sand.

MORTAR:

Color: Original mortar tan; new mortar gray.
Hardness: Moderately hard.
Luster: Dull.
Cracks/Large Voids: Many large voids noted.

REMARKS: Joint saw-cut is 64 mm (2 1/2 in) deep, with silicone seal attached. Crack below saw-cut is roughly vertical.

PETROGRAPHIC EXAMINATION OF HARDENED CONCRETE

PROJECT: DTFH61-93-C-00133 "Physical and Mechanical Properties of Recycled PCC Aggregate Concrete"
SAMPLE: Concrete Core-half from Approach Side of Joint
SAMPLE NAME: KS 1-2J1
SOURCE PAVEMENT OF SAMPLE: KS 1, K-7 in Johnson County
SOURCE PAVEMENT TYPE: Recycled PCC (See remarks)
CORING LOCATION ON SOURCE PAVEMENT: Sta. 5+57, 0.6 m (2 ft) from EOP
CONDITION RATING OF SOURCE PAVEMENT: Good

SAMPLE:

Dimensions: Diameter, 150 mm (6 in); Length, 250 mm (9 3/4 in).
Top Surface: Smooth, no coarse aggregate exposed.
Crack Surface: Moderate textural relief, very few coarse aggregate particles, 13 mm (1/2 in) exposed. Crack surface is coated with gray clay soil.
Bottom Surface: Rough, with traces of bituminous material and some rounded coarse aggregate particles attached.
Reinforcement: None present.
ASR Deposits: Few small U-V green patches in mortar and coarse aggregate on core side. Minor ASR indicated.
Other Deposits: None noted.

AGGREGATES:

Coarse: Recycled PCC with tan, very fine grained limestone original coarse aggregate particles. Stated top size, 19 mm (3/4 in). Coarse aggregate particles are angular and evenly distributed.
Fine: Quartzose natural sand.

MORTAR:

Color: Original mortar tan; new mortar gray.
Hardness: Moderately hard.
Luster: Dull.
Cracks/Large Voids: Many large voids noted.

REMARKS: Joint saw-cut is 76 mm (3 in) deep, with silicone seal attached. Crack below saw-cut is roughly vertical. Specimen slice prepared for linear traverse contains exposure of recycled PCC.

PETROGRAPHIC EXAMINATION OF HARDENED CONCRETE

PROJECT: DTFH61-93-C-00133 "Physical and Mechanical Properties of Recycled PCC Aggregate Concrete"
SAMPLE: Concrete Core-half from Approach Side of Crack
SAMPLE NAME: MN 1-1C1
SOURCE PAVEMENT OF SAMPLE: MN 1, I-94 near Brandon
SOURCE PAVEMENT TYPE: Recycled PCC
CORING LOCATION ON SOURCE PAVEMENT: Sta. 6+10, 0.9 m (3 ft) from EOP
CONDITION RATING OF SOURCE PAVEMENT: Good

SAMPLE:

Dimensions: Diameter, 150 mm (6 in); Length, 290 mm (11 3/8 in).
Top Surface: Rough, with few irregular striations, 32 mm (1 1/4 in) apart.
Crack Surface: Moderate textural relief, few coarse aggregate particles, 19 mm (3/4 in) and 13 mm (1/2 in) exposed. Tan to dark gray clay soil attached to crack surface.
Bottom Surface: Rough, with pebbly sand attached.
Reinforcement: Longitudinal 6 mm (1/4 in) diam. wire at 130 mm (5 1/4 in) depth, rusted through on crack surface.
ASR Deposits: Few U-V green color fine and coarse aggregate particles on crack surface and on core side. Minor ASR indicated.
Other Deposits: None noted.

AGGREGATES:

Coarse: Recycled PCC with igneous and sedimentary, predominantly carbonate, original gravel coarse aggregate particles. Stated top size, 19 mm (3/4 in). Original coarse aggregates are angular and evenly distributed.
Fine: Quartzose natural sand.

MORTAR:

Color: Tan, original and new mortar.
Hardness: Moderately hard.
Luster: Dull.
Cracks/Large Voids: Considerable large voids noted.

REMARKS: Crack is roughly vertical.

PETROGRAPHIC EXAMINATION OF HARDENED CONCRETE

PROJECT: DTFH61-93-C-00133 "Physical and Mechanical Properties of Recycled PCC Aggregate Concrete"
SAMPLE: Concrete Core-half from Approach Side of Joint
SAMPLE NAME: MN 1-1J2
SOURCE PAVEMENT OF SAMPLE: MN 1, I-94 near Brandon
SOURCE PAVEMENT TYPE: Recycled PCC
CORING LOCATION ON SOURCE PAVEMENT: Sta. 4+87, 1.2 m (4 ft) from EOP
CONDITION RATING OF SOURCE PAVEMENT: Good

SAMPLE:

Dimensions: Diameter, 150 mm (6 in); Length, 290 mm (11 1/4 in).
Top Surface: Shallow transverse striations, few long score lines.
Crack Surface: Moderate textural relief, few coarse aggregate particles, 13 mm (1/2 in) exposed. Crack surface is coated with tan clay soil.
Bottom Surface: Rough, with rounded gravel attached.
Reinforcement: None present.
ASR Deposits: Small U-V green patches in mortar on core side. Minor ASR indicated.
Other Deposits: None noted.

AGGREGATES:

Coarse: Recycled PCC with gravel original coarse aggregate containing igneous and sedimentary particles, predominantly carbonates. Stated top size, 19 mm (3/4 in). Original coarse aggregate particles are predominantly rounded and evenly distributed.
Fine: Natural sand with igneous, metamorphic, and sedimentary particles.

MORTAR:

Color: Tan, original and new mortar.
Hardness: Moderately hard.
Luster: Dull.
Cracks/Large Voids: Considerable large voids noted.

REMARKS: Joint saw-cut is approximately 92 mm (3 5/8 in) deep, with neoprene joint seal attached. Crack below saw-cut is roughly vertical.

PETROGRAPHIC EXAMINATION OF HARDENED CONCRETE

PROJECT: DTFH61-93-C-00133 "Physical and Mechanical Properties of Recycled PCC Aggregate Concrete"
SAMPLE: Concrete Core-half from Approach Side of Joint
SAMPLE NAME: MN 1-2J1
SOURCE PAVEMENT OF SAMPLE: MN 1, I-94 near Brandon
SOURCE PAVEMENT TYPE: PCC
CORING LOCATION ON SOURCE PAVEMENT: Sta. 2+84, 0.9 m (3 ft) from EOP
CONDITION RATING OF SOURCE PAVEMENT: Good

SAMPLE:

Dimensions: Diameter, 150 mm (6 in); Length, 280 mm (11 in).
Top Surface: Moderately smooth, with coarse aggregate exposed.
Crack Surface: Moderately high textural relief, with many coarse aggregates, 13 mm (1/2 in), exposed. Crack surface has small amount of tan clay soil attached.
Bottom Surface: Rough, with rounded gravel attached.
Reinforcement: None present.
ASR Deposits: Few U-V green void fillings. Minor ASR indicated.
Other Deposits: None noted.

AGGREGATES:

Coarse: Mottled gray or pink and dark green crushed granite-diorite igneous particles. Stated top size, 19 mm (3/4 in). Coarse aggregate particles are angular and evenly distributed.
Fine: Natural sand with igneous, metamorphic, and sedimentary particles.

MORTAR:

Color: Tan.
Hardness: Moderately hard.
Luster: Dull.
Cracks/Large Voids: Considerable large voids noted.

REMARKS: Joint saw-cut is approximately 92 mm (3 5/8 in) deep, with neoprene seal attached. Crack face below saw-cut is roughly vertical.

PETROGRAPHIC EXAMINATION OF HARDENED CONCRETE

PROJECT: DTFH61-93-C-00133 "Physical and Mechanical Properties of Recycled PCC Aggregate Concrete"
SAMPLE: Concrete Core-half from Approach Side of Crack
SAMPLE NAME: MN 2-1C1
SOURCE PAVEMENT OF SAMPLE: MN 2, I-90 near Beaver Creek
SOURCE PAVEMENT TYPE: Recycled PCC
CORING LOCATION ON SOURCE PAVEMENT: Sta. 3+88, 0.9 m (3 ft) from EOP
CONDITION RATING OF SOURCE PAVEMENT: Cracked

SAMPLE:

Dimensions: Diameter, 150 mm (6 in); Length, 240 mm (9 3/8 in).
Top Surface: Moderately smooth, with one transverse score mark.
Crack Surface: Moderate textural relief, few coarse aggregate particles, 13 mm (1/2 in) and 9.5 mm (3/8 in) exposed. Crack surface is clean, with eroded appearance.
Bottom Surface: Rough, with eroded appearance.
Reinforcement: None present.
ASR Deposits: Few U-V green patches in mortar and fine aggregate on core side. Minor ASR indicated.
Other Deposits: None noted.

AGGREGATES:

Coarse: Recycled PCC with gravel original coarse aggregate containing igneous and sedimentary particles. Stated top size, 19 mm (3/4 in). Original coarse aggregate particles are rounded to angular and evenly distributed.
Fine: Natural sand with igneous, metamorphic, and sedimentary particles.

MORTAR:

Color: Tan, original mortar and new mortar.
Hardness: Moderately hard.
Luster: Dull.
Cracks/Large Voids: Moderate large voids noted.

REMARKS: Crack is roughly vertical.

PETROGRAPHIC EXAMINATION OF HARDENED CONCRETE

PROJECT: DTFH61-93-C-00133 "Physical and Mechanical Properties of Recycled PCC Aggregate Concrete"
SAMPLE: Concrete Core-half from Approach Side of Joint
SAMPLE NAME: MN 2-1J3
SOURCE PAVEMENT OF SAMPLE: MN 2, I-90 near Beaver Creek
SOURCE PAVEMENT TYPE: Recycled PCC
CORING LOCATION ON SOURCE PAVEMENT: Sta. 7+51, 0.9 m (3 ft) from EOP
CONDITION RATING OF SOURCE PAVEMENT: Cracked

SAMPLE:

Dimensions: Diameter, 150 mm (6 in); Length, 230 mm (9 in).
Top Surface: Moderately smooth, with traces of score marks.
Crack Surface: Moderately high textural relief, some coarse aggregate particles, 13 mm (1/2 in) exposed. Crack surface partly coated with tan clay soil, and partly clean, with eroded appearance.
Bottom Surface: Rough, with dark gray appearance.
Reinforcement: Epoxy-coated 25 mm (1 in) diam. dowel protruding from crack surface at 100 mm (4 in) depth. Epoxy coating is cracked and flaked, exposing deep corrosion on dowel.
ASR Deposits: Few U-V green void fillings. Minor ASR indicated.
Other Deposits: None noted.

AGGREGATES:

Coarse: Recycled PCC with gravel original coarse aggregate containing igneous and sedimentary gravel particles. Stated top size, 19 mm (3/4 in). Original coarse aggregate particles are rounded to angular and evenly distributed.
Fine: Natural sand with igneous, metamorphic, and sedimentary particles.

MORTAR:

Color: Tan, original mortar and new mortar.
Hardness: Moderately hard.
Luster: Dull.
Cracks/Large Voids: Few large voids noted.

REMARKS: Joint saw-cut is approximately 54 mm (2 1/8 in) deep. Crack below saw-cut is roughly vertical.

PETROGRAPHIC EXAMINATION OF HARDENED CONCRETE

PROJECT: DTFH61-93-C-00133 "Physical and Mechanical Properties of Recycled PCC Aggregate Concrete"
SAMPLE: Concrete Core-half from Approach Side of Joint
SAMPLE NAME: MN 3-1J2
SOURCE PAVEMENT OF SAMPLE: MN 3, US 59 near Worthington
SOURCE PAVEMENT TYPE: Recycled PCC
CORING LOCATION ON SOURCE PAVEMENT: Sta. 2+92, 0.3 m (1 ft) from EOP
CONDITION RATING OF SOURCE PAVEMENT: Other

SAMPLE:

Dimensions: Diameter, 150 mm (6 in); Length, 200 mm (8 in).
Top Surface: Moderately smooth, with faint longitudinal striations.
Crack Surface: Moderately high textural relief, many coarse aggregate particles, 13 mm (1/2 in) and 9.5 mm (3/8 in) exposed. Dark gray clay soil attached to crack surface; crack surface appears eroded below 180 mm (7 in) depth.
Bottom Surface: Rough, eroded.
Reinforcement: None present.
ASR Deposits: Some U-V green patches in mortar on core-side. Many dull green U-V coarse aggregates. Moderate ASR indicated.
Other Deposits: None noted.

AGGREGATES:

Coarse: Recycled PCC with gravel original coarse aggregate containing igneous, metamorphic, and sedimentary particles. Stated top size, 19 mm (3/4 in). Original coarse aggregates are rounded to angular and evenly distributed.
Fine: Natural sand with igneous, metamorphic, and sedimentary particles.

MORTAR:

Color: Tan, original and new mortar.
Hardness: Moderately hard.
Luster: Dull.
Cracks/Large Voids: Moderate large voids noted.

REMARKS: Joint saw-cut is approximately 51 mm (2 in) deep. Crack below saw-cut is roughly vertical.

PETROGRAPHIC EXAMINATION OF HARDENED CONCRETE

PROJECT: DTFH61-93-C-00133 "Physical and Mechanical Properties of Recycled PCC Aggregate Concrete"
SAMPLE: Concrete Core-half from Approach Side of Crack
SAMPLE NAME: MN 4-1C3
SOURCE PAVEMENT OF SAMPLE: MN 4, US 52 near Zumbrota
SOURCE PAVEMENT TYPE: Recycled PCC
CORING LOCATION ON SOURCE PAVEMENT: Sta. 7+78, 0.6 m (2 ft) from EOP
CONDITION RATING OF SOURCE PAVEMENT: Cracked

SAMPLE:

Dimensions: Diameter, 150 mm (6 in); Length, 240 mm (9 1/4 in).
Top Surface: Smooth, with trace of striations and few transverse score lines.
Crack Surface: Moderately high textural relief, few coarse aggregate particles, 13 mm (1/2 in) exposed. Crack surface is clean, with etched appearance.
Bottom Surface: Rough and clean, with eroded appearance.
Reinforcement: Transverse 6 mm (1/4 in) diam. wire at 110 mm (4 1/4 in) depth, deeply corroded and rusted.
ASR Deposits: Some small U-V green patches in mortar on core-side. Minor ASR indicated.
Other Deposits: None noted.

AGGREGATES:

Coarse: Recycled PCC with gravel original coarse aggregate containing predominantly igneous and metamorphic gravel particles. Stated top size, 25 mm (1 in). Original coarse aggregate particles are rounded to angular and evenly distributed.
Fine: Quartzose natural sand.

MORTAR:

Color: Original mortar tan; new mortar gray.
Hardness: Moderately hard.
Luster: Dull.
Cracks/Large Voids: Few large voids noted.

REMARKS: Crack is roughly vertical.

PETROGRAPHIC EXAMINATION OF HARDENED CONCRETE

PROJECT: DTFH61-93-C-00133 "Physical and Mechanical Properties of Recycled PCC Aggregate Concrete"
SAMPLE: Concrete Core-half from Approach Side of Joint
SAMPLE NAME: MN 4-1J1
SOURCE PAVEMENT OF SAMPLE: MN 4, US 52 near Zumbrota
SOURCE PAVEMENT TYPE: Recycled PCC
CORING LOCATION ON SOURCE PAVEMENT: Sta. 1+37, 0.6 m (2 ft) from EOP
CONDITION RATING OF SOURCE PAVEMENT: Cracked

SAMPLE:

Dimensions: Diameter, 150 mm (6 in); Length, 230 mm (9 in).
Top Surface: Smooth, with trace of striations and few transverse score lines.
Crack Surface: High textural relief, many coarse aggregate particles, 19 mm (3/4 in) and 13 mm (1/2 in) exposed. Crack surface is clean, with etched appearance.
Bottom Surface: Rough.
Reinforcement: Epoxy-coated 25-mm (1-in) diam. dowel protruding from crack surface at 100 mm (4 in) depth. Epoxy coating is cracked, exposing corrosion on dowel.
ASR Deposits: Some small U-V green patches in mortar on core side. Minor ASR indicated.
Other Deposits: None noted.

AGGREGATES:

Coarse: Recycled PCC with gravel original coarse aggregate containing predominantly igneous and metamorphic particles. Stated top size, 25 mm (1 in). Coarse aggregate particles are angular to rounded and evenly distributed.
Fine: Natural sand with igneous, metamorphic, and sedimentary particles.

MORTAR:

Color: Original mortar tan; new mortar gray.
Hardness: Moderately hard.
Luster: Dull.
Cracks/Large Voids: Moderate large voids noted.

REMARKS: Crack is roughly vertical.

PETROGRAPHIC EXAMINATION OF HARDENED CONCRETE

PROJECT: DTFH61-93-C-00133 "Physical and Mechanical Properties of Recycled PCC Aggregate Concrete"
SAMPLE: Concrete Core-half from Approach Side of Crack
SAMPLE NAME: MN 4-2C2
SOURCE PAVEMENT OF SAMPLE: MN 4, US 52 near Zumbrota
SOURCE PAVEMENT TYPE: PCC
CORING LOCATION ON SOURCE PAVEMENT: Sta. 3+69, 0.6 mm (2 ft) from EOP
CONDITION RATING OF SOURCE PAVEMENT: Cracked

SAMPLE:

Dimensions: Diameter, 150 mm (6 in); Length, 230 mm (9 in).
Top Surface: Moderately smooth, with faint striations and one transverse groove.
Crack Surface: Moderate textural relief, few coarse aggregate particles, 19 mm (3/4 in) and 13 mm (1/2 in) exposed. Crack surface is coated with tan to gray clay soil.
Bottom Surface: Rough, with natural sand attached.
Reinforcement: Longitudinal 6-mm (1/4-in) diam. wire protruding from crack surface at 110 mm (4 1/2 in) depth, rusted through.
ASR Deposits: Faint U-V green patches in mortar and coarse aggregate on core side. Minor ASR indicated.
Other Deposits: None noted.

AGGREGATES:

Coarse: Tan, very fine grained dolomite. Stated top size, 19 mm (3/4 in). Coarse aggregate particles are angular and evenly distributed.
Fine: Natural sand with igneous, metamorphic, and sedimentary particles.

MORTAR:

Color: Tan.
Hardness: Moderately hard.
Luster: Dull.
Cracks/Large Voids: Moderate large voids noted.

REMARKS: Crack is roughly vertical.

PETROGRAPHIC EXAMINATION OF HARDENED CONCRETE

PROJECT: DTFH61-C-93-00133 "Physical and Mechanical Properties of Recycled PCC Aggregate Concrete"
SAMPLE: Concrete Core-half from Leave Side of Joint
SAMPLE NAME: MN 4-2J3
SOURCE PAVEMENT OF SAMPLE: MN 4, US 52 near Zumbrota
SOURCE PAVEMENT TYPE: PCC
CORING LOCATION ON SOURCE PAVEMENT: Sta. 6+82, 0.6 m (2 ft) from EOP
CONDITION RATING OF SOURCE PAVEMENT: Cracked

SAMPLE:

Dimensions: Diameter, 150 mm (6 in); Length, 230 mm (9 in).
Top Surface: Moderately smooth, with polished coarse aggregates exposed.
Crack Surface: Moderately high textural relief, few coarse aggregate particles, 19 mm (3/4 in) and 13 mm (1/2 in) exposed. Crack surface is coated with tan to gray clay soil.
Bottom Surface: See remarks.
Reinforcement: None present.
ASR Deposits: Faint U-V green patches in mortar on core side. Minor ASR indicated.
Other Deposits: None noted.

AGGREGATES:

Coarse: Tan, very fine grained dolomite. Stated top size, 19 mm (3/4 in). Coarse aggregate particles are angular and evenly distributed.
Fine: Natural sand with igneous, metamorphic, and sedimentary particles.

MORTAR:

Color: Tan.
Hardness: Moderately hard.
Luster: Dull.
Cracks/Large Voids: Moderate large voids noted.

REMARKS: Joint saw-cut is approximately 64 mm (2 1/2 in) deep. Crack below saw-cut is roughly vertical to depth of 180 mm (7 in). Bottom surface of core is an angled fracture surface roughly parallel to an angled crack at 180 mm (7 in) depth. Bottom surface of pavement is not present.

PETROGRAPHIC EXAMINATION OF HARDENED CONCRETE

PROJECT: DTFH61-93-C-00133 "Physical and Mechanical Properties of Recycled PCC Aggregate Concrete"
SAMPLE: Concrete Core-half from Approach Side of Crack
SAMPLE NAME: WI 1-1C1
SOURCE PAVEMENT OF SAMPLE: WI 1, I-94 near Menomonie
SOURCE PAVEMENT TYPE: Recycled PCC
CORING LOCATION ON SOURCE PAVEMENT: Sta. 3+41, 0.6 m (2 ft) from EOP
CONDITION RATING OF SOURCE PAVEMENT: Cracked

SAMPLE:

Dimensions: Diameter, 150 mm (6 in); Length, 270 mm (10 1/2 in).
Top Surface: Longitudinally re-grooved.
Crack Surface: Moderately high textural relief, few coarse aggregate particles, 13 mm (1/2 in) exposed. Dark gray clay soil attached to crack surface, top 76 mm (3 in).
Bottom Surface: Eroded.
Reinforcement: None Present.
ASR Deposits: Some U-V green patches in mortar; some U-V green chert fine aggregate particles. Minor ASR indicated.
Other Deposits: None noted.

AGGREGATES:

Coarse: Recycled PCC with gravel original coarse aggregate containing igneous and metamorphic particles. Stated top size, 38 mm (1 1/2 in). Original coarse aggregates are angular and evenly distributed.
Fine: Natural sand with igneous, metamorphic, and sedimentary particles.

MORTAR:

Color: Tan, original and new mortar.
Hardness: Soft to moderately hard. Fine aggregate particles readily dislodged when scratched.
Luster: Dull.
Cracks/Large Voids: Few large voids noted.

REMARKS: Crack is angled from vertical.

PETROGRAPHIC EXAMINATION OF HARDENED CONCRETE

PROJECT: DTFH61-93-C-00133 "Physical and Mechanical Properties of Recycled PCC Aggregate Concrete"
SAMPLE: Concrete Core-half from Approach Side of Joint
SAMPLE NAME: WI 1-1J3
SOURCE PAVEMENT OF SAMPLE: WI 1, I-94 near Menomonie
SOURCE PAVEMENT TYPE: Recycled PCC
CORING LOCATION ON SOURCE PAVEMENT: Sta. 6+16, 0.6 m (2 ft) from EOP
CONDITION RATING OF SOURCE PAVEMENT: Cracked

SAMPLE:

Dimensions: Diameter, 150 mm (6 in); Length, 270 mm (10 5/8 in).
Top Surface: Longitudinally re-grooved.
Crack Surface: Moderately high textural relief, few coarse aggregate particles, 19 mm (3/4 in) and 13 mm (1/2 in) exposed. Dark gray clay soil attached to crack surface, top 190 mm (7 5/8 in); tan clay soil attached, bottom 76 mm (3 in). Some eroded patches present in top 200 mm (7 5/6 in).
Bottom Surface: Rough, eroded.
Reinforcement: None Present.
ASR Deposits: Some U-V green patches in mortar; some U-V green chert fine aggregate particles. Minor ASR indicated.
Other Deposits: None noted.

AGGREGATES:

Coarse: Recycled PCC with gravel original coarse aggregate containing igneous and metamorphic particles. Stated top size, 38 mm (1 1/2 in). Original coarse aggregate particles are rounded to angular and evenly distributed.
Fine: Natural sand with igneous, metamorphic, and sedimentary particles.

MORTAR:

Color: Tan, original and new mortar.
Hardness: Moderately hard.
Luster: Dull.
Cracks/Large Voids: Moderate large voids noted.

REMARKS: Joint saw-cut is approximately 64 mm (2 1/2 in) deep. Crack below saw-cut is roughly vertical.

PETROGRAPHIC EXAMINATION OF HARDENED CONCRETE

PROJECT: DTFH61-93-C-00133 "Physical and Mechanical Properties of Recycled PCC Aggregate Concrete"
SAMPLE: Concrete Core-half from Approach Side of Crack
SAMPLE NAME: WI 1-2C1
SOURCE PAVEMENT OF SAMPLE: WI 1, I-94 near Menomonie
SOURCE PAVEMENT TYPE: Recycled PCC
CORING LOCATION ON SOURCE PAVEMENT: Sta. 2+92, 0.6 m (2 ft) from EOP
CONDITION RATING OF SOURCE PAVEMENT: Cracked

SAMPLE:

Dimensions: Diameter, 150 mm (6 in); Length, 250 mm (10 in).
Top Surface: Smooth, with two transverse grooves.
Crack Surface: Moderate textural relief, some coarse aggregate particles, 19 mm (3/4 in) and 13 mm (1/2 in) exposed. Dark gray clay soil attached to crack surface, top 180 mm (7 in); crack surface below 180 mm (7 in) is clean, with eroded appearance.
Bottom Surface: See remarks.
Reinforcement: None Present.
ASR Deposits: Many U-V green patches in mortar; some U-V green chert fine aggregate particles. Moderate ASR indicated.
Other Deposits: None noted.

AGGREGATES:

Coarse: Recycled PCC with gravel original coarse aggregate containing igneous and metamorphic particles. Stated top size, 38 mm (1 1/2 in). Original coarse aggregate particles are rounded to angular and somewhat unevenly distributed.
Fine: Natural sand with igneous, metamorphic, and sedimentary particles.

MORTAR:

Color: Tan, original and new mortar.
Hardness: Moderately hard.
Luster: Dull.
Cracks/Large Voids: Moderate large voids noted.

REMARKS: Crack is roughly vertical to 180 mm (7 in) depth, then angled to side of core. Bottom surface of pavement not present.

PETROGRAPHIC EXAMINATION OF HARDENED CONCRETE

PROJECT: DTFH61-93-C-00133 "Physical and Mechanical Properties of Recycled PCC Aggregate Concrete"
SAMPLE: Concrete Core-half from Approach Side of Joint
SAMPLE NAME: WI 1-2J1
SOURCE PAVEMENT OF SAMPLE: WI 1, I-94 near Menomonie
SOURCE PAVEMENT TYPE: Recycled PCC
CORING LOCATION ON SOURCE PAVEMENT: Sta. 2+87, 0.6 m (2 ft) from EOP
CONDITION RATING OF SOURCE PAVEMENT: Cracked

SAMPLE:

Dimensions: Diameter, 150 mm (6 in); Length, 280 mm (10 7/8 in).
Top Surface: Moderately smooth, with 25-mm (1-in) spaced transverse grooves.
Crack Surface: Moderately high textural relief, few coarse aggregate particles, 19 mm (3/4 in) and 13 mm (1/2 in) exposed. Dark gray clay soil attached to crack surface; some eroded patches present.
Bottom Surface: Rough, eroded.
Reinforcement: None present.
ASR Deposits: Many U-V green patches in mortar and bright yellow color rimming some coarse aggregate particles on core-side. Moderate ASR indicated.
Other Deposits: None noted.

AGGREGATES:

Coarse: Recycled PCC with gravel original coarse aggregate containing igneous and metamorphic particles. Stated top size, 38 mm (1 1/2 in). Original coarse aggregates are rounded to angular and somewhat unevenly distributed.
Fine: Natural sand with igneous, metamorphic, and sedimentary particles.

MORTAR:

Color: Tan, original and new mortar.
Hardness: Moderately hard to soft in eroded patches.
Luster: Dull.
Cracks/Large Voids: Few large voids noted.

REMARKS: Joint saw-cut is approximately 67 mm (2 5/8 in) deep. Crack below saw-cut is roughly vertical.

PETROGRAPHIC EXAMINATION OF HARDENED CONCRETE

PROJECT: DTFH61-93-C-00133 "Physical and Mechanical Properties of Recycled PCC Aggregate Concrete"
SAMPLE: Concrete Core-half from Approach Side of Crack
SAMPLE NAME: WI 2-1C1
SOURCE PAVEMENT OF SAMPLE: WI 2, I-90 near Beloit
SOURCE PAVEMENT TYPE: Recycled PCC
CORING LOCATION ON SOURCE PAVEMENT: Sta. 5+42, 0.6 m (2 ft) from EOP
CONDITION RATING OF SOURCE PAVEMENT: Other

SAMPLE:

Dimensions: Diameter, 150 mm (6 in); Length, 260 mm (10 1/4 in).
Top Surface: Deep transverse score marks.
Crack Surface: Moderate textural relief. Few coarse aggregate particles, 13 mm (1/2 in) and 9.5 mm (3/8 in) exposed. Brown to tan sand and clay soil attached to crack surface.
Bottom Surface: Rough, with tan film attached.
Reinforcement: Imprint of approximately 19 mm (3/4 in) diam. longitudinal deformed steel reinforcing bar at 130 mm (5 in) depth.
ASR Deposits: Many U-V green patches in mortar on core-side and on delamination crack surface at steel depth. Considerable ASR indicated.
Other Deposits: None noted.

AGGREGATES:

Coarse: Recycled PCC with gravel original coarse aggregate containing igneous, metamorphic, and sedimentary particles. Stated top size, 38 mm (1 1/2 in). Original coarse aggregate particles are rounded to angular and evenly distributed.
Fine: Natural sand with igneous, metamorphic, and sedimentary particles.

MORTAR:

Color: Tan, original and new mortar.
Hardness: Moderately hard.
Luster: Dull.
Cracks/Large Voids: Moderate large voids noted.

REMARKS: Crack is roughly vertical.

PETROGRAPHIC EXAMINATION OF HARDENED CONCRETE

PROJECT: DTFH61-93-C-00133 "Physical and Mechanical Properties of Recycled PCC Aggregate Concrete"
SAMPLE: Concrete Core-half from Leave Side of Crack
SAMPLE NAME: WI 2-2C1
SOURCE PAVEMENT OF SAMPLE: WI 2, I-90 near Beloit
SOURCE PAVEMENT TYPE: Recycled PCC
CORING LOCATION ON SOURCE PAVEMENT: Sta. 2+60, 0.6 m (2 ft) from EOP
CONDITION RATING OF SOURCE PAVEMENT: Other

SAMPLE:

Dimensions: Diameter, 150 mm (6 in); Length, 250 mm (10 in).
Top Surface: Deep transverse score marks and shallow longitudinal striations.
Crack Surface: Moderate to high textural relief. Moderate coarse aggregate particles, 13 mm (1/2 in) and 9.5 mm (3/8 in) exposed. Brown clay soil attached to crack surface above steel depth; tan clay soil attached to crack surface below steel depth.
Bottom Surface: Rough, with rounded gravel attached.
Reinforcement: Imprint of approximately 19 mm (3/4 in) diam. longitudinal deformed steel reinforcing bar at 120 mm (4 3/4 in) depth.
ASR Deposits: Many U-V green patches in mortar and bright yellow color rimming some coarse aggregate on core-side. Considerable ASR indicated.
Other Deposits: None noted.

AGGREGATES:

Coarse: Recycled PCC with gravel original coarse aggregate containing igneous, metamorphic, and sedimentary particles. Stated top size, 38 mm (1 1/2 in). Original coarse aggregates are rounded to angular and evenly distributed.
Fine: Natural sand with igneous, metamorphic, and sedimentary particles.

MORTAR:

Color: Tan, original and new mortar.
Hardness: Moderately hard.
Luster: Dull.
Cracks/Large Voids: Moderate large voids noted.

REMARKS: Crack is roughly vertical.

PETROGRAPHIC EXAMINATION OF HARDENED CONCRETE

PROJECT: DTFH61-93-C-00133 "Physical and Mechanical Properties of Recycled PCC Aggregate Concrete"
SAMPLE: Concrete Core-half from Approach Side of Joint
SAMPLE NAME: WY 1 -1J3
SOURCE PAVEMENT OF SAMPLE: WY 1, I-80 near Pine Bluffs
SOURCE PAVEMENT TYPE: Recycled PCC
CORING LOCATION ON SOURCE PAVEMENT: Sta. 7+55, 0.6 m (2 ft) from EOP
CONDITION RATING OF SOURCE PAVEMENT: Other

SAMPLE:

Dimensions: Diameter, 150 mm (6 in); Length, 250 mm (10 in).
Top Surface: Moderately smooth, with shallow transverse grooves.
Crack Surface: Moderate textural relief. Moderate coarse aggregate particles, 13 mm (1/2 in) and 9.5 mm (3/8 in) exposed. Clean, etched appearance.
Bottom Surface: Rough, with rounded gravel attached.
Reinforcement: None present.
ASR Deposits: Many small U-V green patches in mortar and bright yellow color rimming some coarse aggregate on core-side. Moderate ASR indicated.
Other Deposits: None noted.

AGGREGATES:

Coarse: Recycled PCC with pink to white carbonate aggregate blended with gravel original coarse aggregate containing pink to white igneous particles. Stated top size, 25 mm (1 in). Original coarse aggregate particles are rounded to angular and evenly distributed.
Fine: Natural sand with igneous and sedimentary particles.

MORTAR:

Color: Tan, original and new mortar.
Hardness: Moderately hard.
Luster: Dull.
Cracks/Large Voids: Few large voids noted.

REMARKS: Joint saw-cut is approximately 60 mm (2 3/8 in) deep. Crack below saw-cut is roughly vertical.

PETROGRAPHIC EXAMINATION OF HARDENED CONCRETE

PROJECT: DTFH61-93-C-00133 "Physical and Mechanical Properties of Recycled PCC Aggregate Concrete"
SAMPLE: Concrete Core-half from Approach Side of Joint
SAMPLE NAME: WY 1 -2J2
SOURCE PAVEMENT OF SAMPLE: WY 1, I-80 near Pine Bluffs
SOURCE PAVEMENT TYPE: PCC
CORING LOCATION ON SOURCE PAVEMENT: Sta. 5+49, 0.6 m (2 ft) from EOP
CONDITION RATING OF SOURCE PAVEMENT: Other

SAMPLE:

Dimensions: Diameter, 150 mm (6 in); Length, 290 mm (11 1/2 in).
Top Surface: Smooth.
Crack Surface: High textural relief. Many coarse aggregate particles, 13 mm (1/2 in) and 9.5 mm (3/8 in) exposed. Considerable tan to white, calcareous deposit attached to crack surface.
Bottom Surface: Rough, with rounded gravel attached.
Reinforcement: None present.
ASR Deposits: Some small U-V green patches in mortar on core-side. Minor ASR indicated.
Other Deposits: None noted.

AGGREGATES:

Coarse: Pink to white gravel igneous particles. Stated top size, 38 mm (1 1/2 in). Coarse aggregate particles are rounded to angular and evenly distributed.
Fine: Natural sand with igneous and sedimentary particles.

MORTAR:

Color: Tan.
Hardness: Moderately hard.
Luster: Dull.
Cracks/Large Voids: Moderate large voids noted.

REMARKS: Joint saw-cut is approximately 57 mm (2 1/4 in) deep. Crack below saw-cut is roughly vertical.

SUMMARY REPORT OF PETROGRAPHIC EXAMINATION FOR TASK C OF PROJECT: DTFH61-93-C-00133

Pavement Section CT 1, I-84 near Waterbury

Recycled Section

Two core-half specimens from this section, CT 1-1C3 obtained at a transverse crack, and CT 1-1J2 obtained at a joint, were examined. The coarse aggregate is composed of recycled concrete containing fragments of mortar and dark gray, fine grained crushed trap rock. The fine aggregate in the recycled and new concrete is natural sand. The recycled and new mortar, both gray in color, have moderate hardness and display dull appearance, with few large voids present. The coarse aggregate, recycled mortar, and new mortar content determined by linear traverse measurements is as follows:

Specimen	Coarse Aggregate, %	Recycled Mortar, %	New Mortar, %
CT 1-1C3	34.6	0.6	64.8
CT 1-1J2	29.7	7.8	62.5

The transverse crack surface of specimen CT 1-1C3 displays high textural relief with many coarse aggregate particles exposed. The crack surface below the transverse joint saw-cut of specimen CT 1-1J2 displays high textural relief with few coarse aggregate particles exposed. Tests for ASR indicated the presence of minor silica gel deposits in the mortar of these specimens.

Control Section

Two core-half specimens from this section, CT 1-2C1 obtained at a crack, and CT 1-2J1 obtained at a joint, were examined. The coarse aggregate is composed of dark gray, fine grained crushed trap rock. The fine aggregate is natural sand. The mortar, gray in color, has moderate hardness and displays dull appearance, with few large voids present. The coarse aggregate and mortar content determined by linear traverse measurements is as follows:

Specimen	Coarse Aggregate, %	Mortar, %
CT 1-2C1	34.6	65.4
CT 1-2J1	42.5	57.5

The transverse crack surface of specimen CT 1-2C1 displays high textural relief with many coarse aggregate particles exposed. The crack surface below the transverse joint saw-cut of specimen CT 1-2J1 displays high textural relief with many coarse aggregate particles exposed. Tests for ASR indicated the presence of minor silica gel deposits in the mortar of these specimens.

SUMMARY REPORT OF PETROGRAPHIC EXAMINATION FOR TASK C OF PROJECT: DTFH61-93-C-00133

Pavement Section KS 1, K-7 in Johnson County

Recycled Section

One core-half specimen from this section, KS 1-1J3 obtained at a joint, was examined. The coarse aggregate is composed of recycled concrete containing fragments of mortar and tan, very fine grained limestone. The fine aggregate in the recycled and new concrete is natural sand. The recycled mortar, tan in color, and the new mortar, gray in color, have moderate hardness and display dull appearance, with a many large voids present. The coarse aggregate, recycled mortar, and new mortar content determined by linear traverse measurements is as follows:

Specimen	Coarse Aggregate, %	Recycled Mortar, %	New Mortar, %
KS 1-1J3	10.8	15.1	74.1

The crack surface below the transverse joint saw-cut of specimen KS 1-1J3 displays moderate textural relief with very few coarse aggregate particles exposed. A test for ASR indicated the presence of a moderate amount of silica gel deposits in the mortar of this specimen.

Control Section

One core-half specimen from this section, KS 1-2J1 obtained at a joint, was examined. The coarse aggregate is composed of tan, very fine grained limestone. The fine aggregate is natural sand. One large particle of recycled concrete containing mortar with tan color was noted in this sample. The recycled mortar, tan in color, and new mortar, gray in color, have moderate hardness and display dull appearance, with many large voids present. The coarse aggregate, recycled mortar, and new mortar content determined by linear traverse measurements is as follows:

Specimen	Coarse Aggregate, %	Recycled Mortar, %	New Mortar, %
KS 1-2J1	17.7	4.4	77.9

The crack surface below the transverse joint saw-cut of specimen KS 1-2J1 displays moderate textural relief with very few coarse aggregate particles exposed. A test for ASR indicated the presence of minor silica gel deposits in the mortar of this specimen.

SUMMARY REPORT OF PETROGRAPHIC EXAMINATION FOR TASK C OF PROJECT: DTFH61-93-C-00133

Pavement Section MN 1, I-94 near Brandon

Recycled Section

Two core-half specimens from this section, MN 1-1C1 obtained at a transverse crack, and MN 1-1J2 obtained at a joint, were examined. The coarse aggregate is composed of recycled concrete containing fragments of mortar and gravel rock particles. The fine aggregate in the recycled and new concrete is natural sand. The recycled and new mortar, both tan in color, have moderate hardness and display dull appearance, with a considerable amount of large voids present. The coarse aggregate, recycled mortar, and new mortar content determined by linear traverse measurements is as follows:

Specimen	Coarse Aggregate, %	Recycled Mortar, %	New Mortar, %
MN 1-1C1	22.9	11.7	65.4
MN 1-1J2	23.8	11.7	64.5

The transverse crack surface of specimen MN 1-1C1 displays moderate textural relief with few coarse aggregate particles exposed. The crack surface below the transverse joint saw-cut of specimen MN 1-1J2 displays moderate textural relief with few coarse aggregate particles exposed. Tests for ASR indicated the presence of minor silica gel deposits in the mortar of these specimens.

Control Section

One core-half specimen from this section, MN 1-2J1 obtained at a joint, was examined. The coarse aggregate is composed of crushed diorite. The fine aggregate is natural sand. The mortar, tan in color, has moderate hardness and displays dull appearance, with a considerable amount of large voids present. The coarse aggregate and mortar content determined by linear traverse measurements is as follows:

Specimen	Coarse Aggregate, %	Mortar, %
MN 1-2J1	34.2	65.8

The crack surface below the transverse joint saw-cut of specimen MN 1-2J1 displays moderately high textural relief with many coarse aggregate particles exposed. Tests for ASR indicated the presence of minor silica gel deposits in the mortar of this specimen.

SUMMARY REPORT OF PETROGRAPHIC EXAMINATION FOR TASK C OF PROJECT: DTFH61-93-C-00133

Pavement Section MN 2, I-90 near Beaver Creek

Recycled Section

Two core-half specimens from this section, MN 2-1C1 obtained at a transverse crack, and MN 2-1J3 obtained at a joint, were examined. The coarse aggregate is composed of recycled concrete containing fragments of mortar and gravel rock particles. The fine aggregate in the recycled and new concrete is natural sand. The recycled and new mortar, both tan in color, have moderate hardness and display dull appearance, with few to a moderate amount of large voids present. The coarse aggregate, recycled mortar, and new mortar content determined by linear traverse measurements is as follows:

Specimen	Coarse Aggregate, %	Recycled Mortar, %	New Mortar, %
MN 2-1C1	20.7	4.2	75.1

Specimen	Coarse Aggregate, %	Total Mortar, %
MN 2-1J3	20.5	79.5

Total mortar content of specimen MN 2-1J3 was determined due to lack of distinguishing features between the recycled and new mortars. The transverse crack surface of specimen MN 2-1C1 displays moderate textural relief with few coarse aggregate particles exposed. The crack surface below the transverse joint saw-cut of specimen MN 2-1J3 displays moderately high textural relief with some coarse aggregate particles exposed. Tests for ASR indicated the presence of minor silica gel deposits in the mortar of these specimens.

Control Section

No control section pavement core specimens were obtained from pavement section MN 2.

SUMMARY REPORT OF PETROGRAPHIC EXAMINATION FOR TASK C OF PROJECT: DTFH61-93-C-00133

Pavement Section MN 3, US-59 near Worthington

Recycled Section

One core-half specimen from this section, MN 3-1J2 obtained at a joint, was examined. The coarse aggregate is composed of recycled concrete containing fragments of mortar and gravel rock particles. The fine aggregate in the recycled and new concrete is natural sand. The recycled and new mortar, both tan in color, have moderate hardness and display dull appearance, with a moderate amount of large voids present. The coarse aggregate, recycled mortar, and new mortar content determined by linear traverse measurements is as follows:

Specimen	Coarse Aggregate, %	Recycled Mortar, %	New Mortar, %
MN 3-1J2	18.0	14.5	67.5

The crack surface below the transverse joint saw-cut of specimen MN 3-1J2 displays moderately high textural relief with many coarse aggregate particles exposed. Tests for ASR indicated the presence of moderate silica gel deposits in the mortar and some coarse aggregate particles of this specimen.

Control Section

No control section pavement core specimens were obtained from pavement section MN 3.

SUMMARY REPORT OF PETROGRAPHIC EXAMINATION FOR TASK C OF PROJECT: DTFH61-93-C-00133

Pavement Section MN 4, US-52 near Zumbrota

Recycled Section

Two core-half specimens from this section, MN 4-1C3 obtained at a transverse crack, and MN 4-1J1 obtained at a joint, were examined. The coarse aggregate is composed of recycled concrete containing fragments of mortar and gravel rock particles. The fine aggregate in the recycled and new concrete is natural sand. The recycled and new mortar, both tan in color, have moderate hardness and display dull appearance, with few to moderate amount of large voids present. The coarse aggregate, recycled mortar, and new mortar content determined by linear traverse measurements is as follows:

Specimen	Coarse Aggregate, %	Recycled Mortar, %	New Mortar, %
MN 4-1C3	16.8	14.4	68.8
MN 4-1J1	16.1	13.3	70.6

The transverse crack surface of specimen MN 4-1C3 displays moderately high textural relief with few coarse aggregate particles exposed. The crack surface below the transverse joint saw-cut of specimen MN 4-1J1 displays high textural relief with many coarse aggregate particles exposed. Tests for ASR indicated the presence of minor silica gel deposits in the mortar of these specimens.

Control Section

Two core-half specimens from this section, MN 4-2C2 obtained at a transverse crack, and MN 4-2J3 obtained at a joint, were examined. The coarse aggregate is composed of gravel rock particles (very fine grained dolomite). The fine aggregate is natural sand. The mortar, tan in color, has moderate hardness and displays dull appearance, with a moderate amount of large voids present. The coarse aggregate and mortar content determined by linear traverse measurements is as follows:

Specimen	Coarse Aggregate, %	Mortar, %
MN 4-2C2	46.2	53.8
MN 4-2J3	50.8	49.2

The transverse crack surface of specimen MN 4-2C2 displays moderate textural relief with few coarse aggregate particles exposed. The crack surface below the transverse joint saw-cut of specimen MN 4-2J3 displays moderately high textural relief with few coarse aggregate particles exposed. Tests for ASR indicated the presence of minor silica gel deposits in the mortar of these specimens.

SUMMARY REPORT OF PETROGRAPHIC EXAMINATION FOR TASK C OF PROJECT: DTFH61-93-C-00133

Pavement Section WI 1, I-94 near Menomonie

Recycled Section, Undoweled

Two core-half specimens from this section, WI 1-1C1 obtained at a transverse crack, and WI 1-1J3 obtained at a joint, were examined. The coarse aggregate is composed of recycled concrete containing fragments of mortar and gravel rock particles. The fine aggregate in the recycled and new concrete is natural sand. The recycled and new mortar, both tan in color, have moderate hardness and display dull appearance, with few to a moderate amount of large voids present. The coarse aggregate, recycled mortar, and new mortar content determined by linear traverse measurements is as follows:

Specimen	Coarse Aggregate, %	Recycled Mortar, %	New Mortar, %
WI 1-1C1	17.2	19.2	63.6
WI 1-1J3	16.9	14.8	68.3

The transverse crack surface of specimen WI 1-1C1 displays moderately high textural relief with few coarse aggregate particles exposed. The crack surface below the transverse joint saw-cut of specimen WI 1-1J3 displays moderately high textural relief with some coarse aggregate particles exposed. Tests for ASR indicated the presence of minor silica gel deposits in the mortar of these specimens.

Control Section

No control section pavement core specimens were obtained from pavement section WI 1.

SUMMARY REPORT OF PETROGRAPHIC EXAMINATION
FOR
TASK C OF PROJECT: DTFH61-93-C-00133

Pavement Section WI 1, I-94 near Menomonie

Recycled Section, Doweled

Two core-half specimens from this section, WI 1-2C1 obtained at a transverse crack, and WI 1-2J1 obtained at a joint, were examined. The coarse aggregate is composed of recycled concrete containing fragments of mortar and gravel rock particles. The fine aggregate in the recycled and new concrete is natural sand. The recycled and new mortar, both tan in color, have moderate hardness and display dull appearance, with few to a moderate amount of large voids present. The coarse aggregate, recycled mortar, and new mortar content determined by linear traverse measurements is as follows:

Specimen	Coarse Aggregate, %	Recycled Mortar, %	New Mortar, %
WI 1-2C1	13.3	13.4	73.3
WI 1-2J1	17.9	15.9	66.2

The transverse crack surface of specimen WI 1-2C1 displays moderate textural relief with some coarse aggregate particles exposed. The crack surface below the transverse joint saw-cut of specimen WI 1-2J1 displays moderately high textural relief with few coarse aggregate particles exposed. Tests for ASR indicated the presence of moderate silica gel deposits in the mortar of these specimens.

Control Section

No control section pavement core specimens were obtained from pavement section WI 1.

SUMMARY REPORT OF PETROGRAPHIC EXAMINATION FOR TASK C OF PROJECT: DTFH61-93-C-00133

Pavement Section WI 2, I-90 near Beloit

Recycled Section

Two core-half specimens from this section, WI 2-1C1 and WI 2-2C1, both obtained at transverse cracks, were examined. The coarse aggregate is composed of recycled concrete containing fragments of mortar and gravel rock particles. The fine aggregate in the recycled and new concrete is natural sand. The recycled and new mortar, both tan in color, have moderate hardness and display dull appearance, with few to a moderate amount of large voids present. The coarse aggregate, recycled mortar, and new mortar content determined by linear traverse measurements is as follows:

Specimen	Coarse Aggregate, %	Recycled Mortar, %	New Mortar, %
WI 2-1C1	21.0	9.7	69.3
WI 2-2C1	13.7	6.7	79.6

The transverse crack surface of specimen WI 2-1C1 displays moderate textural relief with few coarse aggregate particles exposed. The transverse crack surface of specimen WI 2-2C1 displays moderate to high textural relief with a moderate amount of coarse aggregate particles exposed. Tests for ASR indicated the presence of considerable silica gel deposits in the mortar of specimen WI 2-1C1. A prominent silica gel deposit was noted on a delamination crack surface at the reinforcing steel. Considerable silica gel deposits were noted in the mortar and rimming some coarse aggregate particles of specimen WI 2-2C1.

Control Section

No control section pavement core specimens were obtained from pavement section WI 2.

SUMMARY REPORT OF PETROGRAPHIC EXAMINATION FOR TASK C OF PROJECT: DTFH61-93-C-00133

Pavement Section WY 1, I-80 near Pine Bluffs

Recycled Section

One core-half specimen from this section, WY 1-1J3 obtained at a joint, was examined. The coarse aggregate is composed of recycled concrete containing fragments of mortar and white to pink, very fine grained dolomitic limestone blended with gravel rock particles. The fine aggregate in the recycled and new concrete is natural sand. The recycled mortar and new mortar, both tan in color, have moderate hardness and display dull appearance, with few large voids present. The coarse aggregate, recycled mortar, and new mortar content determined by linear traverse measurements is as follows:

Specimen	Coarse Aggregate, %	Recycled Mortar, %	New Mortar, %
WY 1-1J3	23.8	7.8	68.4

The crack surface below the transverse joint saw-cut of specimen WY 1-1J3 displays moderate textural relief with a moderate amount of coarse aggregate particles exposed. A test for ASR indicated the presence of a moderate amount of silica gel deposits in the mortar of this specimen.

Control Section

One core-half specimen from this section, WY 1-2J2 obtained at a joint, was examined. The coarse aggregate is composed of gravel rock particles. The fine aggregate is natural sand. The mortar, tan in color, has moderate hardness and displays dull appearance, with a moderate amount of large voids present. The coarse aggregate and mortar content determined by linear traverse measurements is as follows:

Specimen	Coarse Aggregate, %	Mortar, %
WY 1-2J2	43.1	56.9

The crack surface below the transverse joint saw-cut of specimen WY 1-2J2 displays high textural relief with many coarse aggregate particles exposed. The crack surface is coated with a tan to white calcareous deposit. A test for ASR indicated the presence of minor silica gel deposits in the mortar of this specimen.

SHEAR STRENGTH ESTIMATIONS

Enclosed are the results of the shearing strength estimations computed for the coarse aggregates contained in the core-half specimens analyzed for Task C of project DTFH61-93-C-00133, "Physical and Mechanical Properties of Recycled PCC Aggregate Concrete."

The projects were ranked according to the shearing strengths of the coarse aggregates using tabulated values for the general rock type categories listed in I. W. Farmer's book - Engineering Properties of Rocks. The listed values for the general rock type categories were used along with composition data from Michigan gravel sources to compute estimated shearing strength factors for the coarse aggregates that were identified in the core-half samples.

The shearing strength values for most of the rock types were used as listed. However, for the absorptive gravel carbonate fractions, typically with greater than 2 percent absorption, a 50 percent reduction was applied to reflect performance in a saturated, worst case moisture condition at cracks and joints.

After computing the shearing strength factors for the coarse aggregates, composition-weighted sample coarse aggregate shearing strength values were calculated using the coarse aggregate compositions determined by linear traverse measurements.

Ranking of the projects according to relative shearing strength was accomplished by using the highest composition-weighted sample coarse aggregate shearing strength value in the data set as unity. The projects are ranked by their departures from unity on a scale from 1.0 to 0.0.

Comments

The ranking of the control versus recycled pavements appears in general to correlate quite well with respect to the shearing strengths of the coarse aggregate components of the concrete. Most of the control pavements with trap rock and limestone or dolomite original coarse aggregate showed relative ranking near unity, whereas the recycled projects ranked much lower, typically less than 0.50, indicating that the recycled concrete has much lower shearing strength than concrete with considerable coarse aggregate content. Refer to table B-1 for the computation of composition-weighted sample coarse aggregate shearing strength, and refer to table B-2 for the rank by relative shearing strength.

Table 86. Computation of composition-weighted sample coarse aggregate shearing strength.

Sample	Sample Type	Coarse Agg.	Sample C.A., %	C.A. S_s Factor, kg/cm^2	Comp.-Wtd. Sample C.A. S_s, kg/cm^2
CT 1-1C3	Recycled	Trap Rock	34.6	600	208
CT 1-1J2	Recycled	Trap Rock	29.7	600	178
CT 1-2C1	Control	Trap Rock	34.6	600	208
CT 1-2J1	Control	Trap Rock	42.5	600	255
MN 1-1C1	Recycled	Gravel	22.9	490	112
MN 1-1J2	Recycled	Gravel	23.4	490	115
MN 1-2J1	Control	Diorite	34.2	550	188
KS 1-1J3	Recycled	Limestone	10.8	500	54
KS 1-2J1	Control	Limestone	17.7	500	89
MN 4-1C3	Recycled	Gravel	16.8	490	82
MN 4-1J1	Recycled	Gravel	16.1	490	79
MN 4-2C2	Control	Dolomite	46.2	500	231
MN 4-2J3	Control	Dolomite	50.8	500	254
MN 2-1C1	Recycled	Gravel	20.7	490	101
MN 2-1J3	Recycled	Gravel	20.5	490	118
WI 1-1C1	Recycled	Gravel	17.2	490	99
WI 1-1J3	Recycled	Gravel	17.0	490	98
WI 1-2C1	Recycled	Gravel	13.3	490	76
WI 1-2J1	Recycled	Gravel	17.9	490	103
MN 3-1J2	Recycled	Gravel	18.0	430	77
WI 2-1C1	Recycled	Gravel	21.0	430	90
WI 2-2C1	Recycled	Gravel	13.7	430	59
WY 1-1J3	Recycled	Gravel + Ls.	23.8	490	117
WY 1-2J2	Control	Gravel	43.1	430	185

Table 87. Rank by relative shearing strength.

Sample	Sample Type	Coarse Agg.	Comp.-Wtd. Sample C.A. S_s, kg/cm^2	Rank by Relative S_s
CT 1-1C3	Recycled	Trap Rock	208	0.82
CT 1-1J2	Recycled	Trap Rock	178	0.70
CT 1-2C1	Control	Trap Rock	208	0.82
CT 1-2J1	Control	Trap Rock	255	1.00
MN 1-1C1	Recycled	Gravel	112	0.44
MN 1-1J2	Recycled	Gravel	115	0.45
MN 1-2J1	Control	Diorite	188	0.74
KS 1-1J3	Recycled	Limestone	54	0.21
KS 1-2J1	Control	Limestone	89	0.35
MN 4-1C3	Recycled	Gravel	82	0.32
MN 4-1J1	Recycled	Gravel	79	0.31
MN 4-2C2	Control	Dolomite	231	0.91
MN 4-2J3	Control	Dolomite	254	1.00
MN 2-1C1	Recycled	Gravel	101	0.40
MN 2-1J3	Recycled	Gravel	118	0.46
WI 1-1C1	Recycled	Gravel	99	0.39
WI 1-1J3	Recycled	Gravel	98	0.38
WI 1-2C1	Recycled	Gravel	76	0.30
WI 1-2J1	Recycled	Gravel	103	0.40
MN 3-1J2	Recycled	Gravel	77	0.30
WI 2-1C1	Recycled	Gravel	90	0.35
WI 2-2C1	Recycled	Gravel	59	0.23
WY 1-1J3	Recycled	Gravel + Ls.	117	0.46
WY 1-2J2	Control	Gravel	185	0.73

TENSILE STRENGTH ESTIMATIONS

Enclosed are the results of the tensile strength estimations computed for the coarse aggregates contained in the core-half specimens analyzed for Task C of project DTFH61-93-C-00133, "Physical and Mechanical Properties of Recycled PCC Aggregate Concrete."

The projects were ranked according to the tensile strengths of the coarse aggregates using tabulated values for the general rock type categories listed in I. W. Farmer's book - Engineering Properties of Rocks (following the same format as that used for computing the shearing strengths of coarse aggregates). The listed values for the general rock type categories were used along with composition data from Michigan gravel sources to compute estimated tensile strength factors for the coarse aggregates that were identified in the core-half samples.

The tensile strength values for most of the rock types were used as listed. However, for the absorptive gravel carbonate fractions, typically with greater than 2 percent absorption, a 50 percent reduction was applied to reflect performance in a saturated, worst case moisture condition at cracks and joints.

After computing the tensile strength factors for the coarse aggregates, composition-weighted sample coarse aggregate tensile strength values were calculated using the coarse aggregate compositions determined by linear traverse measurements.

Ranking of the projects according to relative tensile strength was accomplished by using the highest composition-weighted sample coarse aggregate tensile strength value in the data set as unity. The projects are ranked by their departures from unity on a scale from 1.0 to 0.0.

Comments

The ranking of the control versus recycled pavements according to the tensile strengths of the coarse aggregates was found to be similar to that of the ranking computed for the shearing strengths of the coarse aggregates. Most of the control pavements with trap rock and limestone or dolomite original coarse aggregate showed relative ranking near unity, whereas the recycled projects ranked much lower, typically less than 0.50, indicating that the recycled concrete has much lower tensile strength than concrete with considerable coarse aggregate content. Refer to table B-3 for the computation of composition-weighted sample coarse aggregate tensile strength, and refer to table B-4 for the rank by relative tensile strength.

Table 88. Computation of composition-weighted sample coarse aggregate tensile strength.

Sample	Sample Type	Coarse Agg.	Sample C.A., %	C.A. S_t Factor, kg/cm^2	Comp.-Wtd. Sample C.A. S_t, kg/cm^2
CT 1-1C3	Recycled	Trap Rock	34.6	350	121
CT 1-1J2	Recycled	Trap Rock	29.7	350	104
CT 1-2C1	Control	Trap Rock	34.6	350	121
CT 1-2J1	Control	Trap Rock	42.5	350	149
MN 1-1C1	Recycled	Gravel	22.9	175	40
MN 1-1J2	Recycled	Gravel	23.4	175	41
MN 1-2J1	Control	Diorite	34.2	300	103
KS 1-1J3	Recycled	Limestone	10.8	250	27
KS 1-2J1	Control	Limestone	17.7	250	44
MN 4-1C3	Recycled	Gravel	16.8	245	41
MN 4-1J1	Recycled	Gravel	16.1	245	39
MN 4-2C2	Control	Dolomite	46.2	250	116
MN 4-2J3	Control	Dolomite	50.8	250	127
MN 2-1C1	Recycled	Gravel	20.7	245	51
MN 2-1J3	Recycled	Gravel	20.5	245	50
WI 1-1C1	Recycled	Gravel	17.2	245	42
WI 1-1J3	Recycled	Gravel	17.0	245	41
WI 1-2C1	Recycled	Gravel	13.3	245	33
WI 1-2J1	Recycled	Gravel	17.9	245	44
MN 3-1J2	Recycled	Gravel	18.0	220	40
WI 2-1C1	Recycled	Gravel	21.0	220	46
WI 2-2C1	Recycled	Gravel	13.7	220	30
WY 1-1J3	Recycled	Gravel + Ls.	23.8	245	58
WY 1-2J2	Control	Gravel	43.1	220	95

Table 89. Rank by relative tensile strength.

Sample	Sample Type	Coarse Agg.	Comp.-Wtd. Sample C.A. S_t, kg/cm^2	Rank by Relative S_t
CT 1-1C3	Recycled	Trap Rock	121	0.81
CT 1-1J2	Recycled	Trap Rock	104	0.70
CT 1-2C1	Control	Trap Rock	121	0.81
CT 1-2J1	Control	Trap Rock	149	1.00
MN 1-1C1	Recycled	Gravel	40	0.27
MN 1-1J2	Recycled	Gravel	41	0.28
MN 1-2J1	Control	Diorite	103	0.69
KS 1-1J3	Recycled	Limestone	27	0.18
KS 1-2J1	Control	Limestone	44	0.30
MN 4-1C3	Recycled	Gravel	41	0.28
MN 4-1J1	Recycled	Gravel	39	0.26
MN 4-2C2	Control	Dolomite	116	0.78
MN 4-2J3	Control	Dolomite	127	0.85
MN 2-1C1	Recycled	Gravel	51	0.34
MN 2-1J3	Recycled	Gravel	50	0.34
WI 1-1C1	Recycled	Gravel	42	0.28
WI 1-1J3	Recycled	Gravel	41	0.28
WI 1-2C1	Recycled	Gravel	33	0.22
WI 1-2J1	Recycled	Gravel	44	0.30
MN 3-1J2	Recycled	Gravel	40	0.27
WI 2-1C1	Recycled	Gravel	46	0.31
WI 2-2C1	Recycled	Gravel	30	0.20
WY 1-1J3	Recycled	Gravel + Ls.	58	0.39
WY 1-2J2	Control	Gravel	95	0.64

COMPRESSIVE STRENGTH ESTIMATIONS

Enclosed are the results of the compressive strength estimations computed for the coarse aggregates contained in the core-half specimens analyzed for Task C of project DTFH61-93-C-00133, "Physical and Mechanical Properties of Recycled PCC Aggregate Concrete."

The projects were ranked according to the compressive strengths of the coarse aggregates using tabulated values for the general rock type categories listed in I. W. Farmer's book - Engineering Properties of Rocks (following the same format as that used for computing the shearing and tensile strengths of coarse aggregates). The listed values for the general rock type categories were used along with composition data from Michigan gravel sources to compute estimated compressive strength factors for the coarse aggregates that were identified in the core-half samples.

The compressive strength values for most of the rock types were used as listed. However, for the absorptive gravel carbonate fractions, typically with greater than 2 percent absorption, a 50 percent reduction was applied to reflect performance in a saturated, worst case moisture condition at cracks and joints.

After computing the compressive strength factors for the coarse aggregates, composition-weighted sample coarse aggregate compressive strength values were calculated using the coarse aggregate compositions determined by linear traverse measurements.

Ranking of the projects according to relative compressive strength was accomplished by using the highest composition-weighted sample coarse aggregate tensile strength value in the data set as unity. The projects are ranked by their departures from unity on a scale from 1.0 to 0.0.

Comments

The ranking of the control versus recycled pavements according to the compressive strengths of the coarse aggregates was found to be similar to that of the ranking computed for the shearing and tensile strengths of the coarse aggregates. Most of the control pavements with trap rock and limestone or dolomite original coarse aggregate showed relative ranking near unity, whereas the recycled projects ranked much lower, typically less than 0.50, indicating that the recycled concrete has much lower compressive strength than concrete with considerable coarse aggregate content. Refer to table B-5 for the computation of composition-weighted sample coarse aggregate compressive strength, and refer to table B-6 for the rank by relative compressive strength.

The similarities in the rankings for shearing, tensile, and compressive strengths of the coarse aggregates using data from the tables in I. W. Farmer's text indicate that the tables are conversions, and that the major rock types are assumed to be similarly related with respect to shearing, tensile, and compressive strength.

Table 90. Computation of composition-weighted sample coarse aggregate compressive strength.

Sample	Sample Type	Coarse Agg.	Sample C.A., %	C.A. S_c Factor, kg/cm^2	Comp.-Wtd. Sample C.A. S_c, kg/cm^2
CT 1-1C3	Recycled	Trap Rock	34.6	3,500	1,211
CT 1-1J2	Recycled	Trap Rock	29.7	3,500	1,040
CT 1-2C1	Control	Trap Rock	34.6	3,500	1,211
CT 1-2J1	Control	Trap Rock	42.5	3,500	1,488
MN 1-1C1	Recycled	Gravel	22.9	2,430	557
MN 1-1J2	Recycled	Gravel	23.4	2,430	569
MN 1-2J1	Control	Diorite	34.2	3,000	1,026
KS 1-1J3	Recycled	Limestone	10.8	2,500	270
KS 1-2J1	Control	Limestone	17.7	2,500	443
MN 4-1C3	Recycled	Gravel	16.8	2,450	412
MN 4-1J1	Recycled	Gravel	16.1	2,450	395
MN 4-2C2	Control	Dolomite	46.2	2,500	1,155
MN 4-2J3	Control	Dolomite	50.8	2,500	1,270
MN 2-1C1	Recycled	Gravel	20.7	2,450	507
MN 2-1J3	Recycled	Gravel	20.5	2,450	502
WI 1-1C1	Recycled	Gravel	17.2	2,450	421
WI 1-1J3	Recycled	Gravel	17.0	2,450	417
WI 1-2C1	Recycled	Gravel	13.3	2,500	333
WI 1-2J1	Recycled	Gravel	17.9	2,500	448
MN 3-1J2	Recycled	Gravel	18.0	2,140	385
WI 2-1C1	Recycled	Gravel	21.0	2,140	450
WI 2-2C1	Recycled	Gravel	13.7	2,140	293
WY 1-1J3	Recycled	Gravel + Ls.	23.8	2,450	583
WY 1-2J2	Control	Gravel	43.1	2,140	922

Table 91. Rank by relative compressive strength.

Sample	Sample Type	Coarse Agg.	Comp.-Wtd. Sample C.A. S_c, kg/cm^2	Rank by Relative S_c
CT 1-1C3	Recycled	Trap Rock	1,211	0.81
CT 1-1J2	Recycled	Trap Rock	1,040	0.70
CT 1-2C1	Control	Trap Rock	1,211	0.81
CT 1-2J1	Control	Trap Rock	1,488	1.00
MN 1-1C1	Recycled	Gravel	557	0.37
MN 1-1J2	Recycled	Gravel	569	0.38
MN 1-2J1	Control	Diorite	1,026	0.69
KS 1-1J3	Recycled	Limestone	270	0.18
KS 1-2J1	Control	Limestone	443	0.30
MN 4-1C3	Recycled	Gravel	412	0.28
MN 4-1J1	Recycled	Gravel	395	0.27
MN 4-2C2	Control	Dolomite	1,155	0.78
MN 4-2J3	Control	Dolomite	1,270	0.85
MN 2-1C1	Recycled	Gravel	507	0.34
MN 2-1J3	Recycled	Gravel	502	0.34
WI 1-1C1	Recycled	Gravel	421	0.28
WI 1-1J3	Recycled	Gravel	417	0.28
WI 1-2C1	Recycled	Gravel	333	0.22
WI 1-2J1	Recycled	Gravel	448	0.30
MN 3-1J2	Recycled	Gravel	385	0.26
WI 2-1C1	Recycled	Gravel	450	0.30
WI 2-2C1	Recycled	Gravel	293	0.20
WY 1-1J3	Recycled	Gravel + Ls.	583	0.39
WY 1-2J2	Control	Gravel	922	0.62

REFERENCES

1. Snyder, M. B., K. D. Smith, J. M. Vandenbossche, and M. Wade, 1994. *Physical and Mechanical Properties of Recycled PCC Aggregate Concrete.* Task A Interim Report (Unpublished), Federal Highway Administration. Washington, DC.

2. Strategic Highway Research Program (SHRP), 1993. *Distress Identification Manual for the Long-Term Pavement Performance Project.* Report No. SHRP-P-338. Strategic Highway Research Program, National Research Council. Washington, DC.

3. Korenev, B. G., 1954. *Problems of Analysis of Beams and Plates on Elastic Foundation.* Gosudarstvennoe Izdatel'stvo Literatury po Stroitel'stvu i Arkhitekture. Moscow, USSR (in Russian).

4. Ioannides, A. M., L. Khazanovich, and J. L. Becque, 1992. "Structural Evaluation of Base Layers in Concrete Pavement Systems." *Transportation Research Record 1370.* Transportation Research Board. Washington, DC.

5. Ioannides, A. M., and L. Khazanovich, 1994. "Backcalculation Procedure For Three-Layered Concrete Pavements." Proceedings, Fourth International Conference on Bearing *Capacity of Roads and Airfields.* Minneapolis, MN.

6. Smith, K. D., M. J. Wade, D. G. Peshkin, L. Khazanovich, H. T. Yu, and M. I. Darter, 1995. *Performance of Concrete Pavements, Volume II—Evaluation of Inservice Concrete Pavements.* Report No. FHWA-RD-95-110. Federal Highway Administration. Washington, DC.

7. Crovetti, J. A., and M. I. Darter, 1985. *Appendix C—Void Detection Procedures.* Appendix to NCHRP Report 281. Transportation Research Board. Washington, DC.

8. Lane, K. R., 1978. *Pavement Recycling—Phase I: Energy, Environmental and Material* Considerations. Connecticut Department of Transportation. Hartford, CT.

9. Lane, K. R., and D. G. Bowers, 1979. *Portland Cement Concrete Pavement* Recycling—*Phase II.* Report No. 79-6. Connecticut Department of Transportation. Hartford, CT.

10. Christman, R., and K. R. Lane, 1979. *Pavement Recycling Bituminous Concrete and Concrete Mix Designs.* Report No. FHWA-CT-79-569-1-10. Federal Highway Administration. Washington, DC.

11. Lane, K. R., 1980. *Construction of a Recycled Portland Cement Concrete Pavement.* Report No. FHWA/CT-80-12. Federal Highway Administration. Washington, DC.

12. Frondistou-Yannas, S., 1977. "Waste Concrete as Aggregate for New Concrete." *ACI Materials Journal.* Volume 74, Number 8. American Concrete Institute. Detroit, MI.

13. Hansen, T. C., and E. Boegh, 1985. "Elasticity and Drying Shrinkage of Recycled Aggregate Concrete." *ACI Materials Journal,* Volume 82, Number 5. American Concrete Institute. Detroit, MI.

14. Mukai, T., T. Kemi, M. Nakagawa, and M. Kikuchi, 1979. "Study and Reuse of Waste Concrete for Aggregate of Concrete." *Proceedings of the Seminar on Energy and Resource Conservation in Concrete Technology.* Japan-US Cooperative Science Program. Washington, DC.

15. Snyder, M. B., and J. M. Vandenbossche, 1993. "New Research and Practice in the Recycling of Concrete." *The Evolving World of Concrete.* Concrete Technology Seminars - 7. Michigan State University. East Lansing, MI.

16. Fager, G., 1986. *Recycling PCC Pavements, Johnson County.* Report No. 7-46-K-0449-02. Kansas Department of Transportation. Topeka, KS.

17. Fergus, J. S., 1980. *The Effect of Mix Design on the Design of Pavement Structures When* Utilizing *Recycled Portland Cement Concrete as Aggregate.* Ph.D. Thesis. Department of Civil Engineering. Michigan State University. East Lansing, MI.

18. Van Krevelen, R. C., 1980. *Recycling D-Cracked Concrete Pavement.* Minnesota Department of Transportation. Maplewood, MN.

19. Halverson, A. D., 1981. *Recycling Portland Cement Concrete Pavement.* Minnesota Department of Transportation. Maplewood, MN.

20. Nelson, L. A., 1981. "Rural Recycling." *Proceedings of the National Seminar on PCC* Pavement *Recycling and Rehabilitation.* Report No. FHWA-TS-82-208. Federal Highway Administration. Washington, DC.

21. Federal Highway Administration, 1990. *Continuously Reinforced Concrete Pavement.* Technical Advisory No. 5080.14. Washington, DC.

22. Kelleher, K. and R. M. Larson, 1989. "The Design of Plain Doweled Jointed Concrete Pavement." *Proceedings, Fourth International Conference on Concrete Pavement Design and Rehabilitation.* Purdue University. West Lafayette, IN.

23. Larson, R. M. and S. D. Tayabji, 1994. "Performance of Continuously Reinforced Concrete Pavements." *Proceedings, Third International Workshop on the Design and Evaluation of Concrete* Pavements. The Netherlands: Centre for Research and Contract Standardization in Civil and Traffic Engineering.

24. Tayabji, S. D., P. J. Stephanos, J. S. Gagnon, and D. G. Zollinger, 1994. *Performance of* CRC Pavements *Volume 2 - Field Investigations of CRC Pavements.* Interim Report. Federal Highway Administration. Washington, DC.

25. Horan, R., 1986. *Recycled Concrete Pavement.* Project Number IR-80-6(91)393. Federal Highway Administration Demonstration Project Number 47. Wyoming Department of Transportation. Cheyenne, WY.

26. Swedeen, K. J., 1989. *Use of Fly Ash in Recycled Concrete Pavement for Control of Alkali-Silica Reaction.* Wyoming Highway Department. Cheyenne, WY.

27. Oyler, W.A., 1989. *Evaluation of Recycled Concrete Pavement.* Wyoming Department of Transportation. Cheyenne, WY.

28. Federal Highway Administration, 1994. *Portland Cement Concrete Mix Design and Field Control.* Technical Advisory No. 5080.17. Washington, DC.

29. Yrjanson, W. A., 1989. *Recycling of Portland Cement Concrete Pavements.* NCHRP Synthesis 154. National Cooperative Highway Research Program. Transportation Research Board. Washington, DC.