PERFORMANCE OF RIGID PAVEMENTS CONTAINING RECYCLED CONCRETE AGGREGATES

BY

JEFFREY R. STURTEVANT

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Thesis Director, David L. Gress, Professor

Jo Sias Daniel, Associate Professor

Charles H. Goodspeed, Associate Professor

Date

DEDICATION

To my mother, father and grandparents, who have always been there with support and guidance throughout my life. To my father and grandfathers, who have shown me for years the excitement and satisfaction that comes from working in the engineering industry.

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- Iowa
- Illinois
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- Wisconsin
- Wyoming

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ABSTRACT

PERFORMANCE OF RIGID PAVEMENTS CONTAINING

RECYCLED CONCRETE AGGREGATES

BY

Jeffrey R. Sturtevant

Thesis Director: David. L. Gress

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With the rising cost and dwindling supply of conventional concrete aggregates, recycled concrete aggregate (RCA) is becoming a viable alternative. A performance study of RCA pavements was done in 1994 on nine different RCA pavement sections with ages ranging from six to fourteen years old. A second study was performed in 2006. In addition to the nine sections studied in 1994, two new RCA pavement sections were analyzed. The purpose of the 2006 study was to revaluate the performance of these aging and highly traveled RCA pavements.

Such factors as ASR, maximum aggregate size, RCA mortar content and load transfer dowels affected pavement performance. Additionally, multiple pavements were rehabilitated since the 1994 study with diamond grinding and retrofitting of dowel bars for load transfer, which had a positive effect on performance. Overall, seven different states built acceptable recycled concrete pavements that performed similar to conventional pavements.

CHAPTER 1

INTRODUCTION

Introduction

Even though sustainability has become one of the largest "green" words in recent years, it is a fact that recycled concrete aggregate (RCA) has been used in the United States for many years now. It has been a popular alternative to purchasing new aggregate and having it transported to the site. This has saved states money because it allows them to reuse old concrete instead of hauling it off site and paying a dumping fee to put it in a landfill, plus the cost of new aggregate. RCA is too good to use as just a base and it brings an added value when it can be effectively used as a substitution for natural aggregates in concrete. From previous studies, it has been found that recycled concrete aggregate bases work just as well as new aggregate bases.[1] Nowadays, with dwindling natural resources and greater concern about the environment, RCA is becoming a more viable alternative in not only pavement bases but also new concrete pavements.

As of 1994, 15 different states had laid multiple roads with recycled concrete pavements. Most recycled pavements have performed well, but others have received national attention for their bad performance. These poor performers, though few, have given all recycled concrete pavements a less than favorable name.

The Federal Highway Administration (FHWA) in 1993 sponsored research to combine field site evaluations with laboratory strength testing, as well as petrographic examinations to investigate why some RCA pavements performed better then others. A study was started in 1994 which included 9 different pavement sections across the United States. These nine projects included in the 1994 study ranged in age from 6 to 14 years. 5 of the 9 sites had both an RCA section and a control section with similar designs.

The FHWA, through the University of New Hampshire Recycled Materials Resource Center (RMRC), sponsored research in 2006 to revisit the 1994 study project sites in order to further evaluate the effectiveness of recycled concrete pavements. In addition to the original 9 sites, two new RCA pavement sites were selected in 2006 to study the performance of RCA pavements.

Project Objectives

Similar to the objectives in the 1994 study, the purpose of the 2006 study was to combine field site evaluations with laboratory durability and strength testing, as well as petrographic examinations on the 9 pavement sections studied in 1994 to determine why some pavements performed better than others. The objective for the two new sites chosen was similar to that of the original 9 pavement sections. The purpose of 2006 study was to also provide a better indication of RCA pavement long-term performance trends and offer further insight into the factors that affect RCA pavement durability and performance.[2]

Each site that had a control section was compared to evaluate the RCA pavement performance. Additionally, the 1994 data and 2006 data were compared to show how well the roads have held up after an additional 12 years of traffic.

Recycled Concrete Aggregate (RCA)

European nations were the first to use RCA. At the end of World War II, quite a few cities were little more than rubble. Countries had to deal with the issue of rebuilding their infrastructure while also finding a suitable place to deposit the rubble.[3] The beginning of RCA usage in the U.S. developed in much the same way. Since most buildings and roads in the US were getting older, they needed to be demolished so that newer and better structures could be built. After demolition, the building and road debris needed to go somewhere. With the increasing costs to transport and landfill the debris, those in the public and private sector must take concrete recycling into consideration.[4] An increase in sustainable construction practices and aggregate prices in the past few years made RCA an even more popular alternative to purchasing new aggregate, for a number of uses. A few examples of uses for RCA are fill, base material, drainage material, noise barriers and road construction.[1] Figure 1 shows the difference between a piece of original aggregate and recycled concrete aggregate (RCA).

Figure 1: Original Aggregate (left) and Recycled Concrete Aggregate (right)

Both aggregates in Figure 1 were retained on a 3/4 inch sieve and are roughly the same volume. The aggregate on the left is new aggregate and has a uniform look to it. The recycled concrete aggregate on the right is not composed entirely of rock and has both smaller pieces of aggregate and mortar attached. The recycled concrete aggregate increases the total mortar content when used in a concrete. This increased mortar content can lead to workability issues and can be minimized by proper crushing techniques. The RCA also demands closer moisture control during mixing and lowers the unit weight of the recycled concrete and typically increases shrinkage.

There are two ways a concrete can be recycled. First, it can be hauled off site to be crushed and screened for reuse. This method is quite popular, even though it does increase transportation costs of the debris. A more sustainable way to recycle concrete is to do it onsite and use it there. This method decreases transportation costs and eliminates wear and tear on trucks and roadways.[4]

Since RCA has a higher absorption and can be more angular than regular aggregates, when used in new concrete, the amount of RCA fines should be limited to give acceptable workability. The substitution of new concrete sand or the addition of more water and cement is a common practice.[5] RCA concrete also does not need as much air entertainment to reach a desired air content as regular concretes. This was concluded by comparing RCA and natural aggregate concrete pavements with the same air entrainment dosage for freeze thaw susceptibility.[3, 6] Since RCA already has entrained and entrapped air in the paste section, it adds to the air entrainment of the concrete. This also means that RCA will typically have a lower bulk specific gravity than a comparable natural aggregate. RCA can also be more susceptible to Alkali Silicate Reaction (ASR) than natural aggregate. If a concrete pavement that already has ASR is recycled and made into RCA, then that new concrete with the RCA may have worse ASR then the original one. When new cement is introduced to an ASR susceptible RCA it supplies more alkalis. An increase in alkalis increases the expansion of an RCA concrete more than one that uses natural aggregate.[5] With all of these differing factors, engineers must learn to adjust mixes so that quality RCA pavements can be produced.

RCA Pavements

RCA has been used for years as a base for new roads. Many states have deemed RCA to be an acceptable natural aggregate substitution, suitable for use in new concrete and asphalt concrete pavements.[3] As of 1994, there were close to 100 different sections of RCA pavement around the United States. Unfortunately, some RCA pavements have failed and this has given RCA pavements a somewhat bad reputation. Such failures usually stem from recurring ASR issues, lack of load transfer devices between slabs, too large of distance between joints and poor mix designs. Since hearing about these failures, states have been concerned about paving new roads with RCA, even though it is more economical and better for the environment then using all natural aggregate.[2]

The purpose of this study is to give states a new outlook on RCA pavements. There are many success stories out there of RCA pavements that have performed as well as or better than regular concrete pavements.

1994 RCA Pavements Study

For the 1994 pavement study 9 different test sections were studied. Project ages ranged from 6 to 14 years at the time of evaluation. At that time there were close to 100 RCA roads in the United States. In order to meet the 1994 project objectives, 9 roads were chosen based on the following factors[2]:

- Pavement Age (Roads were to be 8 or more years old)
- Pavement Type (A balance between JPCP and JRCP roads 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, relative condition of the existing pavement, became the principal factor in choosing the roads to be studied in 1994. Having pavements with either good performance, structural problems or other distresses was desired. This was done so that pavements with a wide range of distresses could be observed and conclusions could be drawn as to why some pavements performed better than others. After investigation of all RCA concrete pavements in the United States for the aforementioned factors, 9 pavements were determined to be suitable to study.[2] These 9 pavements and their factors are summarized in Table 1.

Field Survey

A field survey was done on each of the 21 pavement sections in the 2006 study. The same surveyed areas from pavements studied in 1994 were again used in 2006 so that values from both could be compared.

A contact was made between the Recycled Materials Resource Center (RMRC) and each state's respective DOT before site survey work commenced. The DOTs supplied traffic control on the surveyed lanes for a particular site and a coring rig with workers to perform the coring work.

Table 1: Summary of Selection Criteria for 1994 Pavement Sections

Category	Location	Climatic Region	Age, years	Control Section	2 Way ADT, veh/day	Pavement Type (% long. reinf.)	Joint Spacing, m	Dowel Diam., mm
1 (Good)	CT 1, I-84 near Waterbury	W-F	14	yes	56,000	230-mm JRCP (0.09 %)	12	38 (I-beam)
	MN 1, I-94 near Brandon	W-F Transition	6	yes	8,170	280-mm JRCP (0.054 %)	8.2	32
	KS 1, K-7 in Johnson County	W-F	9	yes	7,310	230-mm JPCP	4.7	none
2 (Structural Problems)	MN 4, U.S. 52 near Zumbrota	W-F	10	yes	7,820	230-mm JRCP (0.065 %)	8.2	25
	MN 2, I-90 near Beaver Creek	W-F Transition	10	no	1,670	230-mm JRCP (0.065 %)	8.2	25
	WI 1, I-94 near Menomonie	W-F	10	no	8,170	280-mm JPCP	3.7-4.0-5.8-5.5	none / 35
3 (Other Distresses)	MN 3, U.S. 59 near Worthington	W-F Transition	14	no	2,150	200-mm JPCP	4.0-4.9-4.3-5.8	none
	WI 2, I-90 near Beloit	W-F	8	no	22,622	250-mm CRCP (0.67 %)	n/a	n/a
	WY 1, I-80 near Pine Bluffs	D-F	9 / 10	yes	4,410 (RCA) 4,280 (Con.)	250-mm JPCP	4.3-4.9-4.0-3.7	none

The sites were surveyed and cored, unless a road was overlaid by asphalt, then only coring was completed. Further description of the field study observations can be found in the field survey section in Chapter 3.

Lab Testing

Various lab tests were done on cores from each of the 21 pavement sections in the 2006 study. The number of cores and locations to take cores were decided upon by the RMRC before arriving onsite. A representative from the RMRC marked areas to be cored with spray paint. The coring rig and workers supplied by each respective DOT then extracted cores. Each core was labeled, sealed, packaged and shipped to the University of New Hampshire for laboratory testing.

Each core was labeled with an ID number and vacuum sealed to keep the core in a constant environment once they arrived at The University of New Hampshire. Laboratory testing was done similar to the 1994 study. Further description of laboratory tests performed on the cores can be found in the laboratory testing section in Chapter 3.

CHAPTER 2

SELECTED SITE LOCATIONS

Introduction

Many factors were evaluated when deciding which recycled concrete pavements would be studied in 2006. Most of the factors used in deciding the roads for the 1994 study were used again in 2006.

At the start of the project, it was decided that the 9 sites studied in 1994 would be revisited in 2006 to determine the deterioration that had occurred. In addition to the 9 original sites, 2 new sites were selected to include in the 2006 study. Factors selected to determine the 2 additional sites included the presence of a control section, pavement type and materials used in the concrete. A summary of the 11 pavement sites studied can be seen in Table 2.

Site Criteria

Climatic Region

Between all of the pavement sites a range of climate was desired. Moisture and temperature are the two most important factors affecting pavement performance in its environment.[7] Knowing the climatic region and typical pavement temperatures are important for back calculating values for Falling Weight Deflectometer testing, as was done in 1994. Distresses can occur in a pavement as a function of increased moisture and

Table 2: Summary of Site Locations for 2006 Pavement Study

Project Location	Route	Site Title	Test Strip Location	Pavement Type
Waterbury, CT	I-84	CT1-1	WB, MP 33.71-33.91	Recycled
		CT1-2	EB, MP 33.94-33.83	Control
Rock Rapids, IA	U.S. 75	IA1-1	n/a	Recycled
		IA1-2	NB, Sta. 1091+00 – 1101+00	Recycled
Effingham, IL	I-57	IL1-1	NB, Sta. 5417+50 – 5427+50	Recycled
		Il1-2	SB, Sta. 5427+50 - 5417+50	Recycled
Johnson Co., KS	K-7	KS1-1	NB, .5 mi. north of 55th St.	Recycled
		KS1-2	SB, 500’ from KS River Bridge	Control
Brandon, MN	I-94	MN1-1	WB, MP 90.9-91.1	Recycled
		MN1-2	WB, MP 87.0-87.2	Control
Beaver Creek, MN	I-90	MN2-1	EB, Sta. 89+90 – 100+16	Recycled
		MN2-2	WB, Sta. 100+00 – 90+00	Recycled
Worthington, MN	U.S. 59	MN3	SB, MP 27.00	Recycled
Zumbrota, MN	U.S. 52	MN4-1	NB, Sta. 983+88 – 994+14	Recycled
		MN4-2	NB, Sta. 1035+01 – 1045+27	Control
Menomonie, WI	I-94	WI1-1	EB, MP 39.6-39.8	Recycled
		WI1-2	EB, MP 40.1-40.3	Recycled
Beloit, WI	I-90	WI2-1	WB, MP 176.8-177.0	Recycled
		WI2-2	WB, MP 176.2-176.4	Recycled
Pine Bluffs, WY	I-80	WY1-1	EB, starts 130’ ft. east of MP 400	Recycled
		WY1-2	WB, ends 159’ W of WY-NE Border	Control

temperature. For example, concrete pavements that are exposed to desert like conditions (high heat and low moisture) experience an increase in transverse fatigue cracking due to positive temperature gradients.[7] Due to thermal contraction, concrete pavements will have less aggregate interlock between joints during the winter, which in turn will lead to a decrease in load transfer capability. Conversely, if a pavement is in a cooler environment it can experience an increase in longitudinal cracking due to negative temperature gradients.[7] Pavements in colder environments can be susceptible to freeze thaw issues as well.

The 11 pavement sites studied were either in a dry freeze (D-F) or wet freeze environment (W-F). The northernmost pavement studied was I-94 in Brandon, MN and the southernmost pavement was K-7 in Johnson County, KS. K-7 is in a D-F region while all other pavement sites were in a W-F region. Each pavement's climate region is presented in Table 3.

Pavement Age

The age of a pavement is a big factor in its performance. As a pavement gets older, its ability to withstand load decreases and its likelihood of developing cracks and faulting increases. When jointed plain concrete pavements rely solely on aggregate interlock at their joints, their load transfer capacity will decrease over time. Longitudinal and transverse joint seal damage is also an issue as the jointing material ages due to weathering.

In the 1994 study, pavements were selected that were at least 8 years of age. In 2006 these pavements were another 12 years older and consequently were expected to show more signs of aging. Pavement ages of the 11 sites are given in Table 3.

Table 3: Summary of 2006 Pavement Sites

Site Title	Construction Date	Climate Region
CT1-1	1980	W-F
CT1-2	1980	W-F
IA1-1	1976	W-F
IA1-2	1976	W-F
IL1-1	1986	W-F
Il1-2	1986	W-F
KS1-1	1985	W-F
KS1-2	1985	W-F
MN1-1	1988	W-F*
MN1-2	1988	W-F*
MN2-1	1984	W-F*
MN2-2	1984	W-F*
MN3	1980	W-F*
MN4-1	1984	W-F
MN4-2	1984	W-F
WI1-1	1984	W-F
WI1-2	1984	W-F
WI2-1	1986	W-F
WI2-2	1986	W-F
WY1-1	1985	D-F
WY1-2	1985	D-F

Note: * W-F Transition Zone

One of the additional sites, Iowa US-75, was partially chosen for its increased age. The pavement is one of the oldest RCA pavements in the US, being constructed in 1976. Furthermore, the recycled concrete aggregate used in its construction came from a pavement that was places in 1935. Table 3 shows the construction dates for each of the pavements in the 2006 study.

Pavements can be rehabilitated through diamond grinding of the surface and retrofitting of dowel bars over original aggregate interlock joints. This was seen at the Wyoming I-80, Minnesota US-52 and Minnesota US-59 pavement sites. As a result of rehabilitation, some of these pavements actually had a better ride quality in 2006 then they did in 1994. Minnesota US-59 actually won an award for its exceptional smooth ride.

Mix Design

Mortar Content

Adjusting the amount of mortar in a recycled concrete pavement will alter its performance. Mortar content affects a concretes workability, strength, shrinkage, coefficient of thermal expansion and modulus of elasticity.[8] This is an important factor to consider when designing recycled concrete pavements because RCA can have extensive old mortar bonded to it. It is a good idea to minimize the amount of mortar stuck to RCA to produce an RCA pavement that performs comparably to a similar natural aggregate pavement.[8] Proper crushing of old concrete to reduce the amount of mortar stuck to the old aggregate is the easiest way to minimize the mortar content.

The Wyoming and Connecticut control and recycled sections both contained similar amounts of mortar (<10% difference). This shows that the crushing method for the recycled concrete was successful in removing most of the mortar from the recycled concrete aggregate.[8] Since Connecticut used a gyratory crusher and Wyoming used a jaw crusher, this shows that both types of crushing can accomplish the task of removing most of the recycled mortar from the recycled aggregate. Both the Connecticut and Wyoming control and recycled sections showed similar strength and performance in the 1994 study.

Aggregates

Grading

Grading of RCA and virgin aggregate is an important factor in the design of recycled concrete pavements. Specific things such as coarse aggregate top size, proportion of RCA fine aggregate and fineness modulus all go into how an RCA pavement performs. For example, a larger top size will reduce the amount of water required for a mix and will in turn reduce the cement content.[9] A larger maximum aggregate size will also reduce shrinkage cracking and moisture content in concrete pavements.[10] With an increase in fineness modulus, a mix will need additional fine aggregate to produce a concrete with the same workability (slump).[9] The absorption capacity of recycled fines is more than natural fines because the recycled fines also contain mortar, which has a higher absorption than aggregate. Consequently, the amount of RCA fine aggregate used in a pavement should be minimized to produce a concrete with good workability.

RCA with Alkali Silicate Reaction (ASR)

Some of the pavements studied used aggregate that was found to have ASR. Left untreated, an ASR susceptible aggregate (or in the case of a recycled pavement, ASR susceptible recycled concrete aggregate) may deteriorate over time. Some known remedies for controlling ASR in concrete are to use non-reactive aggregates, limiting alkali loading, adding lithium or pozzolanic material or replacing 30% of aggregate with crushed limestone.[10]

In 1987 the Wyoming DOT decided to recycle a stretch of concrete pavement that was known to have ASR. In the new pavement they specified new limestone, low alkali cement and class F fly ash along with the recycled concrete aggregate.[11]

Fly Ashes

Fly ashes, especially class F, lower the pH of a concrete mixture and make it more resistant to sulfate attack and ASR. They also increase the strength of a concrete and will give it better workability. If a fly ash is added into the mix design then the w/c ratio can be decreased while maintaining constant slump.[12] Fly ashes also will decrease shrinkage cracks in concrete pavements.[10]

Pavement Design

Pavement Type

There were three types of pavements studied; jointed plain concrete pavement (JPCP), jointed reinforced concrete pavements (JRCP) and continuously reinforced concrete pavement (CRCP). JPCP is the least costly type of pavement to construct because it does not have temperature or reinforcing steel in its panels. The jointed plain concrete pavements chosen for study in 2006 were KS1, MN3, WI1 and WY1. Of these pavements, only WI1-2 had load transfer dowel bars originally placed between slabs. JRCP is similar to JPCP, except wire mesh is used to reinforce the slabs. As a result of the added steel, JRCP joint spacing can be increased from that of JPCP.[13] Of the pavements chosen for study in 2006, CT1, MN1, MN2, MN4 and IA1 were JRCP's.

Unlike JPCP and JRCP, CRCP is continuously reinforced and does not have transverse joints. From its initial design in 1921, CRCP's were popular because they eliminated the need for transverse joints, which were thought to cause pavement issues.[13] After the placing of a CRCP, small transverse cracks form in the pavement every meter or so. Due to the presence of strong reinforcing steel, as long as these cracks form close to one another, they will pose no issue to ride quality or allow intrusion of debris.[13] Of the pavements chosen for the 2006 study only WI2 and IL1 were JRCP's.

Joint Spacing

Proper joint spacing in JPCP and JRCP is important for good load transfer and preventing transverse cracking.[13] JPCP joint spacing is typically between 4.6 and 9.1 meters while JRCP can have greater joint spacing, typically between 9 and 30 meters.[13] Joint faulting and transverse cracks are common with JPCP, especially if a pavement's joint spacing is long and it solely relies on aggregate interlock for load transfer between slabs. For jointed pavements, the general rule is that the ratio of slab length, L, to radius of relative stiffness, ℓ, (L/ℓ) should be more than 5.[13] From the 1994 study, it was found that acceptable panel lengths were those that had an L/ℓ greater than 4 for a stabilized base and L/ℓ greater than 6 for a granular base.[8]

Of the sites studied in 2006 there was a variety of joint spacings. Some pavements even had skewed joints with random or equal joint spacing. A summary of joint spacings for each site can be found in Table 4.

Transverse Dowels

While not required in JPCP, transverse dowels will greatly increase proper load transfer between slabs and decrease slab faulting.[2] Since JRCP's have larger joint spacings, transverse dowel bars are required.

Most of the JPCP pavements studied in 1994 did not have transverse dowels, with the exception of WI1-2. Both of the WI1 RCA pavements were designed similarly except WI1-1 did not have transverse dowel bars. From the 1994 study, the difference between the faulting of WI1-1 and WI1-2 was found to be quite large.[2] In the 2006 study, some of the JPCPs were retrofitted with dowel bars to improve load transfers between slabs. This was observed to have a good effect on ride quality.

Table 4: Summary of Primary Pavement Features for 2006 Sites

Site Title	Joint Spacing (m)	Slab (cm)	Dowel Diameter (mm)	Agg. Top Size (mm)	RCA Fines (%)	w/cm	Base** (cm)	Shoulder Type
CT1-1	12	23	38 (I-beam)	38	20	0.4	25	AC
CT1-2	12	23	38 (I-beam)	51	0	0.45	25 and 46	AC
IA1-1	6.1	23	None	38	16	0.54	15	PCC
IA1-2	6.1	23	None	38	30	0.54	15	PCC
IL1-1	CRCP	25	n/a	38	35	0.37*	18	AC
Il1-2	CRCP	25	n/a	38	36	0.40*	18	AC
KS1-1	4.7	23	None	19	25	0.41	10 CTB	AC
KS1-2	4.7	23	None	38	0	0.41	10 CTB	AC
MN1-1	8.2	28	32	19	0	0.47	15	AC
MN1-2	8.2	28	32	19	0	n/a	15	AC
MN2-1	8.2	23	25	19	0	0.46	8	AC
MN2-2	8.2	23	25	19	0	0.46	8	AC
MN3	4.0-4.9-4.3-5.8	20	None	19	0	0.44	3	AC
MN4-1	8.2	23	25	38	0	0.44	13	AC
MN4-2	8.2	23	25	25	0	0.47	13	AC
WI1-1	3.7-4.0-5.8-5.5	28	None	38	0	n/a	15 over 23	PCC
WI1-2	3.7-4.0-5.8-5.5	28	35	38	0	n/a	15 over 23	PCC
WI2-1	CRCP	25	n/a	38	0	n/a	15 over 23	PCC
WI2-2	CRCP	25	n/a	38	0	n/a	15 over 23	PCC
WY1-1	4.3-4.9-4.0-3.7	25	None	38	22	0.38	10	PCC
WY1-2	4.3-4.9-4.0-3.7	25	None	25	0	0.44	n/a	PCC

Note: * Water/(Cement + Fly Ash) Ratio
** Aggregate base unless otherwise specified

CHAPTER 3

METHODS

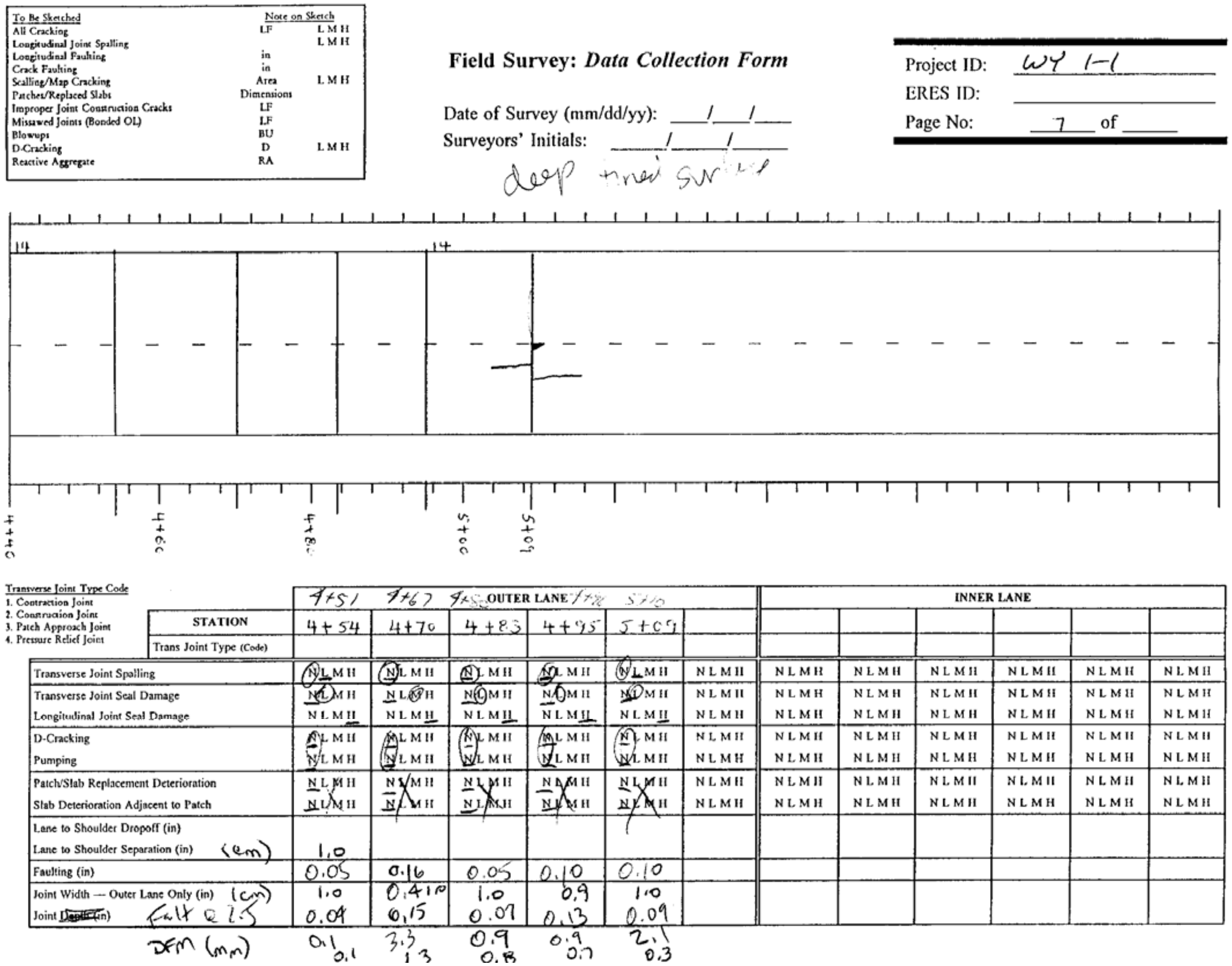
Introduction

This research investigated various properties of RCA pavements in the field and laboratory. The field survey followed Federal Highway Administration (FHWA) guidelines contained in Reference 14. In the lab, ASTM procedures were followed for the preparation, handling, storage and testing of all concrete cores. Typical field survey observations were made and recorded on standardized field survey data collection forms. Laboratory testing was done similar to the 1994 study in order to gain a better understanding of the properties of RCA pavements over time.

Testing Overview

A general number of cores to be extracted and any special precautions were decided upon by all parties involved in the study before any site visits took place. For the 2006 study, the test sites that were part of the 1994 study were used. The two new additional sites were later determined. For divided highways the test strip was always the outside lane.

All State DOT's provided traffic control for each test area to ensure the safety of the survey crew. An example of a survey data sheet is presented in Figure 2. Information from the roads in 1994 were checked and updated on the survey sheets if needed (i.e. a transverse crack in 1994 was determined to be of low severity, but in 2006 it had developed to high severity).



To Be Sketched	Note on Sketch	
All Cracking	LF	L M H
Longitudinal Joint Spalling		L M H
Longitudinal Faulting	in	
Crack Faulting	in	
Scaling/Map Cracking	Area	L M H
Patches/Replaced Slabs	Dimensions	
Improper Joint Construction Cracks	LF	
Missawed Joints (Bonded OL)	LF	
Blowups	BU	
D-Cracking	D	L M H
Reactive Aggregate	RA	

Field Survey: ***Data Collection Form***

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

Surveyors' Initials: ___/___/___

deep tined surface

Project ID: WY 1-1

ERES ID:

Page No: 7 of

14

14-

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

OUTER LANE 4+51 4+67 4+8[illegible] [illegible]

STATION	4+54	4+70	4+83	4+95	5+09	
Trans Joint Type (Code)						
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
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
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
D-Cracking	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
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
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
Lane to Shoulder Dropoff (in)						
Lane to Shoulder Separation (in) (cm)	1.0					
Faulting (in)	0.05	0.16	0.05	0.10	0.10	
Joint Width — Outer Lane Only (in) (cm)	1.0	0.41P	1.0	0.9	1.0	
Joint Depth (in) Falt @ 2.5	0.04	0.15	0.07	0.13	0.09	
DFM (mm)	0.1 / 0.1	3.3 / 1.3	0.9 / 0.8	0.9 / 0.7	2.1 / 0.3	

INNER LANE

Figure 2: Typical Survey Data Sheet

The same methods used in 1994 for measuring and recording pavement distress were used in the 2006 study. While 6” cores were requested, some DOT’s only had 4” or 4.75” core barrels. Due to the variation in core diameters, some alterations had to be made for laboratory testing. Any modifications made are described in their respective description section.

Pavement Survey

Transverse Joint Spalling

Transverse joint spalling was noted on the field survey collection form. Transverse joint spalling consists of damage that is close to the transverse joint such as cracking, breaking and chipping.[14] The amount and degree of the spalling at each transverse joint was recorded as low, medium or high severity. An example of high severity joint spalling is shown in Figure 3. If no spalling was present then non-existent was recorded. If a patch was put over the joint then this was also noted on the form. The condition of the patch was noted in the patch/slab deterioration section.[14] This was done visually and if severe spalling was present then it was documented on the form and a photograph was taken. Transverse joint spalling in the other lanes of a two or more lane road would only be noted if severe issues were present.

Transverse Joint Seal Damage

Transverse joint seal damage was noted as present if the joint seal allowed any foreign objects, such as water or sand to enter. Some examples of damage include grass growing through the joint and the extrusion, splitting or the absence of joint material itself.[14]

Figure 3: Severe Transverse Joint Spalling

Similar to transverse joint spalling, the amount and degree of damage was recorded as low, medium or high severity. If the seal had totally disintegrated or if the joint was never sealed then it was noted on the form. This test was done visually and if severe joint damage was present then it was documented on the form and a photograph was taken. A couple photographic examples of transverse joints with seal damage are shown in Figure 4.

Figure 4: Transverse Joints with Seal Damage

Both pavements shown in Figure 4 were photographed during the summer with temperatures ranging between 70-95 degrees. The joint of the left is doweled and the joint on the right is undowled. If a photograph was taken then a nickel or a quarter was typically used to give a reference size. Transverse joint seal damage in the other lanes of a two or more lane road would only be noted if severe issues were present.

Longitudinal Joint Seal Damage

Similar to transverse joint seal damage, longitudinal joint seal damage was noted if the seal was missing pieces, split or not bonded tight to the pavement allowing foreign objects to enter. Some examples of damage include grass growing through the joint and the extrusion, splitting or the absence of joint material itself.[14] For divided highways the longitudinal joint studied was the one between the slow lane and the next lane over. For two lane highways the longitudinal joint studied was in between the test strip and the opposing traffic lane. The amount and degree of damage was recorded as low, medium or high severity. If the seal had totally disintegrated or was not present then it was noted on the form. This was done visually and if severe joint damage was present then it was documented on the form and a photograph was taken. A photographic example of a longitudinal joint with seal damage is shown in Figure 5.

Figure 5: Longitudinal Joint with Seal Damage

Since the longitudinal joints studied bordered on lanes that did not have traffic control a safe and quick observation had to be made. If the road had more than 2 lanes, the other longitudinal joints were noted only if severe issues were present.

D-Cracking

D (Durability)-cracking usually occurs near cracks, joints or the edge of a pavement slab and will usually propagate from a corner. These hairline cracks typically are crescent shaped or in the shape of a "D" and have a darker color to them.[14] If D-cracking was present then a sketch was be made on the affected area on the road plan. The severity of the D-cracking would then be noted as either low, medium or high. An example of D-cracking is shown in Figure 6.

Figure 6: Severe D-Cracking at a Transverse Joint

Pumping

Pumping occurs when a crack in a pavement allows water and fines from below to be pushed up through it and onto the surface of a pavement. These fines or a stain left behind from them can sometimes be seen around the crack on the pavement surface.[14] If pumping was found it was be noted on the road plan alongside the crack at which it occurred. The severity of the pumping was then noted as either low, medium or high.

Slab/Patch Deterioration

Slab/Patch deterioration occurs when a patch that was placed on part or all of a concrete slab starts to show signs of wear.[14] For this study there were a couple ways of surveying this. First, if a patch was put down and noted in the 1994 study then it was analyzed for distress in 2006. Second, if a patch was new to the 2006 study then it was sketched out on the road plan and then evaluated for distress. The degree of distress for both cases would be recorded as low, medium or high. An example of a pavement patch with high distress is shown in Figure 7.

Figure 7: Slab Patch Intersection with Distress

Lane to Shoulder Drop off

Lane to shoulder drop off was measured as the vertical distance between the surface of a pavement slab and the surface of the adjoining shoulder.[14] This value was measured using a crack comparator card, as seen is Figure 8.

Figure 8: Crack Comparator Card

A lane to shoulder drop off measurement was usually taken once every 3 or 4 slabs or whenever a severe drop off was noticed. Figure 9 shows a severe drop off.

Figure 9: Severe Lane to Shoulder Drop off

The shoulder and pavement in Figure 9 both had the same aggregate base. The measurement was typically recorded to the nearest tenth of a centimeter.

Lane to Shoulder Separation

The distance between the edge of the shoulder to the edge of the pavement was recorded as the lane to shoulder separation value.[14] Similar to the lane to shoulder drop off, this value was measured with a crack comparator card. A lane to shoulder separation measurement was usually taken once every 3 or 4 slabs, along with a lane to shoulder drop off reading, or when a large separation was noticed. The measurement was typically given to the nearest tenth of a centimeter. Some typical lane to shoulder separations are shown in Figure 10.

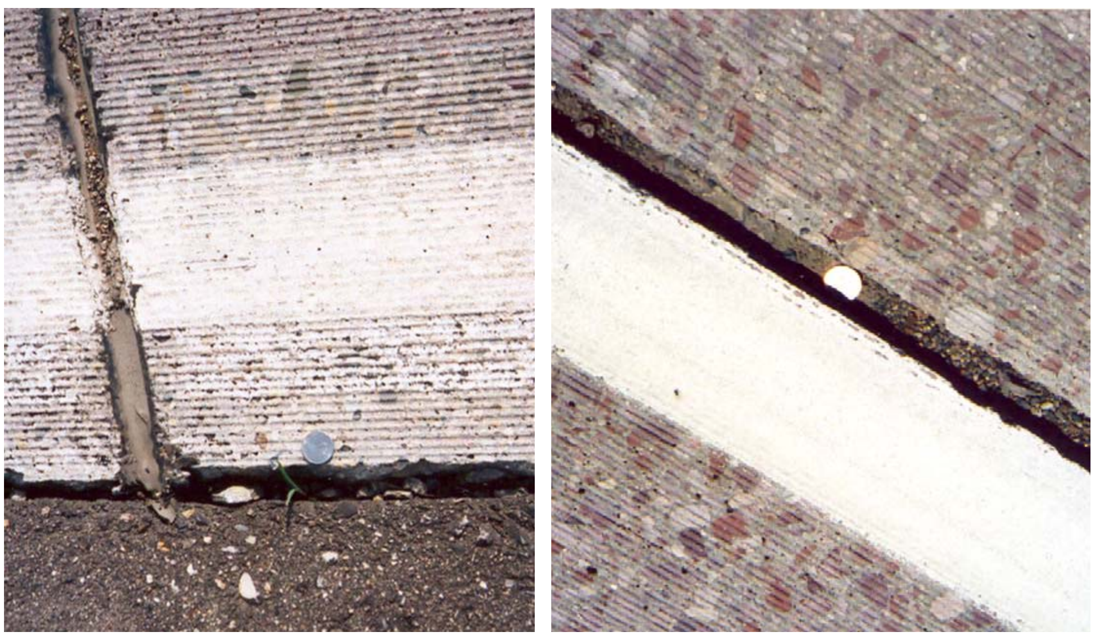


Figure 10: Typical Lane to Shoulder Separations

Faulting Between Panels

The faulting between panels was measured as the vertical distance between the surfaces of two slabs, measured over their common transverse joint.[14] Figure 11 shows an example of extremely large faulting between panels. The white arrow represents the direction of traffic. Some lateral movement over the longitudinal joint can also be seen.

Figure 11: Large Faulting Between Panels

The faulting between panels value was measured in the outer wheel path of the lane (typically 30 – 45 cm from the outer edge of the pavement). Each transverse joint between panels was measured. A value was given to the nearest tenth of a millimeter. To measure this value, a Georgia Faultmeter was used, as shown in Figure 12.

Figure 12: Georgia Faultmeter

Before testing commenced onsite, the Georgia Faultmeter was zeroed out on a calibrated block. When measuring with the fault meter, if the front slab was higher then the back slab a positive faulting value was recorded, vice versa if the front slab was lower. A few tests were taken at each joint and were averaged together. If a joint was patched or otherwise destroyed, then no readings were taken and the issue was noted.

Joint Width

The joint width was measured as the horizontal distance between the transverse edges of two abutting slabs.[14] Similar to the faulting between panels measurement, this value was measured in the outer wheel path of the lane (typically 30 – 45 cm from the outer edge of pavement). A crack comparator card was used to measure this value. Each transverse joint between panels was measured. A value was given to the nearest tenth of a centimeter. Figure 13 shows an example of a typical transverse joint width.

Figure 13: Typical Transverse Joint Width

If a joint was patched or otherwise destroyed, then no reading was taken and the observation was noted.

Longitudinal Cracking

Longitudinal cracks are cracks that generally run parallel to the flow of traffic or a roads centerline.[14] The severity of a longitudinal crack was recorded as low, medium or high. The crack's length and position was estimated and sketched on a road plan supplied in the survey sheet. Figure 14 shows an example of a medium severity longitudinal crack.

Figure 14: Medium Severity Longitudinal Crack

If a longitudinal crack was noticed on a slab it was checked against the survey book to see if it was present in 1994. If a crack was present in 1994 then it was checked to see if it grew in length and if its severity level had changed. If either had changed it was noted on the data form. If not present in 1994, the crack was sketched and labeled with the observed severity level. If a patch was put over the crack then it was noted on the form. The condition of the patch was noted in the patch/slab deterioration section. If severe longitudinal cracking was present then it was documented by taking a photograph. If a road had more than 2 lanes then longitudinal cracking was noted only if severe cracking was present.

Transverse Cracking

Transverse cracks are cracks that generally run perpendicular to the flow of traffic or a roads centerline.[14] The severity of a transverse crack was recorded as low, medium or high. The cracks length and position was estimated and sketched on a road plan supplied in the survey sheet. Figure 15 shows an example of a high severity transverse crack.

Figure 15: High Severity Transverse Cracks

If a transverse crack was noticed on a slab it was checked against the survey book to see if it was present in 1994. If a crack was present in 1994 then it was checked to see if it grew in length and if its severity level had changed. If either had changed it was noted on the data form. If not present in 1994, the crack was sketched and labeled with a severity level. If a patch was put over the crack then it was noted on the form. The condition of the patch was noted in the patch/slab deterioration section. If severe transverse cracking was present then it was documented by taking a photograph. If a road had more than 2 lanes then transverse cracking was noted only if severe issues were present.

Present Serviceability Rating (PSR)

The Present Serviceability Rating (PSR) is an empirical value given to the feel of a road when driving at a rate of 55 miles per hour (MPH). A pavement is rated on a scale of 0 to 5, 0 standing for impassable pavements and 5 is for a perfect pavement.[13] For each test section there were at least 3 people rating it. A line was marked at the start and end of the section so that the judges could know which section to analyze. Once the vehicle hits the start it is kept at 55 MPH under cruise control until the end. Each judge's value was then collected and the average taken. This average value was recorded as the PSR.

International Roughness Index (IRI)

The International Roughness Index (IRI) is a value given to a pavement that defines its roughness. This index was created by the World Bank in the 1980's.[15] A vehicle or trailer is equipped with equipment such as lasers, GPS, transducers or ultrasonic sensors to measure the roughness of a road due to joint faulting, cracking, etc.

The sum of the suspension movements of the testing vehicles is recorded. This value leads to the average rectified slope (ARS), which is the filtered ratio of the testing vehicle's accumulated suspension movement (typ. in. or mm) divided by the length of road traveled during the test (ft or m).[15]

Since IRI was not done in the field during the 2006 study, a correlation developed by the Minnesota Department of Transportation (MNDOT) between PSR and IRI was used to convert from one to the other. MNDOT did field studies to determine a direct correlation between PSR and the IRI (as measured with their Pathways van).[16] They found that IRI can be related to PSR with the equation:

$$IRI = ((PSR-6.6341)/-2.813)^2$$

Figure 16 shows a scale for how IRI values can be interpreted.

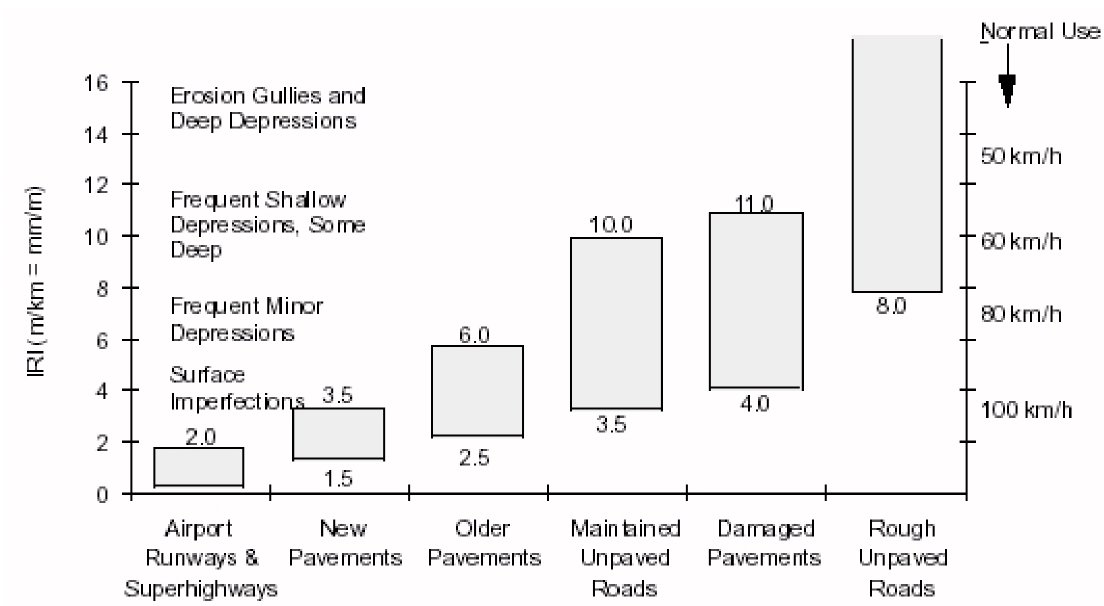


Figure 16: IRI Roughness Scale[15]

Core Extraction

Pavement cores were taken along each of the test strips studied. Each state DOT provided a coring rig and crew to extract cores from the pavement sections and did so pro bono. Similar to the 1994 study, these cores were taken at various locations along the test strip (i.e. mid-panel, joint). For sections tested in 1994, the locations of the 2006 cores were selected to be as close to those of the 1994 study as possible. The coring locations were marked with spray paint and were noted on the road plan. Crew members extracted cores while the pavement survey data were recorded. Figure 17 shows crew members from the Wyoming Department of Transportation extracting a core.

Figure 17: WYDOT Crew Members Extracting a Pavement Core

The cores ranged in diameter from 4” to 6”. The lengths of each core also varied with pavement depths.

After each core was extracted it was labeled and bagged to minimize moisture loss. Since there was no easy way of determining rebar location in the slabs, some of the extracted cores contained rebar. After all site work was completed the cores were packaged to minimize damage and shipped to The University of New Hampshire for laboratory testing.

Laboratory Work on Pavement Cores

Core Sealing

Once the cores from each site arrived at UNH they were vacuum sealed to keep them at a constant environment. Each core was wrapped with bubble warp to keep sharp edges from breaking the vacuum seal. Cores were then inserted into a vacuum bag and vacuum sealed to 99.9% in a General Services Incorporated MVS 45 industrial vacuum sealer. A core set up to undergo vacuum sealing is shown in Figure 18.

Figure 18: Core in Vacuum Sealing Machine

Core extraction information was gathered from all of the sites and an ID number was assigned to each core. A total of 112 cores were extracted from the 11 sites. Each core was assigned an ID number. A list of cores extracted is presented in the Appendix.

Core Testing

ASTM C 496 (Splitting Tension Testing)

A cylinder is laid on its side and two 0.25" thick strips of plywood are put above and below it. A compressive force is then applied to the top side and bottom side of the cylinder. By applying the load this way the vertical section of the cylinder was put in pure tension.[17] Figure 19 shows a cylinder set up and ready to undergo testing on an hydraulically controlled INSTRON® testing machine.

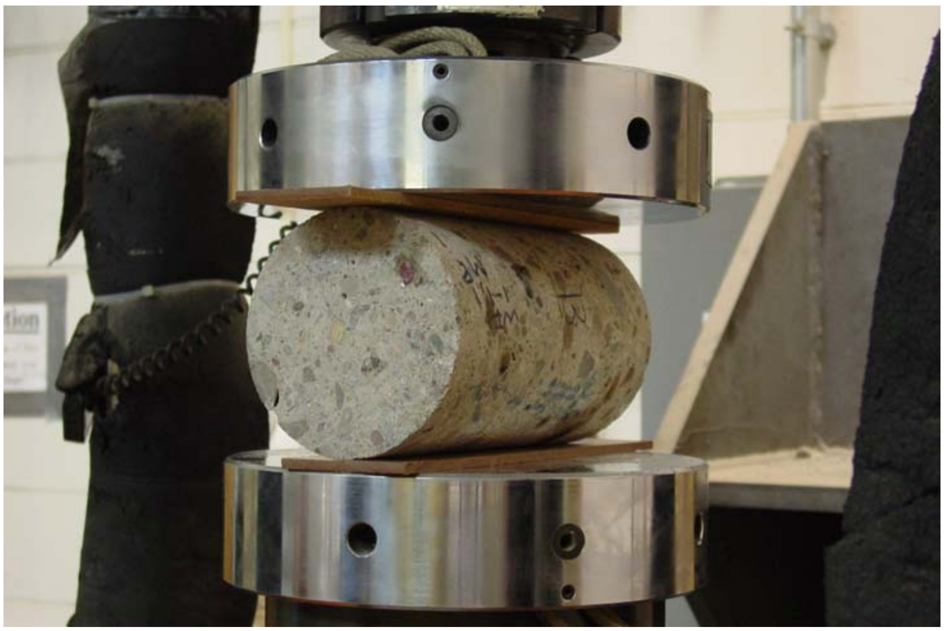


Figure 19: Core set up for Splitting Tensile Strength Testing

Before splitting tension testing commenced, all of the cores were taken out of their vacuum sealed bags. The ends of each core were cut with a concrete saw so that they were flat and perpendicular to the sides. After cutting the cores they were put into a curing room at 20°C and 100% relative humidity for at least 2 days to normalize the amount of moisture on their outer surface, as shown in Figure 20.

Figure 20: Cores in Curing Room

Once the cores were done sitting in the fog room they were taken out and their diameter and length were measured using a caliper, as shown in Figure 21.

Figure 21: Measuring the Diameter of a Cylinder with a Caliper

Once a core's dimensions were recorded it was then placed into the loading machine. An average rate of loading of 175 lb/sec was utilized to conform to the ASTM specifications (11,500 – 23,000 Pa/sec).[17] The cores were loaded until ultimate failure. Once failed, the maximum load attained was recorded and the splitting tension value for each core was calculated. The pieces from the broken core were saved and used for uranyl acetate testing.

ASTM C 39 (Compression Testing)

A cylinder is set upright on top of a neoprene padded metal base. This padded metal base was leveled on top of a ball and socket apparatus. A neoprene padded cap was then placed on top of the cylinder. Figure 22 shows a cylinder set up and ready to undergo compression testing on a hydraulic 300 kip capacity Young® testing machine.

Figure 22: Core set up for Compression Testing

Before compression testing commenced all of the cores were taken out of their vacuum sealed bags, ends cut with a concrete saw so that they were flat and perpendicular to the sides and then placed into a curing room for more than 2 days to normalize the amount of moisture on their outer surface. A compressive axial force was applied to the cylinder using a testing machine. An average rate of loading of 15,000 Pa/sec was used when loading the cylinders, per ASTM specifications.[18] The cores were loaded until ultimate failure. Once failed, the maximum load attained was recorded. The compressive strength value for each core was then determined. A correction factor was applied to the compressive strength for cores with a length to diameter ratio other than 2:1, as per ASTM C39.

ASTM C 856 (Uranyl Acetate)

The uranyl acetate test is a test used to tell if a concrete has Alkali Silica Reaction gel. The test is very useful in that the test itself only takes a couple minutes to perform and can be done in the laboratory or out in the field. When applied to concrete adsorbs into the surface of silica, a component of ASR gel. The uranyl acetate glows a neon green when introduced to ultraviolet (UV) light. Even though uranyl acetate only emits low radioactivity, this test itself is not allowed by many agencies.

The uranyl acetate testing was performed after the splitting tension test so new fractured surfaces could be evaluated. Research shows that smooth surfaces or saw cut surfaces are not good to use for uranyl acetate testing because the gel gets removed and or smeared across a prepared surface.[19]

Before testing started on the fractured face of a core, the piece was lightly wetted with tap water. The uranyl acetate was then sprayed onto the wetted surface and was allowed to set for 3-5 minutes, prior to being flushed with water. Under UV light, areas on the fractured face that produced a green or bright yellow color contained ASR gel.

Any naturally fluorescent aggregates were noted. A rating system was created to give a rating for the different intensities of aggregate and crack light up (Low, Medium or High). Figure 23 shows this rating system.

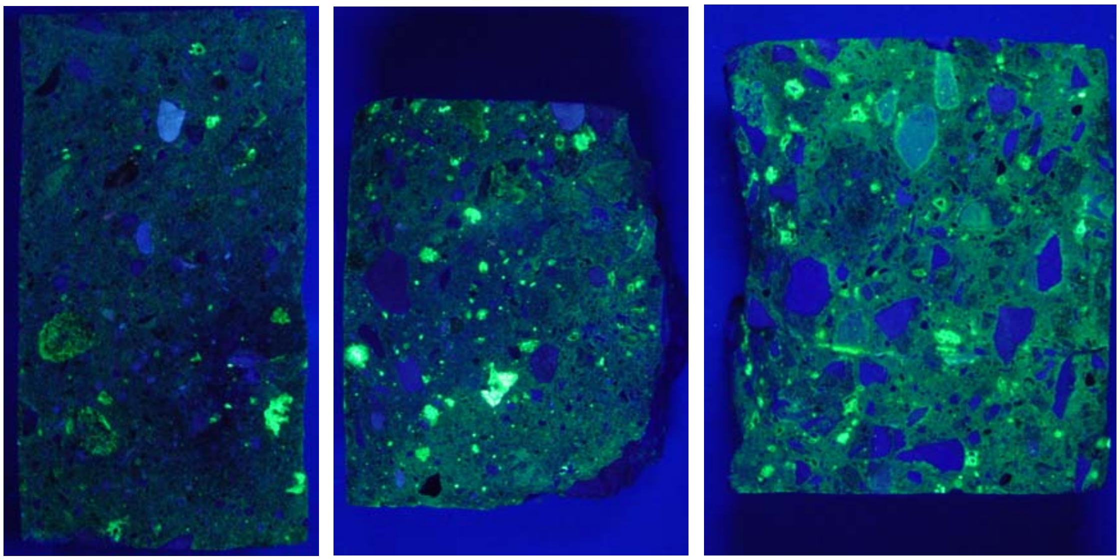


Figure 23: Uranyl Acetate Rating System, Low (left), Medium (center) and High (right)

Modified ASTM C 1293 (Electric Cylinder)

ASTM C 1293 is a common test used to test concrete for ASR susceptibility. A molded concrete prism is placed into a sealed container that keeps the specimen at 100% humidity. The sealed containers are stored in an oven at 38 °C.[20]

Since the ASTM C 1293 test states that cast concrete prisms should be used for testing, some modification had to be made for the pavement cores. First, cores longer than 25 cm had to be cut down in order to properly stud them as the stud fitting jig was designed to create specimens 29.5 cm long, to fit a standard dilatometer. The jig is composed of a base and 2 side metal pieces that are attached with screws so that a core can be extracted after studding. Each side has a similar bushing where the stubs are stuck through to keep them parallel inside the core. The distance from edge of bushing to edge of bushing is exactly 29.5 cm. Figure 24 shows a core being studded inside of the jig.

Figure 24: Core Studding Jig

A 0.95 cm wide by 1.9 cm deep hole was drilled into the center of each end, then the concrete cores were placed in the jig and two studs of proper length were grouted into the ends. Studs were threaded on one end to assure bonding when grouted into the cylinder. The jig was set up so that the length from end of stud to end of stud was exactly 29.5 cm. A studded core can be seen in Figure 25.

Since expansion occurs over at a slow rate with the ASTM C 1293 test a 1 milliamp electrical current was introduced to the cylinders once testing started. Studies have shown that when ASR susceptible concrete is introduced to an electrical current expansion rates increase.[5] The expansion rate increase is due to the migration of hydroxyl ions. With the addition of a small electrical current, the hydroxyl ions are better able to infiltrate into reactive aggregate and accelerate expansion.[5]

Figure 25: Studded Core for Modified ASTM C 1293 Testing

To promote a good flow of electrical current, the two ends of each core were painted with conductive carbon paint, as shown in Figure 26.

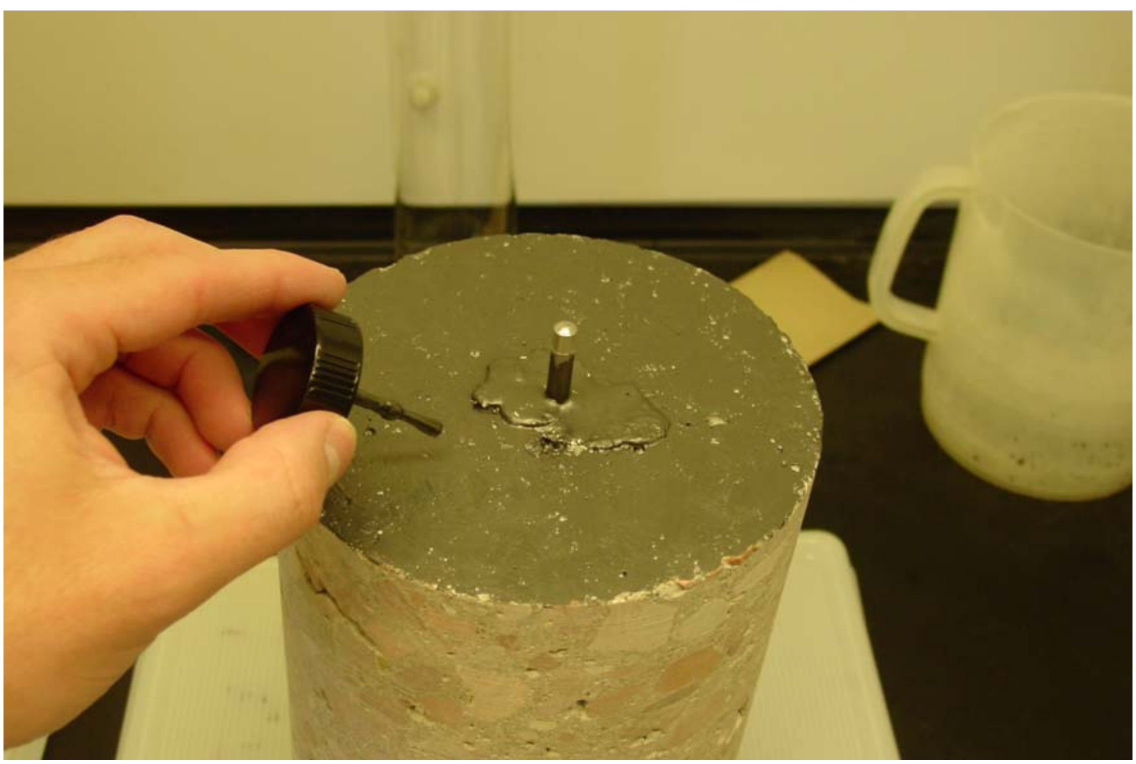
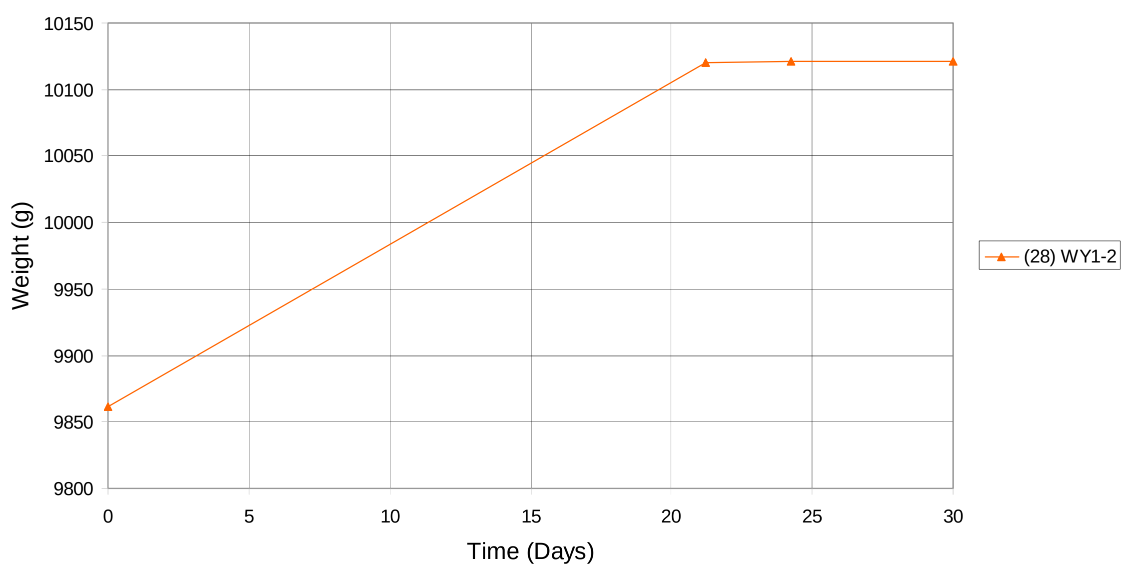


Figure 26: Core with Conductive Carbon Paint

To decrease the likelihood of alkalis washing out of the concrete, the cores were vacuum sealed with only enough water to saturate the pores. They were cut out of their vacuum bags about once a week to record their length and weight. After measurements were complete the cores were vacuum sealed again in their original bags, with their original water. This was done to ensure that any leached out alkalis from a core would remain with it in a constant environment. Measurements were done until each core's length and weight values remained the same. This was done to ensure that cores reached a constant moisture content. Figure 27 shows a typical time vs. weight plot.

Figure 27: Typical Time vs. Weight Plot for Modified ASTM 1293 Core Saturation



Once time vs. expansion and weight gain became asymptotic it was assumed that saturation was attained. A core under vacuum is shown in Figure 28. After the cores were determined to be at a constant moisture they were stripped from the vacuum sealed bags and their lengths and weights were recorded.

Figure 28: Vacuum Saturated Core

The cores were then prepped for testing by attaching an alligator clip to each stud, as shown in Figure 29.

Figure 29: Modified ASTM C 1293 (Electric Cylinder) Test Setup

An inch of water was then placed into the storage container, the core was set up vertical and the container was sealed and placed into the 38 °C oven. Each cylinder was then supplied a constant 1 milliamp of current. When a measurement was taken the current to the cylinders was stopped and the containers were taken out of the oven and left out overnight to cool to room temperature, as per ASTM C 1293. Length and weight measurements were taken at 3, 7, 14, 38, 60, 90 and 108 days. Core storage in the oven is shown in Figure 30.

Figure 30: Modified ASTM C 1293 Core Storage Containers in Oven

ASTM C 469 (Young's Modulus Testing)

A cylinder is set upright on top of a neoprene padded metal base. This padded metal base was placed on top of a ball and socket apparatus. Next, a compressometer was set up on the core to measure its deformation under load. A neoprene padded cap was then placed on top of the cylinder. A compressive axial force was then applied to the cylinder. The load was applied until it reached 40% of the ultimate load, where the axial strain was measured.[21] In addition to 40%, at approximately 10%, 20% and 30% of the ultimate load the axial strain was recorded. Once at 40% of the ultimate load the load was taken off. The deformation returned to zero and the test was performed at least two more times for reputability. Figure 31 shows a cylinder set up and ready to undergo Young's Modulus testing.

Figure 31: Core set up for Young's Modulus Testing

Before Young's Modulus testing was done, cores were taken out of their sealed bags and put into a fog room for over 2 days to normalize moisture. Next, a core from each of the 21 sites was broken in compression. The load that corresponded to 40% of the ultimate failure load was then calculated. An average rate of loading of 35 psi/sec was used when loading the cylinders, per ASTM specifications.[21]

Volumetric Surface Texture

The volumetric surface texture (VST) test was created at The University of Minnesota and is composed of a laser or spring loaded probe that measures the distance in the z axis from a set datum to the joint surface of a core at any given point. The laser probe setup that was used for this study is shown in Figure 32.

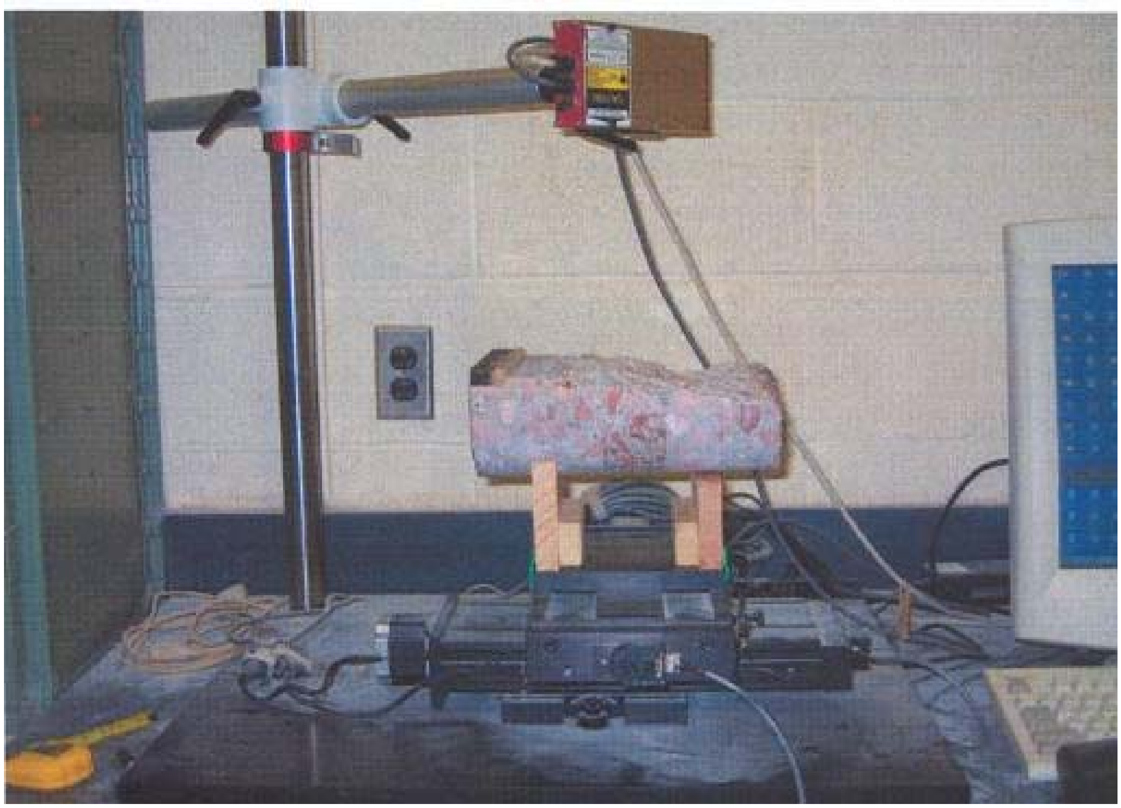


Figure 32: Volumetric Surface Texture Test Setup[22]

This test was developed at the University of Minnesota during the 1994 pavement study as a means of analyzing aggregate interlock for load transfer between panels. The test is used to measure a concrete aggregate's ability to withstand abrasion at a joint or crack.

Before the test was started the two sides of the joint core were pulled apart and a sample area was assigned to each half on the joint face, as seen in Figure 33.

Figure 33: Typical Joint Face Area used for VST Testing[22]

Next, a grid (x,y) was set in centimeters and the laser or probe was run transverse and longitudinal to the joint surface. After all z values were record in centimeters along the x,y grid, the z values were averaged and an average distance from the datum to the surface of the joint was calculated. That value was then subtracted from each z value, making some z values negative and others positive. The new z value was then multiplied by the area traversed for that point (x*y). That gave a volume in cm^3, which was either negative or positive depending if the surface point was above or below the average. The sum of the absolute volume values was the total volume of surface texture (VST). Figure 34 gives a graphical representation of the calculations.

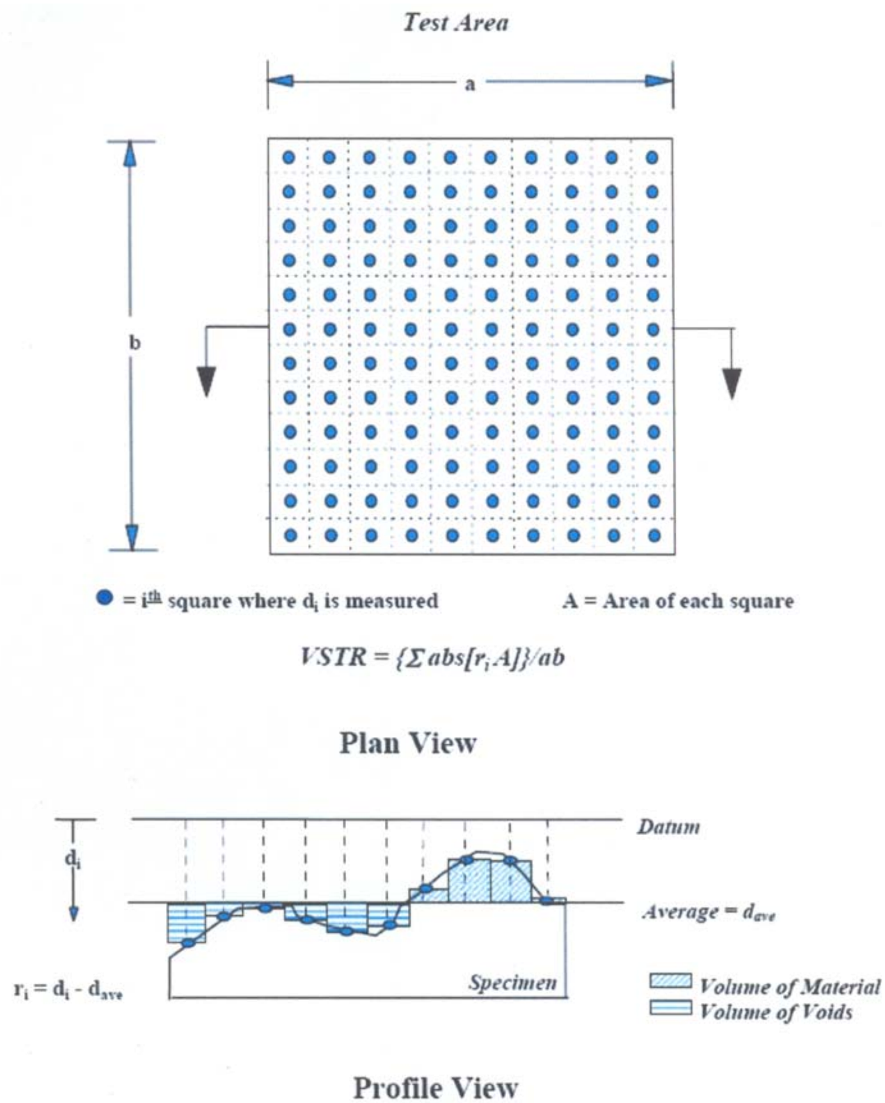


Figure 34: Graphical Representation for VST Calculations[22]

The VST value represented the volume of voids below the average z distance plus the volume of solid material above the average z distance. The VST value was then divided by the overall grid area to obtain the volumetric surface texture ratio (VSTR). Since the VSTR value factors in area, it can be used to compare VST values when grid areas vary.[22] The higher the VSTR value the greater the load transfer capacity for that joint. VSTR values drop over time as concretes experience more load cycles in the field.

ASTM C 856 (Petrographic Study)

ASTM C 856 is a common test used to analyze concrete at the microscopic level. A concrete can be studied to find causes of distress or deterioration. A couple of common purposes of a petrographic examination are to determine if ASR or sulfate attack has taken place. Also, petrographic examinations are useful in verifying that design specs of a concrete were met, such as proper air entrainment.[23]

For this project, a petrographic study was used mainly to identify ASR inside of a concrete's matrix. Before putting a core from each of the 21 test sections through a splitting tensile test, the top 1" of each core was severed. The inside part of this 1" thick piece was then polished on a polishing wheel using 240 to 1000 grit. After polishing, the polished surface of each core cap was scanned into a computer using a flat bed scanner. The images were saved as jpeg image files and printed out for future use in the petrographic study.

After analyzing the uranyl acetate and modified ASTM 1293 results, concretes that were determined to have ASR were put through additional testing. The corresponding caps for each of these concretes were then studied under an Olympus® SZH10 stereo microscope. A picture of the microscope setup can be seen in Figure 35.

Under the microscope, each cap was scanned for micro cracks and voids filled with ASR gel. Aggregate inside of the matrix was also scanned for discolored rings around the outside and cracks running through them containing ASR gel.

Figure 35: Olympus® SZH10 Stereo Microscope used for Petrographic Study

If any abnormalities associated with ASR were found on a cap it was noted on the jpeg image. Each abnormality associated with ASR found on the core caps was documented by photographing, using a microscopic camera.

Next, the area on each polished sample with the most intense abnormalities was identified and a 2” x 3” glass slide was epoxyed to it using 5 minute epoxy. The sample with slide attached was then shaped and cut down using a wet saw so that the core had the area of the glass slide and a height of approximately 1/4”. Figure 36 shows a core cap and a core cap cut down to the aforementioned size.

Figure 36: Original Core Cap and Thin Section Expoxyed to Glass Slide

After each concrete slide was cut on the wet saw it was then ground down to approximately 1/16”. This process was done on a Buehler® thin sectioning machine. Since water removed alkalis out of concrete, the standard water cooled thin sectioning machine was altered. Isopropyl alcohol was used as the coolant for the grinding. The set up of the thin sectioning machine used to grind down the concrete slide can be seen in Figure 37.

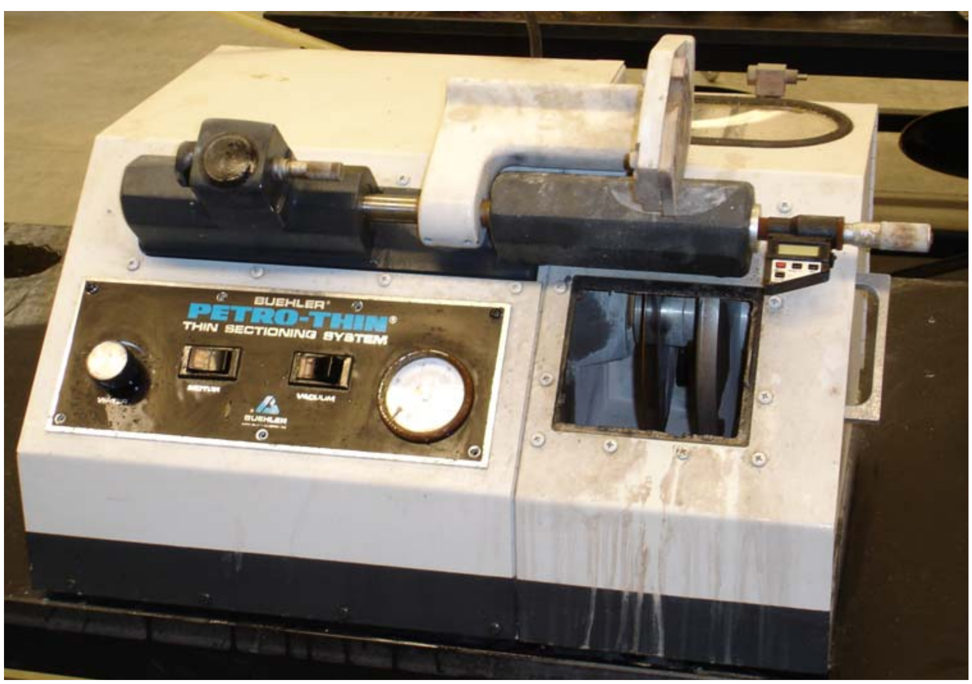


Figure 37: Buehler® Thin Sectioning Machine

Similar to the core caps, once at 1/16” thickness, the slides were polished on a polishing wheel using 240 to 1000 grit. The polished concrete surface was then viewed under the stereo microscope again to find voids and micro cracks filled with ASR gel. Since the glass slide was expoxyed on the polished side of the core cap the top became the bottom. When looking under a microscope at the slides a reverse image of the area was seen. If a concrete showed signs of distress, then it was evaluated with a Scanning Electron Microscope (SEM). Figure 38 shows some stubs set up to be viewed under an SEM.

Figure 38: SEM Stubs with Affixed Concrete Specimens

The stubs of concrete were sputter coated before they were analyzed in a SEM. The SEM was set to use Energy Dispersive Spectroscopy (EDS), which uses an x-ray to analyze the elemental makeup of a specimen.[24] A count of 200 seconds was used for obtaining elemental analysis data on selected areas. The voltage applied to each specimen for surface interpretation was 20 kV. A spectrum and table with the element’s proportion were then produced. Each area that was analyzed using EDS was photographed for visual interpretation.

The graphed images were in black and white and saved as a tif picture file. While the graph resembles how the surface of the specimen looks, it can have variations in brightness due to over charged areas with varying conductivity. Figure 39 shows an example of a surface that was evaluated.

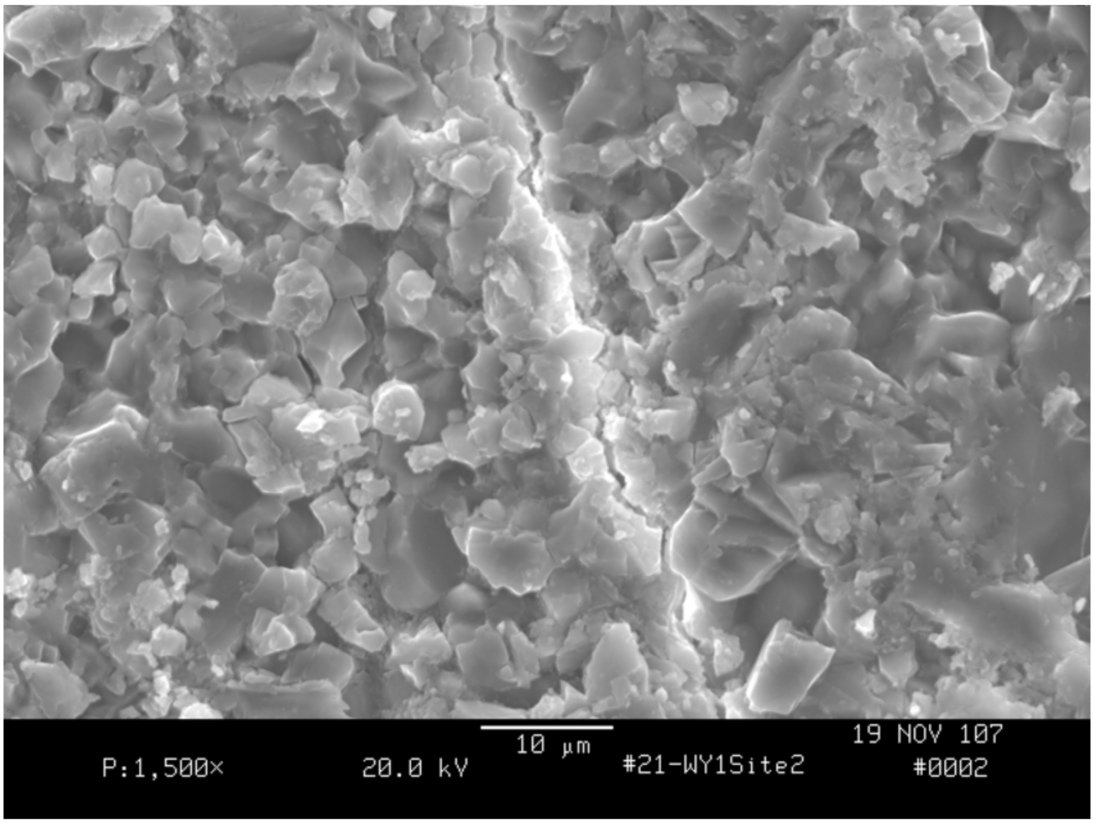


Figure 39: Typical SEM Evaluated Surface

CHAPTER 4

RESULTS

Introduction

The following are results for both recycled and non-recycled concrete pavements. Results contained within this chapter are those from the current (2006) study.

1994 Testing Results

During the 1994 study a variety of tests were done in addition to the tests performed in the 2006 study. Specifically, deflection testing was performed on the pavements to measure the deflection from loads on the midslab, joints, cracks and edges. Also, dynamic modulus and crack and lab fractured surface VSTR testing was performed on concrete cores. Limited resources made it impossible to perform a detailed study in 2006; however, the essential tests to determine the performance of the selected sites were conducted. 1994 testing results can be found in Reference #2.

Results from the tests that were performed in 1994 and 2006 are incorporated in the discussion chapter of this report. The 1994 testing values are compared to the 2006 testing values so that conclusions can be drawn on the durability of the concrete pavements over the course of 12 additional years of service.

Pavement Survey

KS1 and WI2 both were overlaid with asphalt since the 1994 study, so a field survey could not be done in 2006. MN3, MN4, WI1 and WY1 were all rehabilitated since 1994.

Additionally, IL1 was continuously reinforced concrete pavement (CRCP), therefore, some joint data was not applicable.

Transverse Joint Spalling

Each pavement was assigned a value in percent of transverse joints with any spalling. The transverse joint spalling values for the studied pavement sections are presented in Table 5.

Transverse Joint Seal Damage

The resulting values are given in percent of transverse joints with any seal damage. The transverse joint seal damage values for the studied pavement sections are presented in Table 5.

Table 5: Percent of Joints with Transverse Joint Spalling and Seal Damage

Project	**Transverse Joint Spalling, % Joints**	**Transverse Joint Seal Damage, % Joints**
CT1-1	92	100
CT1-2	66	94
IA1-1	100	100
IA1-2	100	100
MN1-1	76	100
MN1-2	54	95
MN2-1	46	100
MN2-2	66	100
MN3-1	89	0
MN4-1	81	100
MN4-2	100	100
WI1-1	98	98
WI1-2	91	100
WY1-1	47	16
WY1-2	77	100

Longitudinal Joint Seal Damage

The resulting values are given in meters of damaged joint (low, medium or high severity) per km of pavement (m/km). For example, if all of a pavement sections longitudinal joint seal was damaged the value for that section would be 1,000 m/km. The longitudinal joint seal damage values for the studied pavement sections are presented in Table 6.

Table 6: Longitudinal Joint Seal Damaged Joints

Project	**Longitudinal Joint Seal Damage, m/km**
IL1-1	1000
IL1-2	1000
IA1-1	1000
IA1-2	1000
MN1-1	1000
MN1-2	1000
MN2-1	1000
MN2-2	1000
MN3-1	1000
MN4-1	973
MN4-2	1000
WI1-1	1000
WI1-2	1000
WY1-1	1000
WY1-2	1000

D-Cracking

The resulting values are given in percent of slabs showing any d-cracking. The d-cracking values for the studied pavement sections are presented in Table 7.

Pumping

All of the pavements except for MN3-1 had no pumping. MN3-1 only had 1% of its slabs showing signs of pumping.

Table 7: Percent of Slabs with D-Cracking

Project	D-cracking, % Slabs
CT1-1	0
CT1-2	0
IL1-1	100
IL1-2	47
IA1-1	15
IA1-2	2
MN1-1	0
MN1-2	0
MN2-1	0
MN2-2	0
MN3-1	0
MN4-1	0
MN4-2	0
WI1-1	0
WI1-2	0
WY1-1	0
WY1-2	0

Slab/Patch Deterioration

The resulting values are given in percent of slabs with any sign of patch deterioration. The slab/patch deterioration values for the studied pavement sections are presented in Table 8.

Lane to Shoulder Drop off

The resulting values are given as the average difference in pavement elevation and shoulder elevation at the lane to shoulder joint. The lane to shoulder drop off values for the studied pavement sections are presented in Table 9.

Table 8: Percent of Slabs that Exhibited Slab/Patch Deterioration

Project	**Slab/Patch Deterioration, % Slabs**
CT1-1	0
CT1-2	0
IL1-1	0
IL1-2	0
IA1-1	2
IA1-2	0
MN1-1	0
MN1-2	0
MN2-1	5
MN2-2	0
MN3-1	0
MN4-1	3
MN4-2	0
WI1-1	0
WI1-2	0
WY1-1	0
WY1-2	0

Table 9: Average Lane to Shoulder Drop off Values

Project	**Avg. Lane to Shoulder Drop off, mm**
IL1-2	8
MN1-1	22
MN1-2	30
MN2-1	11
MN2-2	13
MN3-1	2
MN4-1	20
MN4-2	11

Lane to Shoulder Separation

The resulting values are given as the average width from the pavement edge to the shoulder edge. The lane to shoulder separation values for the studied pavement sections are presented in Table 10.

Table 10: Average Lane to Shoulder Separation Values

Project	Avg. Lane to Shoulder Separation, mm
CT1-1	15
CT1-2	19
IL1-1	3
IL1-2	12
MN1-1	2
MN1-2	2
MN2-1	2
MN2-2	4
MN3-1	2
MN4-1	4
MN4-2	4
WY1-1	11
WY1-2	14

Faulting Between Panels

The resulting values are given as the average height differentiation between abutting panels along the outer wheel path. The average faulting between panels values for the studied pavement sections are presented in Table 11.

Joint Width

The resulting values are given as the average of the width between panels longitudinal to the outer wheel path. The average joint width values for the studied pavement sections are presented in Table 11.

Table 11: Average Faulting Between Panels and Joint Width

Project	Avg. Faulting between Panels, mm	Avg. Joint Width, mm
CT1-1	1.0	13
CT1-2	1.1	14
IA1-1	2.2	18
IA1-2	3.6	17
MN1-1	0.9	11
MN1-2	1.3	10
MN2-1	0.6	12
MN2-2	0.5	13
MN3-1	0.3	18
MN4-1	0.9	12
MN4-2	0.9	11
WI1-1	2.3	9
WI1-2	0.5	11
WY1-1	0.7	10
WY1-2	0.6	10

Longitudinal Cracking

The resulting values are given in meters of longitudinal cracks (low, medium or high severity) per km of pavement (m/km). The longitudinal cracking values for the studied pavement sections are presented in Table 12.

Transverse Cracking

Due to the importance of transverse cracking in pavement performance, the results from this part of the study have been put together in multiple ways. First, the percent slabs with transverse cracking are shown for each section. Second, the amount of deteriorated transverse cracks (medium or high severity) per km of road are presented. Finally, the total amount of transverse cracks per km of road are given for each section. Transverse cracking values are presented in Table 12.

Table 12: Longitudinal and Transverse Cracking Values

Project	Longitudinal Cracking, m/km	Transverse Cracking, % Slabs	Deteriorated Transverse Cracks/km	Total Transverse Cracks/km
CT1-1	0	68	42	82
CT1-2	0	93	3	38
IL1-1	1252	n/a	0	0
IL1-2	527	n/a	n/a	59
IA1-1	12	29	36	49
IA1-2	0	2	3	3
MN1-1	0	31	35	38
MN1-2	0	0	0	0
MN2-1	26	90	112	112
MN2-2	0	92	112	115
MN3-1	0	12	26	26
MN4-1	17	92	125	131
MN4-2	0	24	26	29
WI1-1	0	35	72	75
WI1-2	0	3	6	6
WY1-1	124	0	0	0
WY1-2	9	0	0	0

Note: n/a data not applicable

Present Serviceability Rating (PSR)

The resulting values are given as a number from 0 to 5, taken out to the tenths place. This average of at least 2 individual's ratings is presented for PSR. The PSR values for the studied pavement sections are presented in Table 13. 3 judges were used when determining the PSR for all sections.

International Roughness Index (IRI)

The resulting values are given as a number greater than 0, taken out to the tenths place. This value was calculated from PSR using a relationship given by the Minnesota Department of Transportation, as presented in Chapter 3. The IRI values for the studied pavement sections are presented in Table 13.

Table 13: Present Serviceability and International Roughness Ratings

Project	IRI	IRI
CT1-1	3.7	1.1
CT1-2	3.2	1.5
MN1-1	3.7	1.1
MN1-2	4.0	0.9
MN2-1	4.0	0.9
MN2-2	3.8	1.0
MN3-1	4.3	0.7
MN4-1	3.0	1.7
MN4-2	3.8	1.0
WI1-1	2.8	1.9
WI1-2	3.7	1.1
WY1-1	4.5	0.6
WY1-2	4.2	0.7

Laboratory Work

ASTM C 496 (Splitting Tension Testing)

The splitting tension values are given in MPa (MN/m^2). The tensile strength values for the studied pavement sections are presented in Table 14.

ASTM C 39 (Compression Testing)

The compression values are given in MPa (MN/m^2). The compressive strength values for the studied pavement sections are presented in Table 14.

Table 14: Tensile and Compressive Strength Values

Project	Tensile Strength, MPa	Compressive Strength, MPa
CT1-1	2.3	39.5
CT1-2	3.2	37.0
IL1-1	1.9	56.0
IL1-2	3.6	55.2
IA1-1	2.5	52.6
IA1-2	2.8	47.6
KS1-1	3.6	47.9
KS1-2	3.7	42.0
MN1-1	2.9	44.9
MN1-2	3.3	59.0
MN2-1	3.7	49.5
MN2-2	2.8	64.1
MN3-1	3.7	52.4
MN4-1	2.4	45.1
MN4-2	2.5	50.7
WI1-1	3.1	37.0
WI1-2	4.3	32.7
WI2-1	3.9	43.9
WI2-2	2.9	45.4
WY1-1	2.9	54.6
WY1-2	3.0	48.8

ASTM C 856 (Uranyl Acetate)

A visual analysis description and image rating for the reaction that the uranyl acetate had on each core is presented in Table 15.

Table 15: Observations from Uranyl Acetate Testing

Project	Visual Analysis Description	Reaction Rating
CT1-1	None	None
CT1-2	None	None
IL1-1	Severe aggregate light up, severe crack light up	High
IL1-2	Severe aggregate light up, severe crack light up	High
IA1-1	None	None
IA1-2	Moderate aggregate light up, minimal crack light up	Low
KS1-1	None	None
KS1-2	None	None
MN1-1	None	None
MN1-2	None	None
MN2-1	Minimal aggregate light up, minimal crack light up	Low
MN2-2	None	None
MN3-1	Minimal aggregate light up, minimal crack light up	Low
MN4-1	None	None
MN4-2	None	None
WI1-1	Minimal aggregate light up	Low
WI1-2	Minimal aggregate light up, minimal crack light up	Low
WI2-1	Minimal aggregate light up	Low
WI2-2	Severe aggregate light up, minimal crack light up	High
WY1-1	Moderate aggregate light up, minimal crack light up	Moderate
WY1-2	Severe aggregate light up, moderate crack light up	Moderate

Modified ASTM C 1293 (Electric Cylinder)

The results for the Modified ASTM 1293 ASR testing varied among the tested sites. Only sections that showed signs of ASR from the uranyl acetate testing were tested. Expansion charts for the 10 sections tested are presented in Figure 40 and weight changes can be found in Figure 41.

Figure 40 shows that some sections expanded more than others. All sections that had a 108 day expansion of greater than 0.1% were put through a petrograpahic study to further analyze the concrete for ASR. Further analysis of the Modified ASTM 1293 results can be found in Chapter 5.

Figure 40: Expansion vs. Time for Modified ASTM 1293

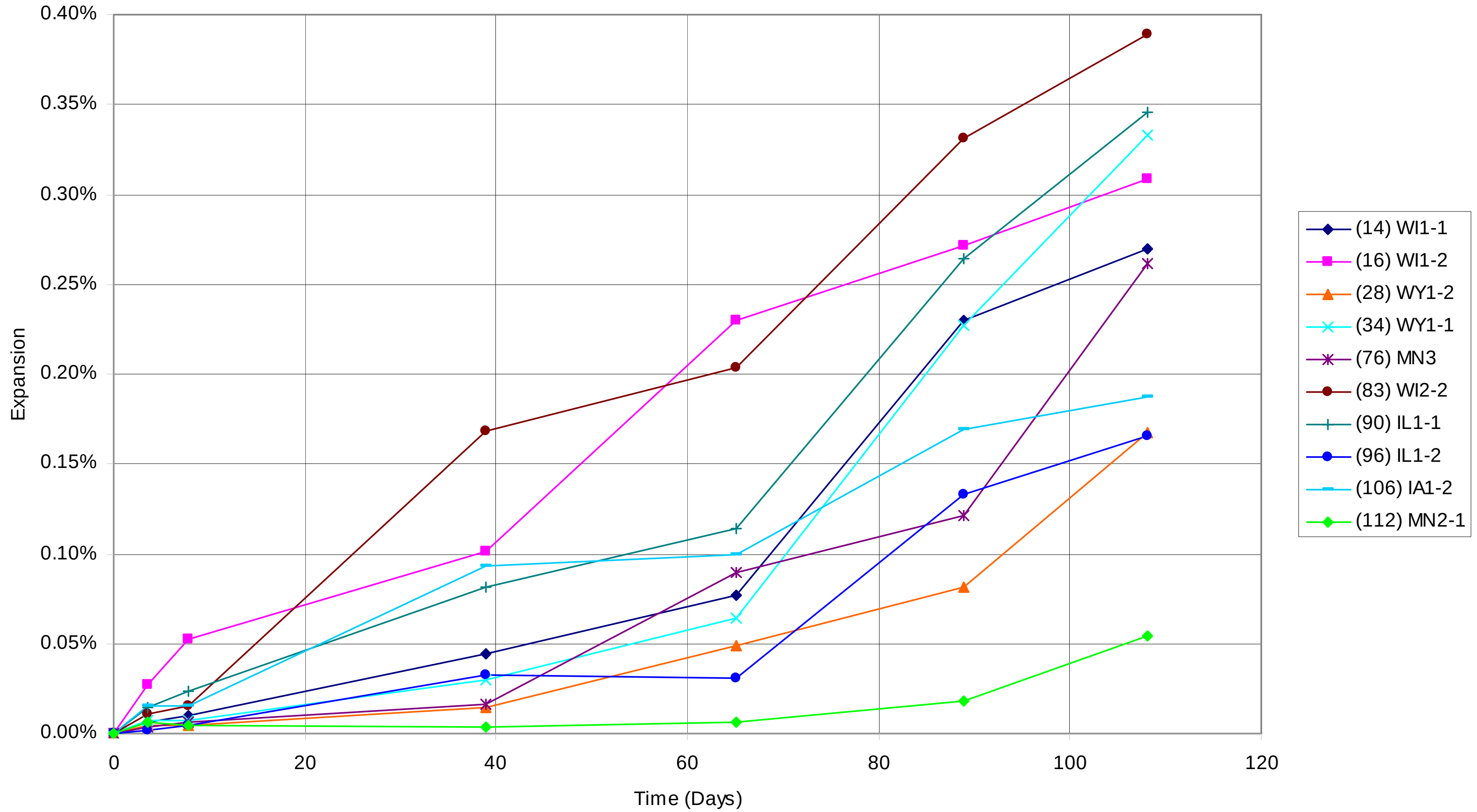
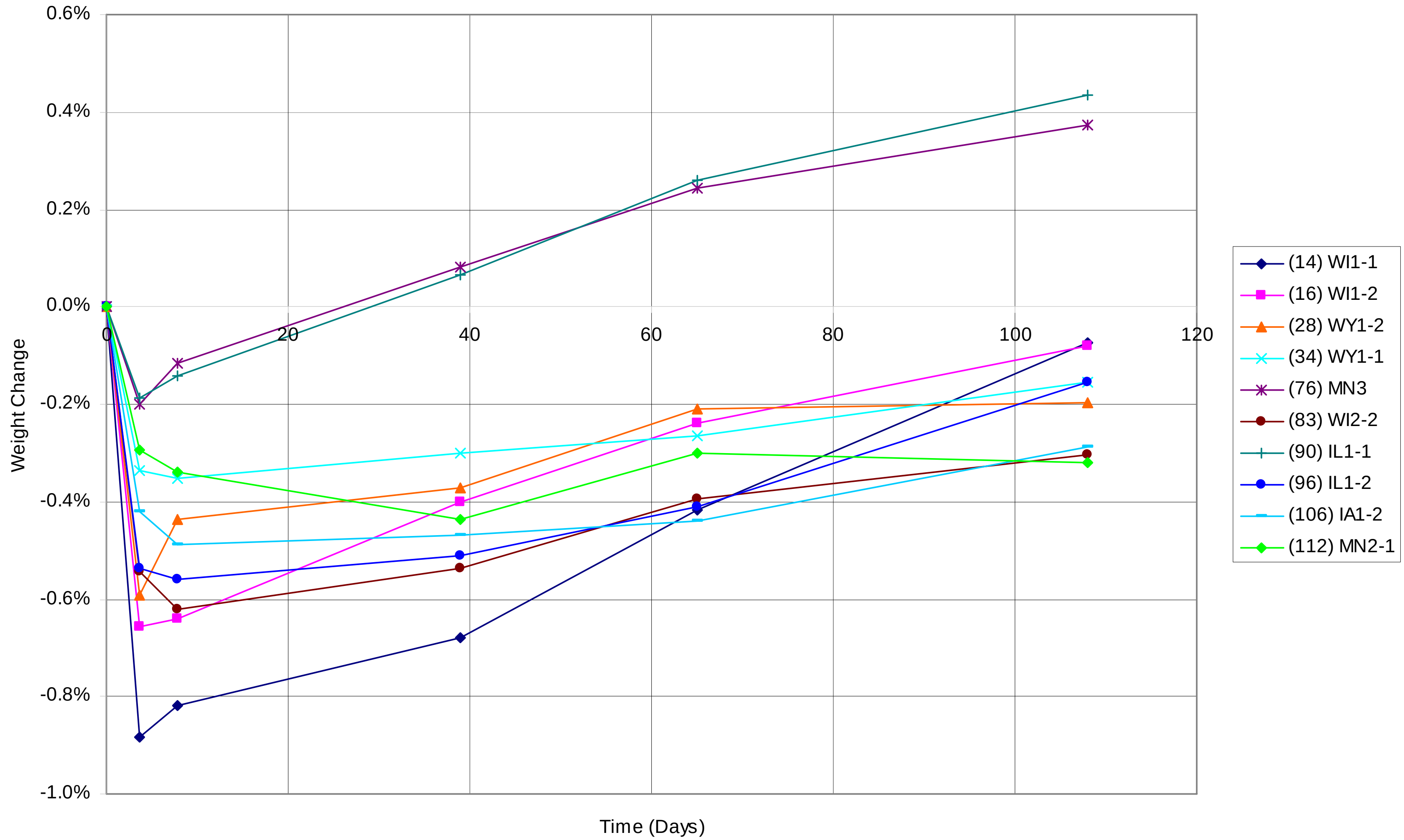


Figure 41: Weight Change vs. Time for Modified ASTM 1293



ASTM C 469 (Young's Modulus Testing)

The Young's Modulus values are given in GPa (GN/m^2) and are presented in Table 16.

Table 16: Young's Modulus Values

Project	Young's Modulus, GPa
CT1-1	24.6
CT1-2	29.7
IL1-1	29.1
IL1-2	26.7
IA1-1	28.3
IA1-2	24.6
KS1-1	30.3
KS1-2	34.3
MN1-1	28.9
MN1-2	33.4
MN2-2	31.1
MN4-1	30.0
MN4-2	43.4
WI1-1	34.2
WI1-2	29.7
WI2-1	25.6
WI2-2	20.5
WY1-1	34.2
WY1-2	29.7

Volumetric Surface Texture

The average VSTR values for the joint faces of the cores are given in Table 17.

Table 17: Joint Volumetric Surface Texture Ratios

Project	Average VSTR (cm^3/cm^2)
MN4-1	0.2902
MN4-2	0.3264
WI2-1	0.4493
WY1-1	0.4131
WY1-2	0.7315

It was not possible to get joint cores from all of the 21 sites, so only the sites that had joint cores extracted were tested. The value given is the average joint volumetric surface texture ratio, measured in cm^3/cm^2. Some of the cores were tested on both joint faces.

ASTM C 856 (Petrographic Study)

Each core's polished cross section was scanned into a flat bed scanner and saved as a jpeg picture file. Such things as mortar content, aggregate top size, aggregate type and macro cracks can be seen at this level. Table 18 shows each pavements polished cross section. Further results from the petrographic study will be incorporated into the discussion section.

Table 18: Polished Core Cross Sections from all 21 Sites

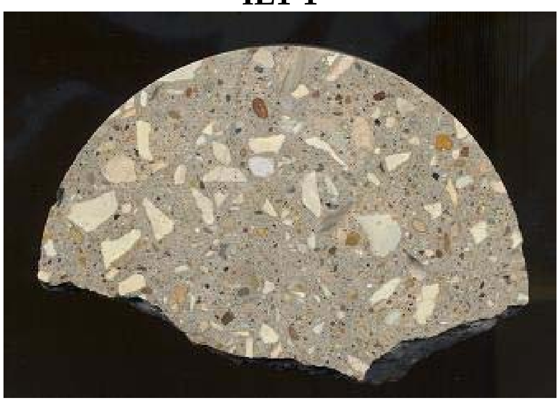
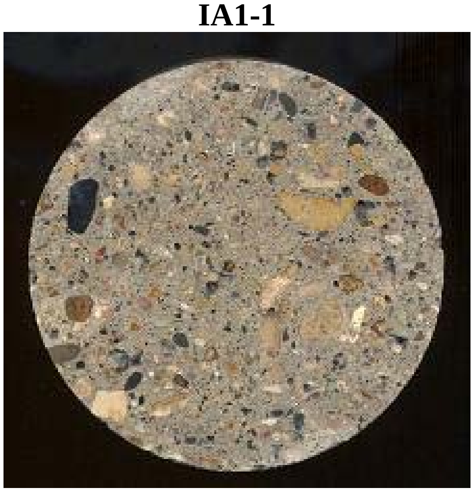
CT1-1 	**CT1-2** 
IL1-1 	**IL1-2** 
IA1-1 	**IA1-2** 

Table 18: Polished Core Cross Sections from all 21 Sites (Cont.)

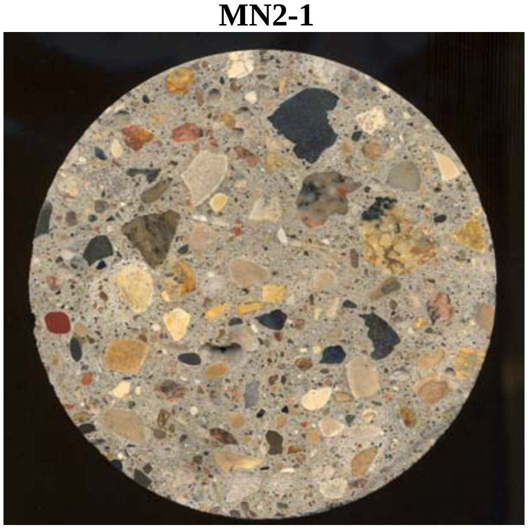
KS1-1 	**KS1-2** 
MN1-1 	**MN1-2** 
MN2-1 	**MN2-2** 

Table 18: Polished Core Cross Sections from all 21 Sites (Cont.)

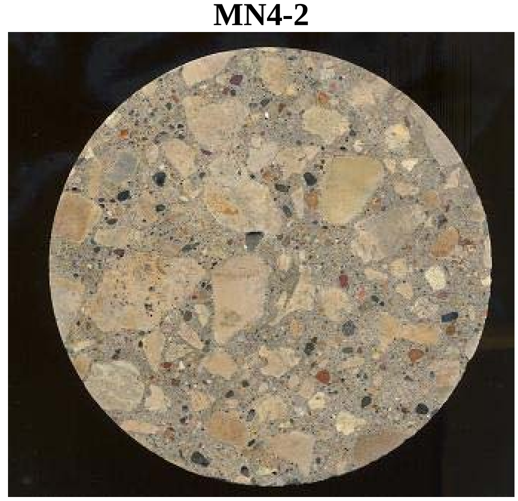
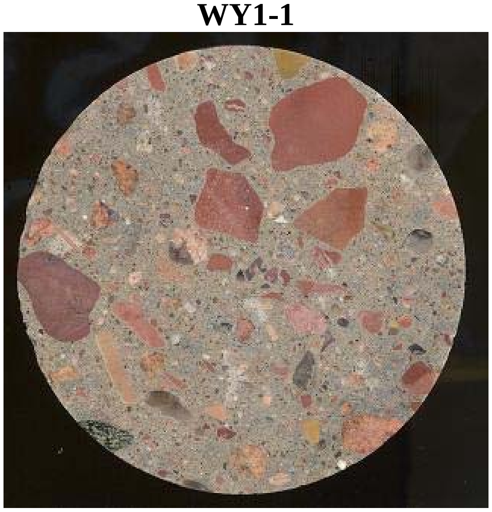
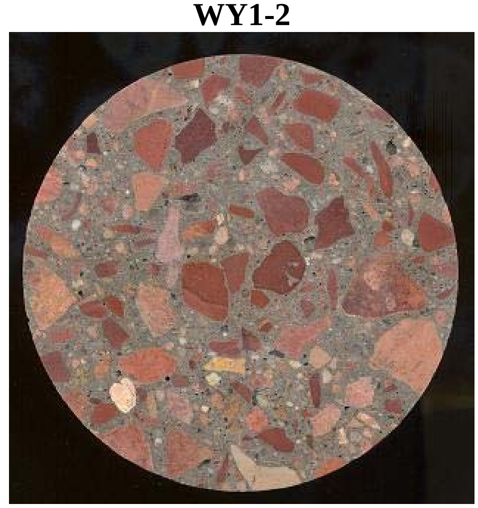
MN3-1 	
MN4-1 	**MN4-2** 
WI1-1 	**WI1-2** 

Table 18: Polished Core Cross Sections from all 21 Sites (Cont.)

WI2-1	WI2-2
	
WY1-1	**WY1-2**
	

CHAPTER 5

DISCUSSION

RCA Sections vs. Control Sections

This section contains a discussion and comparison of the 5 locations that had both a control and recycled sections. Results contained in this section are only from the 2006 study, however where appropriate, the 1994 data are discussed. Discussion and conclusions from the complete 1994 study that compared control and recycled concrete pavements sections can be found in Reference #2.

K-7 Johnson County, KS

Since the 1994 study, both the control and recycled concrete sections of K-7 were overlaid with asphalt. Consequently, only laboratory data are available for these sections. Laboratory testing data comparisons are presented in Table 19.

Table 19: KS 1-1 and KS 1-2 Laboratory Testing Data

Test and Value	**KS 1-1 (Recycled)**	**KS 1-2 (Control)**	**Difference (recycled vs. control)**	**Best**
Aggregate Top Size, mm	19	38	-50%	C
Recycled Fines, %	25	0	25%	C
Tensile Strength, MPa	3.6	3.7	-3%	C
Compressive Strength, MPa	47.9	42.0	14%	R
Uranyal Acetate Reaction	None	None	None	=
Young's Modulus, GPa	30.3	34.3	-12%	C

The tensile strength of the control section was comparable to the recycled. The compressive strength of the recycled concrete was 14% greater than that of the control, which reflects the results found for KS1 in the 1994 study, where the RCA was 10% greater than the control. The decrease in Young's Modulus in KS1-1 may have come from the use of 20% recycled fines.

I-80 Pine Bluffs, WY

Since the 1994 study, both the control and recycled sections of I-80 were rehabilitated (including diamond grinding). Consequently, field performance data such as slab faulting and PSR were positively affected. The 2006 field and laboratory testing data comparisons are presented in Table 20.

The recycled section had a substantial amount of joint spalling compared to the control (47% vs. 7%). This does not reflect what was found in 1994, where only minimal joint spalling was present. While the severity level of the joint spalling was low, the increase from the recycled to control may be attributed to the concrete expanding longitudinally due to ASR. Expansion from ASR and hot pavement temperatures might have led to abutting panels contacting one another at transverse joints, which could have created high stresses and edge failure. Transverse joint seal damage was found to be 84% higher on the control than on the recycled. Faulting was only 17% greater (0.1mm) in the recycled versus the control. Overall, the recycled section was determined to have a higher serviceably and lower roughness rating than the control. The roughness rating was 43% lower in the recycled then in the control.

Laboratory testing showed that the compressive strength of the recycled concrete was higher than the control. The difference (12%) is marginally larger than the 8.0% precision of the test itself.[18]

Table 20: WY 1-1 and WY 1-2 Field and Laboratory Performance Data

Test and Value	WY 1-1 (Recycled)	WY 1-2 (Control)	Difference (recycled vs. control)	Best
Aggregate Top Size, mm	38	25	52%	R
Recycled Fines, %	22	0	22%	C
Transverse Joint Spalling, % Joints	47	7	40%	C
Transverse Joint Seal Damage, % Joints	16	100	-84%	R
Longitudinal Joint Seal Damage, m/km	1000	1000	0%	=
D-cracking, % Slabs	0	0	0%	=
Pumping, % Slabs	0	0	0%	=
Slab/Patch Deterioration, % Slabs	0	0	0%	=
Avg. Lane to Shoulder Separation, mm	11	14	-21%	R
Avg. Faulting between Panels, mm	0.7	0.6	17%	C
Avg. Joint Width, mm	10	10	0%	=
Longitudinal Cracking, m/km	124	9	1278%	C
Transverse Cracking, % Slabs	0	0	0%	=
Deteriorated Transverse Cracks/km	0	0	0%	=
Total Transverse Cracks/km	0	0	0%	=
PSR	4.5	4.2	7%	R
IRI	0.6	0.7	-14%	R
Tensile Strength, MPa	2.9	3.0	-3%	C
Compressive Strength, MPa	54.6	48.8	12%	R
Uranyal Acetate Reaction	Medium	Medium	None	=
Modified ASTM 1293, % Expansion at 108 Days	0.333	0.167	99%	C
Young's Modulus, GPa	34.2	29.7	9%	R
Average VSTR (cm3/cm2)	0.4131	0.7315	-44%	C

Young's Modulus values were a little higher in the recycled than in the control as well. This was the case in 1994 and was attributed to the recycled pavements lower water to cement ratio, higher cement content and addition of recycled fines to the mixture.[2] The VSTR value obtained for the recycled section was 44% lower than that of the control section. This mirrors the results found in the 1994 study which were attributed to the control section aggregate's high strength, high proportion in the mix and larger top size.[2] Uranyl acetate testing showed ASR gel was present in the aggregate and paste of both the control and recycled sections. Figure 42 shows the uranyl acetate test of the control section and Figure 43 shows the test on the recycled section.

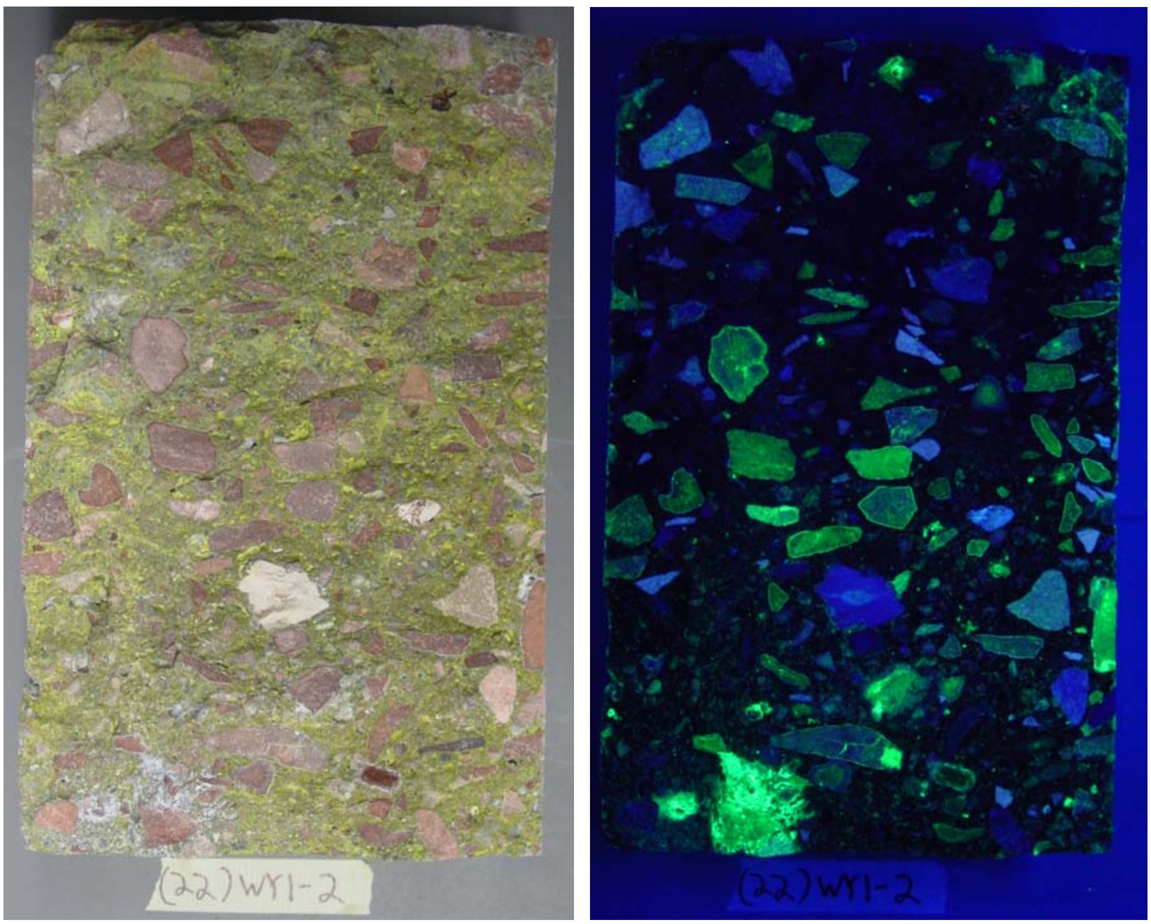


Figure 42: Fractured Core from WY control coated with Uranyl Acetate Dihydrate under UV light (right) showing ASR gel and under regular light (left)

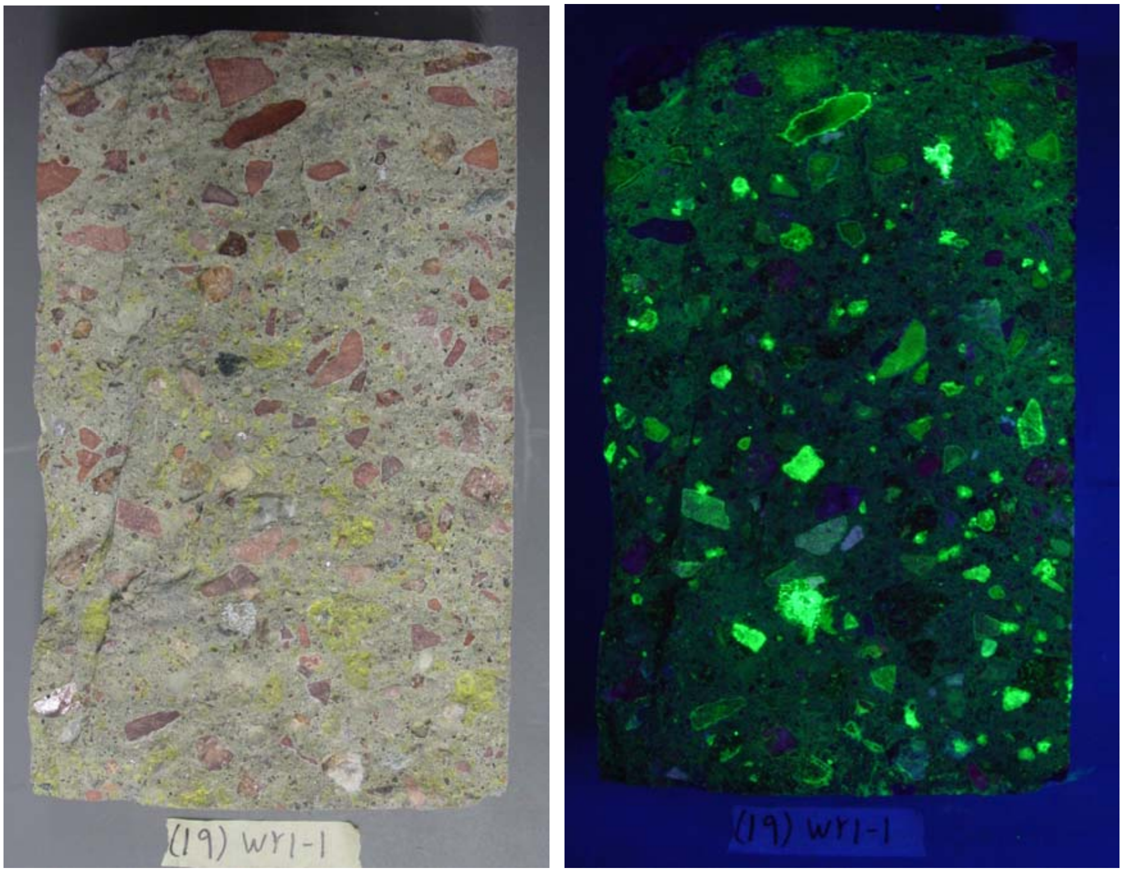


Figure 43: Fractured Core from WY recycled coated with Uranyl Acetate Dihydrate under UV light (right) showing ASR gel and under regular light (left)

Some of these aggregates have natural fluorescence and light up blue under UV light. These aggregates should not be misconstrued as ASR. The Modified ASTM 1293 testing showed the expansion of the recycled concrete section was over double that of the control at 108 days (0.333% vs. 0.167%), both of which indicate a high expansion potential.

Both the control and recycled sections were petrographicly analyzed. The control section had many aggregate pieces with ASR gel deposits. The aggregates also had many micro cracks with ASR gel inside of them. An example of a typical aggregate crack with ASR gel inside that was found in the control section is shown in Figure 44.

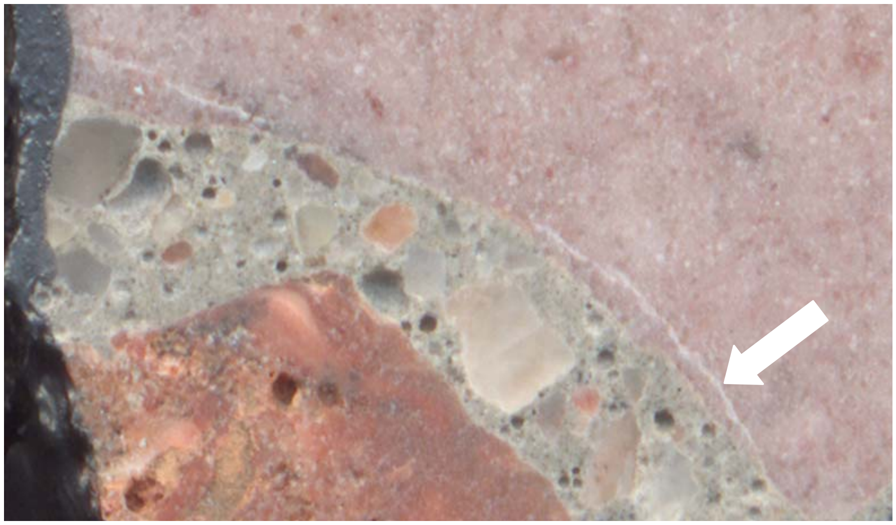


Figure 44: Typical WY1-2 (Control) Aggregate Crack with ASR Gel Deposit

Similar cracking and ASR gel deposits were seen in the recycled section as well. ASR gel could also be seen at the RCA and new mortar border. An example of a typical aggregate crack with deposits of ASR gel inside and around the RCA is shown in Figure 45.

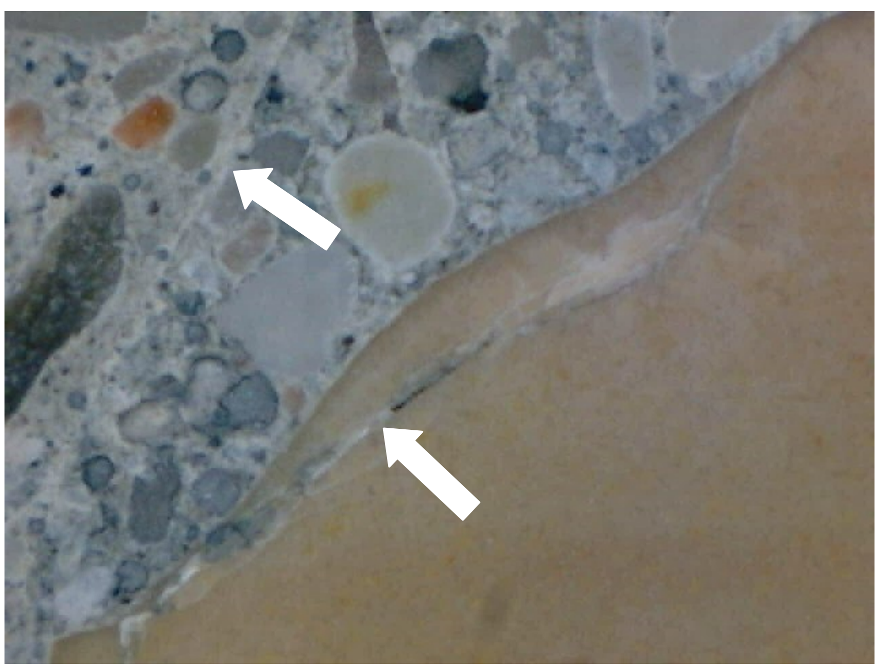
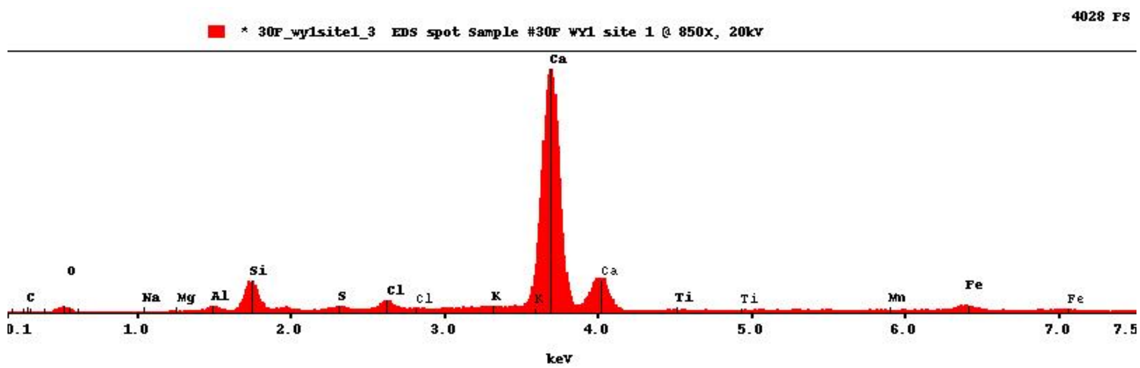


Figure 45: WY1-1 (Recycled) Aggregate Crack and RCA border with ASR Gel Deposits

To gain an understanding of the elemental makeup of the unreacted fly ash in the recycled section a Energy Dispersive Spectroscopy (EDS) scan was performed using the Scanning Electron Microscope (SEM). A substantial amount of particles scanned contained high amounts of calcium, as shown in Figure 46.



Sample /xd1/window1/#1,/30F_wy1site1_3.spt
Accelerating Voltage: 20.00 keV
Takeoff Angle: 30.00 degrees
Library for system standards: /imix/quant/efficiency/default.dir

Elm	Rel. K	ZAF	Norm wt%	Prec.	Atomic %	Standard	Intensity
C	0.0013	3.6236	0.48	0.55	1.37	(S)C_K	36 ?
Mg	0.0008	2.2574	0.17	0.53	0.19	(S)Mg_K	22 ?
Al	0.0072	1.7562	1.27	0.36	1.57	(S)Al_K	454
Si	0.0472	1.4266	6.74	0.55	7.87	(S)Si_K	3624
S	0.0055	1.1827	0.65	0.19	0.66	(S)S_K	477
Cl	0.0172	1.1225	1.93	0.36	1.79	(S)Cl_K	1019
Ca	0.6876	1.0449	71.85	1.78	59.00	(S)Ca_K	36696
Fe	0.0319	1.2118	3.87	0.74	2.27	(S)Fe_K	908
Na	0.0002	3.3885	0.08	1.07	0.10	(S)Na_K	4 ?
K	0.0066	0.9361	0.62	2.70	0.54	(S)K_K	35 ?
Ti	0.0037	1.4495	0.53	0.00	0.34	(S)Ti_K	0
Mn	0.0000	1.2718	0.00	0.00	0.00	(S)Mn_K	77 ?
O	0.0090	13.0844	11.81	2.90	24.26	(S)O_K	435
Total			100.00		99.96		

Figure 46: Elemental Analysis showing high Calcium Content

Figure 47 shows the SEM interpretation of the fly ash particle's surface. The Fly ash particle shown in Figure 47 is from a high calcium fly ash. Since Class C fly ash typically has high alkali content with increased alkali solubility, it would be expected to accelerate ASR, rather then mitigate it, as would have a class F fly ash.

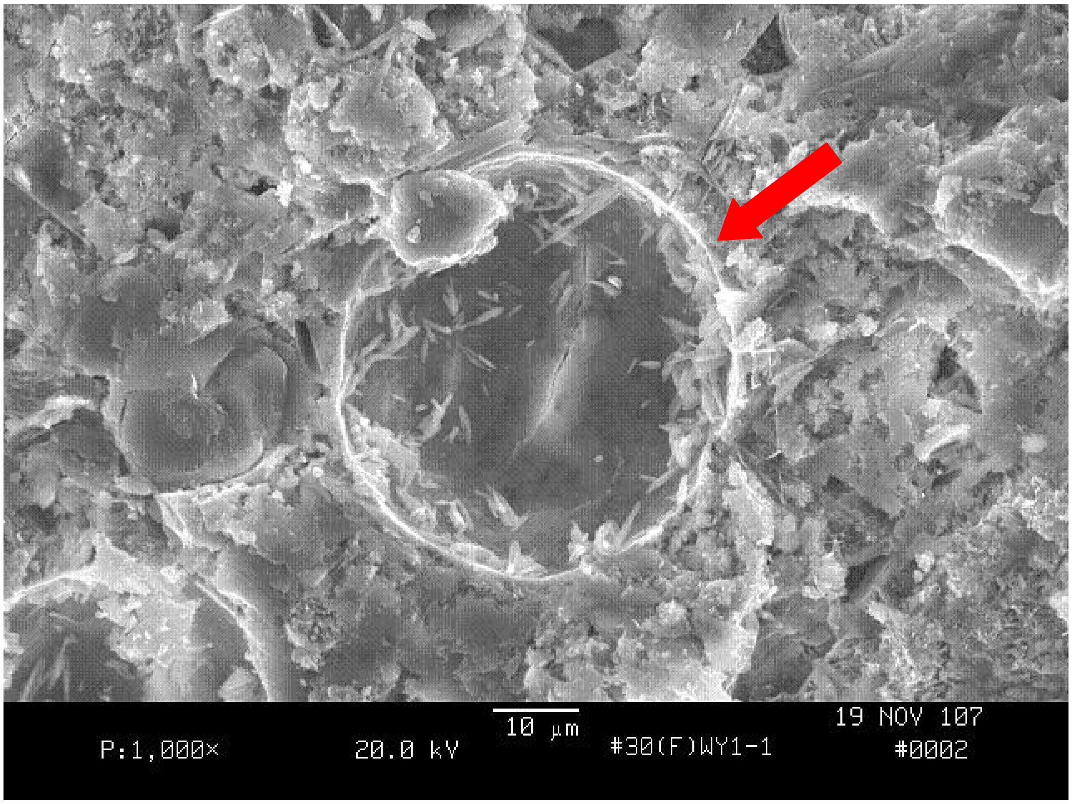


Figure 47: SEM Interpretation of a Fly Ash Particle's Surface (WY1-1)

This might explain why the recycled section is experiencing greater expansion due to ASR than the control.

From modified ASTM 1293 testing, visual analysis in the field and the petrographic study, it became obvious that both WY1-1 and WY1-2 are experiencing ASR and will continue to deteriorate. The RCA section is expected to undergo more expansion than the control due to the presence of the high calcium fly ash. Its higher amount of joint spalling and longitudinal cracking is most likely due to ASR. Both of the sections should be further examined in the future to determine the rate of deterioration that will occur on the concrete from the ASR.

I-84 Waterbury, CT

The 2006 field and laboratory testing data comparisons are presented in Table 21.

Table 21: CT 1-1 and CT 1-2 Field and Laboratory Performance Data

Test and Value	CT 1-1 (Recycled)	CT 1-2 (Control)	Difference (recycled vs. control)	Best
Joint Spacing, m	12	12	0%	=
Aggregate Top Size, mm	38	51	-34%	C
Recycled Fines %	20	0	20%	C
Transverse Joint Spalling, % Joints	92	66	26%	C
Transverse Joint Seal Damage, % Joints	100	94	6%	C
D-cracking, % Slabs	0	0	0%	=
Pumping, % Slabs	0	0	0%	=
Slab/Patch Deterioration, % Slabs	0	0	0%	=
Avg. Lane to Shoulder Separation, mm	15	19	-21%	R
Avg. Faulting between Panels, mm	1.0	1.1	-9%	R
Avg. Joint Width, mm	13	14	-7%	R
Longitudinal Cracking, m/km	0	0	0%	=
Transverse Cracking, % Slabs	68	93	-25%	R
Deteriorated Transverse Cracks/km	42	38	11%	C
Total Transverse Cracks/km	82	131	-37%	R
PSR	3.7	3.2	16%	R
IRI	1.1	1.5	-36%	R
Tensile Strength, MPa	2.3	3.2	-28%	C
Compressive Strength, MPa	39.5	37.0	5%	R
Uranyal Acetate Reaction	None	None	None	=
Young's Modulus, GPa	24.6	26.7	-8%	C

Even though both section had an extremely high amount of joint spalling, there was 26% more in the recycled section than in the control. This was also found to be the case in the 1994 study and was attributed to the recycled concrete's higher coefficient of thermal expansion (due to the addition of 20% fines) plus a high amount of transverse joint seal damage, which could have led to the expansion joints filling with debris.[2] The recycled concrete did however perform better than the control in the areas of lane to shoulder separation and joint faulting. The control section had 37% more transverse cracks per km than the recycled. This is what probably led to a lower PSR rating in the control section than in the recycled.   The test strip was located on a down grade to an off ramp, so the force from vehicles braking could have intensified movement of the transverse cracks, as shown in Figure 48.

Figure 48: Shift in Outside Lane Panels (CT)

Both sections exhibited many high severity transverse cracks, some over 2” wide. The high amount of transverse cracking in both sections may be attributed to the road having an ADT of 56,000 veh/day, which was on average more than 7 times the traffic loading of any other pavement studied. Additionally the pavement had 12m joint spacing, which was the longest of all of the pavements studied. The long panel lengths and high range of temperatures in that region could have induced the transverse cracks. Figure 49 shows an up close view of a typical transverse crack found in both sections. The cracks were very wide and typically had debris inside of them.

Figure 49: Typical Transverse Crack found on CT1-1 and CT1-2

Contrary to the results of the 1994 study, the recycled section had a 28% lower tensile strength than the control section, suggesting more fatigue cracking than the control. The difference in compressive strengths in 2006 was similar to those in 1994.

US 52 Zumbrota, MN

Both the control and recycled sections of US 52 were rehabilitated (including diamond grinding) since the 1994 study. Consequently, field performance data such as slab faulting and PSR were potentially affected. The 2006 field and laboratory testing data comparisons are presented in Table 22.

Cracking in the recycled section, both longitudinal and transverse, exceeded the control. The recycled section had transverse cracks in 92% of its slabs while the control only had them in 24%. The recycled section's transverse cracks were more deteriorated as well, having 125 deteriorated transverse cracks per km of road versus the control section's 26 cracks per km. A similar difference was also reported in the 1994 study.

Table 22: MN 4-1 and MN 4-2 Field and Laboratory Performance Data

Test and Value	MN 4-1 (Recycled)	MN 4-2 (Control)	Difference (control vs. recycled)	Best
Aggregate Top Size, mm	38	25	52%	R
Recycled Fines, %	0	0	0%	=
Transverse Joint Spalling, % Joints	81	100	-19%	R
Transverse Joint Seal Damage, % Joints	100	100	0%	=
Longitudinal Joint Seal Damage, m/km	973	1000	<-1%	R
D-cracking, % Slabs	0	0	0%	=
Pumping, % Slabs	0	0	0%	=
Slab/Patch Deterioration, % Slabs	3	0	3%	C
Avg. Lane to Shoulder Drop off, mm	20	11	81%	C
Avg. Lane to Shoulder Separation, mm	4	4	0%	=
Avg. Faulting between Panels, mm	0.9	0.9	0%	=
Avg. Joint Width, mm	12	11	9%	C
Longitudinal Cracking, m/km	17	0	>100%	C
Transverse Cracking, % Slabs	92	24	68%	C
Deteriorated Transverse Cracks/km	125	26	381%	C
Total Transverse Cracks/km	131	29	352%	C
PSR	3.0	3.8	-21%	C
IRI	1.7	1.0	70%	C
Tensile Strength, MPa	2.4	2.5	-4%	C
Compressive Strength, MPa	45.1	50.7	-11%	C
Uranyal Acetate Reaction	None	None	None	=
Young's Modulus, GPa	30.0	43.4	-31%	C
Average VSTR (cm^3/cm^2)	0.2902	0.3264	-11%	C

There are a few possible causes for the recycled sections increase in cracking. First, from the 1994 study, the recycled pavement's foundation stiffness was calculated to be 30% less than that of the control section.[2] Decreasing foundation support increases pavement stress, making it more susceptible to cracking from traffic loading. Second, the control section used a 1.5" (38 mm) aggregate top size while the recycled used a 1.0" (25 mm) top size, which would be expected to increase shrinkage cracking. Additionally, the mortar content used in the recycled section was higher than the control (recycled had 83.6% and the control had 51.5%).[2] As shown from 2006 strength testing, the control section was stronger and had a higher Young's Modulus than the recycled. As in 1994, the high severity of the recycled pavement's transverse cracks was probably the principal factor which made the PSR of the control section almost 1 rating higher, since both sections faulting values were equal.

The compressive strength of 45.1 MPa for the recycled was comparable to other recycled sections. The recycled section, having a higher amount of recycled fines, made its Young's Modulus value 31% lower than the control sections.

I-94 Brandon, MN

The 2006 field and laboratory testing data comparisons of MN1-1 and MN1-2 are presented in Table 23. The amount of transverse joint spalling in the control was 54% while the recycled was 76%. There was 8% more joint spalling in the recycled section then in the control in 1994. While both sites had moderate amounts of joint spalling, all of it was of low severity. The recycled section's average joint faulting was only 0.9 mm, while the control's was 1.3 mm. 31% of the recycled section's slabs had transverse cracking.

The recycled had a total mortar content of 76.7 percent while the control only had 65.7%, which could have made the recycled section crack more. Ultimately, transverse cracking probably led to the recycled section's lower PSR. The laboratory strength and rigidity testing shows a drop from the control to the recycled. The recycled sections compressive strength was 44.9 MPa, 25% lower than the control section's.

Table 23: MN 1-1 and MN 1-2 Field and Laboratory Performance Data

Test and Value	MN 1-1 (Recycled)	MN 1-2 (Control)	Difference (control vs. recycled)	Best
Aggregate Top Size, mm	19	19	0%	=
Recycled Fines, %	0	0	0%	=
Transverse Joint Spalling, % Joints	76	54	22%	C
Transverse Joint Seal Damage, % Joints	100	95	5%	C
Longitudinal Joint Seal Damage, m/km	1000	1000	0%	=
D-cracking, % Slabs	0	0	0%	=
Pumping, % Slabs	0	0	0%	=
Slab/Patch Deterioration, % Slabs	0	0	0%	=
Avg. Lane to Shoulder Drop off, mm	22	30	-27%	R
Avg. Lane to Shoulder Separation, mm	2	2	0%	=
Avg. Faulting between Panels, mm	0.9	1.3	-31%	R
Avg. Joint Width, mm	11	10	10%	C
Longitudinal Cracking, m/km	0	0	0%	=
Transverse Cracking, % Slabs	31	0	31%	C
Deteriorated Transverse Cracks/km	35	0	n/a	C
Total Transverse Cracks/km	38	0	n/a	C
PSR	3.7	4.0	-8%	C
IRI	1.1	0.9	22%	C
Tensile Strength, MPa	2.9	3.3	-12%	C
Compressive Strength, MPa	44.9	59.0	-24%	C
Uranyal Acetate Reaction	None	None	None	=
Young's Modulus, GPa	28.9	33.4	-13%	C

Nevertheless, 44.9 MPa is comparable to other recycled pavement's compressive strengths.

2006 Additional RCA Sections

This section contains a discussion and comparison of the 5 locations that had 2 recycled sections. Results contained in this section are only from the 2006 study.

I-94 Menomonie, WI

Since the 1994 study, both recycled sections of I-94 were rehabilitated (including diamond grinding). Consequently, field performance data such as slab faulting and PSR was positively affected. The 2006 field and laboratory testing data comparisons are presented in Table 24.

Table 24: WI 1-1 and WI 1-2 Field and Laboratory Performance Data

Test and Value	WI 1-1 (RCA 1)	WI 1-2 (RCA 2)	Difference (RCA 1 vs. RCA 2)	Best
Aggregate Top Size, mm	38	38	0%	=
Recycled Fines, %	0	0	0%	=
Transverse Joint Spalling, % Joints	98	91	7%	2
Transverse Joint Seal Damage, % Joints	98	100	-2%	1
Longitudinal Joint Seal Damage, m/km	1000	1000	0%	=
D-cracking, % Slabs	0	0	0%	=
Pumping, % Slabs	0	0	0%	=
Slab/Patch Deterioration, % Slabs	0	0	0%	=
Avg. Faulting between Panels, mm	2.3	0.5	360%	2
Avg. Joint Width, mm	9	11	-18%	1
Longitudinal Cracking, m/km	0	0	0%	=
Transverse Cracking, % Slabs	35	3	32%	2
Deteriorated Transverse Cracks/km	72	6	1100%	2
Total Transverse Cracks/km	75	6	1150%	2
PSR	2.8	3.7	-24%	2
IRI	1.9	1.1	73%	2
Tensile Strength, MPa	3.1	4.3	-28%	2
Compressive Strength, MPa	37.0	32.7	13%	1
Uranyal Acetate Reaction	Low	Low	None	n/a
Modified ASTM 1293, % Expansion at 108 Days	0.269	0.308	-15%	1
Young's Modulus, GPa	20.1	21.2	-5%	2

The fact that WI1-1 does not have dowel bars for load transfer makes it difficult to compare with WI1-2. Other then differing load transfer, both pavements have the same mix design and layout. The increased amount of transverse joint spalling, faulting and transverse cracks can be directly attributed to poor interlock between panels in WI1-1. The maximum joint width for adequate aggregate interlock is 0.76 mm and since WI1-1 had an average joint width of 9 mm, proper aggregate interlock between panels would virtually be nonexistant.[2] By not having transverse dowel bars between slabs, the PSR of WI1-1 was almost 1 rating lower than it's counterpart, which had dowel bars. Additionally, WI1-1 had a high average joint faulting value of 2.5 mm, while WI1-2 only had 0.5 mm. This project is a good example of the importance of using mechanical load transfer devices between slabs in recycled concrete pavements and not solely relying on load transfer from aggregate interlock.

Splitting tensile strengths were different between the two RCA sections. Uranyl acetate testing showed minimal amounts of ASR gel in the concrete. On the other hand, from modified ASTM 1293 testing, both sections had high expansion due to ASR, so the difference between the tensile strengths might be due to one core having more ASR then the other. ASR might also be a reason why the Young's Modulus values for both sections was low compared to other RCA sections. While ASR may not immediately decrease compressive strength it does decrease tensile strength. It can be hypothesized that the reaction of ASR in the pavement is taking its time as the expansion rate of the two samples in the modified ASTM 1293 test only show the potential to expand.

Positive results of the uranyl acetate test suggested that both sections should be petrographicly analyzed. Both sections contained many aggregate particles with ASR gel deposits. The aggregates and paste also had many micro cracks with ASR gel. An example of a typical ASR gel deposit along a RCA and new paste border can be seen in Figure 50.

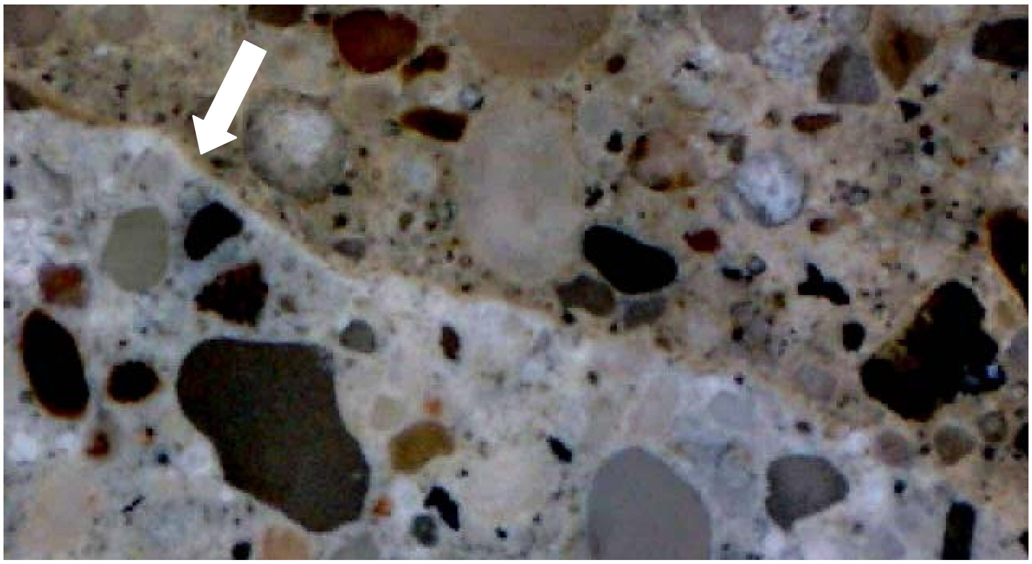


Figure 50: W11-2 RCA Border with ASR Gel Deposit

A few instances of cracks propagating from RCA into new paste were also found, as shown in Figure 51.

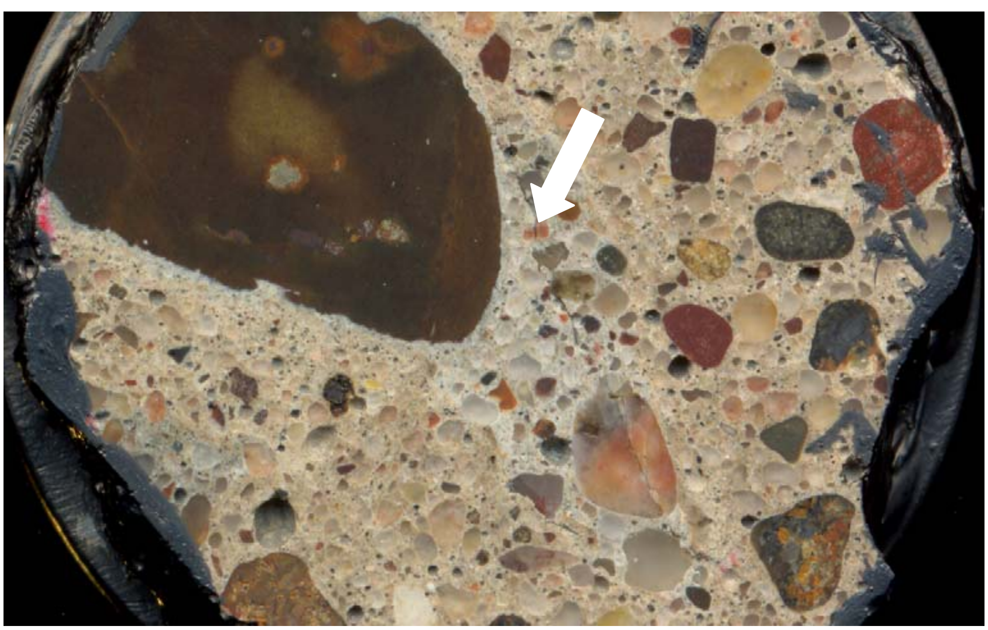


Figure 51: W11-1 Crack Propagating from RCA

According to the modified ASTM 1293 testing, the two RCA sections of WI1 were some of the most ASR reactive concretes studied, suggesting high potential for continued expansion in the field.

I-90 Beloit, WI

Since the 1994 study, both recycled concrete sections of I-90 were overlaid with asphalt. Consequently, field performance data were not available for these sections in the 2006 study. Laboratory testing data comparisons are presented in Table 25.

Table 25: WI 2-1 and WI 2-2 Laboratory Testing Data

Test and Value	WI 2-1 (RCA 1)	WI 2-2 (RCA 2)	Difference (RCA 1 vs. RCA 2)	Best
Aggregate Top Size, mm	38	38	0%	=
Recycled Fines, %	0	0	0%	=
Tensile Strength, MPa	3.9	2.9	35%	1
Compressive Strength, MPa	43.9	45.4	-3%	2
Uranyal Acetate Reaction	Low	High	2 Levels	1
Modified ASTM 1293, % Expansion at 108 Days	n/a	0.389	n/a	1
Young's Modulus, GPa	25.6	20.5	25%	1
Average VSTR (cm^3/cm^2)	0.4493	n/a	n/a	n/a

WI2-1 performed better in most of the strength categories. Uranyl acetate testing showed that there was a lot of ASR gel in WI2-2 and a small amount in WI2-1. The higher amount of ASR in WI2-2 may be the reason why the tensile strength and rigidity values for WI2-1 are 25% and 35% higher. However, Young's Modulus values for both sections are low compared to other recycled sections. Due to the lack of cylinders retrieved from WI2-1, it was not possible perform a modified ASTM 1293 test for that section. A cylinder for WI2-2 was however set up for modified ASTM 1293 testing.

The WI2-2 pavement exhibited the most expansion in the modified ASTM 1293 test, showing a very high potential for expansion. A petrographic study was done on the concrete to evaluate for ASR. Under the microscope, many aggregate pieces were found to have ASR gel deposits. The aggregates also had cracks running into them with ASR gel deposits inside. An example of a piece of aggregate with ASR gel filled cracks is presented in Figure 52.

Figure 52: W12-2 Aggregate Crack with ASR Gel Deposit

The Wisconsin DOT was contacted following the study to find out why the pavement was overlaid with asphalt. The state reported that the road was overlaid with 4" of asphalt between 2004 and 2005 because they noticed a substantial amount of d-cracking, which was creating pop outs. The road is slated to be replaced in the next 10-15 years.[25] From the petrographic study there were so signs of micro cracks radiating from aggregate, which would show d-cracking. The DOT probably did not realize that the concrete was actually suffering from ASR.

The ASR reactivity in both sections of the concrete should be further studied since there was a lack of samples available for modified ASTM 1293 testing during the 2006 study. Since being overlaid, the pavement's ASR reaction can be expected to increase due to the higher moisture and temperature that the somewhat impermeable asphalt membrane traps at the concretes surface.

I-90 Rock Co., MN

The 2006 field and laboratory testing data comparisons are presented in Table 26. The field performance of the two recycled sections in MN2 were similar, which was expected as both sections used the same mix and design. Both had a high amount of transverse cracking, with MN2-1 having 92% of its slabs containing transverse cracks and MN1-1 having 90%. The high amount of transverse cracking may be due to a high mortar content and low aggregate top size. The mortar content of both sections was reported to be 79.0% from the 1994 study.[2] The aggregate top size of 3/4" was low compared to other recycled sections. The higher mortar content and lower top aggregate size may have made both sections more susceptible to panel cracking due to increased shrinkage and the lowered strength and rigidity. MN 2-1 did have 26 m of longitudinal cracking per km of road as well. The PSRs of 4.0 for MN2-1 and 3.8 for MN2-2 were similar and showed that both pavements were still at a serviceable level.

Results from compression, splitting tension and Young's Modulus testing on both sections showed that they were comparable to other recycled pavements.

Table 26: MN 2-1 and MN 2-2 Field and Laboratory Performance Data

Test and Value	MN 2-1 (RCA 1)	MN 2-2 (RCA 2)	Difference (RCA 1 vs. RCA 2)	Best
Aggregate Top Size, mm	19	19	0%	=
Recycled Fines, %	0	0	0%	=
Transverse Joint Spalling, % Joints	46	66	-20%	1
Transverse Joint Seal Damage, % Joints	100	100	0%	=
Longitudinal Joint Seal Damage, m/km	1000	1000	0%	=
D-cracking, % Slabs	0	0	0%	=
Pumping, % Slabs	0	0	0%	=
Slab/Patch Deterioration, % Slabs	5	0	5%	2
Avg. Lane to Shoulder Drop off, mm	11	13	-15%	1
Avg. Lane to Shoulder Separation, mm	2	4	-50%	1
Avg. Faulting between Panels, mm	0.6	0.5	20%	2
Avg. Joint Width, mm	12	13	-8%	1
Longitudinal Cracking, m/km	26	0	<100%	2
Transverse Cracking, % Slabs	90	92	-2%	1
Deteriorated Transverse Cracks/km	112	112	0%	=
Total Transverse Cracks/km	112	115	-3%	1
PSR	4.0	3.8	5%	1
IRI	0.9	1.0	-10%	1
Tensile Strength, MPa	3.7	2.8	32%	1
Compressive Strength, MPa	49.5	64.1	-23%	2
Uranyal Acetate Reaction	Low	None	1 Level	2
Modified ASTM 1293, % Expansion at 108 Days	0.054	n/a	n/a	2
Young's Modulus, GPa	n/a	31.1	n/a	n/a

Both MN2-1 and MN2-2 were tested for ASR using uranyl acetate, but only MN2-1 showed signs of ASR. From modified ASTM 1293 testing it became apparent that the rate of expansion for the concrete was very small and that cracking issues in the pavement cannot mainly be attributed to ASR.

I-57 Effingham, IL

The 2006 field and laboratory testing data comparisons are presented in Table 27.

Table 27: IL 1-1 and IL 1-2 Field and Laboratory Performance Data

Test and Value	IL 1-1 (RCA 1)	IL 1-2 (RCA 2)	Difference (RCA 1 vs. RCA 2)	Best
Aggregate Top Size, mm	38	38	0%	=
Recycled Fines, %	35	36	-1%	1
Longitudinal Joint Seal Damage, m/km	1000	1000	0%	=
D-cracking, % Slabs	100	47	53%	2
Pumping, % Slabs	0	0	0%	=
Slab/Patch Deterioration, % Slabs	0	0	0%	=
Avg. Lane to Shoulder Drop off, mm	n/a	8	n/a	n/a
Avg. Lane to Shoulder Separation, mm	3	12	-75%	1
Longitudinal Cracking, m/km	1252	527	138%	2
Deteriorated Transverse Cracks/km	0	n/a	n/a	n/a
Total Transverse Cracks/km	0	59	<-100%	1
Tensile Strength, MPa	1.9	3.6	-47%	2
Compressive Strength, MPa	56.0	55.2	2%	1
Uranyal Acetate Reaction	High	High	None	=
Modified ASTM 1293, % Expansion at 108 Days	0.345	0.166	98%	2
Young's Modulus, GPa	29.1	26.7	9%	1

Illinois was the first RCA pavement site studied in 2006 that was not part of the 1994 study. I-57 was chosen since it was a CRCP and it had 2 recycled sections with different mixtures. Both sections used mixtures which had a high amount of RCA fines. There was also literature available that described its design and gave some field and laboratory testing values.

Both IL1-1 and IL1-2 were paved using RCA from an old pavement constructed in 1964. The pavement was a 100' jointed pavement and at the time of recycling had 2 to

3 badly faulted transverse cracks per slab.[26] The pavement did not show any signs of D-cracking. To crush the pavement a jaw and roll crusher were used. Before use, the RCA was tested using Illinois DOT's sodium sulfate test. The test results showed that the RCA failed the test, so the requirement was waived for the RCA.[26]

Both sections used 15% class C fly ash. The original mix design of the northbound lanes used a high amount of recycled fines. After poor workability, the initial mixed was revised.[26] More fly ash and more virgin fine aggregate were added, while the amount of recycled fines was lowered to improve workability. 35% of the total fine aggregate was RCA fines for revised design for the northbound lanes. The southbound lanes mix design varied somewhat. It had a higher water to cement ratio, but also used more recycled fines (36% of the total fines content). The northbound lanes were paved with RCA pavement in 1986 and southbound in 1987. The Illinois DOT reported that while the workability of the mix was poor (due to the high amount of RCA fines), using the class C fly ash improved workability.[26]

A site survey was done by the Illinois DOT in 1990. The primary distress they reported seeing was random low severity longitudinal cracks. The southbound lanes exhibited 25 meters of longitudinal cracking per kilometer of road.[26] The longitudinal cracking was attributed to the contractor not cutting the longitudinal joint in a timely manner.[26] In 1992, the study was done again and 36 meters of longitudinal cracking per kilometer of road was found. In 2006, the southbound lane had 526 meters of longitudinal cracking per km of road. The increase in longitudinal cracking was substantial between 1992 and 2006. The northbound section (IL1-1) also exhibited a significant amount of longitudinal cracking, more than twice the amount found in the southbound lane (IL1-2).

From testing in 2006, both sections were found to have D-cracking. IL1-1 had 100% of the pavement experiencing d-cracking while IL1-2 had 47%. No D-cracking was reported in the 1990 and 1992 study by the Illinois DOT.[26] Only IL1-2 exhibited transverse cracking in 2006, with 59 cracks per km of pavement.

Compressive strengths and Young's Modulus values for each section were either better than or comparable to other recycled pavements in the study. Uranyl acetate testing indicated a large amount of ASR gel inside the concrete of both sections. Figure 53 shows the uranyl acetate test on section 1 of IL.

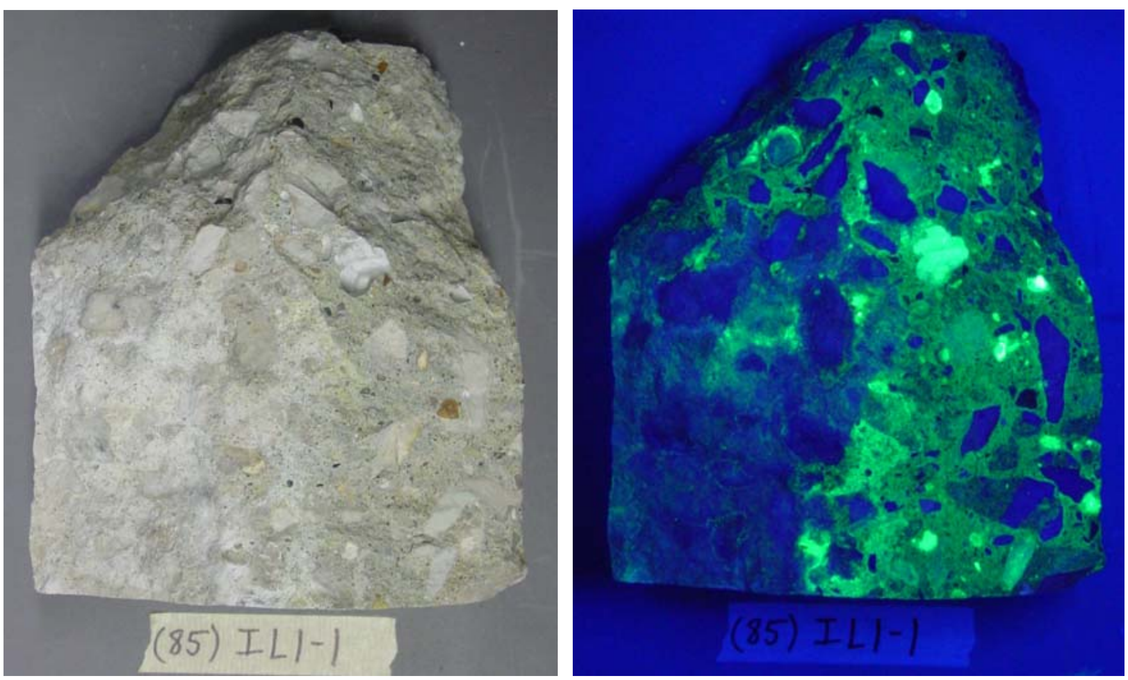


Figure 53: Fractured Core 85 (IL 1-1) coated with Uranyl Acetate Dihydrate under UV light (right) showing ASR gel and under regular light (left)

Some of these aggregates have natural fluorescence and light up blue under UV light. These aggregates should not be misconstrued as ASR.

Figures 54 and 55 show the uranyl acetate test on IL1-2.

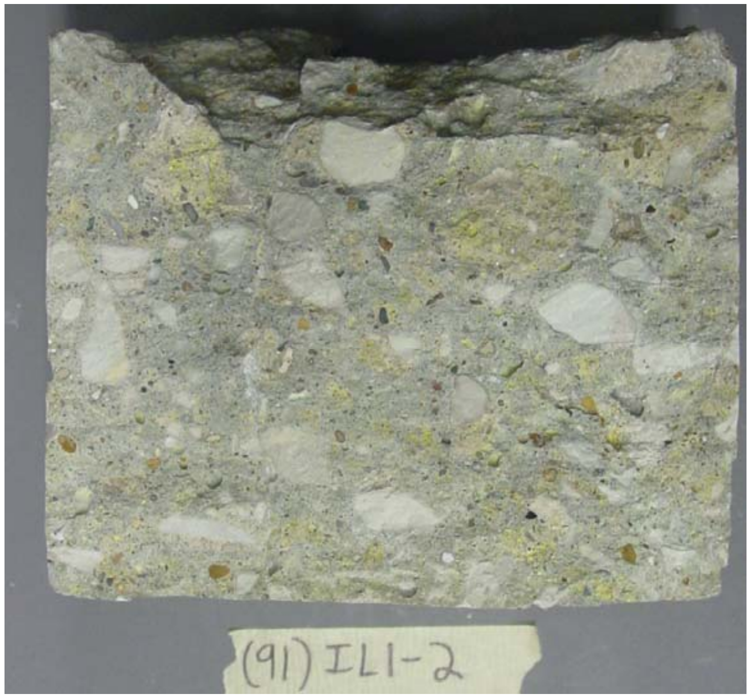


Figure 54: Fractured Core 91 (IL 1-2) coated with Uranyl Acetate Dihydrate under regular light

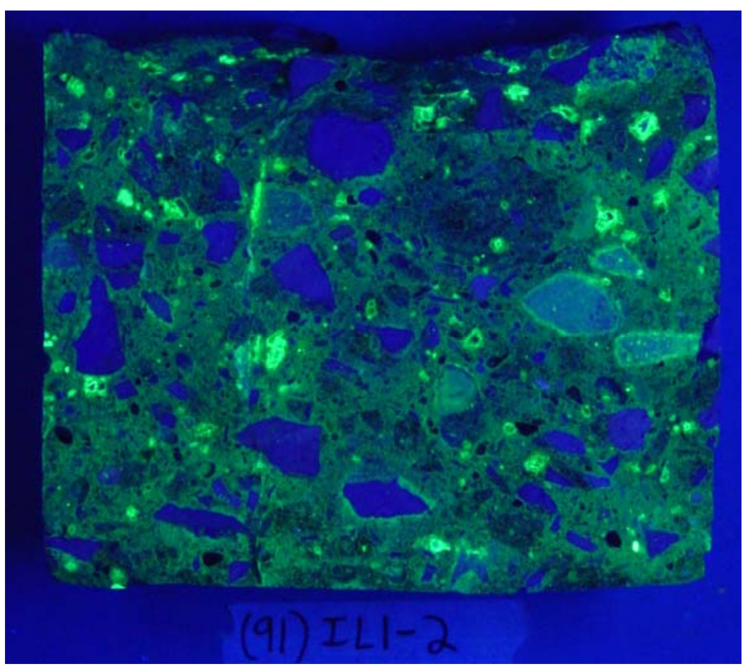


Figure 55: Fractured Core 91 (IL 1-2) coated with Uranyl Acetate Dihydrate under UV light

When the cores from IL were shipped to the laboratory it was noted that their quality was poor. Most of the cylinders had small cracks in them and some were reduced to rubble during shipping. Both uranyl acetate and modified ASTM 1293 testing showed that ASR was an issue with both sections. A petrograhpic examination of each section was performed to analyze for ASR. Both sections showed many micro cracks with ASR gel deposits in them, as well as many voids filled with ASR gel. Furthermore, both showed cracks propagating out from RCA into new paste. Figure 56 shows a crack going through both RCA and new paste found on a sample from IL1-1. Figure 57 shows a void filled with ASR gel found on a sample from IL1-2.

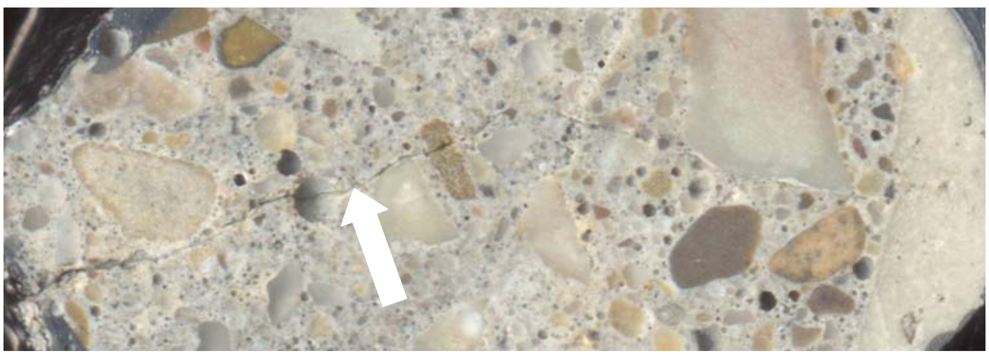


Figure 56: IL1-1 Crack Propagating from RCA

Figure 57: IL1-2 Void filled with ASR Gel

Figure 58 shows a SEM interpretation of the crack surface shown in Figure 56 indicating the different areas that were analyzed.

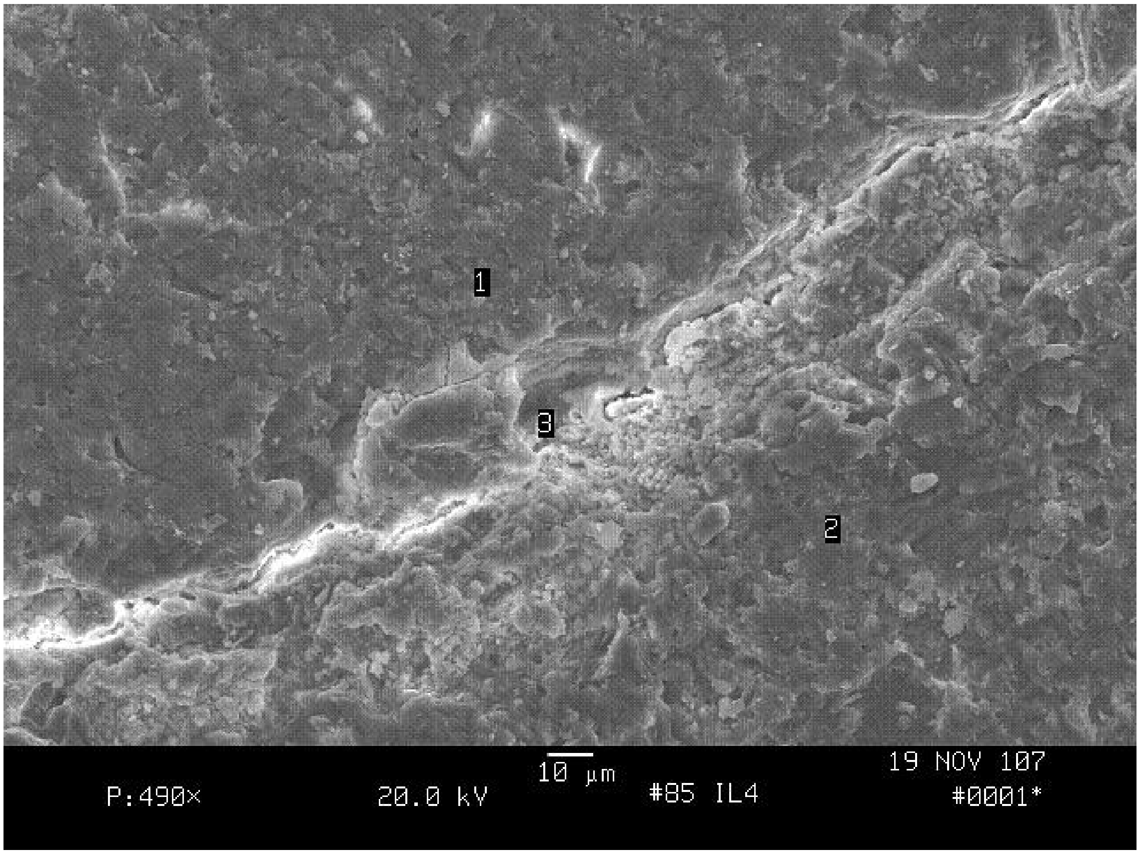


Figure 58: SEM Interpretation of Surface around Crack (IL1-1)

Area 2 was a piece of recycled aggregate and Area 3 was the inside of the crack. Table 28 shows the EDS for each of the 2 areas. The EDS showed that the crack had a higher amount of sodium, silica and calcium, which shows that the substance filled in the crack could possibly be ASR gel and/or calcium hydroxide.

From the field survey and ASR testing, it becomes apparent that both sections of IL1 are experiencing ASR, IL1-1 more so than IL1-2. The fact that the original pavement recycled had substantial cracking and distress suggests that it might have been experiencing ASR.

Table 28: Elemental Analysis for ASR Crack (IL1-1)

Sample Area 2
Accelerating Voltage: 20.00 keV
Takeoff Angle: 30.00 degrees
Library for system standards: /imix/quant/efficiency/default.dir

Elm	Rel. K	ZAF	Norm wt%	Prec.	Atomic %	Standard	Intensity	
C	0.0107	4.3594	4.66	0.59	8.37	(S)C_K	1799	
Mg	0.0413	2.1824	9.02	0.29	8.01	(S)Mg_K	24205	
Al	0.0108	1.9863	2.15	0.17	1.73	(S)Al_K	5768	
Si	0.0340	1.5936	5.42	0.20	4.17	(S)Si_K	19248	
S	0.0012	1.2828	0.16	0.08	0.10	(S)S_K	616	
Cl	0.0012	1.2170	0.15	0.07	0.09	(S)Cl_K	655	
Ca	0.2922	1.0816	31.60	0.43	17.03	(S)Ca_K	122979	
Fe	0.0106	1.1872	1.26	0.17	0.49	(S)Fe_K	2312	
Na	0.0008	3.2647	0.27	0.00	0.25	(S)Na_K	0	
K	0.0133	1.0445	1.39	0.13	0.77	(S)K_K	5047	
Ti	0.0018	1.3058	0.24	0.11	0.11	(S)Ti_K	582	
Mn	0.0006	1.2274	0.07	0.12	0.03	(S)Mn_K	131	?
O	0.0631	6.9121	43.61	1.33	58.85	(S)O_K	24090	
Total			100.00		100.00			

Sample Area 3
Accelerating Voltage: 20.00 keV
Takeoff Angle: 30.00 degrees
Library for system standards: /imix/quant/efficiency/default.dir

Elm	Rel. K	ZAF	Norm wt%	Prec.	Atomic %	Standard	Intensity	
C	0.0033	4.4349	1.46	0.48	3.54	(S)C_K	280	
Mg	0.0020	2.3848	0.48	0.18	0.56	(S)Mg_K	442	
Al	0.0107	1.8381	1.96	0.21	2.10	(S)Al_K	2786	
Si	0.0691	1.4953	10.33	0.37	10.67	(S)Si_K	18372	
S	0.0011	1.2692	0.14	0.06	0.13	(S)S_K	448	
Cl	0.0126	1.1913	1.50	0.16	1.23	(S)Cl_K	3226	
Ca	0.4855	1.0637	51.64	0.77	37.38	(S)Ca_K	97905	
Fe	0.0852	1.1878	10.12	0.58	5.26	(S)Fe_K	7837	
Na	0.0003	3.6461	0.12	0.41	0.15	(S)Na_K	29	?
K	0.0101	1.0050	1.02	0.15	0.76	(S)K_K	2099	
Ti	0.0016	1.3645	0.22	0.12	0.14	(S)Ti_K	343	
Mn	0.0000	1.2383	0.00	0.00	0.00	(S)Mn_K	338	
O	0.0214	9.8159	21.01	1.70	38.09	(S)O_K	3593	
Total			100.00		100.01			

Since Illinois did not know their pavement had ASR there was no mitigation strategy employed when using the RCA in the new pavement. The pavement is expected to continue cracking and deteriorating in the future. Further field surveys and ASR testing should be done to determine the rate and extent of distress caused by the ASR.

US 75 Rock Rapids, IA

The 2006 field and laboratory performance data comparisons are presented in Table 29.

Table 29: IA 1-1 and IA 1-2 Field and Laboratory Performance Data

Test and Value	IA 1-1 (RCA 1)	IA 1-2 (RCA 2)	Difference (RCA 1 vs. RCA 2)	Best
Aggregate Top Size, mm	38	38	0%	=
Recycled Fines, %	16	30	-14%	1
Transverse Joint Spalling, % Joints	100	100	0%	=
Transverse Joint Seal Damage, % Joints	100	100	0%	=
Longitudinal Joint Seal Damage, m/km	1000	1000	0%	=
D-cracking, % Slabs	15	2	13%	2
Pumping, % Slabs	0	0	0%	=
Slab/Patch Deterioration, % Slabs	2	0	>100%	1
Avg. Faulting between Panels, mm	2.2	3.6	-39%	1
Avg. Joint Width, mm	18	17	6%	2
Longitudinal Cracking, m/km	12	0	>100%	2
Transverse Cracking, % Slabs	29	2	27%	2
Deteriorated Transverse Cracks/km	36	3	1100%	2
Total Transverse Cracks/km	49	3	1533%	2
Tensile Strength, MPa	2.5	2.8	-11%	2
Compressive Strength, MPa	52.6	47.6	11%	1
Uranyal Acetate Reaction	None	Low	1 Level	1
Modified ASTM 1293, % Expansion at 108 Days	n/a	0.187	n/a	1
Young's Modulus, GPa	28.3	24.6	15%	n/a

Iowa was the second RCA pavement site studied in 2006 that was not part of the 1994 study. US 75 was chosen since it was one of the oldest RCA pavements in the US, it had 2 recycled sections with different mixtures and each mixture had a different amount of RCA fines. The pavements were constructed in 1976 and there was literature available that described their design.

Both IA1-1 and IA1-2 were paved using RCA from an old pavement constructed in 1936. The pavement was a one mile segment of 10" thick pavement.[27] The pavement had been overlaid with asphalt at some point. The asphalt had to be stripped off before the pavement was broken up for crushing. Crushing was performed using a jaw crusher to minimize the amount of fines produced.

The Iowa DOT wanted to study the effect of workability by varying the amount of coarse and fine aggregates.[27] It was decided to produce 2 mixtures, the first with a 35/65 RCA:FA ratio with 16% recycled fines and the second with 50/50 RCA:FA with 30% recycled fines.[27] Both mixes had a 38 mm (1.5") aggregate top size. A third composite mix was done, but was not included in the 2006 study. The Iowa DOT reported that the mix with 65% fines was too heavily sanded.[27]

Transverse joint spalling was present at all of the joints for both sections, but for the most part is was of low severity. IA1-1 had 15% of its slabs experiencing d-cracking while IA1-2 had none. IA1-2 did show an average faulting of 3.6 mm, while IA1-1's average faulting was 2.2 mm (39% lower). IA1-1 had more transverse cracks and some longitudinal cracking. Since the design of the pavements was similar, the variation in performance can probably be attributed to the mix design. However, no correlation could be made between the field and lab data and the amount of recycled fines added to each section. IA1-2 had 30% fines and showed less cracking than IA1-1, which had only 16% fines. This is opposite of what was expected since higher fines usually increases shrinkage cracking and will lower strength and rigidity.

Strength data were similar for the two recycled sections. IA1-2's splitting tensile strength was 11% higher than IA1-1's. The decrease in compressive strength and Young's Modulus from IA1-2 might be attributed to it having 30% recycled fines, while IA1-1 only used 16%. Both sections compressive strengths were comparable to other recycled sections. During uranyl acetate testing, IA1-2 showed ASR. A core was put through modified ASTM 1293 testing and expanded 0.187% at 108 days, indicating a potential for continued expansion in the field. ASR does not however explain why IA1-1 had more cracking than IA1-2.

Other than faulting between panels, both of the Iowa sections performed fairly well for a pavement of their age. Their cracking values were comparable to other recycled sections studied. Their faulting values could easily be reduced through road refurbishing, including diamond grinding. Future studies should be done on US 75 to determine the pavements rate of deterioration.

All 2006 Studied Sites

This section provides a final comparison between all recycled and control sections studied in 2006. While this comparison is necessary it pools all data, so variables such as climate conditions, traffic loadings, pavement ages, internal distresses, etc are included.

There were a total of 5 control sections and 16 recycled sections that were part of the 2006 study. All but 3 recycled sections (KS1-1, WI2-1 and WI2-2) and 1 control section (WI2-1) were used for compiling average field performance values. All 21 pavement sections were used when compiling average lab testing values. Table 30 presents data comparisons for the average recycled and control sections.

Table 30: Averaged Data Comparisons for 2006 Control and Recycled Sections

Test	Average Recycled (2006)	Average Control (2006)	Difference (Recycled vs. Control)	Best
Aggregate Top Size, mm	32	32	0%	=
Recycled Fines, %	12%	0%	12%	C
Transverse Joint Spalling, % Joints[a]	80%	74%	6%	C
Transverse Joint Seal Damage, % Joints[a]	83%	97%	-14%	R
Longitudinal Joint Seal Damage, m/km[a]	997 (99%)	1000 (100%)	-1%	R
D-cracking, % Slabs[b]	13%	0%	13%	C
Pumping, % Slabs[a]	0%	0%	0%	=
Slab/Patch Deterioration, % Slabs[a]	1%	0%	1%	C
Avg. Lane to Shoulder Drop off, mm[a]	13	21	-8 (-38%)	R
Avg. Lane to Shoulder Separation, mm[a]	6	10	-4 (-40%)	R
Avg. Faulting between Panels, mm[a]	1.2	1.0	0.2 (20%)	C
Avg. Joint Width, mm[a]	13	11	2 (18%)	C
Longitudinal Cracking, m/km[b]	326 (33%)	2 (<1%)	324 (33%)	C
Transverse Cracking, % Slabs[a]	45%	30%	15%	C
Deteriorated Transverse Cracks/km[a]	52	7	45	C
Total Transverse Cracks/km[a]	58	17	41	C
PSR[a]	3.7	3.8	-0.1 (-3%)	C
IRI[a]	1.1	1.0	0.1 (10%)	C
Tensile Strength (MPa)[a]	3.1	3.1	0	=
Compressive Strength (MPa)[a]	48.0	47.5	0.5 (1%)	R
Uranyal Acetate Reaction[a]	Low	None	1 Level	C
Modified ASTM 1293, % Expansion at 108 Days[a]	0.2569%	0.1670%	0.0899 (54%)	C
Young's Modulus (GPa) [a]	28.4	34.1	-5.7 (-17%)	C
Average VSTR (cm^3/cm^2) [a]	0.3842	0.5290	-0.1448 (-27%)	C

Note: [a] Statistically no difference at the 5% α level
[b] Statistically different at the 5% α level

A T-test was used for statistical analysis of the averaged values for each test, with an α level of 0.05. For most tests the difference between the average control and average recycled was statistically insignificant. D-cracking was one of two performance properties that were determined to be statistically different from recycled to control.

IL1-1 and IL1-2 were both recycled sections and 100% and 47% of their slabs had d-cracking, respectively. Both recycled sections in Iowa had D-cracking as well, with IA1-1 having 15% and IA1-2 having 2%. All other sites studied in 2006 did not have any d-cracking. This is a property of the aggregate, so proper testing for d-cracking in RCA should be done prior to its use.

The second statistically different test was longitudinal cracking. The recycled sections did not perform as well as the controls in this category. The most likely reason for this is because of ASR. 10 sections were found to have ASR, 9 of which were recycled sections and only one was a control (WY1-2). Of the 10 sections that had ASR, all but 3 were found to have longitudinal cracking, a prime indicator of ASR expansion. Once expansion is produced in the longitudinal direction it closes up transverse joints, so further expansion is easier in the transverse direction. This results in longitudinal tension, which will in turn create longitudinal cracking. A classical example of this was Il1-1, which had the second highest expansion at 108 days in the modified ASTM test and had the most longitudinal cracking of any other section (1252 m/km). Another example is WY1-1 which had the third highest expansion at 108 days from the modified ASTM test. It also had the third highest amount of longitudinal cracking (124 m/km).

Only 2 other sections (IA1-1 and MN4-1) had longitudinal cracking but did not have ASR. Both IA1-1 and MN4-1 had the lowest values of longitudinal cracking found during the study, 12 m/km and 17 m/km, respectively. The area around the longitudinal cracking in MN4-1 was noticed to be settling, so differential settlement could have created the cracking. In IA1-1, there was only 1 crack and it was less than 1 m long and of low severity.

Values from both of these statistically significant tests could have easily been reduced or eliminated by proper material testing and mitigation of the recycled concrete. This comparison shows the importance of testing RCA for ASR susceptibility.

1994 Results vs. 2006 Results

This section contains a discussion and comparison of the 9 locations that were studied in both 1994 and 2006. Discussion and conclusions from the 1994 study that compared control and recycled concrete pavement sections can be found in Reference 2. Factors not common to both the 1994 and 2006 studies were excluded from this section.

K-7 Johnson County, KS

Since the 1994 study, both the control and recycled concrete sections of K-7 were overlaid with asphalt. As a result, only strength and durability data from lab testing on extracted cores can be compared. 1994 and 2006 laboratory testing data comparisons are presented in Table 31.

Laboratory strength testing showed that the recycled section performed better over time then the control did. The compressive strength for the recycled section remained at 47.9 MPa from 1994 to 2006 while the compressive strength for the control decreased 4% to 42.0 MPa.

I-90 Beloit, WI

Both recycled concrete sections of I-90 were overlaid with asphalt. As a result, field performance data is not available for these sections in the 2006 study. 1994 and 2006 laboratory testing data comparisons are presented in Table 32.

Table 31: KS 1-1 and KS 1-2 Laboratory Testing Data (1994 and 2006)

Test and Value	KS 1-1 (1994)	KS 1-1 (2006)	Change (1994 to 2006)	KS 1-2 (1994)	KS 1-2 (2006)	Change (1994 to 2006)	Difference (KS1-1 vs. KS 1-2)	Best
Tensile Strength, MPa	3.2	3.6	12%	3.6	3.7	3%	9%	R
Compressive Strength, MPa	47.9	47.9	0	43.7	42.0	-4%	4%	R

Table 32: WI 2-1 and WI 2-2 Laboratory Testing Data (1994 and 2006)

Test and Value	WI 2-1 (1994)	WI 2-1 (2006)	Change (1994 to 2006)	WI 2-2 (1994)	WI 2-2 (2006)	Change (1994 to 2006)	Difference (WI 2-1 vs. WI 2-2)	Best
Tensile Strength, MPa	3.5	3.9	11%	4.1	2.9	-29%	40%	2-1
Compressive Strength, MPa	55.5	43.9	-20%	44.3	45.4	3%	-23	2-2

The splitting tensile strength of WI2-1 actually went up 11%, while WI2-2 went down 29% to 2.9 MPa, which is still comparable to other recycled sections. WI2-2's tensile strength dropped and its low Young's modulus value of 20.5 GPa is likely attributed to its very high ASR reactivity. The opposite was true for the compressive strengths. WI1-1 dropped 20% to 43.9 MPa, which is lower than most recycled pavements.

I-94 Menomonie, WI

Since the 1994 study, both recycled sections of I-94 were rehabilitated (including diamond grinding). Consequently, field performance data such as slab faulting and PSR were positively affected. 1994 and 2006 field and laboratory performance data comparisons are presented in Table 33.

From 1994 to 2006, the undoweled section of I-94 (WI1-1) had a lower rate of deterioration than the doweled section (WI1-2). This was due to WI1-1 benefiting more from the rehabilitation than WI1-2 because it was so deteriorated. Even though WI1-1's joint faulting value went down from 2.8 to 2.3 after refurbishing, it is still high. The transverse cracking on WI1-1 also increased 2.5 times more than WI1-2, which was probably due to WI1-1 not having load transfer dowels. Overall, the PSR for WI1-2 only went down 0.1 to 3.7 over 12 years, while WI1-1 went down 1.3 to 2.8 (even with the pavement being refurbished). The results from this pavement survey show the importance of using dowel bars between transverse joints for load transfer.

Table 33: WI 1-1 and WI 1-2 Field and Laboratory Performance Data (1994 and 2006)

Test and Value	WI 1-1 (1994)	WI 1-1 (2006)	Change (1994 to 2006)	WI 1-2 (1994)	WI 1-2 (2006)	Change (1994 to 2006)	Difference (WI 1-1 vs. WI 1-2)	Best
Transverse Joint Spalling, % Joints	100	98	-2%	23	91	68%	-70%	1-1
Transverse Joint Seal Damage, % Joints	100	98	-2%	100	100	0%	-2%	1-1
Longitudinal Joint Seal Damage, m/km	1000	1000	0%	1000	1000	0%	0%	=
D-cracking, % Slabs	0	0	0%	0	0	0%	0%	=
Pumping, % Slabs	0	0	0%	0	0	0%	0%	=
Slab/Patch Deterioration, % Slabs	0	0	0%	0	0	0%	0%	=
Avg. Faulting between Panels, mm	2.8	2.3	-18%	0.5	0.5	0%	-18%	1-1
Avg. Joint Width, mm	10	9	-10%	11	11	0%	-10%	1-1
Longitudinal Cracking, m/km	0	0	0%	0	0	0%	0%	=
Transverse Cracking, % Slabs	8	35	27%	2	3	1%	26%	1-2
Deteriorated Transverse Cracks/km	0	72	>100%	0	6	>100%	0%	=
Total Transverse Cracks/km	16	75	369%	3	6	100%	269%	1-2
PSR	4.1	2.8	-32%	3.8	3.7	-3%	-29%	1-2
IRI	0.8	1.9	138%	1.0	1.1	10%	128%	1-2
Tensile Strength, MPa	3.0	3.1	3%	3.0	4.3	43%	-40%	1-2
Compressive Strength, MPa	34.2	37.0	8%	35.1	32.7	-7%	15%	1-1
Young's Modulus, GPa	29.0	20.1	-31%	28.0	21.2	-24%	-7%	1-2

Young's Modulus values of WI1-1 and WI1-2 both went down about the same amount since 1994. The decrease may be attributed to the increase in pavement age as concrete pavements loose tensile strength due to fatigue micro cracking from traffic loading. The tensile strength drop may also be caused by the ASR. Both recycled sections compressive strengths were lower than most of the other recycled sections, which might be due to ASR.

I-80 Pine Bluffs, WY

Since the 1994 study, both the control and recycled sections of I-80 were rehabilitated (including diamond grinding). As a result, some field performance data were positively affected. 1994 and 2006 field and laboratory performance data comparisons are presented in Table 34.

The recycled section of WY1 fared better than the control did over 12 years. While transverse joint spalling went up 61% in the control section, it only went up 22% in the recycled. Even though both sections of pavement were refurbished between 1994 and 2006, the control section had an increase in transverse joint seal damage while the recycled had a substantial decrease. As a result of retrofitting dowel bars and the refurbishing of the pavements, the faulting between panels for both sections was substantially improved. The recycled section's faulting went down to 0.7 from 2.0 mm and the controls went down to 0.6 from 2.0 mm. The only issue was that the recycled section had an increase in longitudinal cracking while the control did not, most likely due to the recycled section expanding from ASR.

Table 34: WY 1-1 and WY 1-2 Field and Laboratory Performance Data (1994 and 2006)

Test and Value	WY 1-1 (1994)	WY 1-1 (2006)	Change (1994 to 2006)	WY 1-2 (1994)	WY 1-2 (2006)	Change (1994 to 2006)	Difference (WY 1-1 vs. WY 1-2)	Best
Transverse Joint Spalling, % Joints	25	47	22%	16	77	61%	-39%	R
Transverse Joint Seal Damage, % Joints	97	16	-81%	96	100	4%	-85%	R
Longitudinal Joint Seal Damage, m/km	100	1000	>100%	0	1000	>100%	0	=
D-cracking, % Slabs	0	0	0	0	0	0	0	=
Pumping, % Slabs	0	0	0	0	0	0	0	=
Slab/Patch Deterioration, % Slabs	0	0	0	0	0	0	0	=
Avg. Lane to Shoulder Separation, mm	10	11	10%	11	14	27%	-17%	R
Avg. Faulting between Panels, mm	2.0	0.7	-65%	2.0	0.6	-70%	5%	C
Avg. Joint Width, mm	9	10	11%	11	10	-9%	20%	C
Longitudinal Cracking, m/km	55	124	125%	14	9	-36%	161%	C
Transverse Cracking, % Slabs	0	0	0	0	0	0	0	=
Deteriorated Transverse Cracks/km	0	0	0	0	0	0	0	=
Total Transverse Cracks/km	0	0	0	0	0	0	0	=
PSR	3.6	4.5	25%	3.6	4.2	17%	8%	R
IRI	1.2	0.6	-50%	1.2	0.7	-42%	-8%	R
Tensile Strength, MPa	3.7	2.9	-22%	3.2	3.0	-6%	-16%	C
Compressive Strength, MPa	48.7	54.6	12%	44.7	48.8	9%	3%	R
Young's Modulus, GPa	33.2	34.2	3%	29.1	29.7	2%	1%	R
Average VSTR (cm3/cm2)	0.2927	0.4131	41%	0.5043	0.7315	45%	-4%	C

A high percentage (92%) of transverse joints had seal damage back in 1994. From that time until the refurbishing it was a possibility that debris filled the cracks, which caused the panels to crack from ASR and thermal expansion. Even so, the PSR for the recycled section went up to 4.5 from 3.6, while the control only went up to 4.2 from 3.6.

The splitting tensile strength decreased more in the recycled section then in the control, possibly due to ASR and micro cracking. The large increase in VSTR values for both sections is most likely due to the type of probe used for VST testing. In 1994 a spring loaded probe was used, while a laser probe was used in 2006. The laser probe has been found to give a VSTR 1.4 times higher on a joint core than if it was tested using a spring loaded probe.[22] With that adjustment applied, the recycled section has had a lower loss of aggregate interlock capability then the control section.

I-84 Waterbury, CT

1994 and 2006 field and laboratory performance data comparisons are presented in Table 35. Transverse joint spalling for the recycled section, while quite high, stayed the same from 1994 to 2006. It did however increase on the control section. Faulting between panels went up from 0.3 to 1.0 mm for the recycled and 1.1 mm for the control. The amount of slabs with transverse cracks remained about the same, but the recycled section experienced a greater increase in deteriorated transverse cracks and total transverse cracks (58% and 26% respectively). As stated in the 2006 data discussion, transverse cracking may be attributed to the high traffic loading, long panel lengths (12 m) and the test strip being in the vicinity of an off ramp.

Table 35: CT 1-1 and CT 1-2 Field and Laboratory Performance Data (1994 and 2006)

Test and Value	CT 1-1 (1994)	CT 1-1 (2006)	Change (1994 to 2006)	CT 1-2 (1994)	CT 1-2 (2006)	Change (1994 to 2006)	Difference (CT 1-1 vs. CT 1-2)	Best
Transverse Joint Spalling,% Joints	92	92	0%	37	66	29%	-29%	R
Transverse Joint Seal Damage, % Joints	88	100	12%	38	94	56%	-44%	R
D-cracking, % Slabs	0	0	0	0	0	0	0	=
Pumping, % Slabs	0	0	0	0	0	0	0	=
Slab/Patch Deterioration,% Slabs	0	0	0	0	0	0	0	=
Avg. Faulting between Panels, mm	0.3	1.0	233%	0.3	1.1	267%	34%	R
Avg. Joint Width, mm	14	13	-7%	13	14	8%	-15%	R
Longitudinal Cracking, m/km	0	0	0%	0	0	0%	0%	=
Transverse Cracking, % Slabs	66	68	2%	93	93	0%	2%	C
Deteriorated Transverse Cracks/km	27	42	56%	33	38	15%	41%	C
Total Transverse Cracks/km	64	82	28%	115	131	14%	14%	C
PSR	3.4	3.7	9%	3.5	3.2	-9%	18%	R
IRI	1.3	1.1	-15%	1.2	1.5	20%	-35%	R
Tensile Strength, MPa	3.8	2.3	-39%	3.8	3.2	-19%	-20%	C
Compressive Strength, MPa	39.2	39.5	1%	35.4	37.0	5%	-4%	C

Overall, the increase in PSR for the recycled section from 1994 to 2006 is probably due to the primitiveness of the test itself because no aspects studied during the field survey improved from 1994 to 2006.

The splitting tensile strength of both sections decreased, probably due to the heavy traffic loading throughout the 26 years it has been in service. Compressive strengths increased for both sections, which was expected since concrete pavement will typically gain compressive strength with age.

US 52 Zumbrota, MN

Since the 1994 study, both the control and recycled sections of US 52 were rehabilitated (including diamond grinding). Consequently, field performance data such as slab faulting and PSR were potentially affected. 1994 and 2006 field and laboratory performance data comparisons are presented in Table 36.

The increase in transverse joint seal damage in the recycled section (88%) was more than the control (13%). Both sections showed 100% of their transverse joints with some level of seal damage in 2006. After rehabilitation of the pavement sections (including retrofitting of dowel bars and diamond grinding), the control section's faulting between panels actually went up .1 mm to 0.9 mm while the recycled sections went down .1 mm to 0.9 mm. The recycled section did start to shown some longitudinal cracking (17 m/km), but both sections had an increase in transverse cracking and the cracks severity levels.

Overall, the PSR of the recycled section dropped 1 rating to 3.0 from 1994 to 2006, while the control dropped from 0.4 to 3.8. This is probably due to the increased amount of deteriorated transverse cracks.

Table 36: MN 4-1 and MN 4-2 Field and Laboratory Performance Data (1994 and 2006)

Test and Value	MN 4-1 (1994)	MN 4-1 (2006)	Change (1994 to 2006)	MN 4-2 (1994)	MN 4-2 (2006)	Change (1994 to 2006)	Difference (MN 4-1 vs. MN 4-2)	Best
Transverse Joint Spalling, % Joints	76	81	5%	92	100	8%	-3%	R
Transverse Joint Seal Damage, % Joints	12	100	88%	87	100	13%	75%	C
Longitudinal Joint Seal Damage, m/km	0	973	>100%	0	1000	>100%	0%	=
D-cracking, % Slabs	0	0	0	0	0	0	0	=
Pumping, % Slabs	0	0	0	0	0	0	0	=
Slab/Patch Deterioration, % Slabs	0	3	3%	0	0	0	3%	C
Avg. Lane to Shoulder Drop off, mm	0	20	>100%	0	11	>100%	0%	=
Avg. Lane to Shoulder Separation, mm	0	4	>100%	0	4	>100%	0%	=
Avg. Faulting between Panels, mm	1.0	0.9	-10%	0.8	0.9	13%	-23%	R
Avg. Joint Width, mm	11	12	9%	11	11	0%	9%	C
Longitudinal Cracking, m/km	0	17	>100%	0	0	0%	>100%	C
Transverse Cracking, % Slabs	88	92	4%	22	24	9%	-5%	R
Deteriorated Transverse Cracks/km	80	125	56%	0	26	>100%	>-100%	R
Total Transverse Cracks/km	115	131	14%	26	29	12%	2%	C
PSR	4.0	3.0	-25%	4.2	3.8	-10%	-15%	C
IRI	0.9	1.7	89%	0.7	1.0	43%	46%	C
Tensile Strength, MPa	4.3	2.4	-44%	4.3	2.5	-42%	-2%	C
Compressive Strength, MPa	42.8	45.1	5%	47.6	50.7	7%	-2%	C
Young's Modulus, GPa	30.1	30.0	0%	33.3	43.4	30%	-30%	C
Average VSTR (cm3/cm2)	0.2372	0.2902	22%	0.2807	0.3264	16%	6%	R

As stated in the 2006 data discussion section, the high amount of deteriorated transverse cracks in the recycled section may be due to the section's comparably small 1.0" aggregate top size and/or its lower foundation support value. The decrease in tensile strength and increase in compressive strength for the recycled and control sections was approximately the same.

I-90 Rock Co., MN

1994 and 2006 field and laboratory performance data comparisons are presented in Table 37. Since both recycled sections of MN2 had the same design the field performance data were relatively similar. MN2-2 did however experience a greater increase in joint spalling than MN2-1, for reasons unknown. Both sites transverse and longitudinal joints had no damage in 1994 and in 2006 all of them were damaged. MN2-1 had the addition of some longitudinal cracking from 1994 to 2006. From uranyl acetate testing, MN2-1 did indicate some ASR, so longitudinal cracking might have occurred from expansion due to ASR (especially since joints seals were damaged and debris might have gotten into them).

Both sites did have a large increase in deteriorated transverse cracks. Even with an increase in pavement issues, both pavements performed fairly well in the PSR test. MN2-1 went down 0.1 to 4.0 and MN2-2 went down 0.5 to 3.8

Since strength data for MN2-2 were not available from 1994, nothing can be said for the difference in deterioration rates between the two recycled sites.

Table 37: MN 2-1 and MN 2-2 Field and Laboratory Performance Data (1994 and 2006)

Test and Value	MN 2-1 (1994)	MN 2-1 (2006)	Change (1994 to 2006)	MN 2-2 (1994)	MN 2-2 (2006)	Change (1994 to 2006)	Difference (MN 2-1 vs. MN 2-2)	Best
Transverse Joint Spalling, % Joints	21	46	25%	15	66	51%	26%	2-1
Transverse Joint Seal Damage, % Joints	0	100	100%	0	100	100%	0%	=
Longitudinal Joint Seal Damage, m/km	0	1000	>100%	0	1000	>100%	0%	=
D-cracking, % Slabs	0	0	0%	0	0	0%	0%	=
Pumping, % Slabs	0	0	0%	0	0	0%	0%	=
Slab/Patch Deterioration, % Slabs	0	5	5%	0	0	0%	-5%	2-2
Avg. Lane to Shoulder Drop off, mm	9	11	22%	10	13	30%	-8%	2-1
Avg. Lane to Shoulder Separation, mm	2	2	0%	4	4	0%	0%	=
Avg. Faulting between Panels, mm	0.8	0.6	-25%	n/a	0.5	n/a	n/a	n/a
Avg. Joint Width, mm	11	12	9%	11	13	18%	-9%	2-1
Longitudinal Cracking, m/km	0	26	>100%	0	0	0%	>100%	2-1
Transverse Cracking, % Slabs	84	90	6%	82	92	10%	-4%	2-1
Deteriorated Transverse Cracks/km	61	112	84%	42	112	167%	83%	2-1
Total Transverse Cracks/km	115	112	-3%	102	115	13%	-16%	2-1
PSR	4.1	4.0	-2%	4.3	3.8	-12%	10%	2-1
IRI	0.8	0.9	13%	0.7	1.0	43%	-30%	2-1
Tensile Strength, MPa	4.1	3.7	-10%	n/a	2.8	n/a	n/a	n/a
Compressive Strength, MPa	39.2	49.5	26%	n/a	64.1	n/a	n/a	n/a
Young's Modulus, GPa	29.2	n/a	n/a	n/a	31.1	n/a	n/a	n/a
Average VSTR (cm3/cm2)	0.2913	n/a	n/a	n/a	n/a	n/a	n/a	n/a

I-94 Brandon, MN

1994 and 2006 field and laboratory performance data comparisons are presented in Table 38. The recycled section did not fare quite as well in most categories as the control did. Transverse joint spalling went up 27% in the recycled section and only 13% in the control. The increase in faulting for the control (0.5 to 1.3 mm) was higher than the recycled (0.5 to 0.9mm). Transverse and longitudinal cracking increased only in the recycled section. The amount of deteriorated cracks in the recycled section increased substantially as well. All joints in both sections had at least low severity joint seal damage. The total mortar content in the recycled section was reported in 1994 as being 11% higher in the recycled section then in the control. Higher shrinkage and coefficient of thermal expansion might have led to this increase in transverse cracking. Those factors plus the 8.2 m panel lengths may have led to the transverse cracking increase in the recycled section.

The decrease in tensile strength for the recycled section (26%) was about the same as the control section (-28%). The compressive strength and Young's Modulus values of the recycled went down while the control went up. The recycled section's compressive strength (44.9 MPa) is still comparable to other recycled sections.

Table 38: MN 1-1 and MN 1-2 Field and Laboratory Performance Data (1994 and 2006)

Test and Value	MN 1-1 (1994)	MN 1-1 (2006)	Change (1994 to 2006)	MN 1-2 (1994)	MN 1-2 (2006)	Change (1994 to 2006)	Difference (MN 1-1 vs. MN 1-2)	Best
Transverse Joint Spalling, % Joints	49	76	27%	41	54	13%	14%	C
Transverse Joint Seal Damage, % Joints	0	100	100%	0	95	95%	5%	C
Longitudinal Joint Seal Damage, m/km	0	1000	>100%	0	1000	>100%	0%	=
D-cracking, % Slabs	0	0	0%	0	0	0%	0%	=
Pumping, % Slabs	0	0	0%	0	0	0%	0%	=
Slab/Patch Deterioration, % Slabs	0	0	0%	0	0	0%	0%	=
Avg. Lane to Shoulder Drop off, mm	20	22	10%	14	30	114%	-104%	R
Avg. Lane to Shoulder Separation, mm	0	2	>100%	2	2	0%	>100%	C
Avg. Faulting between Panels, mm	0.5	0.9	80%	0.5	1.3	160%	-80%	R
Avg. Joint Width, mm	11	11	0%	9	10	11%	-11%	R
Longitudinal Cracking, m/km	0	0	0%	0	0	0%	0%	=
Transverse Cracking, % Slabs	1	31	30%	0	0	0	30%	C
Deteriorated Transverse Cracks/km	3	35	>100%	0	0	0	>100%	C
Total Transverse Cracks/km	3	38	>100%	0	0	0	>100%	C
PSR	3.9	3.7	-5%	4.0	4.0	0%	-5%	C
IRI	0.9	1.1	22%	0.9	0.9	0%	22%	C
Tensile Strength, MPa	3.9	2.9	-26%	4.6	3.3	-28%	2%	R
Compressive Strength, MPa	47.3	44.9	-5%	46.5	59.0	27%	-32%	C
Young's Modulus, GPa	31.4	28.9	-8%	32.1	33.4	4%	-12%	C

US 59 Worthington, MN

Since the 1994 study, the recycled section of US-59 was rehabilitated (including diamond grinding). Consequently, field performance data such as slab faulting and PSR were positively affected. 1994 and 2006 field and laboratory performance data comparisons are presented in Table 39.

Table 39: MN 3 Field and Laboratory Performance Data (1994 and 2006)

Test and Value	**MN 3 (1994)**	**MN 3 (2006)**	**Change (1994 to 2006)**
Transverse Joint Spalling, % Joints	71	89	18%
Transverse Joint Seal Damage, % Joints	76	0	-76%
Longitudinal Joint Seal Damage, m/km	0	1000	>100%
D-cracking, % Slabs	0	0	0%
Pumping, % Slabs	2	0	-2%
Slab/Patch Deterioration, % Slabs	0	0	0%
Avg. Lane to Shoulder Drop off, mm	24	2	-92%
Avg. Lane to Shoulder Separation, mm	6	6	0%
Avg. Faulting between Panels, mm	6.1	0.3	-95%
Avg. Joint Width, mm	20	18	-10%
Longitudinal Cracking, m/km	19	0	-100%
Transverse Cracking, % Slabs	2	12	10%
Deteriorated Transverse Cracks/km	3	26	>100%
Total Transverse Cracks/km	3	26	>100%
PSR	3.0	4.3	43%
IRI	1.7	0.6	-65%
Tensile Strength, MPa	4.1	3.7	-10%
Compressive Strength, MPa	44.1	52.4	19%

An originally D-cracked pavement was broken up into RCA and mixed with all natural fine aggregate to pave US 59 in 1980. Other then CT1 and IA1, MN3 was the oldest recycled pavement of the 2006 study. Unfortunately, no control section was ever found, so MN3 only had one recycled section to study. MN3 was by far the most improved recycled pavement studied. In addition to diamond grinding, dowel bars were retrofitted for load transfer. Transverse joint seals were also replaced.

In the field study MN3 performed better than it did back in 1994 in every category except for transverse joint spalling, longitudinal joint seal damage and transverse cracking. The most astonishing decrease in distress was the drop in faulting between panels. The average faulting between panels dropped from 6.1 mm in 1994 to only 0.3 mm in 2006. Additionally, the lane to shoulder drop off decrease from 24 mm to 2 mm was quite amazing. The large drop in slab faulting (6.1 mm to 0.3 mm) was the biggest reason behind the PSR improvement of 3.0 in 1994 to 4.3 in 2006. If the amount of transverse cracks had not increased from 2% slabs in 1994 to 12% slabs in 2006, then the PSR probably would have been closer to 4.5.

The tensile strength went down 10% to 3.7 MPa 2006, which is acceptable since the roads is 36 years old and has had a fairly large traffic loading since its inception. Young's Modulus testing was not possible as all of the cores were too short for evaluation.

Overall Deterioration

This section provides a final comparison between the deterioration of all recycled and control sections studied in 1994 and 2006. This comparison cannot account for different climate conditions, traffic loadings, pavement ages, internal distresses, etc due to the low number of observations.

There were a total of 5 control sections and 12 recycled sections that were part of the 1994 and 2006 study. All but 3 recycled sections (KS1-1, WI2-1 and WI2-2) and 1 control section (WI2-1) were used for compiling overall deterioration for field performance. All 17 pavement sections studied in 1994 and 2006 were used when compiling average lab testing values.

The factor that had the most impact on the deterioration was pavement refurbishing. First, a comparison was done between pavements there were not refurbished between 1994 and 2006. Table 40 presents data comparisons for the average rates of deterioration for recycled and control sections not refurbished between 1994 and 2006.

Overall, there was little difference between the recycled and control sections. The increase in longitudinal cracking was higher for the recycled pavement than the control. This can be attributed to the high amount of recycled roads experiencing ASR as expansion from it may make panels crack longitudinally. The increase for deteriorated transverse cracks was also higher in the recycled sections. This can be attributed to low aggregate top size, which decreases the strength of a pavement and increased shrinakge. Also, one recycled section with substantial cracking was reported as having a foundation support value that was 30% less than its similarly designed control section.

Faulting between panels only increased 0.3 mm for the recycled section, whereas it increased 0.8 mm for the control. Tensile strengths decreased equally for both sections, which is expected since pavements loose their tensile strength as they age due to traffic loading and micro cracking. Compressive strengths increased more for the control section since the recycled sections typically had higher mortar content and a lower top aggregate size, making them less rigid and strong. Overall, even without refurbishing, most aspects of the recycled concrete pavements held up as well as the control sections.

Table 40: Average Deterioration for Recycled and Control Sections not Refurbished between 1994 and 2006

Test	Average Recycled Change (1994 to 2006)	Average Control Change (1994 to 2006)	Difference in Change (Recycled vs. Control)	Best
Transverse Joint Spalling, % Joints	33%	31%	2%	C
Transverse Joint Seal Damage, % Joints	78%	76%	2%	C
Longitudinal Joint Seal Damage, m/km	100%	100%	0%	=
D-cracking, % Slabs	0%	0%	0%	=
Pumping, % Slabs	0%	0%	0%	=
Slab/Patch Deterioration, % Slabs	1%	0%	1%	C
Avg. Faulting between Panels, mm	96% (+0.3 mm)	100% (+0.8 mm)	-4% (-0.5 mm)	R
Avg. Joint Width, mm	5% (+0.5 mm)	10% (+1.0 mm)	-5%	R
Longitudinal Cracking, m/km	25%	0%	25%	C
Transverse Cracking, % Slabs	12%	0%	12%	C
Deteriorated Transverse Cracks/km	85%	8%	77%	C
Total Transverse Cracks/km	35%	7%	28%	C
IRI	16%	10%	6%	C
Testing Average	37%	26%	11%	C
PSR	-3%	-5%	2%	R
Tensile Strength	-19%	-24%	5%	R
Compressive Strength	1%	16%	-15%	C
Testing Average	-7%	-4%	-3%	C

Next, a comparison was done between pavements there were refurbished between 1994 and 2006. Table 41 presents data comparisons for the average deterioration of recycled and control sections refurbished between 1994 and 2006.

Table 41: Average Deterioration of Recycled and Control Sections
Refurbished between 1994 and 2006

Test	Average Recycled Change (1994 to 2006)	Average Control Change (1994 to 2006)	Difference in Change (Recycled vs. Control)	Best
Transverse Joint Spalling, % Joints	22%	35%	-13%	R
Transverse Joint Seal Damage, % Joints	-14%	9%	-23%	R
Longitudinal Joint Seal Damage, m/km	60%	100%	-40%	R
D-cracking, % Slabs	0%	0%	0%	=
Pumping, % Slabs	0%	0%	0%	=
Slab/Patch Deterioration, % Slabs	1%	0%	1%	C
Avg. Lane to Shoulder Drop off, mm	4%	100%	-96%	R
Avg. Lane to Shoulder Separation, mm	37%	64%	-27%	R
Avg. Faulting between Panels, mm	-38% (-1.5 mm)	-29% (-0.7 mm)	-9% (-0.8 mm)	R
Avg. Joint Width, mm	0% (-0.2 mm)	-5% (-0.5 mm)	5% (+0.3 mm)	C
Longitudinal Cracking, m/km	20%*	-18%*	38%	C
Transverse Cracking, % Slabs	11%*	5%*	6%	C
Deteriorated Transverse Cracks/km	87%*	50%*	37%	C
Total Transverse Cracks/km	63%*	6%*	57%	C
IRI	24%	0%	24%	C
Testing Average	18%	21%	-3%	R
PSR	2%	4%	-2%	C
Tensile Strength	-6%	-24%	18%	R
Compressive Strength	7%	8%	-1%	C
Young's Modulus	-13%	-7%	-6%	C
Testing Average	-3%	-5%	2%	R

Note: * Crack repair was not part of the rehabilitation

Overall, the recycled sections benefited more from refurbishing than the control sections. The most significant difference between the control and RCA sections was longitudinal and transverse cracking (which was not part of the refurbishing). The difference is easily explained by ASR, aggregate top size and RCA fines used.

Faulting decreased to an average of 1.5 mm for refurbished recycled pavements while the refurbished controls only decreased 0.7 mm. Strength value changes were similar between recycled and control sections and were what was to be expected from a typical concrete pavement that has been subjected to traffic and environmental effects.

This comparison shows that a recycled concrete pavement will benefit more from refurbishing (retrofitting of dowel bars in undowled joints, diamond grinding and/or replacement of joint seals) then a control section will. These are essential issues to consider as pavements around the United States continue to age.

Finally, a deterioration comparison was done for all recycled and control pavements studied in 1994 and 2006. Table 42 presents data comparisons for the average deterioration of recycled and control sections between 1994 and 2006.

Overall, the recycled sections deteriorated the same as the control sections. Some categories the recycled pavement actually deteriorated less than the control sections, were faulting between panels (23% lower) and transverse joint spalling (5% lower).

On the other hand, increase of transverse and longitudinal cracking in recycled sections was higher than the control sections. A few recycled sections also had lower top aggregate size, which also contributed to a weaker pavement. An astonishing 9 out of 10 recycled sections evaluated had ASR, resulting in an increase of longitudinal cracking for the recycled sections.

Table 42: Average Deterioration of Recycled and Control Sections between 1994 and 2006

Test	Average Recycled Change (1994 to 2006)	Average Control Change (1994 to 2006)	Difference in Change (Recycled vs. Control)	Best
Transverse Joint Spalling, % Joints	27%	33%	-5%	R
Transverse Joint Seal Damage, % Joints	27%	42%	-15%	R
Longitudinal Joint Seal Damage, m/km	71%	100%	-29%	R
D-cracking, % Slabs	0%	0%	0%	=
Pumping, % Slabs	0%	0%	0%	=
Slab/Patch Deterioration, % Slabs	1%	0%	1%	C
Avg. Lane to Shoulder Drop off, mm	10%	100%	-90%	R
Avg. Lane to Shoulder Separation, mm	35%	42%	-7%	R
Avg. Faulting between Panels, mm	13% (-0.9 mm)	36% (+0.8 mm)	-23% (-1.7 mm)	R
Avg. Joint Width, mm	2% (+0.2 mm)	3% (+0.3 mm)	-1% (-.1 mm)	R
Longitudinal Cracking, m/km	28%	-9%	37%	C
Transverse Cracking, % Slabs	10%	2%	8%	C
Deteriorated Transverse Cracks/km	85%	29%	56%	C
Total Transverse Cracks/km	58%	7%	51%	C
IRI	21%	5%	16%	C
Testing Average	26%	26%	0%	=
PSR	0%	-1%	1%	R
Tensile Strength	-12%	-18%	6%	R
Compressive Strength	4%	9%	-5%	C
Young's Modulus	-12%	12%	-24%	C
Testing Average	-5%	1%	-4%	C

Figures 59 and 60 show graphically the difference in the average recycled sections and control sections change in distresses from 1994 to 2006 for all pavements, including those rehabilitated, respectively.

Figure 59: Recycled Pavement Distresses (Avg. Percent Change from 1994 to 2006)

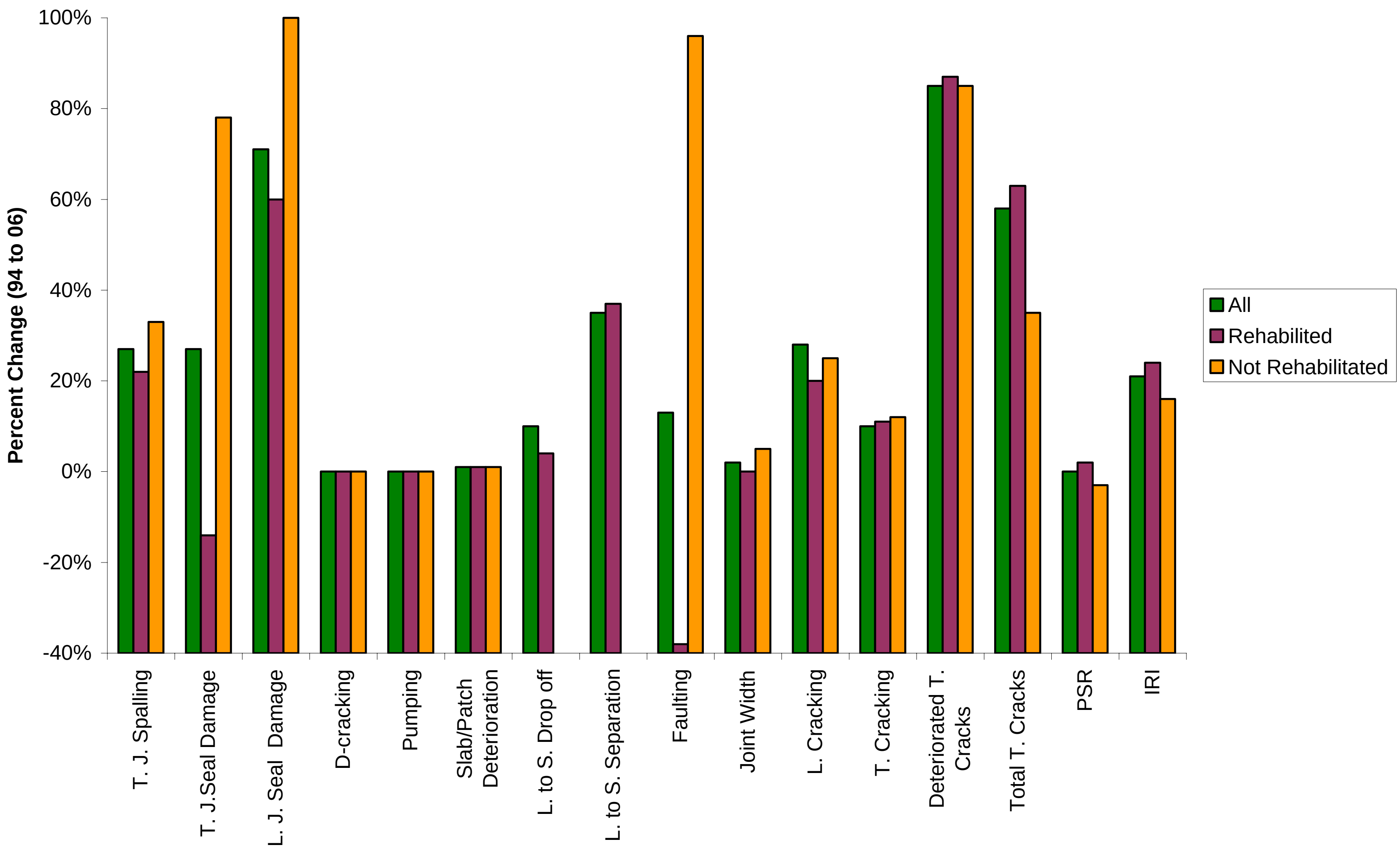
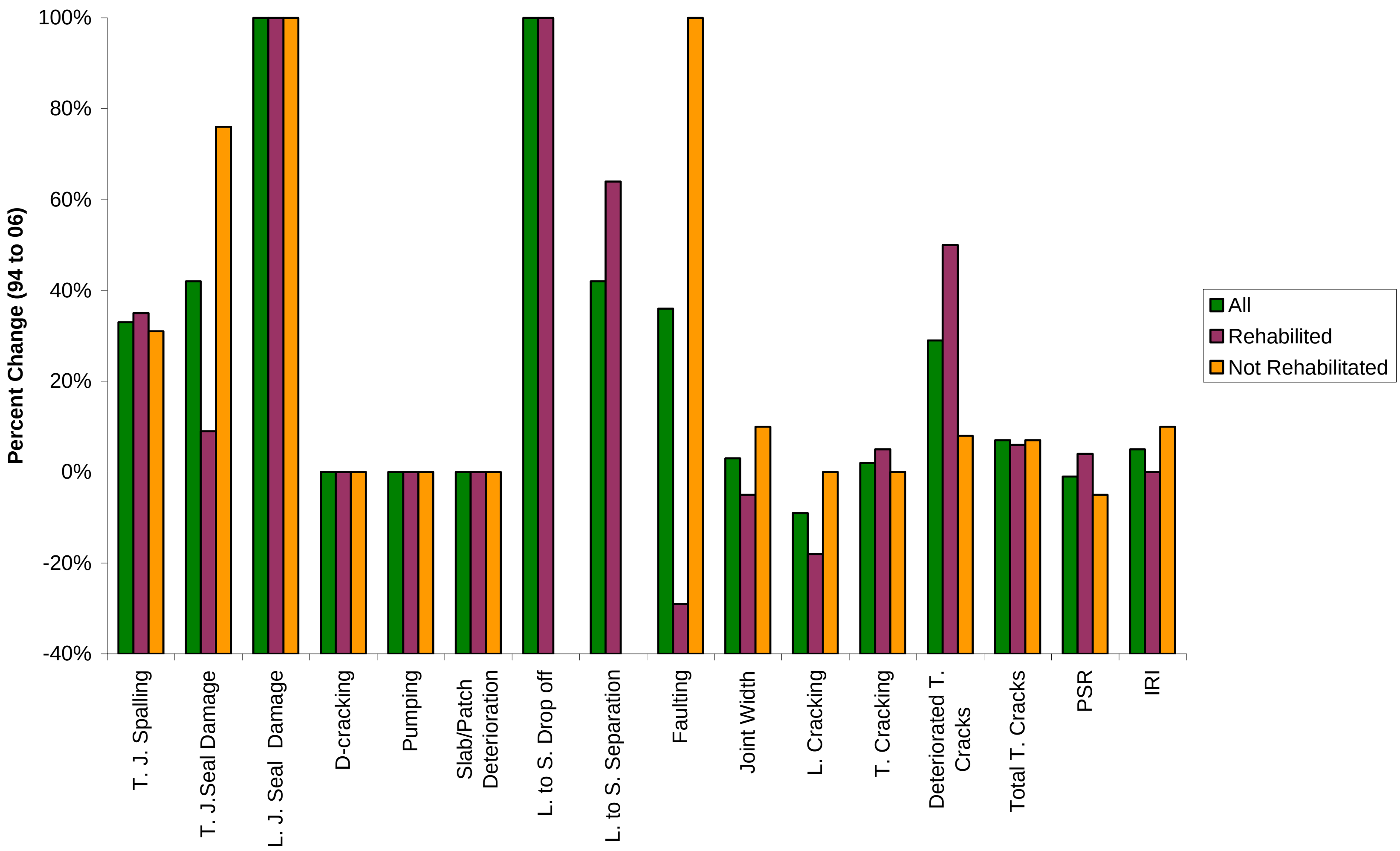


Figure 60: Control Pavement Distresses (Avg. Percent Change from 1994 to 2006)



CHAPTER 6

CONCLUSIONS

All Pavement Sections

Overall, the recycled sections performed comparably to the control sections. Both types of pavements field and laboratory values were found to be similar in the 2006 study. The amount of deterioration from 1994 to 2006 was also consistent between control and recycled pavements. Even thought 10 out of 16 recycled sections had ASR and were not mitigated, they still performed as well as the control sections.

Such factors as lower aggregate top size, higher mortar content and ASR gave some recycled pavements higher transverse and longitudinal cracking. For example, the average increase from 1994 to 2006 for % of slabs with transverse cracking was 10% for all the recycled sections, but only 2% for the control. Similarly, the average increase from 1994 to 2006 for m/km of longitudinal cracking in recycled sections was 28%, while the control sections actually decreased 9%. These losses could have easily been prevented in RCA sections if the amount of mortar on the RCA had been restricted and the aggregate top size kept the same as control sections. Additionally, longitudinal cracking values could have been prevented if the RCA was tested for ASR reactivity and properly mitigated if present.

Overall, the 2006 study on recycled pavements was well worth it. It allowed for a better understanding of how different aspects of the pavement and mix designs affected different RCA pavement's performance.

Based on data from the 2006 study the following conclusions, relative to the performance of RCA pavements, seem reasonable. These conclusions, although they appear to be appropriate for the data, may or may not apply to other RCA pavements:

- Load transfer devices improve performance of RCA pavements.
- In that 10 out of 16 of the RCA pavements studied were found to have ASR, it is prudent to test for ASR and mitigate as required to prevent the reoccurrence of ASR in the RCA pavement.
- RCA with lower mortar contents showed higher performance.
- RCA pavements with maximum aggregate sizes less than their control showed overall lower performance.
- RCA pavements can be effectively rehabilitated resulting in equal or better PSR ratings than conventional pavements.

Future Recommendations

- As with any conventional pavement, the use of dowel bars for load transfer should be done on all recycled JPCPs, no matter how low the traffic loading or how small the panel length.
- Even with dowel bars for load transfer, panel lengths should be kept as short as economically possible. CT1 showed that 12 m panel lengths produced transverse cracks that were in upwards of 70 mm wide.
- Before RCA is used in a concrete mix it must be tested for potential ASR. If the pavement is potentially reactive then different mitigation strategies should be tested or another RCA source should be considered. This is impetrative as the

most likely candidate roads for recycling are ones that have deteriorated, possibly due to ASR.

- Proper care should be taken when crushing concrete so that the mortar content can be minimized.
- As concrete crushing is expensive, a cost benefit analysis should be done to determine an allowable amount of RCA mortar that will still produce an acceptable concrete.
- Further research into the amount of RCA fines in a recycled concrete versus pavement performance should be carried out as a correlation could not be found in this study.
- Since RCA is composed of both mortar and aggregate, the need for thorough material properties testing before use in a mix is essential. RCA should be considered an engineered material and design of recycled pavements needs to take that into account.
- Future field studies should be done to evaluate these recycled concrete pavements when they have had been exposed to even more traffic loading.

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APPENDIX: Core Data

ID Number	Pavement Section	Core Number (As Marked)	Station	Core Notes
1	KS1-1	1-1	3+30	TJ
2	KS1-1	1-2	3+30	1' FROM N. TRANS JT
3	KS1-1	1-3	3+30	CENTER PANEL
4	KS1-1	2-1	7+80	18' FROM LS, TRANS JT
5	KS1-1	2-2	7+81.5	18' FROM LS
6	KS1-1	2-3	7+87	MID-PANEL
7	KS1-2	3-1	6+75	TJ
8	KS1-2	3-2	6+76.5	WP
9	KS1-2	3-2	6+82	MID-PANEL
10	KS1-2	4-1	9+60	TJ
11	KS1-2	4-2	9+61.5	WP
12	KS1-2	4-3	3+67	MID-PANEL
13	WI1-1	1-1	3+76	
14	WI1-1	1-2	3+84	JT Partially Destroyed
15	WI1-1	1-3	n/a	
16	WI1-2	2-1	4+50	JT
17	WI1-2	2-2	4+66	
18	WI1-2	2-3	5+10	
19	WY1-1	1	2+69	MID-SPAN
20	WY1-1	1	2+75	JOINT
21	WY1-2	2	5+71	CONTROL (DESTROYED)
22	WY1-2	2	7+90	
23	WY1-2	2	7+94	JT
24	WY1-2	2	2+38	SUB-PANEL, MD-PANEL
25	WY1-2	2	2+43	MD-PANEL
26	WY1-2	2	2+50	JT
27	WY1-2	2	n/a	3RD PANEL, MD-PANEL
28	WY1-2	2	n/a	MD-PANEL
29	WY1-1	1	5+56	3RD PANEL, JT
30	WY1-1	1	5+60	MD-PANEL (DESTROYED)
31	WY1-1	1	5+64	JT
32	WY1-1	1	3+50	MID-MARKED
33	WY1-1	1	3+54	MID #2
34	WY1-1	1	3+60	JT
35	WY		n/a	MP 386, LI SECT. (4" CORE)
36	WY		n/a	MP 384.25, LI SECT. (4" CORE)
37	WY		n/a	MP 384.75, LI SECT. (4" CORE)
38	CT1-2	C#1	0+46	
39	CT1-1	R#2	2+05	
40	CT1-1	R#1	0+80	
41	CT1-1	R#3	6+66	
42	CT1-1	R#4	7+67	

ID Number	Pavement Section	Core Number (As Marked)	Station	Core Notes
43	CT1-1	R#5	9+78	
44	CT1-2	C#2	2+31	
45	CT1-2	C#3	n/a	
46	CT1-2	C#9	n/a	OUT OF SECTION
47	MN4-1	R#1	1+30	
48	MN4-1	R#2	3+18	
49	MN4-1	R#3	4+10	
50	MN4-1	R#4	4+16	CRACK
51	MN4-1	R#5	5+98	
52	MN4-1	R#6	6+21	JOINT
53	MN4-1	R#7	7+36	
54	MN4-2	C#1	2+48	
55	MN4-2	C#2	3+24	JOINT
56	MN4-2	C#3	3+44	
57	MN4-2	C#4	5+27	
58	MN4-2	C#5	6+60	
59	MN2-2	R#1	2+66	(4.75" CORE)
60	MN2-2	R#2	3+45	(4.75" CORE)
61	MN2-2	R#3	5+90	(4.75" CORE)
62	MN2-2	R#4	n/a	(4.75" CORE)
63	MN1-1	R#1	3+72	
64	MN1-1	R#2	5+04	
65	MN1-1	R#3	6+66	
66	MN1-1	R#4	n/a	
67	MN1-2	C#1	2+76	
68	MN1-2	C#2	3+83	
69	MN1-2	C#3	5+05	
70	MN1-2	C#4	6+94	
71	MN-3	RC#1	2+82	(4.75" CORE)
72	MN-3	RC#2	2+92	(4.75" CORE)
73	MN-3	RC#3	3+71	(4.75" CORE)
74	MN-3	RC#4	3+77	(4.75" CORE) (DESTROYED)
75	MN-3	RC#5	4+94	(4.75" CORE)
76	MN-3	RC#6	5+02	(4.75" CORE)
77	MN-3	RC#7	7+06	(4.75" CORE)
78	MN-3	RC#8	7+16	(4.75" CORE)
79	WI2-1	1-1	4+52	
80	WI2-1	1-2	4+68	
81	WI2-1	1-3	5+08	JT
82	WI2-2	2-1	n/a	
83	WI2-2	2-2	n/a	
84	WI2-2	2-3	n/a	
85	IL1-1	1-C1	3+15	
86	IL1-1	1-C2	5+27	

ID Number	Pavement Section	Core Number (As Marked)	Station	Core Notes
87	IL1-1	1-C3	7+21	
88	IL1-1	1-M1	3+02	
89	IL1-1	1-M2	5+00	
90	IL1-1	1-M3	7+00	
91	IL1-2	2-C1	3+25	
92	IL1-2	2-C2	5+18	
93	IL1-2	2-C3	7+13	
94	IL1-2	2-M1	3+00	
95	IL1-2	2-M2	5+00	
96	IL1-2	2-M3	7+00	
97	IA1-1	1-M1	n/a	(4" CORE)
98	IA1-1	1-M2	n/a	(4" CORE)
99	IA1-1	1-M3	n/a	(4" CORE)
100	IA1-1	1-M4	n/a	(4" CORE)
101	IA1-1	1-M5	n/a	(4" CORE)
102	IA1-1	1-UNMRK	n/a	(4" CORE)
103	IA1-2	2-M1	n/a	(4" CORE)
104	IA1-2	2-M2	n/a	(4" CORE)
105	IA1-2	2-M3	n/a	(4" CORE)
106	IA1-2	2-M4	n/a	(4" CORE)
107	IA1-2	2-UNMRK-A	n/a	(4" CORE)
108	IA1-2	2-UNMRK-B	n/a	(4" CORE)
109	MN-2	RC#1	3+18	(4.75" CORE)
110	MN-2	RC#2	3+84	(4.75" CORE)
111	MN-2	RC#3	5+15	(4.75" CORE)
112	MN-2	RC#4	6+16	(4.75" CORE)