

Highway Concrete Pavement Technology Development and Testing: Volume II— Field Evaluation of Strategic Highway Research Program (SHRP) C-203 Test Sites (Freeze-Thaw Resistance)

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Foreword

This report documents results from 2 field sites, which include a total of 18 test sections in Minnesota and Ohio. The sites were monitored for long-term performance to verify the effectiveness of freeze-thaw resistance technology. The performance monitoring included annual distress surveys and physical testing of cores taken from the concrete slabs at both sites. Wave velocity was measured at joints at the Ohio site to evaluate some treatments for D-cracking mitigation.

The field test sites were installed in 1992 as a part of Strategic Highway Research Program (SHRP) Contract C-203, "Resistance of Concrete to Freezing and Thawing." The purpose of both installations was to validate the current frost resistance technology for new concrete in a field environment. In addition, at the Ohio site an additional evaluation was conducted to examine the effectiveness of methods to treat existing D-cracked concrete pavements to mitigate the effect of future D-cracking.

Results from the field and laboratory testing over a 6-year period after construction (1992 to 1998) showed very little visual distress at the surface or physical distress of portland cement concrete (PCC) cores taken from the interior portions of the test sections. A description of the field sites and monitoring results are provided. Conclusions reached and recommendations for further study are then presented.

Gary L. Henderson
Director
Office of Infrastructure
Research and Development

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16. Abstract Field test sections were constructed during 1992 as part of the Strategic Highway Research Program (SHRP) investigation of the frost resistance of concrete. The first freeze-thaw-related deterioration expected for pavement concrete exposed to de-icing salt would be salt scaling. Unfortunately, the test sections constructed in Ohio were diamond-ground between construction and the first visit of the monitoring team. The diamond-ground surface did not deteriorate over time. Internal deterioration of the Ohio test sections was either not detected or was believed to be caused by a mechanism other than freeze-thaw. Freeze-thaw deterioration was not noticed, either, in the Minnesota test sections (not exposed to deicing salts), though freeze-thaw tests conducted on specimens cut from the test sections 6 years after construction showed significantly different performance than specimens prepared and tested at the time of test section construction. For both the Ohio and Minnesota test sections, only 6 years of winter exposure would not be adequate to evaluate potential long-term performance thoroughly. Though the Ohio sections have been overlaid, making further monitoring impossible, the Minnesota sections are still exposed. Additional monitoring of these sections is recommended, along with providing salt exposure to the sections to determine their resistance to salt scaling. The D-cracking mitigation study indicated that in many cases the D-cracking returned after 6 years, independent of the mitigation technique tried. Additional testing would be required to make further evaluations.			
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SI* (MODERN METRIC) CONVERSION FACTORS				
APPROXIMATE CONVERSIONS TO SI UNITS				
Symbol	When You Know	Multiply By	To Find	Symbol
LENGTH				
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
AREA				
in ²	square inches	645.2	square millimeters	mm ²
ft ²	square feet	0.093	square meters	m ²
yd ²	square yard	0.836	square meters	m ²
ac	acres	0.405	hectares	ha
mi ²	square miles	2.59	square kilometers	km ²
VOLUME				
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft ³	cubic feet	0.028	cubic meters	m ³
yd ³	cubic yards	0.765	cubic meters	m ³
NOTE: volumes greater than 1000 L shall be shown in m ³				
MASS				
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
TEMPERATURE (exact degrees)				
°F	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	°C
ILLUMINATION				
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m ²	cd/m ²
FORCE and PRESSURE or STRESS				
lbf	poundforce	4.45	newtons	N
lbf/in ²	poundforce per square inch	6.89	kilopascals	kPa
APPROXIMATE CONVERSIONS FROM SI UNITS				
Symbol	When You Know	Multiply By	To Find	Symbol
LENGTH				
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
AREA				
mm ²	square millimeters	0.0016	square inches	in ²
m ²	square meters	10.764	square feet	ft ²
m ²	square meters	1.195	square yards	yd ²
ha	hectares	2.47	acres	ac
km ²	square kilometers	0.386	square miles	mi ²
VOLUME				
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m ³	cubic meters	35.314	cubic feet	ft ³
m ³	cubic meters	1.307	cubic yards	yd ³
MASS				
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
TEMPERATURE (exact degrees)				
°C	Celsius	1.8C+32	Fahrenheit	°F
ILLUMINATION				
lx	lux	0.0929	foot-candles	fc
cd/m ²	candela/m ²	0.2919	foot-Lamberts	fl
FORCE and PRESSURE or STRESS				
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in ²
*SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380. (Revised March 2003)				

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CHAPTER 1. INTRODUCTION

This report documents results from 2 field sites, which include a total of 18 test sections:

- Maintenance area of the Minnesota Road Research Project (Mn/ROAD) test site, about 40 miles northwest of Minneapolis, MN (8 test sections non-trafficked).
- Route 2 in Erie and Lorain Counties near Vermilion, OH (10 test sections placed as full-depth patches in an old D-cracked pavement).

The sites were monitored for long-term performance to verify the effectiveness of freeze-thaw resistance technology. The performance monitoring included annual distress surveys and physical testing of cores taken from the concrete slabs at both sites. Wave velocity was measured at joints at the Ohio site to evaluate some treatments for D-cracking mitigation.

The field test sites were installed in 1992 in Minnesota and Ohio as a part of Strategic Highway Research Program (SHRP) Contract C-203, "Resistance of Concrete to Freezing and Thawing." The purpose of both installations was to validate the current frost resistance technology for new concrete in a field environment. In addition, at the Ohio site another evaluation was conducted to examine the effectiveness of methods to treat existing D-cracked concrete pavements to mitigate the effect of future D-cracking.

Briefly, results from the field and laboratory testing over a 6-year period after construction (1992 to 1998) showed very little visual distress at the surface or physical distress of portland cement concrete (PCC) cores taken from the interior portions of the test sections. Deterioration of some of the Ohio patches (especially in the wheel-path near transverse joints) was observed, but this is not believed to be due to a direct freeze-thaw mechanism. Some deterioration from D-cracking was observed at the joints of the original Ohio pavement. Further monitoring of the Minnesota site is recommended to obtain long-term distress and PCC deterioration information for use in validating the C-203 recommendations.

A description of the Minnesota field sites and monitoring results are provided first, followed by a description of the Ohio field sites and monitoring results. Conclusions reached and recommendations for further study are then presented.

CHAPTER 2. MINNESOTA FREEZING AND THAWING FIELD SECTIONS

The purpose of the test sections installed in Minnesota was to determine the influence of varying fly ash contents and air contents on the frost resistance of concrete. Fly ash contents of 15 and 30 percent (by mass) cementitious material as well as zero-fly ash mixtures were placed at various air contents. These sections were placed in October of 1992 and were monitored from 1994 through 1998. An additional survey was conducted in 2001 to obtain up-to-date performance information.

Caution should be exercised in evaluating results because the sections were not on trafficked pavement and did not experience de-icing salt application. However, snow was removed from the sections on a regular basis, so this pavement was certainly subjected to normal freeze-and-thaw cycles.

MINNESOTA WINTER WEATHER CONDITIONS

A summary of climatic exposure conditions for the Minnesota sections is given in table 1. These sections were subjected to severe winter freezing and thawing temperatures. The air freeze-thaw cycles range from 140 to 170 per year over this time period.

DESCRIPTION OF MINNESOTA SECTIONS

The Minnesota sections consist of 8 concrete pads of 4.6 meter (m) by 6.1 m at the Mn/ROAD test site maintenance yard. The sections were placed at the maintenance yard so that they would be subjected to environmental loads only. Each pad was quartered by making two saw cuts; hence, each section consists of 4 panels that are 2.3 m by 3.0 m.

The eight pads comprise eight test sections constructed of eight different mixes. These installations were developed to look at pozzolan content and air-void parameters. These experimental variables included 1.5, 2.5, and 5.5 percent air entrainment, and 0 and 30 percent fly ash by weight of cementitious material. A section containing a proprietary admixture that produces high-carbon concrete was also included (section 8). A summary of the various mix types is provided in table 2. One of the sections was the same as the conventional paving mix used on many of the Mn/ROAD concrete pavement sections (section 1).

Table 1. Summary of winter weather conditions, Minnesota sections.*

Month/Year	Monthly Low Temperature, °C	Days Below 0 °C (Air)	Days Below -18 °C (Air)
October 1992	-6	11	—
November 1992	-9	25	—
December 1992	-24	29	6
January 1993	-27	30	12
February 1993	-24	28	9
March 1993	-19	30	7
April 1993	-8	9	—
October 1993	-3	11	—
November 1993	-16	25	—
December 1993	-26	30	7
January 1994	-33	31	17
February 1994	-27	28	12
March 1994	-12	27	—
April 1994	-8	13	—
October 1994	-2	4	—
November 1994	-13	20	—
December 1994	-19	31	1
January 1995	-24	30	5
February 1995	-24	28	9
March 1995	-20	18	4
April 1995	-15	9	—
October 1995	-4	3	—
November 1995	-17	29	—
December 1995	-23	31	5
January 1996	-31	31	18
February 1996	-36	28	8
March 1996	-23	28	8
April 1996	-6	20	—
October 1996	-9	9	—
November 1996	-24	27	4
December 1996	-33	31	9
January 1997	-28	31	16
February 1997	-21	28	3
March 1997	-20	28	1
April 1997	-13	16	—
October 1997	-11	8	—
November 1997	-20	26	2
December 1997	-21	30	1
January 1998	-31	31	8
February 1998	-14	18	—
March 1998	-17	24	—
April 1998	-2	3	—

*From *Climatological Data*. Minneapolis, MN, National Oceanic and Atmospheric Administration, www.noaa.gov.

Table 2. Summary of Minnesota sections and PCC mixtures.

Mixture Number	Fly Ash Content, %	Fresh Air Content, %	Hardened Air Content, % (Entrained)*	28-day Comp. Strength, megapascals (MPa)	Dynamic E, MPa
MN1**	15	3.9	2.9	30.5	39,000
MN2	15	2.7	0.9	26.5	39,000
MN3	15	2.0	0.7	38.5	42,000
MN4	0	2.5	1.4	36.5	39,000
MN5	0	1.5	1.7	40.5	42,000
MN6	30	3.8	1.4	29.0	38,000
MN7	30	1.4	1.5	26.5	44,000
MN8***	HCC	5.6	4.4	23.0	27,000

*From *Resistance of Concrete to Freezing and Thawing*, SHRP-C-391, Transportation Research Board.

**Minnesota Department of Transportation (MnDOT) Standard, except for air content.

***Proprietary high-carbon concrete admixture.

Additional data on the mixtures are given in tables 3 and 4.

CONDITION SURVEYS OF MINNESOTA SECTIONS, 1994 THROUGH 1998

The Minnesota sections were constructed in October 1992 and first surveyed in October 1994. After 2 years of exposure, the test sections appeared just like new. Even the section with low air content and no fly ash (MN4 and MN5) showed no signs of freeze-thaw distress (no D-cracking, scaling, or popouts). The only minor distress present was some loss of fines from surface (not extensive enough to be called scaling) and a marred surface on sections MN6 and MN7, which appeared to be construction related as shown in figure 1. Most of the sections also had occasional minor joint spalling, which again appears to be construction related (i.e., sawing damage). Photos of each section in 1994 are provided in appendix A.



Figure 1. Test panels for mixture MN7 showing cores taken in 1994.

Table 3. Summary of laboratory durability test results from the Minnesota site—air entrainment.

Mixture	Description	Air Void/Water Pore System Characteristics												
		Fresh Air (FA) (%)	Hardened Air (%)	Hardened Air (ent, %)	Spacing Factor (mm)*	Spacing Factor (ent, mm)	Specific Surface (mm-1)	Specific Surface (ent, mm-1)	Voids/mm	Voids/mm (ent)	Avg. Ch. Length (mm)	Avg. Ch. Length (ent, mm)	Paste-to-Air Ratio	28-day f'c (MPa)
MN1	MnDOT standard	3.9	3.5	2.9	0.249	0.228	24.5	29.0	0.216	0.213	0.163	0.138	9.2	30.5
MN2	MN1 w/marginal air	2.7	1.2	0.9	0.353	0.306	28.0	36.2	0.080	0.079	0.143	0.111	27.7	26.5
MN3	MN1 w/low air	2.0	1.0	0.7	0.907	0.780	11.3	15.3	0.029	0.027	0.353	0.262	30.6	38.5
MN4	MN1 w/marginal air, no FA	2.5	1.7	1.4	0.377	0.344	21.8	26.1	0.094	0.092	0.183	0.153	18.2	36.5
MN5	MN1 w/low air, no FA	1.5	2.2	1.7	0.378	0.336	19.4	24.7	0.109	0.105	0.206	0.162	13.9	40.5
MN6	MN1 w/marginal air, 30% FA	3.8	1.6	1.4	0.277	0.261	31.2	35.0	0.128	0.126	0.128	0.114	20.2	29.0
MN7	MN1 w/low air, 30% FA	1.4	2.7	1.5	0.390	0.304	17.6	28.9	0.116	0.108	0.228	0.138	11.9	26.5
MN8	MnDOT experimental mixture	5.6	7.3	4.4	0.215	0.166	20.7	33.9	0.379	0.370	0.190	0.118	4.6	23.0

*mm—millimeters

Note 1: All specimens cured 28 days (including 27 days in lime water).

Note 2: All air void measurements performed at Michigan State University.

Table 4. Summary of laboratory durability test results from the Minnesota site—freeze-thaw testing.

Mixture	Description	Durability Factor		Cycles to 60% RDM*		Cycles to 0.1% Dilation		% Mass Loss @ 60% RDM or 300 Cycle		% Dilation @ 60% RDM or 300 Cycle	
		Mean	Std. Dev.	Mean	Std. Dev.	Mean	Std. Dev.	Mean	Std. Dev.	Mean	Std. Dev.
MN1	MnDOT standard	98.9	0.4	300+	—	300+	—	0.1001	0.0066	-0.0090	0.0216
MN2	MN1 w/marginal air	98.4	0.6	300+	—	300+	—	0.1028	0.0206	0.0019	0.0019
MN3	MN1 w/low air	97.6	1.1	300+	—	300+	—	0.1185	0.0083	0.0088	0.0019
MN4	MN1 w/marginal air, no FA	97.9	0.3	300+	—	300+	—	0.0989	0.0205	0.0084	0.0024
MN5	MN1 w/low air, no FA	92.5	3.2	300+	—	300+	—	0.0460	0.0084	0.0267	0.0142
MN6	MN1 w/marg. air, 30% FA	99.2	0.5	300+	—	300+	—	0.1054	0.0123	0.0030	0.0050
MN7	MN1 w/low air 30% FA	91.4	6.5	300+	—	300+	—	0.0731	0.0154	0.0209	0.0029
MN8	MnDOT experimental mixture	98.4	0.3	300+	—	300+	—	0.0467	0.0506	0.0087	0.0053

Note 1: All specimens cured 28 days (27 days in lime water).

Note 2: All frost-resistance testing conducted in accordance with the American Association of State Highway and Transportation Officials (AASHTO)

T161 (American Society for Testing and Materials (ASTM) C 666) Procedure B, modified (use of cloth wraps).

RDM = Minnesota Road Design Manual

These sections were visually surveyed again in September 1995. There were virtually no observable changes in condition from 1994 other than the appearance of two popouts, one in MN3 and one in MN4. Both these sections have low air content, but the popouts are very minor. No signs of scaling or other deterioration were observed. One other change of note is the dense encroachment of vegetation all around the test pads.

Subsequent visual surveys of all test sections were conducted in 1996, 1997, and 1998. Very little deterioration occurred over this time period. The results of the final survey in 1998 are summarized as follows:

Section 1—15 percent fly ash, 3.9 percent air content—Very light surface scaling (visible sand particles are about 0.2 mm diameter) over the whole surface. See figure 2 for 1998 photograph of section 1.

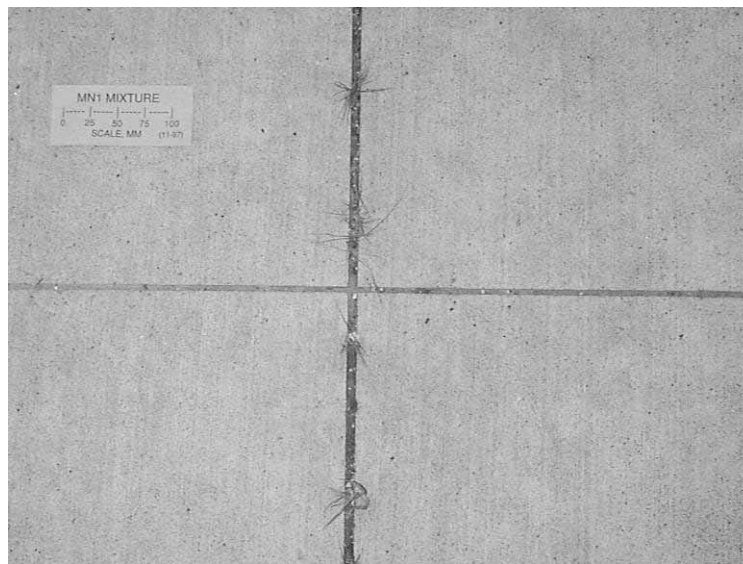


Figure 2. Photograph of Minnesota test section 1 (1998). Most of the surface texture (brooming) is still visible. Sand particles up to 2 mm are visible in a very lightly scaled surface.

Section 2—15 percent fly ash, 2.7 percent air content—Same visual condition as section 1.

Section 3—15 percent fly ash, 2.0 percent air content—Similar very light surface scaling as sections 1 and 2, with a band about 400-500 mm wide in the direction of brooming having visibly less scaling.

Section 4—No fly ash, 2.5 percent air content—Similar very light scaling, with a band about 600 mm wide having less scaling.

Section 5—No fly ash, 1.5 percent air content—Light scaling, with enough scaling in a few places to obscure the brooming marks. See figure 3 for photograph of section 5 in 1998.



**Figure 3. Photograph of Minnesota test section 5 (1998).
Scaling is slightly heavier than figure 2, and in some areas
the surface texture (brooming) is obscured.**

Section 6—30 percent fly ash, 3.8 percent air content—Scaling is generally very light, but about 5-10 percent of the total section area is scaled enough to obscure the brooming marks. The visible sand particles in these areas are in the 0.5-2 mm size range.

Section 7—30 percent fly ash, 1.4 percent air content—Light scaling over entire area. Streaks of scaling heavy enough to obscure broom marks are present throughout the section to a total of about 20 percent of the section area; these are generally adjacent to areas showing excessive paste (possibly an artifact of plastic placed on the concrete for curing before the concrete was initially set).

Section 8—High-carbon concrete (MnDOT experimental), 5.6 percent air content—Very light scaling throughout the section, with a few small streaks of unscaled concrete and also a few streaks of slightly heavier scaling.

It is important to note that none of these sections received any traffic loadings or de-icing salts over the 6-year period. The lack of de-icing salts may be of particular impact, as nearly all highways in freeze areas received large amounts, and this has been shown to have a very large effect on freeze-thaw scaling. (Okamoto et al., 1993)

TESTING OF CORES FROM MINNESOTA SECTIONS, 1994 TO 1998

Since the initial monitoring in 1994, two cores have been taken every year from each test section for the purpose of measuring their dynamic modulus. These cores were generally taken away from the joints and are not representative of concrete near the joints. The cores were dried at approximately 50 percent relative humidity and 22 °C to constant mass and were tested in accordance with ASTM C 215 to determine the dynamic modulus of elasticity. The results are summarized in table 5. There does not appear to be any significant change in the condition of the

interior concrete as measured by changes in core stiffness. This would suggest that there has not been any internal deterioration in the concrete from construction to 1998 despite the harsh winter conditions at this Minnesota site. It is not known what damage existed near the joints where more water could saturate the concrete.

Table 5. Dynamic modulus of Minnesota cores.

SHRP C-203 Designation	Core Dynamic Modulus, MPa				
	1994	1995	1996	1997	1998
MN1	38,829	36,274	38,125	38,601	38,453
MN2	39,248	39,229	40,411	39,630	38,317
MN3	42,286	40,652	41,676	41,740	41,902
MN4	38,710	39,201	38,088	40,689	38,741
MN5	41,745	40,513	41,289	40,127	41,410
MN6	38,363	40,473	40,602	40,027	40,941
MN7	44,231	44,038	44,879	44,283	47,191
MN8	27,519	27,376	28,435	29,068	28,553

The Minnesota sections are exposed to environmental effects only, and they do not show any significant degradation in condition. Minor scaling was visible on most of the slabs, consisting mostly of loss of fines from the surface. The level of scaling was similar in most sections, but at least two sections seemed to exhibit somewhat noticeable differences in condition. Section 8, which is the MnDOT experimental mix that had the highest fresh air content (5.6 percent), did not show any signs of scaling. Section 5 appeared to have slightly more scaling than other sections. Section 5 contains only 1.5 percent air and no fly ash. The field observations were generally consistent with what may be expected based on mix design, but section 7, which had the lowest fresh air content (1.4 percent), had very little scaling. Section 7, however, did contain twice the amount of fly ash provided in the control mix (30 percent).

FOLLOWUP FREEZE-THAW TESTING OF SPECIMENS FROM MINNESOTA SECTIONS

Cores were taken from Minnesota test panels MN1, MN2, and MN3. The cores were allowed to air dry for 8 months and were then cut into freeze-thaw test specimens. The specimens were soaked in calcium-saturated water for 7 days and then tested in rapid freezing and thawing (AASHTO T161, Procedure C). Results of the freeze-thaw tests are presented in table 6, along with the original freeze-thaw results (from specimens prepared during test section construction) and the air-void parameters measured at the time of construction.

Table 6. Results of followup freeze-thaw testing of Minnesota sections.

Mixture	Plastic Air %	Hardened Air %	Specific Surface mm²/mm³	Spacing Factor mm	Original Durability Factor	Followup Durability Factor
MN1	3.9	3.5	24.5	0.249	99	100
MN2	2.7	1.2	28.0	0.353	98	39
MN3	2.0	1.0	11.3	0.907	98	16

CHAPTER 3. OHIO TEST SECTIONS

OHIO WINTER WEATHER CONDITIONS

A summary of climatic exposure conditions for the Ohio sections is given in table 7. These sections were subjected to severe winter freezing and thawing temperatures. The number of air freeze-thaw cycles is about 110 per year (less than Minnesota, with about 160 per year).

DESCRIPTION OF THE OHIO EARLY-STRENGTH FREEZE-THAW AND D-CRACKING MITIGATION EXPERIMENTAL SECTIONS

The purpose of this experiment was to investigate the effect of the entrained air-void system on resistance to freezing and thawing. The Ohio sections consist of 10 lane-width 1.8-m full-depth repairs placed on State Route 2 in an old D-cracked jointed concrete pavement. Six different early-strength mixtures (designated PC, RSC, HES, VES, FT, and ODOT) were placed to determine whether these mixtures had different resistance to freezing and thawing than conventional mixtures. One of the mixtures was placed in six replicates (standard mixture, with replicates I1 through I5 having different air contents than the standard ODOT mixture) with varying air contents to examine the effects of air-void parameters on frost resistance.

These sections were placed during the summer of 1992 and were first monitored for evidence of deterioration from freezing and thawing in November of 1994. The sections were part of the trafficked pavement, and de-icing salt was applied when necessary. Snow was removed from the sections as a part of normal highway maintenance. However, the test section in Ohio was diamond-ground between construction in 1992 and the first monitoring visit in 1994. This removed the original surface texture, making the quantification of scaling almost impossible. Laboratory data from these sections are given in tables 8 and 9.

The experiment at the Ohio site also included the evaluation of the effectiveness of various types of sealers in mitigating D-cracking in the old adjacent concrete slabs. The saw-cut face of the original pavement exposed by the removal of the concrete in the repair area was cleaned and treated with four different types of sealers prior to the placement of the repair concrete for this experiment. The sealers and control joints included the following:

- A penetrating oil sealer (LS).
- A water-based silane sealer (WS).
- A solvent-based silane sealer (SS).
- A two-part urethane sealer (2P).
- Control joints with no sealer treatment (N).

Twenty joint repairs were made at the Ohio site, providing 40 joint faces in the original concrete and as indicated, control joints were left with no sealer treatment.

Table 7. Summary of winter weather conditions, Ohio sections.

Month/Year	Monthly Low Temperature, °C	Days Below 0 °C	Days Below -18 °C
October 1992	-3	6	—
November 1992	-7	10	—
December 1992	-9	22	—
January 1993	-13	26	—
February 1993	-21	28	4
March 1993	-16	18	—
October 1993	-1	2	—
November 1993	-6	12	—
December 1993	-18	21	1
January 1994	-28	30	8
February 1994	-20	25	1
March 1994	-10	22	—
October 1994	1	—	—
November 1994	-7	9	—
December 1994	-10	20	—
January 1995	-16	25	—
February 1995	-18	25	1
March 1995	-13	17	—
October 1995	1	—	—
November 1995	-6	19	—
December 1995	-16	29	—
January 1996	-18	28	1
February 1996	-23	20	4
March 1996	-14	29	—
October 1996	0	1	—
November 1996	-10	20	—
December 1996	-13	20	—
January 1997	-20	27	3
February 1997	-13	22	—
March 1997	-8	22	—
October 1997	0	1	—
November 1997	-8	14	—
December 1997	-11	24	—
January 1998	-11	23	—
February 1998	-7	15	—
March 1998	-8	20	—

*From *Climatological Data*. Cleveland, OH, National Oceanic and Atmospheric Administration, www.noaa.gov.

Table 8. Summary of laboratory durability test results on Ohio test sections—air entrainment.**Air Void/Water Pore System Characteristics**

Mixture	Description	Fresh Air (%)	Hardened Air (%)	Hardened Air (ent, %)	Spacing Factor (mm)	Spacing Factor (ent, mm)	Specific Surface (mm-1)	Specific Surface (ent, mm-1)	Voids/mm	Voids/mm (ent)	Avg. Ch. Length (mm)	Paste-to-Air Ratio	28-day f'_c (MPa)
H1	ODOT Class FS (morning)	3.8	4.9	2.6	0.246	0.179	24.2	44.2	0.298	0.286	0.165	8.7	50.6
H1	ODOT Class FS (morning, 14-day)	4.5	3.4	2.4	0.222	0.187	30.8	42.4	0.260	0.250	0.130	11.8	Note 4
H2	ODOT Class FS (afternoon)	6.5	4.7	2.6	0.350	0.360	25.2	43.5	0.296	0.287	0.159	9.0	n/a
I1	H1 w/0.8*A/E dosage	7.0	4.1	2.8	0.157	0.128	42.1	61.0	0.431	0.425	0.095	11.0	44.6
I2	H1 w/0.6*A/E dosage	5.6	4.4	3.0	0.176	0.147	36.1	50.6	0.393	0.386	0.111	10.1	50.7
I3	H1 w/0.4*A/E dosage	6.2	4.1	2.6	0.470	0.387	14.0	20.7	0.143	0.135	0.286	10.8	48.3
I4	H1 w/0.2*A/E dosage	5.8	5.2	4.5	0.213	0.200	27.7	31.3	0.359	0.355	0.144	8.5	50.2
I5	H1 w/0.05*A/E dosage	3.1	2.7	1.8	0.382	0.318	20.2	28.8	0.138	0.132	0.198	15.5	57.7

Note 1: All specimens cured 28 days (including 27 days in lime water) unless noted otherwise.

Note 2: All air void measurements for 14-day cures performed at the University of Washington.

Note 3: All air void measurements for 28-day cures performed at Michigan State University.

Note 4: Compression tests not performed on day 14.

Note 5: RSC2 data obtained from contractor's report; separate traverse not conducted for this project.

Note 6: 28-day Pyrament[®] #1 linear traverse data not available.

Table 9. Summary of laboratory durability test results on Ohio test sections—freeze-thaw testing.

Durability Test Results											
Mix	Description	Durability Factor		Cycles to 60% RDM		Cycles to 0.1% Dilation		% Mass Loss @ 60% RDM or 300 Cycles		% Dilation @ 60% RDM or 300 Cycles	
		Mean	Std. Dev.	Mean	Std. Dev.	Mean	Std. Dev.	Mean	Std. Dev.	Mean	Std. Dev.
H1	ODOT Class FS (morning)	17.2	1.7	87	6.2	72	2.1	-0.72	0.01	0.179	0.006
H1	ODOT Class FS (morning, 14-day)	13.0	0.6	65	2.9	62	8.1	-0.86	0.11	0.127	0.025
H2	ODOT Class FS (afternoon)	18.5	0.5	92	2.5	73	3.0	-0.51	0.01	0.180	0.037
I1	H1 w/0.8*A/E dosage	18.8	1.0	95	5.3	82	8.2	-0.32	0.07	0.151	0.029
I2	H1 w/0.6*A/E dosage	14.0	0.8	71	3.3	66	4.1	-0.83	0.05	0.144	0.016
I3	H1 w/0.4*A/E dosage	17.6	2.4	89	10.4	75	6.1	-0.87	0.05	0.170	0.017
I4	H1 w/0.2*A/E dosage	12.8	0.4	64	0.7	56	1.0	-0.90	0.04	0.150	0.010
I5	H1 w/0.05*A/E dosage	7.75	0.4	41	2.3	33	0.9	-0.69	0.03	0.156	0.007

Note 1: All specimens cured 28 days unless noted otherwise.

Note 2: All frost resistance testing was accomplished in accordance with AASHTO T161 (ASTM C 666) Procedure B, except for use of cloth wraps and cure periods noted.

Note 3: All frost-resistance testing for 14-day cures performed at the University of Washington.

Note 4: All frost-resistance testing for 28-day cures performed at Michigan State University.

CONDITION SURVEY OF OHIO SECTIONS, 1994 THROUGH 1998

The Ohio sections were constructed in September 1992 and first surveyed in November 1994. As previously stated, all of these test sections were diamond-ground between 1992 and 1994, which made it impossible to observe scaling in the original surface. However, it was possible to observe the ground surface for deterioration. No evidence of surface scaling or other deterioration has been seen on the diamond-ground surface over the monitoring period.

All patches in the Ohio section developed a longitudinal crack shortly after construction (within 2 months). High temperature gradients that developed in the repairs during curing are thought to be responsible for the longitudinal cracking. One repair in the FS section also developed a transverse crack.

However, other than these cracks, in 1994 the Ohio sections were distress free. No signs of scaling or D-cracking were present on either the repair concrete or the surrounding concrete. Faulting and cracking were very low. Results from the 1994 survey are given in tables 10 and 11.

Table 10. Distress summary of Ohio sections.

ERES ID	Repair Type	Station	Longitudinal						Transverse					
			SHRP	1994	1995	1996	1997	1998	SHRP	1994	1995	1996	1997	1998
FS—3 to 8 hr														
H-2	FS	152+53	L	L	L	L	L	L		L	L	L	L	H
H-3	FS	151+11	L	L	L	L	L	L						
H-4	FS	149+48	L	L	L	L	L	L						
H-5	FS	148+66	L	L	L	L	L	L						
H-6	FS	147+57	L	L	L	L	L	L						
H-7	FS	147+03	L	L	L	L	L	L						
H-8	FS	146+68	L	L	L	L	L	L						
H-9	FS	145+04	L	L	L	L	L	L						
H-10	FS	144+25	L	L	L	L	L	L						L
Percent Slabs Cracked			100	100	100	100	100	100	0	11	11	11	11	22
FS—Freeze-Thaw														
I-1	I-3	152+14	L	L	L	L	L	L		L	L	L	L	L
I-2	I-4	150+76	L	L	L	L	L	L						
I-3	I-5	150+28	L	L	L	L	L	L						
I-4	I-1	149+97	L	L	L	L	L	L						
I-5	I-5	149+14	L	L	L	L	L	L						
I-6	I-2	147+82	L	L	L	L	L	L						
I-7	I-3	147+43	L	L	L	L	L	L						
I-8	I-4	147+17	L	L	L	L	L	L						
I-9	I-2	145+04	L	L	L	L	L	L						
I-10	I-1	144+64	L	L	L	L	L	L						
Percent Slabs Cracked			100	100	100	100	100	100	0	10	10	10	10	10

Table 11. Faulting summary of Ohio sections.

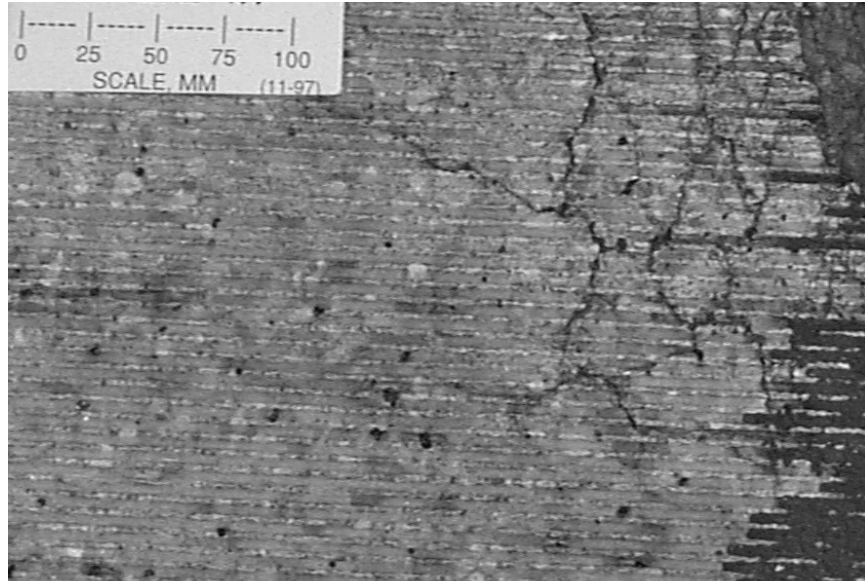
			Faulting									
			Approach Joint					Leave				
								1994	1995	1996	1997	1998
ERES ID	Repair Type	Station	1994	1995	1996	1997	1998	1994	1995	1996	1997	1998
FS—3 to 8 hr												
H-2	FS	152+53	0.01	0.00	0.01	0.02	n/a	0.06	0.02	0.05	0.06	n/a
H-3	FS	151+11	0.05	0.03	0.02	0.03	0.01	0.11	0.07	0.10	0.05	0.07
H-4	FS	149+48	0.04	0.02	0.04	0.01	n/a	0.06	0.04	0.03	0.04	0.00
H-5	FS	148+66	0.04	0.02	0.01	0.01	n/a	0.00	0.00	0.01	-0.01	0.00
H-6	FS	147+57	0.02	0.00	0.00	0.00	0.03	0.05	0.00	0.01	0.02	0.04
H-7	FS	147+03	0.02	0.00	0.00	-0.01	-0.01	0.00	-0.01	-0.01	0.00	-0.01
H-8	FS	146+68	0.01	0.00	0.02	-0.02	0.10	0.04	0.03	0.08	0.07	0.07
H-9	FS	145+04	0.00	0.02	-0.01	-0.02	0.01	0.04	0.00	-0.01	0.02	0.00
H-10	FS	144+25	0.05	0.04	0.03	n/a	n/a	0.04	0.03	0.01	0.02	n/a
Average			0.03	0.01	0.01	0.00	0.03	0.04	0.02	0.03	0.03	0.02
FS—Freeze-Thaw												
I-1	I-3	152+14	0.03	-0.01	0.00	0.03	0.01	0.01	0.01	0.03	0.03	-0.01
I-2	I-4	150+76	0.03	0.03	0.03	0.01	0.02	0.02	-0.02	-0.01	-0.02	-0.04
I-3	I-5	150+28	-0.02	0.00	0.00	0.00	-0.01	0.05	0.05	0.04	0.04	0.05
I-4	I-1	149+97	0.02	0.01	0.03	-0.01	-0.02	0.02	-0.03	-0.03	0.00	0.02
I-5	I-5	149+14	0.00	0.00	0.01	0.0	0.0	0.0	-0.01	0.01	-0.01	0.00
I-6	I-2	147+82	0.04	0.03	0.03	0.0	0.0	0.0	0.0	0.05	0.03	0.03
I-7	I-3	147+43	0.00	-0.01	-0.02	-0.02	0.0	-0.01	-0.02	-0.01	-0.03	n/a
I-8	I-4	147+17	0.02	0.00	0.03	0.0	0.0	0.0	-0.02	0.00	-0.03	-0.03
I-9	I-2	145+04	0.03	-0.02	-0.01	0.0	0.0	0.0	-0.01	-0.01	0.00	-0.01
I-10	I-1	144+64	0.01	0.02	-0.02	n/a	n/a	-0.01	-0.02	-0.02	-0.06	-0.06
Average			0.02	0.01	0.01	0.00	0.01	0.01	-0.01	0.0	-0.01	-0.01

The Ohio sections were surveyed again in November 1995. There were no significant changes in condition from 1994. No new distresses developed, and faulting remained about the same level. Some faulting values actually decreased. The decreased faulting is most likely due to measurement variability, but moisture and temperature conditions can also affect faulting measurements. Again, no signs of scaling or D-cracking were observed.

The patches were examined annually from 1994 through 1998. No additional surface defects were noted in any of the patches from 1994 through 1996. In 1997, surface map-pattern cracking was noted in varying degrees (see figure 4). The cracking was generally worse in the wheelpath near the transverse joints between the patches and the original pavement (figure 5). By 1998, the severity of the cracking had increased; some patches had been repaired with asphalt concrete where the deterioration had produced potholes.

Samples were taken from portions of the patches, and microscopic examination of the samples indicated that both cracks and air voids in the concrete were filled with ettringite.

The entrained air-void system appears to have had no effect on the deterioration of the repairs with the diamond ground surface (Stark et al., 1998, and Heinz et al., 1989).



**Figure 4. Surface cracking in patch, mixture I3
(parallel lines on surface are the result of
surface diamond grinding soon after construction).**



**Figure 5. Deterioration in patch (left) in wheelpath, mixture
I4 (surface has been diamond ground, and note longitudinal
paint strip along pavement edge at bottom of photo).**

TESTING OF CORES FROM OHIO SECTIONS, 1994 TO 1998

Cores were taken from the Ohio test sections between 1994 and 1998 for dynamic modulus testing. The cores were dried at approximately 50-percent relative humidity and 22 °C to constant mass and were tested in accordance with ASTM C 215 to determine the dynamic modulus of elasticity. The results are summarized in table 12, along with strengths and stiffness determined as parts of the SHRP C-203 and C-206 projects.

Table 12. Ohio early-strength sections.

SHRP C-203 Designation	Strength 28-day lab, MPa	SHRP 2-month Strength	C-206 Cores, MPa Stiffness	Core Dynamic Modulus, MPa				
				1994	1995	1996	1997	1998
PC1	58.9	43.9	26,546	–	27,125	–	–	27,608
PC2	50.5	55.4	24,822	29,631	26,876	26,699	–	25,340
RSC1	–	41.4	21,030	–	30,968	–	–	–
RSC2	–	38.7	20,685	29,482	30,834	33,218	–	24,657
HES	43.3	42.0	19,306	–	22,899	–	–	23,600
VES	37.3	38.6	26,546	16,937	18,826	17,832	–	20,131
FT1	–	39.4	19,996	–	25,249	–	–	28,281
ODOT	50.6	51.9	21,719	–	–	18,433	–	21,748
I1	44.6	–	–	–	20,601	–	–	–
I3	48.3	–	–	–	18,217	21,139	–	27,773
I4	50.2	–	–	–	20,164	22,160	–	24,907
I2	50.7	–	–	–	22,281	–	–	–
I5	57.7	–	–	–	22,661	27,647	–	28,925

With the exception of the VES and RSC2 mixtures, the core stiffnesses do not suggest any significant evidence of internal deterioration. The change in dynamic modulus of the VES section between the SHRP C-206 2-month followup coring and the 1994-98 coring indicates that this mixture may have suffered some internal deterioration, but the relatively constant stiffness from 1994-98 suggests that the deterioration is not continuing. The drop in stiffness in 1998 for the RSC2 core may not be significant, as there was no other evidence of deterioration in this section. Additional coring would be required to verify whether this drop is significant or just sampling variability.

OHIO D-CRACKING MITIGATION SECTIONS

The original Ohio pavement on which the test sections were installed was constructed with D-cracking susceptible aggregate. In an effort to determine whether the existing D-cracking could be mitigated, the D-cracked concrete near each of the joints was removed and replaced with a full-depth patch. Prior to placing the patches, the sawn faces of the existing concrete were treated with one of four sealers (or none for control) in an attempt to prevent the lateral movement of water from the transverse joint into the pavement. It was believed that this might prevent or at least delay the continuation of the D-cracking. The treatments consisted of either: (1) a penetrating oil sealer (LS), (2) a water-based silane sealer (WS), (3) a solvent-based silane sealer (SS), or 4) a two-part urethane sealer (2P). Control sections were also left with no sealer

treatment (N). Table 13 summarizes the sealer treatments along with the subbase type (asphalt-treated base (ATB) or granular) and the maximum aggregate size in the existing pavement.

Table 13. Ohio D-cracking mitigation sections.

Station No.	Aggregate Size	Base Type	Patch No.	Edge Treatment
152+94	25 mm	ATB		None
152+94	25 mm	ATB	H-1	SS
152+53	25 mm	ATB		WS
152+53	25 mm	ATB	H-2	LS
152+14	25 mm	ATB	I-1	2P
152+14	25 mm	ATB	(I3)	None
151+11	25 mm	ATB		None
151+11	25 mm	ATB	H-3	WS
150+76	25 mm	ATB	I-2	SS
150+76	25 mm	ATB	(I4)	2P
150+28	19 mm	crushed	I-3	LS
150+28	19 mm	stone	(I5)	None
149+97	19 mm	crushed	I-4	SS
149+97	19 mm	stone	(I1)	WS
149+48	19 mm	crushed		LS
149+48	19 mm	stone	H-4	2P
149+14	19 mm	crushed	I-5	None
149+14	19 mm	stone	(I5)	LS
148+66	19 mm	crushed		2P
148+66	19 mm	stone	H-5	None
147+82	19 mm	crushed	I-6	WS
147+82	12 mm	stone	(I2)	SS
147+57	12 mm	crushed		None
147+57	12 mm	stone	H-6	LS
147+43	12 mm	crushed	I-7	LS
147+43	12 mm	stone	(I3)	WS
147+17	12 mm	crushed	I-8	None
147+17	12 mm	stone	(I4)	None
147+03	12 mm	crushed		WS
147+03	12 mm	stone	H-7	SS
146+68	12 mm	crushed		SS
146+68	12 mm	stone	H-8	2P
145+97	12 mm	crushed	I-9	2P
145+97	12 mm	stone	(I2)	None
145+04	25 mm	crushed		WS
145+04	25 mm	stone	H-9	2P
144+64	25 mm	crushed	I-10	2P
144+64	25 mm	stone	(I1)	None
144+25	25 mm	crushed		None
144+25	25 mm	stone	H-10	WS

Cores have been taken periodically from selected sections of the D-cracking mitigation test section. The cores were identified by the station of the original joint (prior to full- depth patching) along with an indication of whether the core was from the approach side of the patch or the leave side. The cores were dried at approximately 50-percent relative humidity and 22 °C to constant mass and were tested in accordance with ASTM C 215 to determine the dynamic modulus of elasticity.

Nondestructive testing of the existing concrete was conducted adjacent to the treated (or untreated) joints using Spectral Analysis of Surface Wave (SASW) methods. This testing was used to determine the average velocity of shear wave propagation in the concrete (Vokes et al., 1997). Because the equipment used to make these measurements in 1994 had an inadequate sampling rate to permit reliable separation of the measured vibrations from background noise, equipment with a higher sampling rate was used for the 1995 through 1998 measurements. These results, along with the dynamic modulus values determined from the cores, are shown in table 14.

As of 1998, the core dynamic modulus values do not show any conclusive trends. D-cracking would be expected to appear first at the intersection of the transverse and lane-shoulder joints. Equipment limitations with the core rig used prevented the cores from being taken exactly at the lane-shoulder joints, and because the cores could not be taken close to the lane-shoulder joints, this area may not show D-cracking-related deterioration for a number of years.

However, an examination of the concrete velocity measurements indicates that many sections have some degree of deterioration. Direct comparisons of the velocity values from year to year cannot be made, probably because temperature effects could not be taken into account (pavement temperature measurements at either the surface or at depth were not part of the scope of work). By normalizing the data to the approximate maximum value recorded each year (assuming the maximum values represent non-deteriorated concrete), the values can be compared. Figure 6 shows the concrete velocities for each test joint for all 4 years of testing. Visual examination during 1997 and 1998 indicated cracking near the transverse joints, which appeared to be D-cracking. These joints are marked with a "D" in figure 6. Also, in 1998 tests could not be conducted at some of the transverse joint locations because the concrete near the joints was missing right adjacent to the joint or had been patched. These are marked with a "P" in figure 6. Tests were conducted (and test values are plotted in figure 6) for these locations, but the actual testing location was a few inches (depending upon the extent of the patching or damaged concrete) further back from the original joint location. Since D-cracking severity decreases with increasing distance from the joint, these test results labeled "P" can be expected to be higher than they would have been if the test could have been conducted right at the joint.

**Table 14. Summary of Ohio core dynamic modulus
and field nondestructive testing (NDT).**

Station No.	Core Dynamic Modulus, MPa					NDT Velocity, m/s			
	1994	1995	1996	1997	1998	1995	1996	1997	1998
152+94	33,700			33,914		2,310	1,950	1,950	2,500
152+94	33,000			30,662		1,810	1,915	1,550	2,050
152+53			34,188			2,280	1,895	1,900	2,500
152+53						2,625	2,200	1,600	2,050
152+14						2,375	1,895	1,850	1,850
152+14						2,750	2,470	2,400	2,500
151+11						2,125	1,940	2,000	2,500
151+11	36,000					2,670	1,930	2,300	2,450
150+76				34,143		2,655	2,440	2,500	2,100
150+76				36,600		2,655	2,470	2,450	2,450
150+28			33,697		34,128	2,690	1,890	1,950	2,550
150+28			34,709		34,637	2,470	2,000	1,950	2,450
149+97		29,500		35,919		2,310	1,940	2,450	2,550
149+97		29,600				2,780	2,470	2,550	2,600
149+48		32,500				2,625	1,870	2,400	2,600
149+48	33,200	36,900				2,655	2,440	2,450	2,550
149+14	32,900	35,100				2,375	2,375	2,400	2,450
149+14			32,972			2,440	2,410	2,350	2,500
148+66						2,430	2,375	2,600	2,050
148+66						2,500	2,440	2,550	2,700
147+82			32,322			2,375	2,500	2,300	2,550
147+82	25,700		30,243			2,470	2,470	2,400	2,000
147+57					32,028	2,440	1,875	2,300	2,050
147+57					30,635	2,280	1,920	1,950	1,300
147+43					25,851	2,340	2,345	2,350	2,450
147+43					30,857	2,250	2,440	2,450	2,550
147+17			28,912	30,913		2,375	2,390	2,300	2,500
147+17	29,800	34,000				2,405	2,440	2,350	2,000
147+03						2,060	2,345	2,300	2,450
147+03				31,879		2,470	1,690	1,950	2,500
146+68	31,600	30,600				2,250	1,970	2,450	2,450
146+68	28,500	34,600				2,400	1,970	2,350	2,550
145+97			33,740			2,390	1,970	2,400	2,500
145+97						2,340	2,030	2,250	2,550
145+04						2,390	2,000	2,200	2,450
145+04					35,283	2,090	1,875	2,300	2,100
144+64				35,415		2,375	2,280	1,800	2,050
144+64						2,375	1,940	1,900	2,200
144+25						2,200	2,000	2,000	2,550
144+25					22,532	2,410	1,940	2,400	2,600

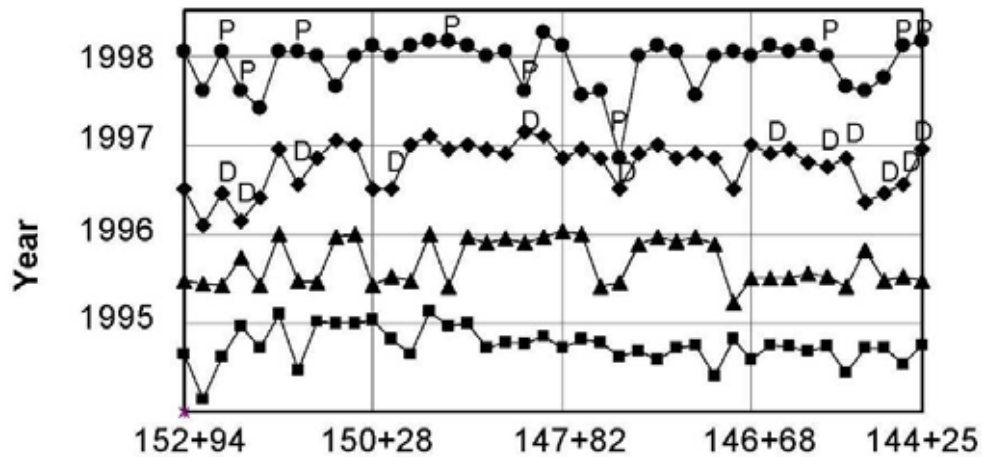


Figure 6. Field NDT velocity measurements.

D-CRACKING MITIGATION RESULTS

D-cracking has reappeared adjacent to some of the patches and can be identified both visually and with the NDT velocities (Vokes et al., 1997). Areas with visual D-cracking are identified with a “D” in figure 6. D-cracking has reappeared in all of the sections made with aggregate having a 25-mm top size, except for three of the sections listed as being on ATB, which may reduce the moisture availability from the subbase. This suggests that none of the edge treatments was successful in significantly delaying the return of D-cracking. D-cracking has returned to some of the sections made with 19-mm aggregate, and the 19-mm sections with no evidence of D-cracking include both treated sections and control sections with no treatment. A few of the 12-mm sections also have D-cracking but, as expected, the smaller aggregate size has reduced the D-cracking potential.

CHAPTER 4. CONCLUSIONS AND RECOMMENDATIONS

Field test sections were constructed during 1992 as part of the SHRP program investigating the frost resistance of concrete. The test sections had three general purposes:

- Determine whether high-performance concrete (high-performance due to rapid strength gain) meeting conventional entrained-air requirements would be frost resistant.
- Determine whether conventional entrained-air requirements were appropriate for pavement concrete.
- Evaluate a potential method for mitigating existing D-cracking. This report documents the results of a 5-year monitoring program conducted for the purpose of evaluating the performance of the test sections.

FREEZE-THAW RESULTS

The Minnesota sections contained concrete slabs having fresh air contents ranging from 1.4 to 5.6 percent (hardened entrained air ranged from 0.7 to 4.4 percent). Other than minor surface scaling, none of the Minnesota freeze-thaw sections shows any evidence of significant deterioration after six Minnesota winters. This is contrary to expectations based on the lack of “adequate” entrained air in most of the sections (Snyder and Janssen, 1994). However, these results must be tempered with the fact that no de-icing salts were used on these sections since they were not trafficked. Also, the monitoring only covered a period of six winter exposures. Additional exposure may be necessary to produce measurable damage. A recent survey conducted in May 2001 showed some additional scaling and popouts on the sections with a lower entrained air content (< 2.5 percent), but the other sections did not show significant further deterioration. With de-icing salts, the deterioration is expected to be much greater for the lower air content concrete (Janssen and Snyder, 1994).

Concrete specimens prepared from cores taken from the Minnesota sections after six winters did deteriorate in laboratory freeze-thaw testing. These results are in contrast to the results of tests conducted when the sections were placed. Table 6 shows that section MN1, with close to the recommended maximum spacing factor of 0.20 mm, performed well in the more recent tests, while sections MN2 and MN3 performed much more poorly. The spacing factors for MN2 and MN3 were much worse than the recommended 0.20 mm; however, all three sections performed well in freeze-thaw tests conducted on concrete that had not received any “real” winter exposure. This difference in test results may be due to shortcomings in the AASHTO T161 test procedure (El-Korchi et al., 1995).

No significant evidence of frost-related deterioration was identified in the diamond-ground Ohio early-strength sections, and these were regularly subjected to de-icing salts. The fresh mix air contents ranged from 1.8 to 4.5 percent, and hardened from 1.8 to 3.3. Some of these sections are cracking, but the cause does not appear to be frost-related.

Surface scaling is the most likely form of freeze-thaw deterioration in a concrete pavement. The diamond grinding of the Ohio sections prevented the detection of scaling in those early-strength

patches. Light scaling was seen in the Minnesota sections, but the severity was too low to describe it as significant deterioration.

Because the Minnesota sections did not receive any de-icing salt, it is recommended that portions of these sections receive de-icing salt for the next 5 years to determine whether the de-icing salt causes significant deterioration.

D-CRACKING MITIGATION

The D-cracking mitigation study indicated that in many cases the D-cracking returned after 6 years, independent of the mitigation technique tried. Whether this is because the edge treatments were not effective in preventing the ingress of moisture through the vertical faces is not known. Cores should be taken at the treated edges and investigated to determine the effectiveness of the aged sealers.

Additional testing would be required with other materials and existing D-cracked concrete to further evaluate the specific mitigation approaches.

APPENDIX A

CONDITION SURVEY OF MN/ROAD PCC TEST PADS (SHRP C-203 PADS) IN 1994, 1998, AND 2001

This appendix describes the condition surveys performed in 1998 and 2001 and includes the photos that were taken in 1994 and 2001. The condition surveys in 1994 showed no deterioration whatsoever. These sections were constructed in 1992, so the 2001 survey represents 9 years of service. Note that no de-icing salts were applied to these concrete pads.

MN1 Section 1—15 percent fly ash, 3.9 percent air content

1998: Very light surface scaling (visible sand particles are about 0.2 mm diameter) over the whole surface.

2001: Very light loss of surface mortar observed throughout the pad—probably consistent with expectations for 9-year-old PCC. There are several skid loader surface scratches present on this pad. Three or four small popouts (< 25.4 mm diameter, < 12.7 mm deep) are present, but there is no significant scaling or spalling. The surface condition is relatively uniform and the pad is generally in excellent condition with respect to durability.

MN2 Section 2—15 percent fly ash, 2.7 percent air content

1998: Very light surface scaling (visible sand particles are about 0.2 mm diameter) over the whole surface.

2001: Five or more popouts are visible, mostly small ones, but one is medium-sized (~31.75 mm in diameter, 12.7 to 19.05 mm deep). There is one large spall adjacent to a core hole from previous test specimen retrieval. There is generally more loss of surface mortar than was observed for MN1, and there is a whitish, crystalline-looking deposit in the surface texture of some of the areas with mortar loss. Some skid loader damage (surface scratches/gouges) runs in the east-west direction. The surface is generally in good condition with respect to durability.

MN3 Section 3—15 percent fly ash, 2.0 percent air content

1998: Similar very light surface scaling as sections 1 and 2, with a band about 400-500 mm wide in the direction of brooming having visibly less scaling.

2001: A few larger popouts are visible on this pad (~38.1 mm in diameter, 12.7 to 19.05 mm deep). More loss of surface mortar is observed throughout the pad, especially at the east end. The surface color and texture of the pad is less uniform, and the original broomed surface texture is difficult to see in some areas of mortar loss. Overall, the surface is in fair to good condition.

MN4 Section 4—No fly ash, 2.5 percent air content

1998: Similar very light scaling, with a band about 600 mm wide having less scaling.

2001: There is one large popout (50.8+ mm in diameter, 19 mm deep) near the west end of the pad. There is a large spall adjacent to a 101.6-mm core hole at the west end of the slab. There is some loss of surface mortar (and evidence of the same whitish crystalline deposits observed in MN2), but texture is generally good and uniform throughout. No significant spalling or scaling is present. Overall condition is good to very good.

MN5 Section 5—No fly ash, 1.5 percent air content

1998: Light scaling, with enough scaling in a few places to obscure the brooming marks.

2001: There is heavier loss of surface mortar throughout, resulting in localized elimination of the broomed surface texture. There is no significant scaling or spalling/fraying of joints. The heaviest loss of mortar and texture is along a .3 m to .6 m band along the south edge of the pad, which is characterized by a darker color. The whitish crystalline deposits are visible in other areas of the pad. There are some skid loader marks oriented in a north-south direction in the southwest quarter of the pad. The overall general condition of the pad is considered good.

MN6 Section 6—30 percent fly ash, 3.8 percent air content

1998: Scaling is generally very light, but about 5 to 10 percent of the total section area is scaled enough to obscure the brooming marks. The visible sand particles in these areas are in the 0.5 mm to 2 mm size range.

2001: There is localized heavy loss of mortar and surface discoloration along various lines that suggest localized water problems under plastic curing sheets. There is a large spall at the southeast corner of the pad. A few small popouts are present throughout the pad. No significant spalling or scaling was observed. The pad is in generally good condition with respect to durability except for the presence of the locally heavy mortar loss probably associated with the plastic curing cover.

MN7 Section 7—30 percent fly ash, 1.4 percent air content

1998: Light scaling over entire area. Streaks of scaling heavy enough to obscure broom marks are present throughout the section to a total of about 20 percent of the section area; these are generally adjacent to areas showing excessive paste (possibly an artifact of plastic placed on the concrete for curing before the concrete was initially set).

2001: There is again heavy mortar loss along various “plastic covering” lines in the east half of the pad. The loss is fairly deep (3.18 to 6.35 mm, locally) and the broomed texture is gone in these areas. There are also areas of what appear to be coring slurry or grey epoxy on the pad surface. A few small popouts are present throughout the slab. There is light fraying (25.4 to 50.8 mm long, < 12.7 mm wide) at two joint locations. There is a large spall in the southeast corner of the pad. Slab texture at the west end of the pad is generally good and uniform (no evidence of plastic sheeting problems).

MN8 Section 8—“High-Carbon Concrete (MnDOT Experimental),” 5.6 percent air content

1998: Very light scaling throughout the section, with a few small streaks of unscaled concrete and also a few streaks of slightly heavier scaling.

2001: There are skid loader marks/gouges present, many in the northeast and southeast corners and one along the south edge of the pad. There are also areas of slurry or gray epoxy on the surface in the east third of the pad. There is moderate mortar loss throughout the pad, especially in the southwest corner of the pad. There is light fraying along the north-south joint between the two panels on the south side of the pad, and one frayed area extends 12.7 to 19 mm from both sides of the joint for a distance of about 76.2 mm. A few small popouts are scattered throughout the pad. The pad surface is generally in good condition with respect to apparent freeze-thaw durability problems.

Summary

From 1998 to 2001 there was heavier loss of surface mortar throughout those sections with the lowest air content (section 3 with 2 percent; section 5 with 1.5 percent; and section 7 with 1.4 percent) resulting in localized elimination of the broomed surface texture. More loss of surface mortar was observed throughout these pads. However, there was no significant scaling or spalling/fraying of joints.

The MnDOT standard mix (except for lower air), section 1 (3.9 percent air), had only a very light loss of surface mortar throughout the pad—probably consistent with expectations for 9-year-old PCC.

Had these sections been subjected to substantial de-icing salts over the past 9 years, it is expected that those with the lowest air voids would have shown much greater surface deterioration.

Note that some of the photos taken in May 2001 could not be taken of the center joint because MnDOT maintenance was storing materials on the pads.



Figure A1. Section MN1—1994.



Figure A2. Section MN1—May 2001.



Figure A5. Section MN3—1994.



Figure A6. Section MN3—May 2001.



Figure A7. Section MN4—1994.



Figure A8. Section MN4—May 2001.



Figure A9. Section MN5—1994.



Figure A10. Section MN5—May 2001.



Figure A11. Section MN6—1994.



Figure A12. Section MN6—May 2001.



Figure A13. Section MN7—1994.



Figure A14. Section MN7—May 2001.



Figure A15. Section MN8—1994.

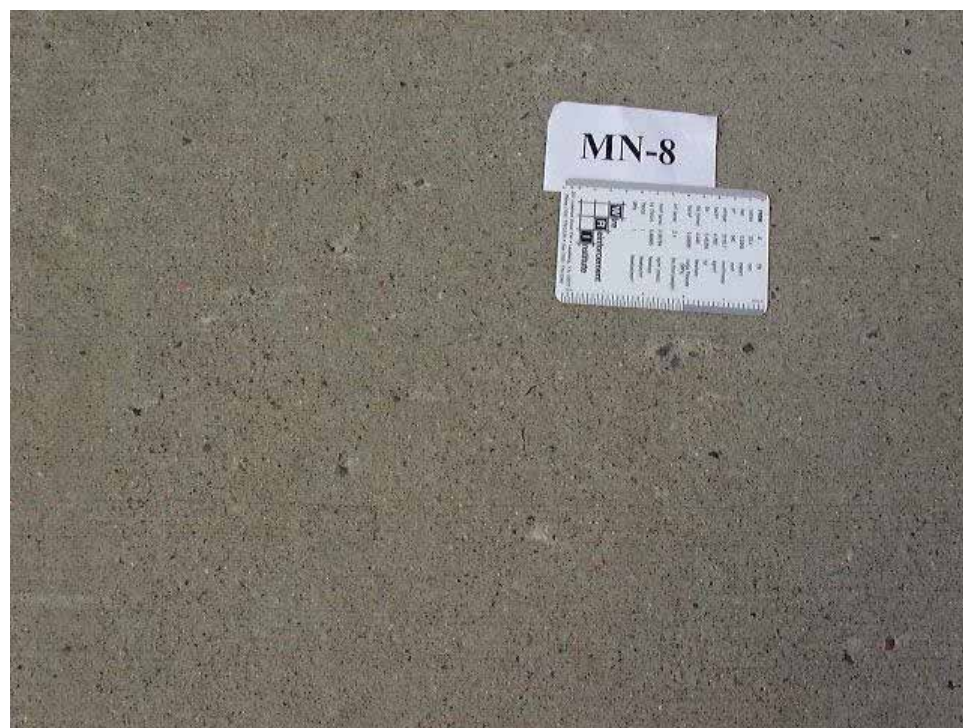


Figure A16. Section MN8—May 2001.

APPENDIX B

INFORMATION AND PHOTOS ON OHIO FREEZE-THAW FULL-DEPTH REPAIRS

The test sections in Ohio were diamond ground between construction and the first monitoring visit (figure B1), so no evidence of surface scaling was visible. However, the diamond-ground surface did not experience any deterioration.

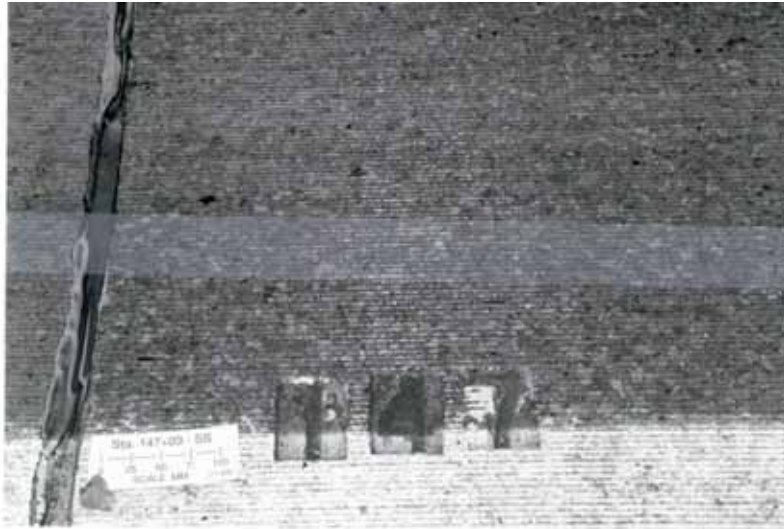


Figure B1. Typical surface textures after diamond grinding, Ohio test sections.

Nondestructive testing of the existing D-cracking susceptible concrete was conducted adjacent to the treated (or untreated) joints using Spectral Analysis of Surface Wave (SASW) methods. A schematic of the SASW test method is shown in figure B2. Actual testing at the Ohio test sections is shown in figure B3.

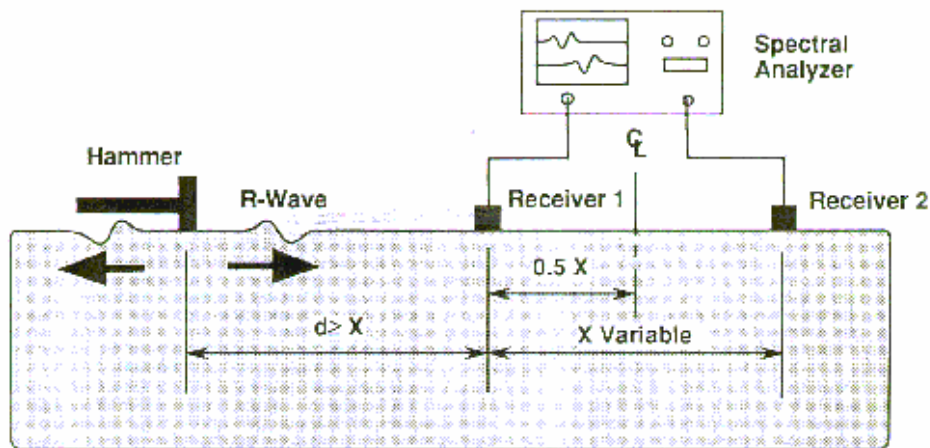


Figure B2. Schematic diagram of SASW testing (from Malhorta and Carino, *Handbook on Nondestructive Testing on Concrete*, CRC Press, New York, 1991.)

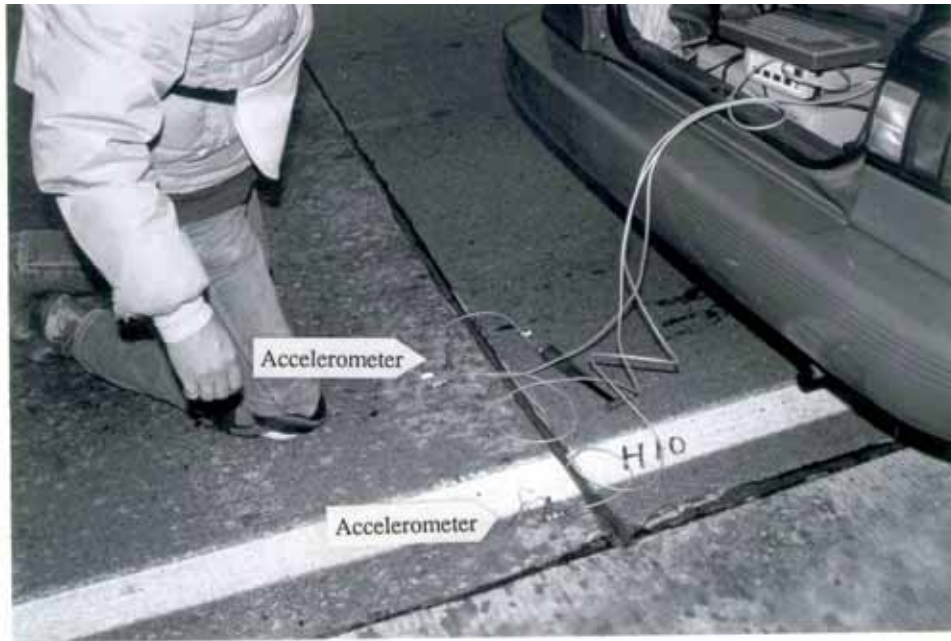


Figure B3. SASW testing at the Ohio test sections.

The method involved impacting the surface of concrete in such a way as to maximize the production of surface waves (R-waves) while minimizing the compression waves (P-waves). Two accelerometers mounted in line about 75 mm from the joint were used to measure the R-wave vibrations. Analysis of the phase shift of the vibrations between the two accelerometers was used to determine the R-wave velocity for various vibration frequencies. High-frequency vibrations are representative of shallow depths, while lower frequencies are more influenced by materials at greater depths in the pavement.

Figure B4 shows a typical R-wave velocity versus vibration frequency plot. The concrete pavement and subbase (base) portion are easily identified. The central section of the graph is dominated by noise resulting from waves reflected by the vertical edge of the pavement. This phenomenon is present in most of the R-wave velocity versus vibration frequency plots from the Ohio testing. Extensive analysis was conducted to filter out this reflection noise from the digital data obtained in Ohio.

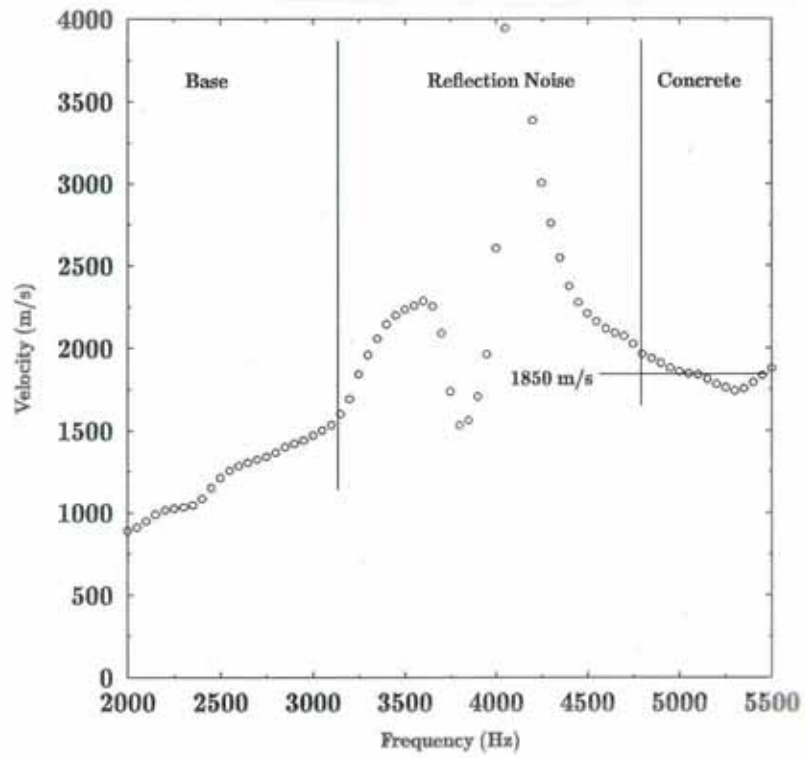


Figure B4. Typical R-wave velocity versus vibration frequency plot.

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