Highway Concrete Technology Development and Testing, Volume III: Field Evaluation of SHRP C-205 Test Sites (High-Performance Concrete)

PUBLICATION NO. FHWA-RD-02-084

MAY 2006

U.S. Department of Transportation
Federal Highway Administration

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

LTPP

Foreword

This research study, sponsored by the Federal Highway Administration, summarizes the field performance of eight high-early-strength (HES) concrete patches between 1994 and 1998. The patches were constructed under the Strategic Highway Research Program (SHRP) between June 1991 and July 1992 and were located in five States (Arkansas, Illinois, Nebraska, New York, and North Carolina) using existing State construction practices. The patches were constructed mainly with Type III cement, four different types of coarse aggregate, and three different types of fine aggregate. Similar types of air entraining admixtures, water reducers, and set accelerators were used at all except the North Carolina site. The patches were located in areas with varying environmental and traffic conditions. The performance criterion of interest was durability. Durability of the HES concrete was quantified over a period of 7 years using various indicators including compressive strength, static elastic modulus, rapid chloride permeability, and asphalt concrete (AC) impedance. The HES patches were also examined visually to locate any material- or durability-related distresses. This report discusses in detail the effects of climate and material properties on the HES concrete durability.

Some of the results of interest include the effect of water reducer type, curing method, and aggregate type on long-term durability. The report also presents comparisons of the rapid chloride permeability and AC impedance test results and the rate of strength gain for the mixes evaluated. Overall, the HES patches performed well with no obvious signs of deterioration. However, the results were not conclusive because the performance-monitoring period was relatively short. There is a need for further research in the areas of long-term HES concrete mechanical properties and durability.

Gary Henderson
Director
Office of Infrastructure
Research and Development

Notice

Quality Assurance Statement

Technical Report Documentation Page

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.
FHWA-RD-02-084		

4. Title and Subtitle	5. Report Date
Highway Concrete Technology Development and Testing, Volume III: Field Evaluation of SHRP C-205 Test Sites (High-Performance Concrete)	May 2006
	6. Performing Organization Code
7. Author(s)	8. Performing Organization Report No.
Jagannath Mallela, Leslie Titus-Glover, H. Thomas Yu	
9. Performing Organization Name and Address	10. Work Unit No. (TRAIS)
ERES Division of Applied Research Associates, Inc. 505 W. University Avenue Champaign, IL 61820-3915	11. Contract or Grant No. DTFH61-94-C-00009
12. Sponsoring Agency Name and Address	13. Type of Report and Period Covered
Office of Engineering and Highway Operations R&D Federal Highway Administration 6300 Georgetown Pike McLean, VA 22101-2296	Final Report
	14. Sponsoring Agency Code

15. Supplementary Notes

FHWA Contracting Officer's Technical Representative: Monte Symons, P.E.

16. Abstract

This research study, sponsored by the Federal Highway Administration, summarizes the field performance of eight high-early-strength (HES) concrete patches between 1994 and 1998. The patches were constructed under the Strategic Highway Research Program (SHRP) between June 1991 and July 1992 and were located in 5 States (Arkansas, Illinois, Nebraska, New York, and North Carolina) using existing State construction practices. The patches were constructed mainly with Type III cement, four different types of coarse aggregate, and three different types of fine aggregate. Similar types of air entraining admixtures, water reducers, and set accelerators were used at all except the North Carolina site. The patches were located in areas with varying environmental and traffic conditions. The performance criterion of interest was durability. Durability of the HES concrete was quantified over a period of 7 years using various indicators including compressive strength, static elastic modulus, rapid chloride permeability, and asphalt concrete (AC) impedance. The HES patches were also examined visually to locate any material- or durability-related distresses. This report discusses in detail the effects of climate and material properties on the HES concrete durability.

Some of the results of interest include the effect of water reducer type, curing method, and aggregate type on long-term durability. The report also presents comparisons of the rapid chloride permeability and AC impedance test results and the rate of strength gain for the mixes evaluated. Overall, the HES patches performed well with no obvious signs of deterioration. However, the results were not conclusive because the performance monitoring period was relatively short. There is a need for further research in the areas of long-term HES concrete mechanical properties and durability.

17. Key Words	18. Distribution Statement
Concrete pavement, durability, compressive strength, rapid chloride permeability test, AC impedance test, life cycle cost	No restrictions. This document is available to the public through the National Technical Information Service, Springfield, Virginia 22161.

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No of Pages	22. Price
Unclassified	Unclassified	69	

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

SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square 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)		
^oF	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	^oC
		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)		
^oC	Celsius	1.8C+32	Fahrenheit	^oF
		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)

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

CHAPTER 1. INTRODUCTION

BACKGROUND

This research study, sponsored by the Federal Highway Administration (FHWA), summarizes the field performance of high-early-strength (HES) concrete patches constructed under Strategic Highway Research Program (SHRP) study C-205. [1]
HES is designed to develop the strength needed for opening in 12 to 24 hours through:

- The use of Type III cement.
- High cement content.
- Low water to cement (w/c) ratio.
- Addition of nonchloride accelerator—calcium nitrite.

A high-range water reducer (HRWR) is used in the mixture to maintain workability at a low w/c ratio.

The test sections were constructed between June 1991 and July 1992 in five States that represent a wide variety of environmental and exposure conditions: Arkansas, Illinois, Nebraska, New York, and North Carolina. The basic configuration of the test sections consist of six 4.6-meter (m) (15-feet (ft)) full-depth patches made with HES. Three of the patches were insulated for more rapid strength gain, and three were uninsulated for normal curing. The test sections in Arkansas, Illinois, and Nebraska are of this basic configuration. In New York, only three insulated patches were placed.

More extensive testing was conducted at the North Carolina site. The North Carolina section consists of 55 m (180 ft) of U.S. 17 on both driving lane and passing lane. The experimental factors for the North Carolina sections include the following:

- Insulation—insulated and uninsulated.
- Aggregate type—crushed gravel and marine marl.
- HRWR type—naphthalene and melamine.

The SHRP-C-205 report discusses aspects such as HES mixture proportioning, mixing and curing, laboratory experiments, and field installations.[1] Laboratory tests were conducted on both fresh or plastic and hardened concrete (both laboratory-cured and field-cored samples). These tests included the determination of fresh concrete properties such as slump, air content, and unit weight, as well as the estimation of the strength, modulus, shrinkage, and freeze-thaw durability for hardened concrete.

SHRP C-205 showed that it is possible to produce HES concrete that will achieve its desired strength (35 megapascals (MPa) or 5000 pounds per square inch (psi) in 24 hours) using conventional materials and equipment. However, more care is needed during batching, placing, and finishing than for normal concrete.

The monitoring of the SHRP C-205 test sections conducted under this project showed that HES concrete patches can perform adequately in the field with no extraordinary signs of durability-related distresses under a wide range of climatic conditions. However, because the study only spanned 7 years into the life of the patches, additional monitoring is needed to evaluate the long-term durability of these materials.

DESCRIPTION OF THE FIELD SECTIONS

High-performance concrete patches constructed at the following sites were monitored as part of this study:

1. I–88 eastbound (EB) near marker 88I-9406-3158 in Otsego County near Worcester, NY
2. U.S. 17 just north of Williamson, NC
3. I–57 about 16 kilometers (km) (10 miles (mi)) north of Effingham, IL
4. I–40 less than 8 km (5 mi) west of Forrest City, AR
5. U.S. 20 at mile marker 361 (Station 435) northwest of Norfolk, NE

Brief descriptions of each patch, highlighting the various section details and other relevant site information, are presented below. Most of the information in this section was obtained from the original SHRP report.[(1)]

New York

This installation was a full-depth patch constructed on Interstate 88, near the town of Worcester (about 82 km (50 mi) west of Albany). The patch is located in the eastbound passing lane near reference marker 88I-9406-3158. The patch was constructed on June 25, 1991. The original pavement was approximately 12 years old at the time the patch was built.

The patch is approximately 18.3 m (60 ft) long, 3.7 m (12 ft) wide, and 225 millimeters (mm) (9 inches (in.)) thick, with doweled joints placed at 6.1-m (20-ft) intervals. Epoxy-coated dowel bars were used, and the patch was provided with welded wire mesh reinforcement. The subbase consists of a 300-mm (12-in.) layer of sand, gravel, slag, and stone. The longitudinal joint with the adjacent lane was greased to act as a bond breaker. This patch was fully insulated to simulate properties of very-early-strength (VES) concrete.

The climatic exposure of the pavement can be described as wet-freeze. The annual average daily traffic (AADT) was approximately 6,200 vehicles with about 20 percent trucks.

Type III cement was used in constructing the patch, and the coarse aggregate was a crushed limestone. The admixtures used in the project included an HRWR, an air-entraining admixture (AEA), and calcium nitrite. The materials were batched in different proportions in the laboratory prior to field installation, and the optimum mix design was selected. However, modifications had to be made in the field to the predetermined quantities to adjust for acceptable fresh concrete properties such as slump and air content.

Table 1 contrasts the laboratory concrete mix proportions with the proportions batched onsite. The field mixture proportions satisfied the laboratory-specified values, with the exception of the amounts of water and HRWR added. The mix water in the laboratory seems higher than the field value. However, this is only an apparent discrepancy since the reported laboratory value was adjusted for free water in the aggregates and the calcium nitrate admixture, whereas the field value was not. The total amounts of water in both the laboratory and field were presumed to be the same if this discrepancy is accounted for. The discrepancy in the HRWR contents was noted in the original SHRP report.[(1)] In the field, the HRWR was reduced to decrease the total slump and air content in the mixture.

Table 1. Comparison of final laboratory and actual field concrete mix proportions for the New York site. [(1)]

Material	Laboratory Mix Proportions	Field Batch Proportions,[1] Average (Range)
Cement (Type III), lb/yd^3	810	816 (815–818)
Water, lb/yd^3	276[2]	190 (190–190)
Coarse aggregate, lb/yd^3	1790	1738 (1658–1776)
Fine aggregate, lb/yd^3	1040	1090 (1077–1108)
HRWR, gal/100 lb of cement	1.33	1.1 (1.0–1.33)
AEA, gal	0.38	0.38 (0.38–0.38)
Calcium nitrite, gal/yd^3	6	6 (6–6)

[1] Determined from the proportions reported from the three concrete delivery trucks.
[2] Adjusted for free aggregate moisture and water in calcium nitrite.
1 pound (lb)/cubic yard (yd^3) = 0.593 kilogram (kg)/m^3; 1 gallon (gal)/yd^3 = 4.94 liter (L)/m^3; gal/100lb = 0.083 L/kg

North Carolina

The experimental placement is on U.S. 17 over the Roanoke River, just north of Willamston. The pavement was constructed between July and August of 1991.

The high-performance concrete pavement was approximately 55 m (180 ft) long and two lanes wide. It was unreinforced and jointed at 4.6-m (15-ft) intervals. However, dowel bars were placed only at the end of each day's placement, with a maximum of 36.6 m (120 ft) between doweled sections. The inside lane was placed first, and its construction was controlled tightly, with emphasis on testing and materials. The lane was placed in three different sections, each 18.3 m (60 ft) long. The outside lane concrete was placed using typical North Carolina Department of Transportation (NCDOT) batching and placement rates, also in three 18.3-m-long (60-ft-long) sections. Here the emphasis was to study the impact of routine construction methods on variations in the product delivered. The minimum depth of the patches is 225 mm (9 in.). The pavement section was built on an asphalt base course.

To mimic the strength development of VES concrete, some of the sections were insulated by first covering them with plastic sheets and subsequently placing 25-mm-thick (1-inch-thick) rigid-foam building insulation on the slab. The insulation was removed after 6 hours. The pavement can be described as being exposed to a mild marine environment.

Type III cement and two coarse aggregate types—crushed granite (CG) and marine marl (MM)—were used in preparing the concrete mixture. The CG aggregate is very hard and tough, whereas the MM is a relatively porous shell limestone. In addition, two different HRWRs were used, one with a melamine base and the other with a naphthalene base. Apart from this, AEA and calcium nitrite were used as admixtures. Taking into account the curing methods employed (insulation versus no insulation) and the different admixtures and aggregates used, a total of eight unique combinations of test sections were built at the North Carolina site. All the mixtures contained the same nominal cement content and had approximately the same w/c ratio. The fine aggregates used in all the mixes were also the same.

Illinois

Two high-performance concrete experimental patches were constructed on Interstate 57, about 16 km (10 mi) north of Effingham. The patches were constructed in October 1991 and are situated in the outside lane of the northbound highway, separated by a distance of 305 m (1000 ft). The original pavement was approximately 24 years old when the patches were built.

The patches are 13.7 m (45 ft) long, 3.7 m (12 ft) wide, and 250 mm (10 in.) thick, with doweled joints placed at 4.6-m (15-ft) intervals. Epoxy-coated dowel bars were used at all transverse joints. Both patches were constructed with welded wire fabric. The patches were placed on the existing granular subbase. One of the patches was fully insulated; the other was not.

The pavement is situated in a wet-freeze environment. The average daily traffic (ADT) for this roadway is approximately 11,800 with approximately 22 percent trucks. Type III cement and limestone coarse aggregate were used in constructing the patch. The admixtures used in the project included an HRWR, an AEA, and calcium nitrite. The materials were batched in different proportions in the laboratory prior to field installation, and the optimum mix design was selected. However, modifications had to be made in the field to the predetermined quantities to adjust for acceptable fresh concrete properties such as slump and air content.

Table 2 contrasts laboratory concrete mix proportions with proportions batched onsite. The onsite proportions were computed by taking averages of quantities used to prepare batches for each of the six trucks that delivered the concrete to the patches. The field mixture proportions satisfied the laboratory-specified values, with the exception of the water content added to the mix. It is assumed that this discrepancy can be attributed to the fact that the field water content reported does not include the water contributed from

Table 2. Comparison of final laboratory and actual field concrete mix proportions for the Illinois site.(1)

Material	Laboratory Mix Proportions	Field Batch Proportions[1] Average (Range)
Cement (Type III), lb/yd^3	870.00	867 (865–869)
Water, lb/yd^3	299.00	194 (183–198)
Coarse aggregate, lb/yd^3	1685.00	1743 (1732–1751)
Fine aggregate, lb/yd^3	1030.00	934 (896–957)
HRWR, gal/100 lb of cement	1.09	1.25 (1.23–1.35)
AEA, gal	0.23	0.23 (0.23–0.23)
Calcium nitrite, gal/yd^3	4.00	4 (4–4)

[1] Determined from the proportions reported from the six concrete delivery trucks.
1 lb/yd^3 = 0.593 kg/m^3; 1 gal/yd^3 = 4.94 L/m^3; gal/100lb = 0.083 L/kg

the calcium nitrite and the aggregates. Although the water content in the field could not be determined for this site due to lack of information about the exact amount of water contributed to the mixture by the aggregate, it was presumed in the original SHRP study that the laboratory and field values match approximately.(1)

Arkansas

The Arkansas installation is on Interstate 40, less than 8 km (5 mi) west of Forrest City. There are two patches in the passing lane of the westbound traffic near mile marker 237. The patches are separated by a distance of about 152 m (500 ft) and were built in November 1991. The original pavement was approximately 25 years old when the patches were built and was built on cement-treated subgrade.

The patches are 13.7 m (45 ft) long, 3.7 m (12 ft) wide, and 250 mm (10 in.) thick, with doweled transverse joints placed at 4.6 m (15 ft) intervals. One of the patches (the easternmost) was insulated with rigid foam insulation for about 3.5 hours after the concrete had set.

The pavement is situated in a wet environment with potential for freeze-thaw cycling. The two-way AADT near the installation was approximately 19,890 vehicles with just over 47 percent trucks.

Type III cement was used in constructing the patch, and the coarse aggregate was limestone. The admixtures used in the project included an HRWR, an AEA, and calcium nitrite. The materials were batched in the laboratory in accordance with the original HES mix design prior to field installation.(1) With the exception of a minor modification to the quantity of AEA, the rest of the original HES mix design was adopted. This adjustment was necessitated to achieve the target air content. Table 3 presents a comparison of the laboratory concrete mix proportions with the proportions batched onsite. The onsite proportions were computed by taking averages of the quantities used to prepare batches for each of the six trucks that delivered the concrete to the patches.

Table 3. Comparison of final laboratory and actual field concrete mix proportions for the Arkansas site.(1)

Material	Laboratory Mix Proportions	Field Batch Proportions[1] Average (Range)
Cement (Type III), lb/yd^3	877.00	871 (866–881)
Water, lb/yd^3	237.00	238 (236–239)
Coarse aggregate, lb/yd^3	1693.00	1702 (1680–1720)
Fine aggregate, lb/yd^3	1080.00	1064 (1060–1080)
HRWR, gal/100 lb of cement	1.23	1.01 (0.97–1.14)
AEA, gal	0.31	0.31 (0.31–0.32)
Calcium nitrite, gal/yd^3	4.00	4.08 (4–4.29)

[1] Determined from the proportions reported from the seven concrete delivery trucks.
1 lb/yd^3 = 0.593 kg/m^3; 1 gal/yd^3 = 4.94 L/m^3; gal/100lb = 0.083 L/kg

Based on the field batching, the w/c ratio was 0.27, and the standard deviation was 0.002 between batches. The average entrained air content, slump, and unit weight were 4.6 percent, 183 mm (7.2 in.), and 2318 kg/m^3 (143 lb/ft^3), respectively.

Nebraska

Two high-performance concrete patches were built in Nebraska on the eastbound lane of U.S. Highway 20 between the towns of Osmond and Plainsview. The patches, constructed in July 1992, are the youngest of all the sections discussed in this report. The original concrete pavement was built in 1957 and consists of a 200-mm (4-in.) granular subbase resting on a silty clay subgrade.

The patches are 14.64 m (48 ft) long, 3.35 m (11 ft) wide, and 200 mm (8 in.) thick. They were built end to end. One patch was insulated during the construction for roughly 5.5 hours after the concrete had set. Transverse joints were sawed at 4.9-m (16-ft) intervals within each patch, and epoxy-coated dowel bars were placed at all transverse joints.

The pavement is situated in a wet environment with a very high potential for freeze-thaw cycling. Approximately 1,500 vehicles per day with about 20 percent trucks constitute the traffic at the site.

Type III cement was used in constructing the patch, and the coarse aggregate was limestone. The admixtures used in the project included an HRWR, an AEA, and calcium nitrite. The materials batched in the laboratory were, for the most part, in accordance with the original HES mix design, but the coarse aggregate and fine aggregate contents were reversed to suit local practice.(1) This adjustment was necessitated to reduce the probability of alkali-silica reactivity (ASR) prevalent in the area.

Table 4 presents a comparison of the laboratory concrete mix proportions with the proportions batched onsite. The onsite proportions were computed by taking averages of quantities used to prepare batches for five of the six trucks that delivered the concrete to the patches. Data from one of the trucks was not used since the slab poured with the concrete mix from this truck was eventually replaced. It can be noted from the table that the field batching followed the laboratory specifications quite well.

Table 4. Comparison of final laboratory and actual field concrete mix proportions for the Nebraska site.[(1)]

Material	Laboratory Mix Proportions	Field Batch Proportions[1] Average (Range)
Cement (Type III), lb/yd^3	873.00	876 (874–881)
Water, lb/yd^3	228.00	214 (201–224)
Coarse aggregate, lb/yd^3	1087.00	1057 (1046–1065)
Fine aggregate, lb/yd^3	1710.00	1747 (1745–1749)
HRWR, gal/100 lb of cement	1.23	1.44 (1.36–1.64)
AEA, gal	0.34	0.33 (0.31–0.34)
Calcium nitrite, gal/yd^3	4.00	4 (4–4)

[1] Determined from the proportions reported from the five of the six delivery trucks.
1 lb/yd^3 = 0.593 kg/m^3; 1 gal/yd^3 = 4.94 L/m^3; gal/100lb = 0.083 L/kg

SUMMARY OF SECTION PROPERTIES—EXPERIMENTAL DESIGN

Table 5 presents a summary of fresh concrete properties and important climatic indicators for each of the five experimental sites considered in this study. Mean values of the concrete w/c ratio, slump, air content, and unit weight are included in the table along with measures of variability of these properties within each site. The data presented in the table will be valuable in interpreting the long-term concrete performance data collected as part of this study and to draw important conclusions about the applicability of high-performance concrete as a patching material.

The data on the average number of freeze-thaw cycles and the average freezing index were not collected at the sites under consideration in this study. They were determined from the Long-Term Pavement Performance (LTPP) climatic database from a weather station closest to the site under consideration. Also, the freeze-thaw cycles reported were based on air temperature and not pavement temperature. The actual pavement freeze-thaw cycles will be lower than this value, the exact magnitude being a function of several other site and pavement variables. Regardless, since the same parameter was used for all the sites, these data should give a relative indication of the climatic conditions at each of the sites.

Table 5. Summary of materials, site factors, and fresh concrete properties for the SHRP C-205 sites.

Site	**Concrete Materials Used in Patch Construction**	**Patch Curing Conditions**	**Climate Indicators**			**Fresh Concrete Properties**			
			LTPP Classification	**Average Air F-T[1] Cycles**	**Freezing Index, °F-days**	**W/C Ratio Mean (SD[2])**	**Slump, in Mean (SD)**	**Air, Percent Mean (SD)**	**Unit Weight, lb/ft^3 Mean (SD)**
New York	Cement—Type III Coarse aggregate—crushed limestone	Insulated	Wet-freeze	113	584	0.34 (0.00)[3]	6.4 (1.6)	8.6 (2.4)	141.0 (4.3)
North Carolina	Cement—Type III Coarse aggregate—crushed granite; marine marl	Insulated and uninsulated	Wet-no freeze	71	45	0.33 (0.015)	4.7 (2.3)	6.6 (1.8)	142.4 (3.6)
Illinois	Cement—Type III Coarse aggregate—crushed limestone	Insulated and uninsulated	Wet-freeze	84	298	0.33 (0.01)[1]	8.9 (1.3)	3.2 (0.5)	139.3 (2.2)
Arkansas	Cement—Type III Coarse aggregate—crushed limestone	Insulated and uninsulated	Wet-no freeze	56	63	0.27 (0.00)	4.6 (1.4)	7.2 (1.3)	142.9 (2.1)
Nebraska	Cement—Type III Coarse aggregate—crushed limestone	Insulated and uninsulated	Wet-freeze	108	581	0.24 (0.01)	4.0 (1.9)	9.4 (3.3)	137.0 (3.9)

[1] F-T stands for air freeze-thaw cycles.
[2] SD stands for standard deviation.
[3] The mean w/c ratio reported is based on laboratory mix proportioning. The standard deviation is based on field batching data.
1 lb/ft^3 = 16.02 kg/m^3; 0 °C = 32 °F

Based on the freeze-thaw cycle information presented in table 5, the Nebraska and New York test pavements are in the harshest cold-weather climate, followed by Illinois, North Carolina, and Arkansas. The North Carolina section uses different coarse aggregates than the others. The mix proportions are relatively uniform (not all mix information is presented in the table), except for some variation in the w/c ratio. The factorial presented in the table forms a good basis for reasonable comparison of the durability of high-performance concrete.

OBJECTIVE AND SCOPE OF THE STUDY

The objective of this study was to perform long-term performance monitoring to verify the effectiveness of high-performance concrete patches constructed in the previously referenced SHRP study. The emphasis during this monitoring effort was on determining the durability and integrity of the concrete over time. The following work items were undertaken to realize the objective of the study:

- Collection of important site information such as weather conditions, traffic, use of deicers, and maintenance history of the pavement sections.
- Visual inspection using standard SHRP distress identification procedures to detect durability problems such as joint spalling and cracking.
- Obtaining photograph and video logs of each patch and the sections within each patch to record the progression of distress.
- Obtaining and testing cores for compressive strength, elastic modulus, rapid chloride permeability, and asphalt concrete (AC) impedance.

Visual inspections of the patches were to be performed once every year for a period of 5 years beginning in 1994. Cores were obtained from the field in alternate years instead of every year, as was intended in the original project plan.

CHAPTER 2. VISUAL EXAMINATION OF SHRP C-205 SITES

Visual examination of the HES concrete patches was conducted each year between 1994 and 1998, usually between the months of August and November. The survey teams consisted of experienced personnel, and care was taken to maintain uniformity in the data collection, as much as possible, over the entire performance monitoring period. The main data items of interest during the visual examination process included recording the progression of pavement distress within each patch, with special focus on materials-related distresses. The distresses were recorded through a variety of means, including mapping distresses, recording any changes in condition, and taking photos and videos of the section. In addition to monitoring the condition of concrete, for the North Carolina section it was also meaningful to collect and analyze pavement performance data such as faulting, transverse cracking, and joint spalling. At each site, the respective State DOT provided necessary traffic control to perform the distress surveys.

Appendix A provides detailed maps showing the locations and magnitudes of the various pavement distresses as they developed during the performance period. Major findings from the visual surveys of each site are summarized below.

ARKANSAS

The HES patches (insulated and uninsulated) in Arkansas could be surveyed only in 1994 and 1995. The patches were overlaid shortly after the 1995 survey as part of a major rehabilitation project on I–40. Each patch consisted of three pavement slabs 4.6 m (15 ft) long. Both patches exhibited minor scaling and map cracking in 1994. However, their condition did not vary significantly between the two survey dates. The joint sealant was in good condition on all sections, with no evidence of durability-related distresses.

ILLINOIS

The Illinois sections exhibit some map cracking. The level of cracking did not change substantially over the monitoring period, although some progression of map cracking was noted over the years. One plausible cause for this distress is the potential use of deicing salts on these patches. Subjectively, at least, the sections generally seemed more deteriorated in 1998 than in 1994 (perhaps the impression is due to more scaling). However, as noted earlier, these changes were difficult to quantify. Figures 35 through 39 in appendix A show that both Illinois sections had some transverse cracking. The structural crack in the IL-2 section existed since 1994, and no significant changes in progression or deterioration of the crack occurred over the monitoring period. Other cracks are a result of the interconnecting of map cracks. A minor progression of these cracks occurred over the monitoring period, but none appeared to have progressed to a working, structural crack.

NEBRASKA

The Nebraska sections have a few transverse cracks, which have existed since 1994 (see figures 40 through 44 in appendix A). A modest increase in map cracking was noticed in 1998. The map cracking occurred mostly along the joints and cracks and around the cores. The pattern cracking appeared characteristic of ASR, which is known to occur in the area. It is interesting to note, however, that map cracking appeared to be a problem only on the repair slabs (not just the SHRP repairs, but also other repairs in the area). The original concrete, placed in 1957, did not exhibit any signs of map cracking.

NEW YORK

The one experimental patch in New York remained in excellent condition with no signs of map cracking or scaling over the entire monitoring period. The scaling and map cracking prevalent in patches located at other sites did not affect this patch. It was reported in the original SHRP study that deicing salts were not used on this section between 1991 and 1993.(1)

NORTH CAROLINA

The North Carolina section is in excellent condition with no signs of map cracking or scaling. The concrete condition did not change over the monitoring period. The North Carolina patch is relatively long compared to the other patches, so monitoring the pavement performance is more meaningful. The pavement performance data from the North Carolina section are summarized in table 6. The transverse cracks present at the time of construction (due to delay in sawing joints) progressed over time from low to high severity in some locations. The section also exhibits some joint spalling and faulting. However, the levels of these distresses remained relatively low and their condition remained virtually unchanged over the entire performance monitoring period.

SUMMARY OF VISUAL EXAMINATION RESULTS

In general, the test sections did not show any significant changes in condition over the monitoring period, especially those in the warmer climates. The sections in freezing climates—the Illinois and Nebraska sections—showed some amount of map cracking and scaling. The severity of map cracking at these two sites increased slightly over the monitoring period, but not significantly enough to report as a different level of severity. Quantifying changes in severity for such distress proved difficult. The sites cannot be mapped accurately enough over the years to note minor changes that occur from year to year, and the photos do not show enough detail to be useful for this purpose.

Table 6. Distress summary of SHRP C-205 section in North Carolina.

Station Number	Transverse Cracking					Joint Spalling					Faulting (inch)				
	1994	1995	1996	1997	1998	1994	1995	1996	1997	1998	1994	1995	1996	1997	1998
0+15							L	L	L	L	-0.01	-0.03	0.01	0.00	0.07
0+30	L[1]	M[1]	M[1]	M[1]	M[1]						-0.03[2]	0.00[2]	0.03[2]	-0.01[2]	0.00[2]
0+45						L	L	L	L	L	0.04	0.05	0.06	0.06	0.06
0+60						L	L	L	L	L	0.00	0.00	-0.01	0.03	-0.01
0+75						L	L	L	L	L	0.10	0.10	0.14	0.10	0.11
0+90						L	L	L	L	L		0.10	0.10	0.10	0.08
1+05	L[1]	M[1]	M[1]	M[1]	M[1]		L	L	L	L	0.07[2]	0.06[2]	0.07[2]	0.05[2]	0.06[2]
1+20	H	H	H	H	H			Patch Joint				Patch Joint			
1+35							L	L	L	L	-0.01	0.00	0.01	0.06	0.08
1+50	M[1]	M[1]	M[1]	M[1]	M[1]		L	L	L	L	0.06[2]	0.07[2]	0.10[2]	0.10[2]	0.10[2]
1+65											0.07	0.08	0.06	0.11	0.18
Percent/Average						**31%**	**54%**	**54%**	**54%**	**54%**	**0.03**	**0.04**	**0.06**	**0.06**	**0.07**

[1] Joint not sawed.
[2] Faulting of the crack.
1 inch = 25.4 mm
L = Low
M = Medium
H = High

CHAPTER 3. CONCRETE CORE TESTING

CORING PLAN

In conjunction with the visual surveys conducted on the various sites, cores were obtained from within each test patch on a regular basis to evaluate the durability of the concrete materials. As noted earlier, the original project plan called for obtaining the concrete cores in each of the 5 years during which performance was monitored. However, to protect the structural integrity of the patches, and to preserve their condition, the coring plan was revised to taking cores only in alternate years. Therefore, cores were obtained only in 1994, 1996, and 1998 from most of the patches with the exception of the patch located in Arkansas. From this section, only two sets of cores could be obtained, one in 1994 and the other in 1995, just before the pavement was overlaid.

The cores were subjected to tests for compressive strength, modulus of elasticity, rapid chloride permeability, and AC impedance. Wherever possible, companion cores were obtained from the patches so that replicate testing could be performed for each test type. Table 7 summarizes the types of tests performed at each location, along with the year in which the tests were performed. (For brevity, the frequency and types of AC impedance testing performed are not shown in the table.) Many AC impedance tests were carried out in this study, using numerous combinations of test parameters. Further discussion of AC impedance testing is provided later in the report.

The gaps in table 7 indicate instances when it was not possible to perform some of the testing. Furthermore, as noted in the table, each test patch in North Carolina was treated separately, and coring was done accordingly. Recall that multiple patches were constructed in North Carolina to determine the effects of different kinds of coarse aggregates and water reducers on the performance of concrete. Naphthalene (N) and melamine (M) water reducers were used along with crushed granite (G) and marine marl (MM) to set up the four experiments: NM, NG, MM, and MG.

Prior to the origination of this contract, cores were obtained at 28 days and 6, 12, and 18 months for comparison with cylinder strengths; their corresponding test results were documented in SHRP C-364. These data were also used in the analysis presented in this report.

SIGNIFICANCE OF PERFORMANCE TESTS AND LABORATORY TEST PROTOCOLS

As noted earlier, four types of laboratory testing were performed on the cores retrieved from the field sites. A brief description of the significance of each test in relation to the main objective of this study is presented below. The protocols that were followed to estimate the respective property values are also presented.

Table 7. Summary of testing performed.

Location	Insulated Sections									Uninsulated Sections								
	Compressive Strength			Modulus of Elasticity			RCPT			Compressive Strength			Modulus of Elasticity			RCPT		
	1995	1997	1999	1995	1997	1999	1995	1997	1999	1995	1997	1999	1995	1997	1999	1995	1997	1999
New York	X	X	X	—	X	X	X	X	X	—	—	—	—	—	—	—	—	—
North Carolina																		
NM	X	X	X		X					X	X	X		X				
NG	X	X	X	—	X	X	X	X	X	X	X	X	—	X	X	X	X	X
MG	X	X	X		X	X	X	X	X	X	X	X		X	X	X	X	X
MM	X	X	X		X	X				X	X	X		X	X			
Illinois	X	X	X	—	X	X	X	X	X	X	X	X	—	X	X	X	X	X
Arkansas[1]	X	X		—	X	—	X	X	—	X	X	—	—	X	—	X	X	—
Nebraska	X	X	X	—	X	X	X	X	X	X	X	X	—	X	X	X	X	X

[1] The cores designated as being tested in 1997 were actually obtained in 1995. This was accounted for in the data analysis.
"**X**" indicates test was performed; "—" indicates test was not performed. Gaps in text indicate instances when it was not possible to perform testing.

Compressive Strength and Elastic Modulus Testing

In this study, the compressive strength and elastic modulus of concrete were used as surrogate indicators of material durability. Sound concrete with no materials-related problems is expected to gain strength and, to a lesser degree, modulus over time because of the continuing hydration process. The onset of any materials durability-related distress such as durability- (D-) cracking or ASR leads to the deterioration of concrete and results in the development of cracks in the cement paste and the aggregates. This causes a drop in the compressive strength and elastic modulus values. Therefore, by periodically monitoring the rate of strength and modulus gain (or drop), any durability-related problems can be detected.

All of the compressive strength and elastic modulus tests were conducted after removing the top and bottom portions of the 100-mm-diameter (4-inch-diameter) cores, to form 200-mm-long (8-inch-long) specimens. The ends were capped with a sulfur-based capping compound and tested to failure in compression in accordance with ASTM C 39 for compressive strength and ASTM C 469 for elastic modulus. To represent their condition in the pavement more accurately, the cores were not soaked before testing.

Rapid Chloride Permeability Testing

The rapid chloride permeability test (RCPT) indicates the ability of the concrete to resist the penetration of chloride ions. Chloride ions may enter into the concrete from three major sources: set accelerating admixtures, deicing salts, and seawater and salt spray.(2) Although the ingress of these ions into concrete is primarily detrimental to the performance of the embedded steel, it can also produce durability problems in concrete such as salt scaling and map cracking. The test measures the amount of charge passed when a 60-volt DC electrical potential is placed across a concrete specimen for a 6-hour period. The greater the charge, the lower is the resistance of the concrete to chloride ion ingress, and therefore the greater the potential for materials-related problems. Therefore, the RCPT values over time can give an indication of the durability of concrete.

The rapid chloride permeability tests were carried out on 100-mm-diameter (4-inch-diameter) specimens cut from the area between 12.5 and 62.5 mm (0.5 and 2.5 in.) from the top of the core, and from 12.5 to 62.5 mm (0.5 to 2.5 in.) from the bottom of the core, where applicable. Note that the bottoms of the cores have only been tested since 1997. The outside of the specimens was coated in epoxy, vacuum-saturated, and tested according to ASTM C 1202.

AC Impedance Testing

AC impedance testing was considered in the original SHRP study as a reasonable alternative to the RCPT for more rapid determination of concrete permeability. The goal was to explore a faster method that could be more versatile and portable.(1) The AC impedance test values have an inverse relationship to the RCPT values.

In the original SHRP study, the AC impedance testing was conducted using a Kohlrausch bridge instrument. The Kohlrausch bridge uses a 1000-Hz AC frequency. The electrical connection between the Kohlrausch bridge terminals and the concrete specimens was ensured using potassium agar gel. The impedance measurement was measured at five random points on each specimen. In the present study, a slightly different test setup used a programmable potentiostat in place of the Kohlrausch bridge. Other setup variables such as the frequency of the current and the electrical coupling between the potentiostat and the concrete specimen were also varied in an effort to establish a reasonable correlation with the RCPT measurements. Based on this experimentation, it was decided that the impedance values obtained at a frequency of 100 Hz AC using a Nilsson soil resistance meter in two-pin mode provided the best correlation with the RCPT measurements. These values are reported in this study.

DESCRIPTION OF METHODS USED FOR DATA ANALYSIS

The results from the various tests conducted are analyzed and presented in this section. In order to make the data more meaningful and to draw broad-based conclusions, the concrete core and cylinder test data from report SHRP-C-364 were merged with the database compiled from this study.[1]

A major task in this study was to conduct field studies to investigate how HES concrete mix properties and field conditions influence durability and, hence, performance. A measure of durability was obtained by monitoring the following properties:

- Concrete compressive strength (ASTM C 39).
- Concrete static modulus of elasticity (ASTM C 469).
- Electrical indication of concrete's ability to resist chloride ion penetration through the use of RCPT (ASTM C1202).
- AC impedance test at 100 and 1000 Hz (SHRP-C-364).

A list of the durability indicator data available and used for investigation and analysis has already been presented in table 7. The first step in assessing the durability of HES concrete was to conduct a comprehensive bivariate analysis. The bivariate analysis consisted of plots of the various durability indicator variables with concrete age. These plots were used for observing trends of the durability indicator variables (dependent variable) over the performance monitoring period, which was approximately 7 to 9 years for the sites evaluated. Bivariate statistics measure the degree of dependence between two variables (dependent and independent). They also show the trends and changes in the value of the dependent variable as the level of the independent variable is varied. For this study, the dependent variable was a durability indicator variable and the independent variables were the climate, construction, or materials-related variables or cluster of variables that could influence the concrete's long-term durability.

Results of the bivariate analysis were presented in the form of plots of the durability indicator variables against age for the different levels of the independent variables being evaluated. Simple statistical regression curves, trend lines, and correlation coefficients were also developed, where necessary. The plots visually show the effect of the independent variables under investigation on the durability indicators. They were used to determine if the trends observed were reasonable and as expected from mechanistic analysis, engineering judgment, and past empirical analyses. However, because bivariate plots present the effects of only a single independent variable on the durability indicator, their effect on the durability indicator variable can be confounded by the effects of other independent variables not considered. This could lead to contradictory and misleading results. Observations from bivariate plots are therefore preliminary in nature and cannot be conclusive.(3)

More sophisticated statistical tools such as analysis of variance (ANOVA) were also used to investigate the effect of the independent variables on durability. ANOVA determines whether differences in observed trends were statistically significant by determining the significance of each independent variable included in a linear regression model used to predict the given durability indicator. The linear regression models used for ANOVA are designed based on the hypothesis to be tested (e.g., the significance of a given independent variable).

ANOVA models are versatile statistical tools for studying the relationship between a dependent variable (durability indicator variable) and one or more independent variables (e.g., climate). They do not require making assumptions about the nature of the statistical relationship, nor do they require that the independent variables be quantitative. ANOVA models are generally used for applications where the effects of one or more independent variables on the dependent variable are of interest. Independent variables in the models for ANOVA are mostly called factors or treatments. For example, for this study, the independent variable (aggregate type) had three factors, levels, or treatments (limestone, marine marl, and crushed granite). The basic ANOVA procedures used in this study for determining the effects of several factors on the dependent variables are summarized as follows(3):

1. Determine all independent variables to be analyzed and transform the continuous independent variables into classification variables.
2. Develop ANOVA models for predicting each durability indicator variable to be analyzed. The models must be based on a specific experimental design to determine the effect of a specific group of independent variables on the durability indicator.
3. Perform ANOVA; using the models developed, test the hypotheses (the effect of curing method on durability, the effect of aggregate type on durability); and then determine if there is a significant difference in RCPT results from the top and bottom portions of a given core specimen.

The procedure outlined is simple and suits the purposes of most simple analysis of variance. Key elements of the ANOVA procedure are further explained in the following sections.

ANOVA Models

ANOVA models are basic, type I statistical models. They are concerned, like regression models, with the statistical relation between one or more independent variables and a dependent variable. Like regression models, ANOVA models are appropriate for both observational data and data based on formal experiments. Further, like the usual regression models, the dependent variable for ANOVA models is a quantitative variable. However, they differ from ordinary regression models in two key respects. First, the independent variables in the ANOVA model can be qualitative (e.g., aggregate type). Second, if the independent variables are quantitative, no assumption is made in the ANOVA models about the nature of the statistical relation between them.[3]

Hypothesis Testing

The goal of ANOVA is to compare means of the response variable (durability indicator) for various combinations of the classification variables (e.g., aggregate type and curing method). The effect and significance of the variables on performance can be confirmed or verified by comparing the level of significance of the variables to a predetermined level of significance, called p-value. ANOVA determines if there is a statistical difference in the mean values of the distress for the different classes of the independent variables in the model. The mean level of the distress for the different regression model classes gives an indication of whether the independent variable has a positive or negative effect on the distress. The following example illustrates the ANOVA technique.

Treatment or factor (aggregate type):	limestone		
	granite		
Durability indicator (compressive strength):	class (limestone)	=	μ_{LS}
	class (granite)	=	μ_{GR}

The significance of the effect of aggregate type on the given durability indicator is determined by the following test of hypothesis:

Null hypothesis,	H_o:	μ_{LS}	$=$	μ_{GR}
Alternative hypothesis,	H_A:	μ_{LS}	$\neq$	μ_{GR}

Based on a significance level (p-value) of 5 percent (0.05), a p-value of less than 0.05 rejects the null hypothesis, whereas a result greater than 0.05 confirms the null hypothesis. A comparison of the magnitude of the means determines the nature of the effect of the independent variable (in this case, aggregate type) on the durability indicator.

DATA ANALYSIS

This section presents the results of both the bivariate analysis and a more detailed analysis of variance used in the investigation of the influence of climate and concrete mix properties on durability.

Experimental Design

For both the bivariate analysis and ANOVA, the effects of climate, construction, and mix properties on durability were investigated by grouping the data according to climate and mix properties. This was because relatively few data were available, as shown in table 7. To keep the individual evaluation data sets to a minimum size, engineering judgment was used to divide the data intelligently. Each of these divisions was specific to the durability indicator being investigated and is described in greater detail in later sections of this chapter.

Climatic region was defined using the mean annual precipitation, mean annual air freeze-thaw cycles, and the mean annual freezing index of the sites; mix properties were defined using the aggregate type. Table 8 presents an example of the matrix of site and material properties used in grouping the test sites for analysis.

Bivariate Analysis—Compressive Strength

Figures 1 through 5 show the strength development history for the five sites analyzed. The plots show the effects of curing condition, specimen type (cylinder or core), strength at key ages, and site or climate. The specimen type describes whether the test data came from cores obtained from the concrete slabs or came from cylinders cast in foam boards using concrete samples obtained during slab placement.

The data presented in figures 1 through 5 show no consistent trend in compressive strength in either insulated or uninsulated sections. However, the compressive strengths from cylinders (insulated or uninsulated) were generally higher than those from cores. The comparison of core and cylinder compressive strength was limited to data representing the age range from 28 days to 18 months because there were no data for cores outside this age range. For North Carolina, compressive strength was plotted according to aggregate type and water reducer types to determine their effect on strength gain, as shown in figure 3.

Table 8. Matrix of climate and material properties
used in grouping the test sites for analysis.

Precipitation (Wet > 20 in./yr)	Temperature (Freeze Index)	Freeze-Thaw Cycles	Aggregate Type	Sites
Wet	Freeze	< 100	Limestone	Illinois
			Crushed granite	—
			Marine marl	—
		>100	Limestone	Nebraska, New York
			Crushed granite	—
			Marine marl	—
	No freeze	< 100	Limestone	Arkansas
			Crushed granite	North Carolina
			Marine marl	North Carolina
		>100	Limestone	—
			Crushed granite	—
			Marine marl	—
Dry	Freeze	< 100	Limestone	Illinois
			Crushed granite	—
			Marine marl	—
		>100	Limestone	Nebraska, New York
			Crushed granite	—
			Marine marl	—
	No freeze	< 100	Limestone	Arkansas
			Crushed granite	North Carolina
			Marine marl	North Carolina
		>100	Limestone	—
			Crushed granite	—
			Marine marl	—

1 inch = 25.4 mm; 0 ^{o}C = 32 ^{o}F

Freeze Index:
Freeze > 277 ^{o}C degree-days
No freeze < 277 ^{o}C degree-days

The number of degree-days is the sum of the total number of degrees below freezing on each day when the average daily temperature is below freezing. For instance, if the average daily temperature is below freezing for 90 days per year, and the number of degrees Celsius below freezing is 2 per day, then the number of degree-days is 180 and the area is considered no freeze. This definition is explained in an equation in the Long-Term Pavement Performance Information Management System *Pavement Performance Database User Reference Guide*, FHWA-RD-03-088—see page 34 of the PDF version of the following: http://www.tfhrc.gov/pavement/ltpp/reports/03088/index.htm

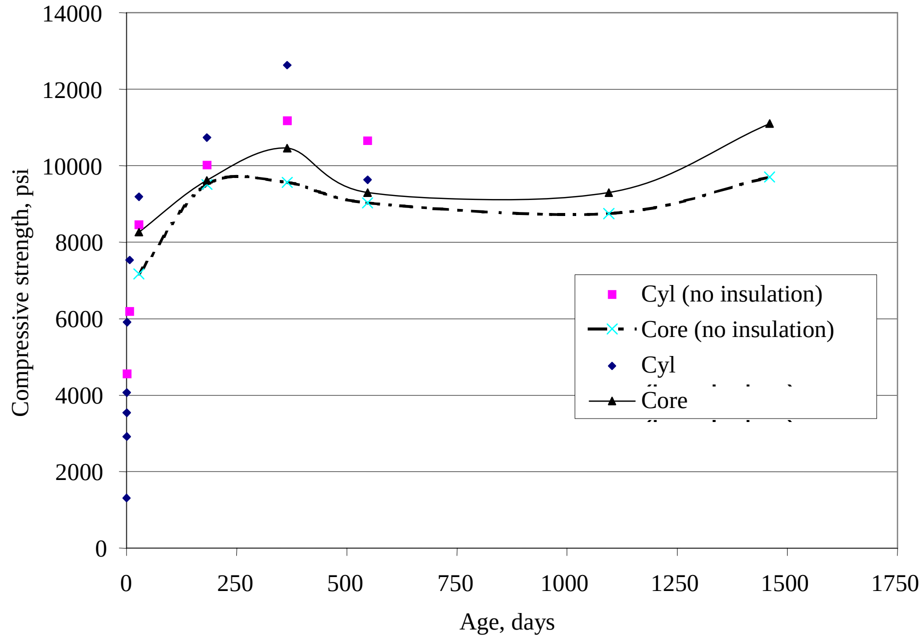


Figure 1. Graph. Plot of compressive strength versus age for test site in Arkansas.

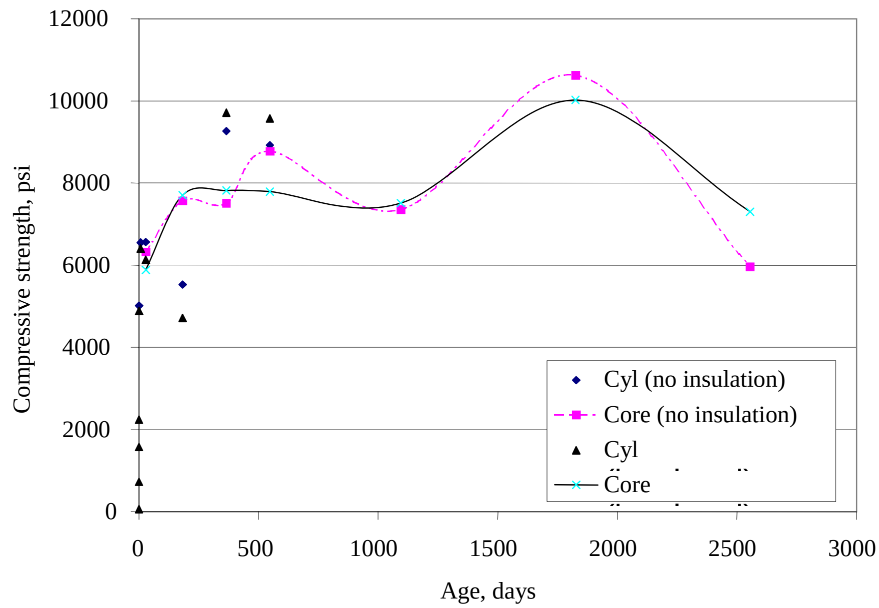


Figure 2. Graph. Plot of compressive strength versus age for test site in Illinois.

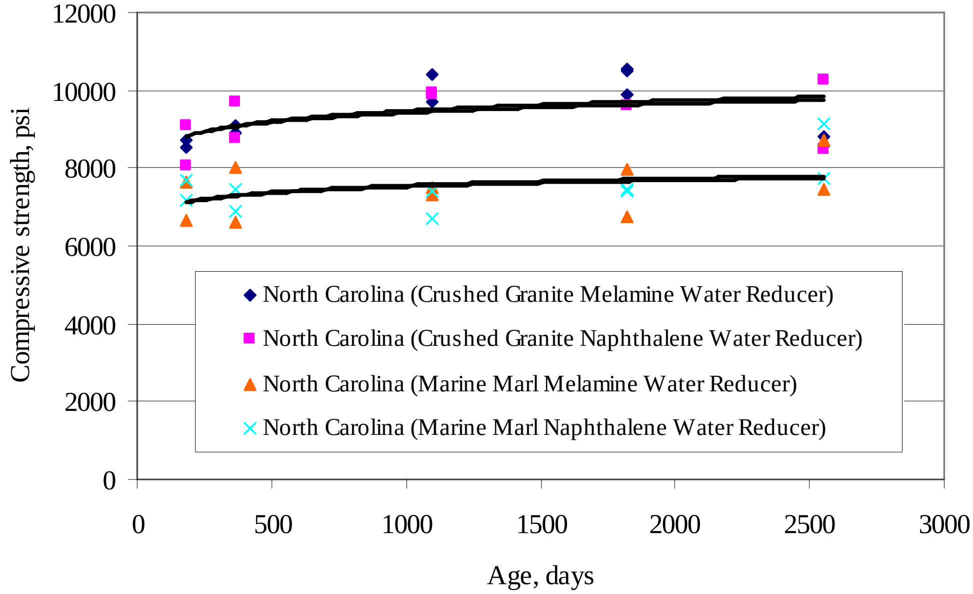


Figure 3. Graph. Plot of strength gain for the different experiments (aggregate type and water reducer type) in North Carolina.

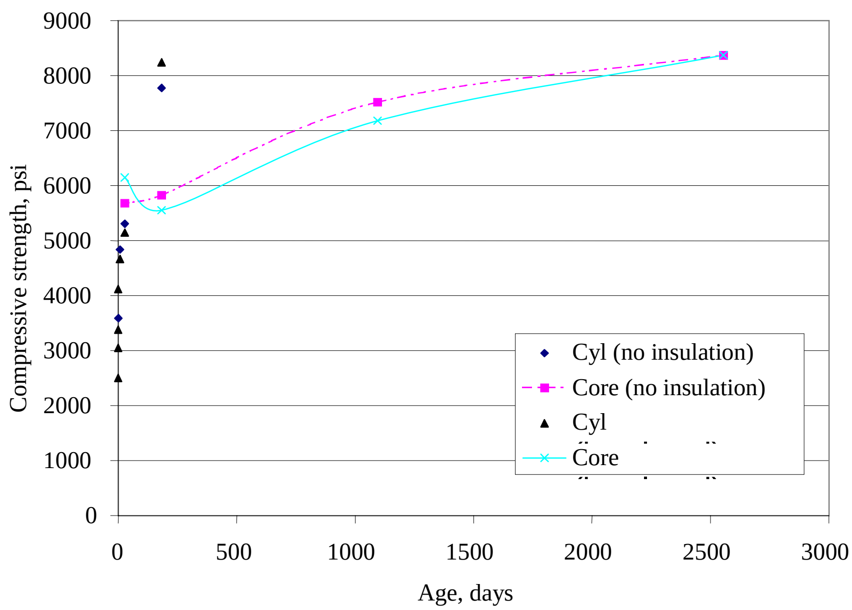


Figure 4. Graph. Plot of compressive strength versus age for test site in Nebraska.

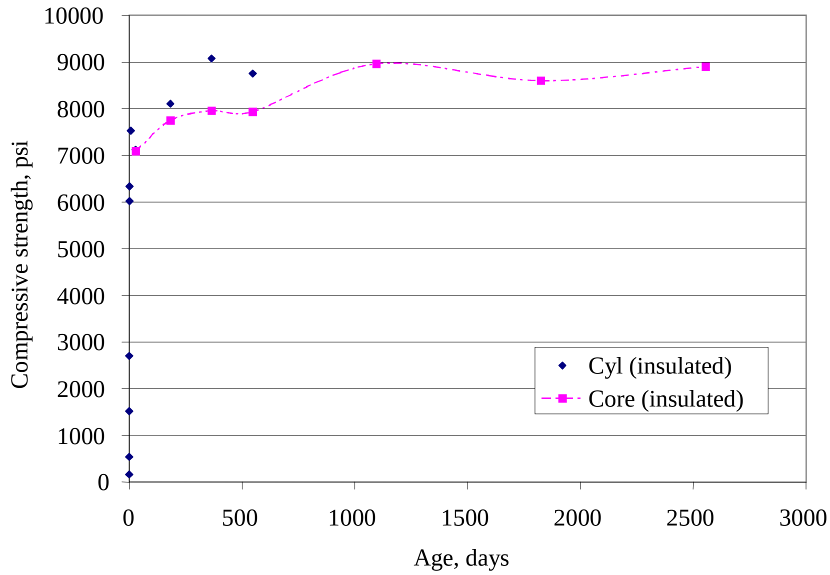


Figure 5. Graph. Plot of compressive strength versus age for test site in New York (data available for insulated sections only).

The plots in figure 3 show that the water reducer type generally has no effect on strength development. For both aggregates types (crushed granite and marine marl), the type of water reducer used (melamine or naphthalene) did not influence the magnitude of compressive strength or the rate of strength gain. However, figure 3 shows that the aggregate type does have an influence on the magnitude of compressive strength achieved (not strength gain). The mixes with crushed granite consistently had 16- to 22-percent higher strength values over the 7-year period evaluated.

As observed from the cores taken from Arkansas and Illinois, medium- to long-term strength development showed significant variations in the post-18-month strength; those from North Carolina, New York, and Nebraska showed relatively little variability and increased with age as expected. In general, the 5- to 7-year strengths for cores were greater than 55.2 MPa (8,000 psi), with the exception of the data from Illinois, which exhibited a significant drop in strength from a high of over 69 MPa (10,000 psi) after 5 years to between 41.4 and 51.7 MPa (6,000 and 7,500 psi) after 7 years. The reason for this drop was not obvious. Table 9 presents a summary of the 1-day to 7-year concrete compressive strength and the 7-year to 28-day mean compressive strength for the combined data (insulated and uninsulated).

For HES concrete, the minimum compressive strength after 24 hours should be 34.5 MPa (5,000 psi). This was achieved for most of the sites examined, with the exceptions of Illinois and Nebraska. The lower strengths in Illinois were caused by problems encountered during concrete placement and construction; those in Nebraska were caused by the use of a low coarse-aggregate content in the mixes to limit potential aggregate-related durability problems.

Table 9 also shows that the rate of strength gain was generally higher for mixes with limestone (10 to 20 percent) than those with crushed granite and marine marl. Nevertheless, the low rate of strength gain could have been caused by many factors, including climate.

Table 9. Summary of strength gain for combined compressive strength test data.

State	**Aggregate Type**	**Mean Core Compressive Strength, psi**			**Percent Strength Gain (28-day to 7-yr)**
		1-day*	**28-day**	**7-yr****	
Arkansas	Limestone	6223	7010	8078	15.22
Illinois	Limestone	4453	5260	6353	20.78
Nebraska	Limestone	3883	4586	5539	20.77
New York	Limestone	5742	6297	7050	11.95
North Carolina	Crushed granite	6831	7385	8136	10.16
	Marine marl	5852	6200	6673	7.62

*1-day compressive strength was backcasted from the longer-term data.
** 7-yr strength for Arkansas was forecasted.
1 kPa = 6.9 psi

The relationship between compressive strength obtained from cores and cylinders was also investigated. Core and cylinder compressive strength data were only available for the test pavement in New York, so the analysis was limited to only this site. The plot of compressive strength from cores and cylinders is presented in figure 6. Figure 6 shows a good relationship between the compressive strength values with a coefficient of determination R^2 value of 49 percent. Even though the magnitudes of compressive strength were close, compressive strength obtained from cores tended to be lower than those from cylinders.

Bivariate Analysis—Static Elastic Modulus

Figure 7 shows the static elastic modulus history for the three aggregate types analyzed. The figure shows that there was a slight decrease in moduli for crushed granite and limestone for the period 5 to 7 years while the moduli for marine marl remained constant. Even though an increase in moduli is typical for the ages evaluated, the data available were limited, and no firm conclusions could be reached.

The relationship between compressive strength and static elastic modulus was also investigated by estimating elastic modulus using the model form[(2)]:

$$E_{PCC} = \alpha f_C^{/0.5} \tag{1}$$

where

E_{PCC} = static modulus of elasticity
α = regression constant (57,000 for conventional concrete)
f_c = compressive strength

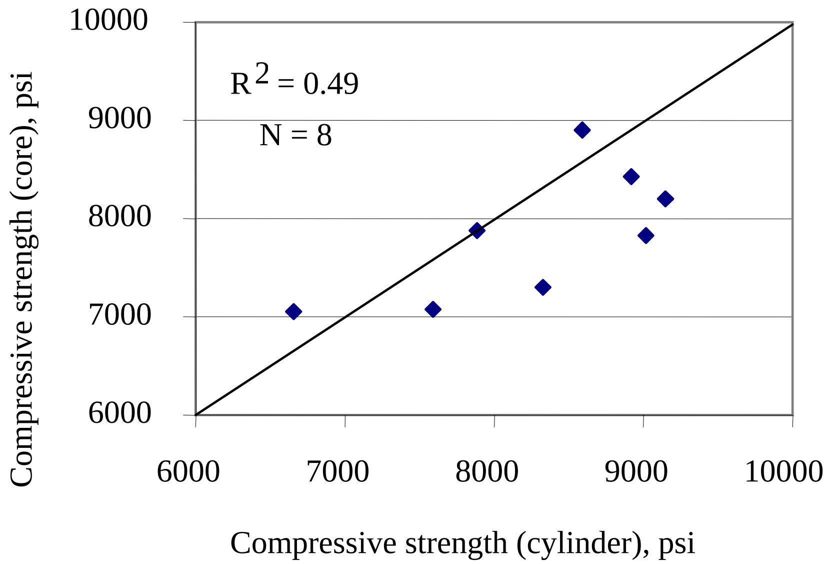


Figure 6. Graph. Plot of cores' compressive strength versus cylinders' compressive strength.

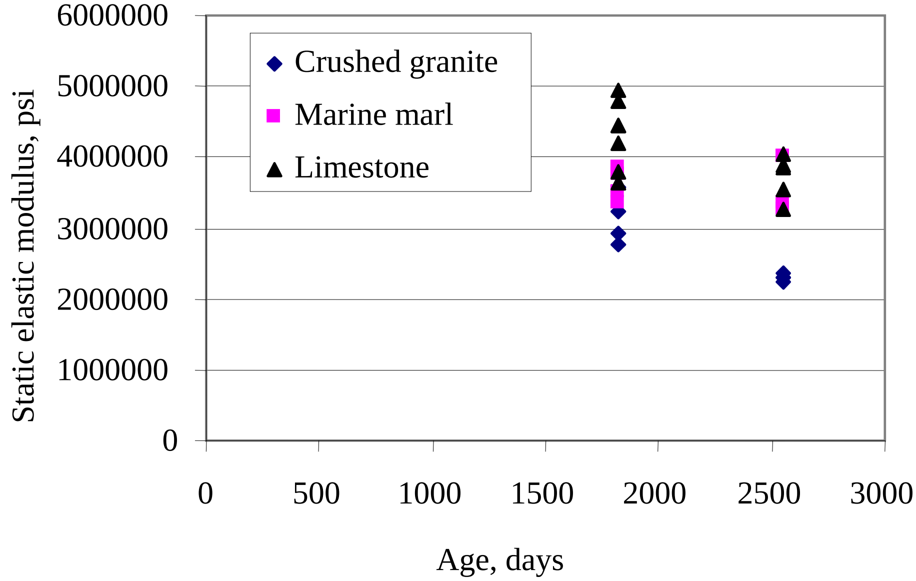


Figure 7. Graph. Plot of static elastic modulus versus age
for the three aggregate types analyzed.

Figures 8 through 10 show plots of the measured and predicted E_{PCC} from compressive strength for crushed granite, marine marl, and limestone. For the aggregate types examined, α ranged from 26,500 to 43,000.(2) This was lower than the typical value of 57,000 used for conventional concrete. The coefficient of determination R^2 ranged from 38 to 67 percent, which was reasonable for field data.

Bivariate Analysis—RCPT and AC Impedance Tests

Rapid Chloride Permeability Testing

RCPT was performed on the core samples to determine chloride permeability and resistivity of the core specimens, respectively. Both RCPT and AC impedance test results provide an indication of the permeability or porosity of the concrete. A less permeable or less porous concrete material is expected to be more durable than one that is permeable or porous because of its susceptibility to moisture penetration. Permeable or porous cores are more likely to deteriorate from the influence of deicing salts as well as from freeze-thaw cycling.

RCPT estimates concrete permeability by estimating the charge that flows through a test specimen over a set period of time. The higher the charge, the more permeable the concrete is. Table 10 presents a summary of the relationship between typical RCPT results (measured as charge in coulombs) and chloride permeability rating.(4) Plots of RCPT test results for the five test sites evaluated are presented in figures 11 through 15. For North Carolina, the test data were divided according to the aggregate type.

In general, the charge passing through the concrete slightly increased with age or remained relatively constant. The only exception to this was for Nebraska. Chloride permeability for the sites evaluated (after 3 to 7 years' placement) was classified using the ratings in table 10 and presented in table 11. Table 11 shows that the chloride permeability rating of the sites ranged from low to moderate. A moderate rating is quite high for a conventionally low water-cement ratio mixture (modified with water reducers). The abnormally high rating may be due to the presence of calcium nitrate (a set accelerating admixture) in the mix and hence does not necessarily mean the concrete mix is porous. In fact, the mix may have a moderate probability for durability-related distress.

As shown in table 11, even though the absolute values of charge varied slightly with time, the chloride permeability rating remained relatively constant. With the exception of North Carolina, the chloride permeability rating of the test sites remained constant. This is in agreement with the observed visual distress that indicates little to no durability-related problems 7 years after concrete placement.

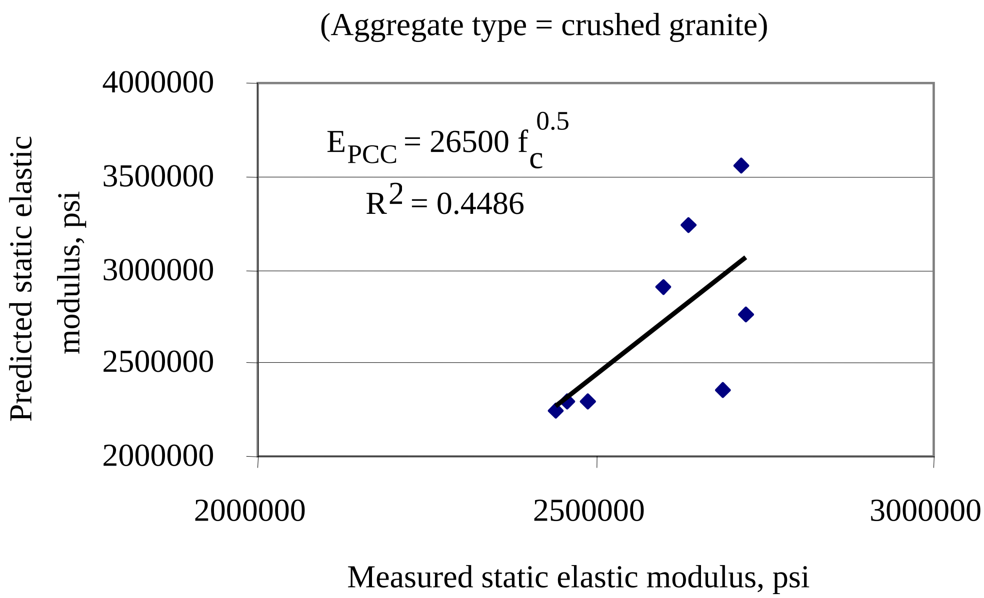


Figure 8. Graph. Plot of the measured and predicted E_{PCC} from compressive strength for crushed granite.

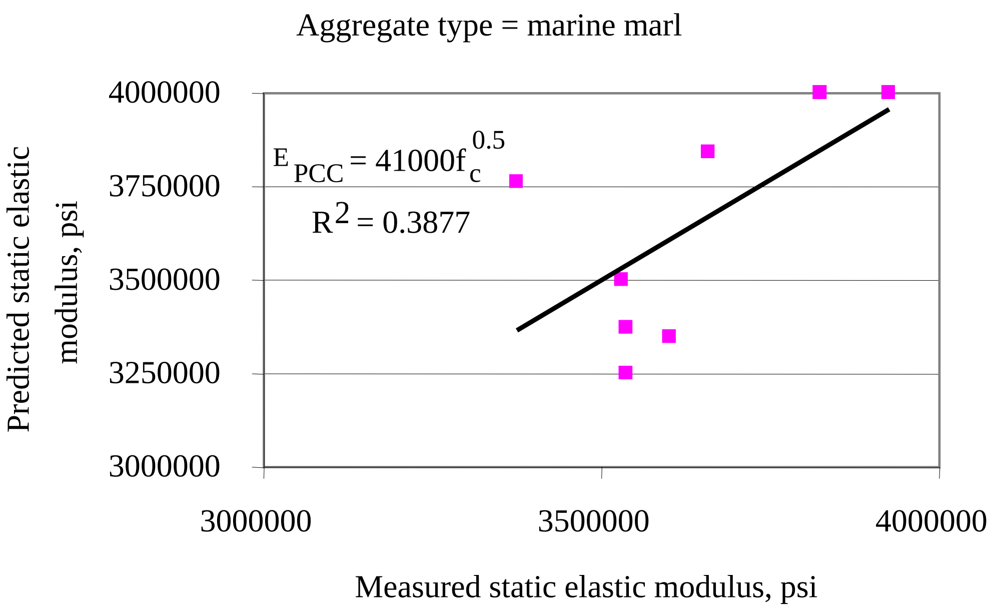


Figure 9. Graph. Plot of the measured and predicted E_{PCC} from compressive strength for marine marl.

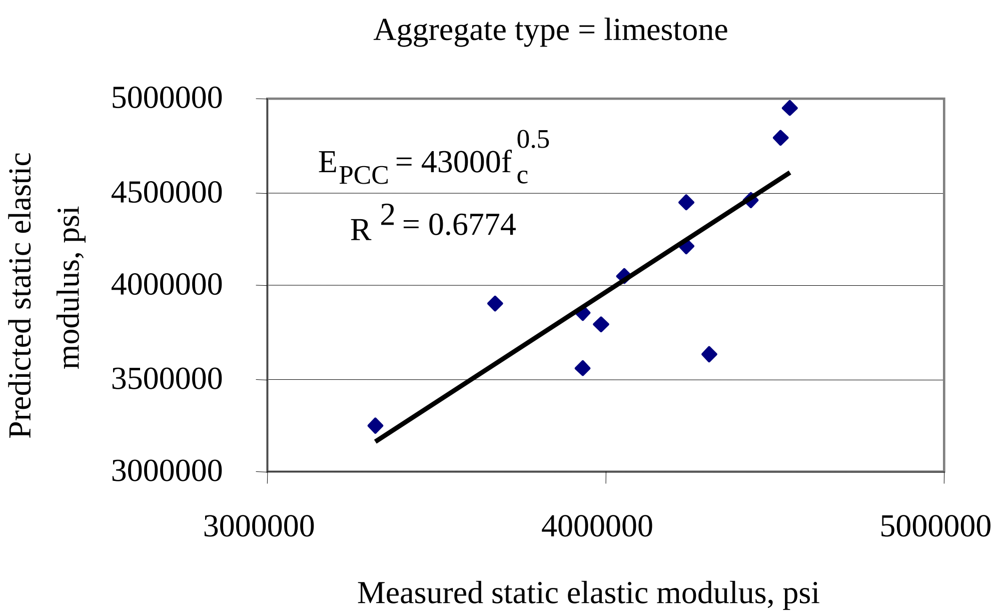


Figure 10. Graph. Plot of the measured and predicted E_{PCC} from compressive strength for limestone.

Table 10. RCPT Chloride permeability ratings.

RCPT Results (coulombs)	**Chloride Permeability**
> 4000	High
2000 to 4000	Moderate
1000 to 2000	Low
100 to 1000	Very low
< 100	Negligible

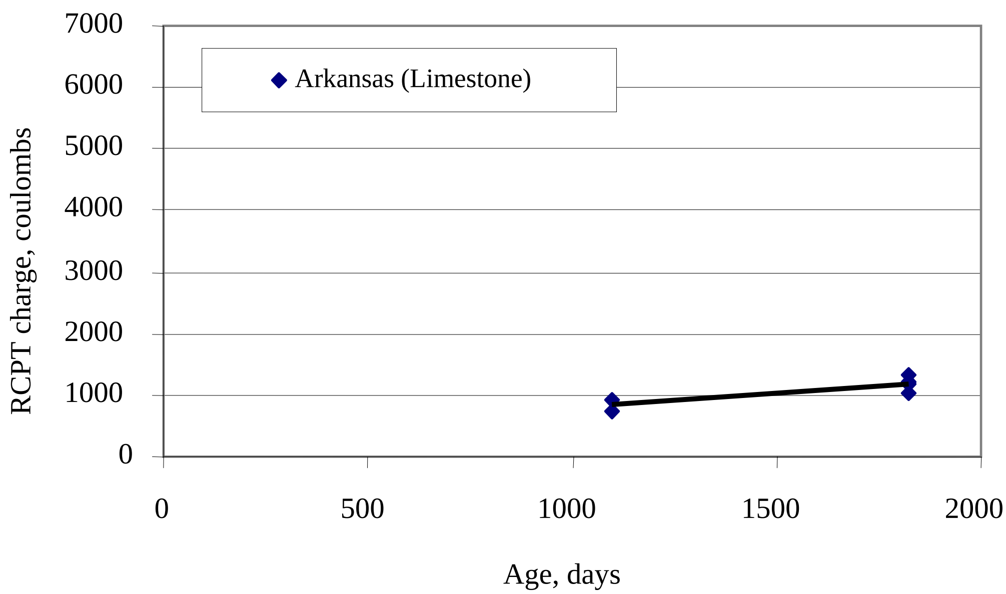


Figure 11. Graph. Plot of charge passed versus concrete age for Arkansas.

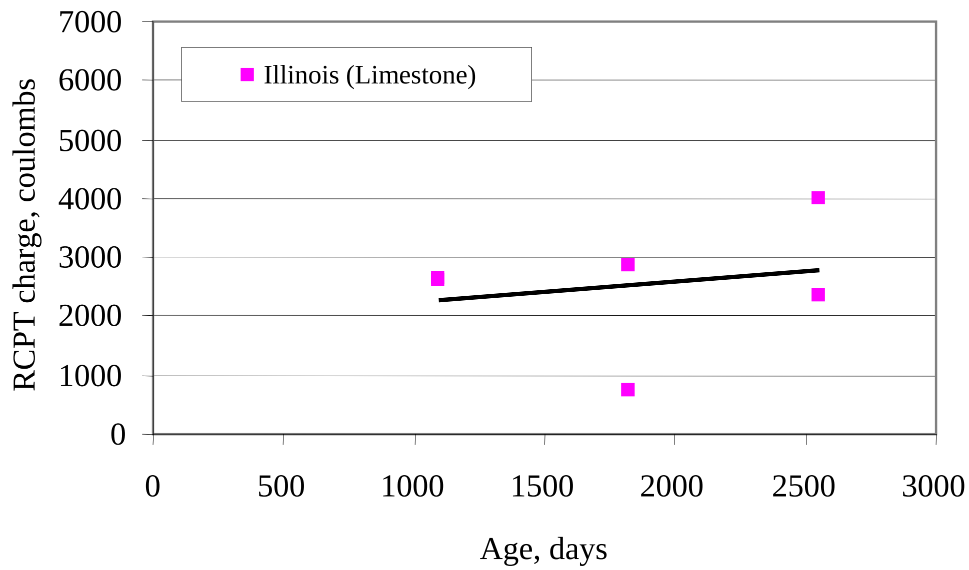


Figure 12. Graph. Plot of charge passed versus concrete age for Illinois.

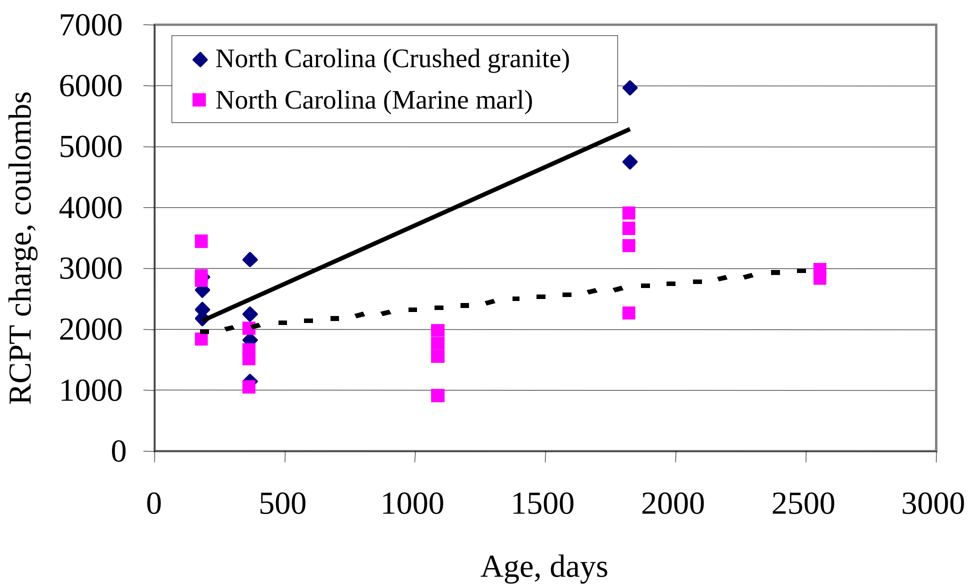


Figure 13. Graph. Plot of charge passed versus concrete age for North Carolina.

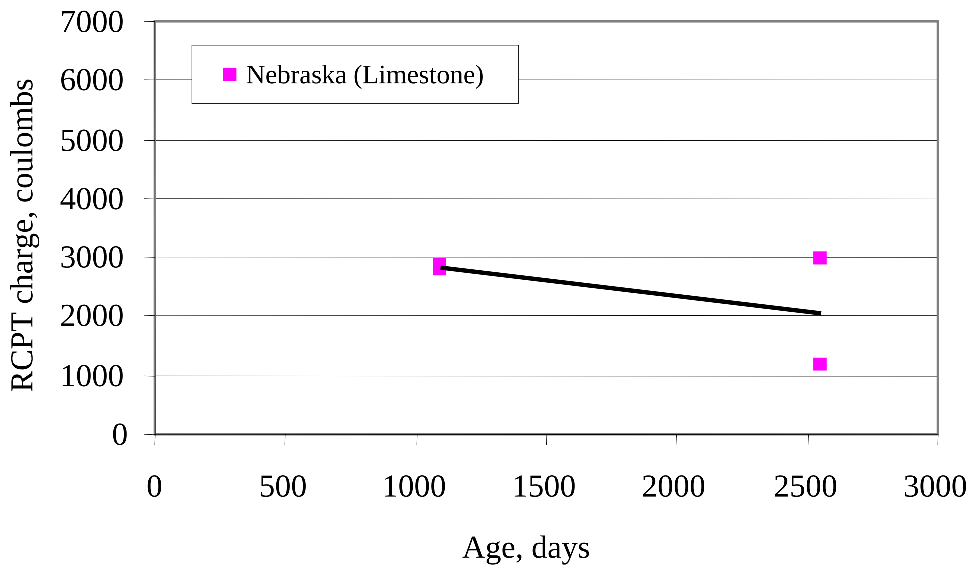


Figure 14. Graph. Plot of charge passed versus concrete age for Nebraska.

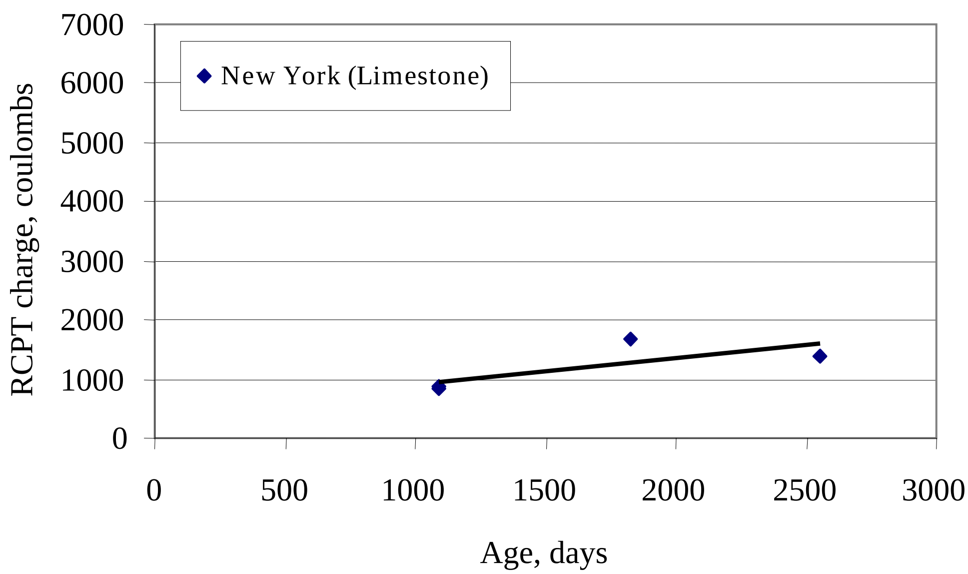


Figure 15. Graph. Plot of charge passed versus concrete age for New York.

Table 11. Summary of chloride permeability for test sites evaluated.

State	Aggregate Type	Chloride Permeability				
		0.5 year	1 year	3 year	5 year	7 year
Arkansas	Limestone	—	—	Low	Low	—
Illinois	Limestone	—	—	Moderate	Moderate	Moderate
North Carolina	Crushed granite	Moderate	Moderate	—	Moderate	—
North Carolina	Marine marl	Moderate	Low	Low	Moderate	Moderate
Nebraska	Limestone	—	—	Moderate		Moderate
New York	Limestone	—	—	Moderate	Moderate	Moderate

Figures 16 through 18 present histograms showing comparisons of RCPT results measured from the top and bottom portions of the core specimens. Evaluating both the top and bottom portions of the slab separately was very important since durability-related distress such as D-cracking could be initiated from either end of the concrete slab. The information presented showed no obvious trend in the RCPT results for the top or bottom of the slab. This finding agreed with observation that showed obvious distress through the core specimens inspected. Because of the importance of the investigation to determine if there are significant differences in the top and bottom RCPT results, ANOVA was performed using the test data. The results are presented later in this chapter.

AC Impedance

The final durability indicator variable to be analyzed was the AC impedance, Z. As mentioned earlier, the impedance test was investigated as a viable alternative to RCPT. The test has a potential advantage over RCPT because it does not induce ionic diffusion within the specimen and, therefore, eliminates the effects of concentration gradients within the concrete.

The AC impedance test was performed at two frequencies—100 and 1,000 Hz. However, because of inconsistencies in the testing procedure for the 1,000-Hz test, no reliable time series data were available. Therefore, the bivariate analysis was limited only to the 100-Hz AC impedance test data conducted using the Nilsson soil resistance meter in two-pin mode.

In general, higher measures of impedance within a concrete material indicate a less porous and more durable material able to withstand the effects of wet-dry, freeze-thaw, and other adverse climatic cycles. Figures 19 through 23 present plots of measured impedance versus age for the concrete samples evaluated from the different test sites. They show that impedance measured at 100 Hz generally remained constant or decreased slightly with age. This observation was similar to the RCPT results, where no obvious trend with time was determined. The observation also agrees with the visual distress data that showed no obvious deterioration in the concrete due to ASR or D-cracking.

Because none of the sites evaluated showed any significant amounts of visual durability-related distress, it would be premature to draw conclusions about whether the 100-Hz test result could accurately indicate the presence of durability-related distress. Even though Illinois and possibly other sites use deicing salts to free the pavements of ice during winter, the test results showed no adverse effect (such as increased variability).

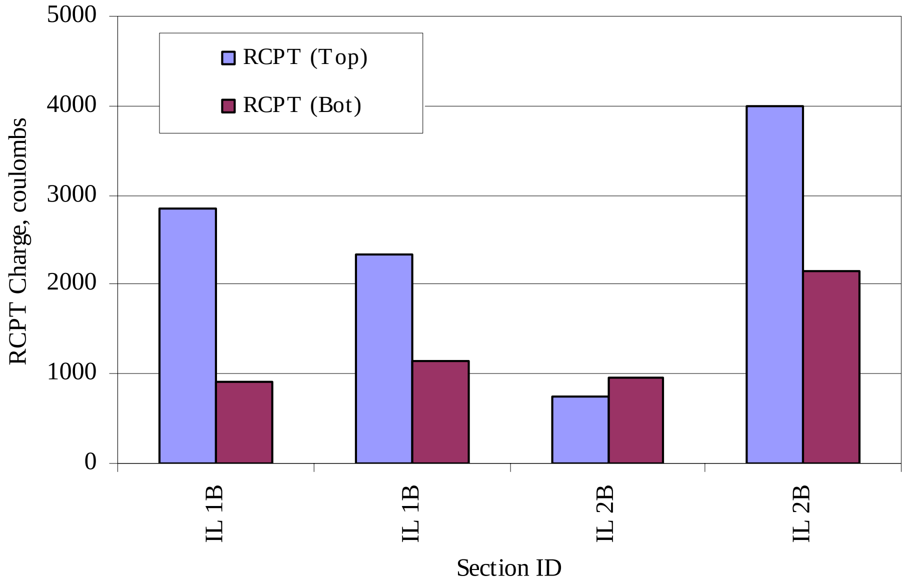


Figure 16. Bar chart. Histogram showing RCPT results from the top and bottom sections of concrete cores (Illinois).

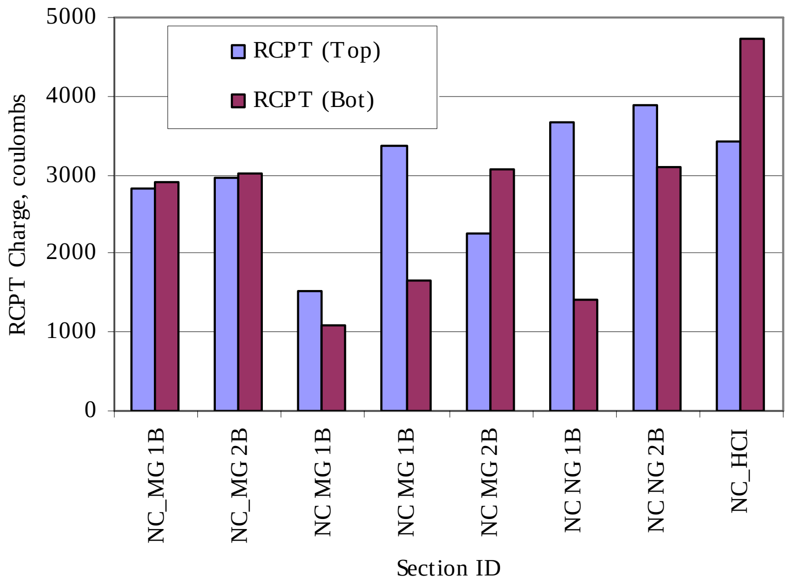


Figure 17. Bar chart. Histogram showing RCPT results from the top and bottom sections of concrete cores (North Carolina).

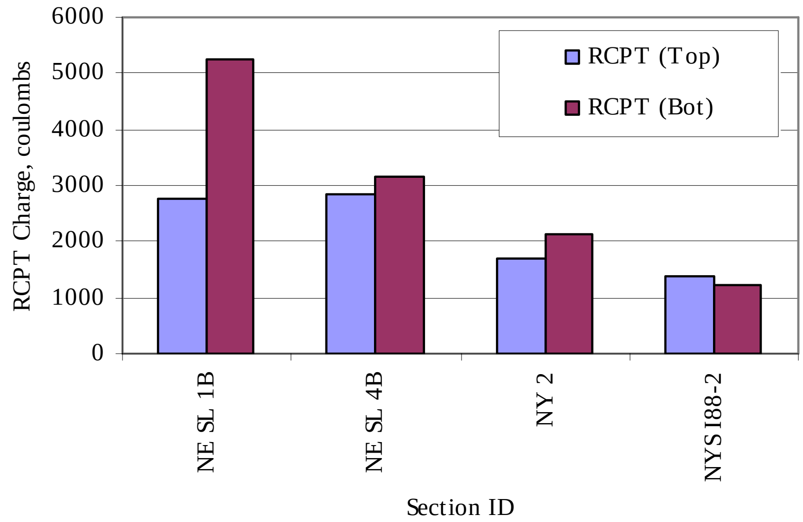


Figure 18. Bar chart. Histogram showing RCPT results from the top and bottom sections of concrete cores (Nebraska and New York).

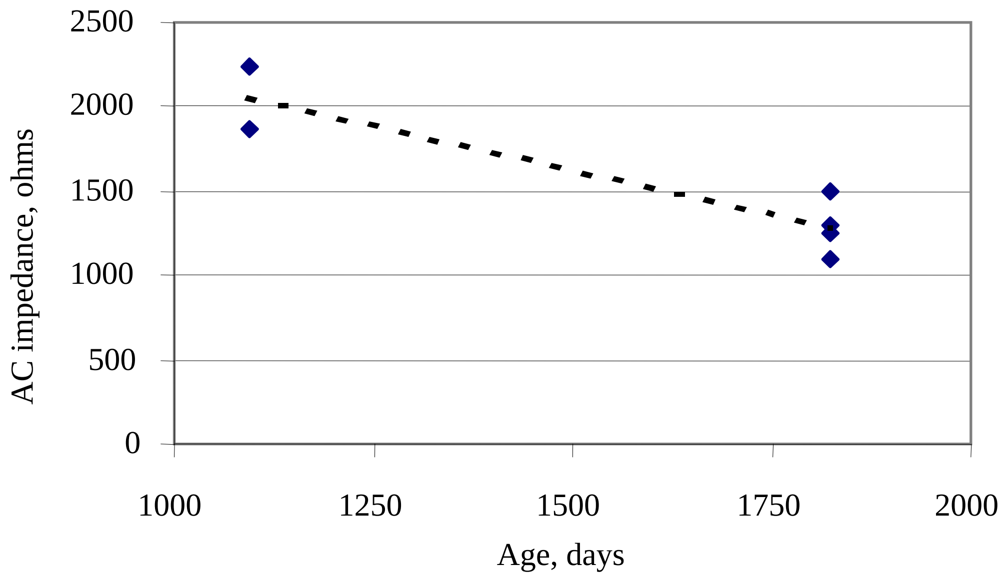


Figure 19. Graph. Plot of AC impedance versus age for concrete cores (Arkansas, limestone).

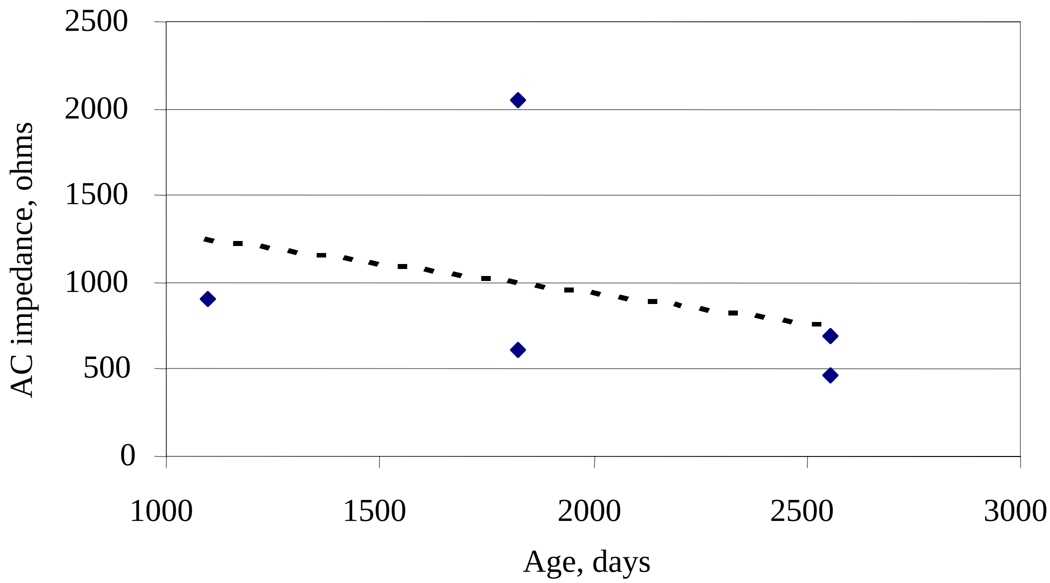


Figure 20. Graph. Plot of AC impedance versus age
for concrete cores (Illinois, limestone).

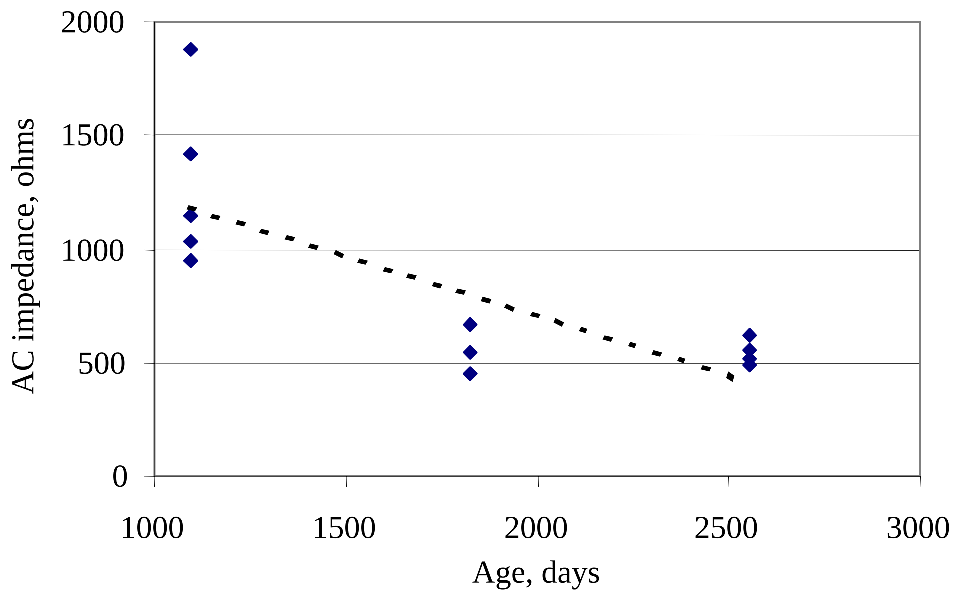


Figure 21. Graph. Plot of AC impedance versus age
for concrete cores (North Carolina, granite).

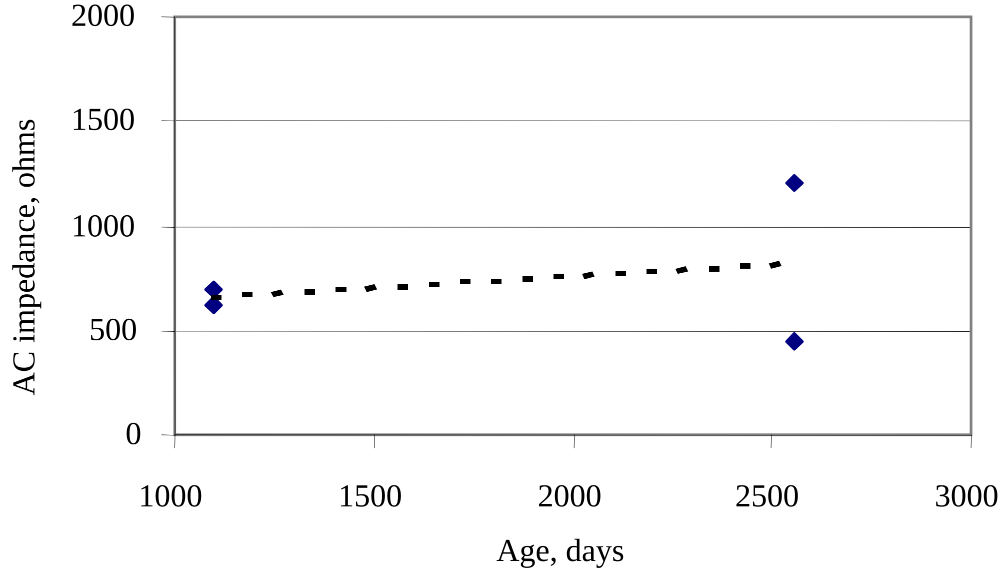


Figure 22. Graph. Plot of AC impedance versus age
for concrete cores (Nebraska, limestone).

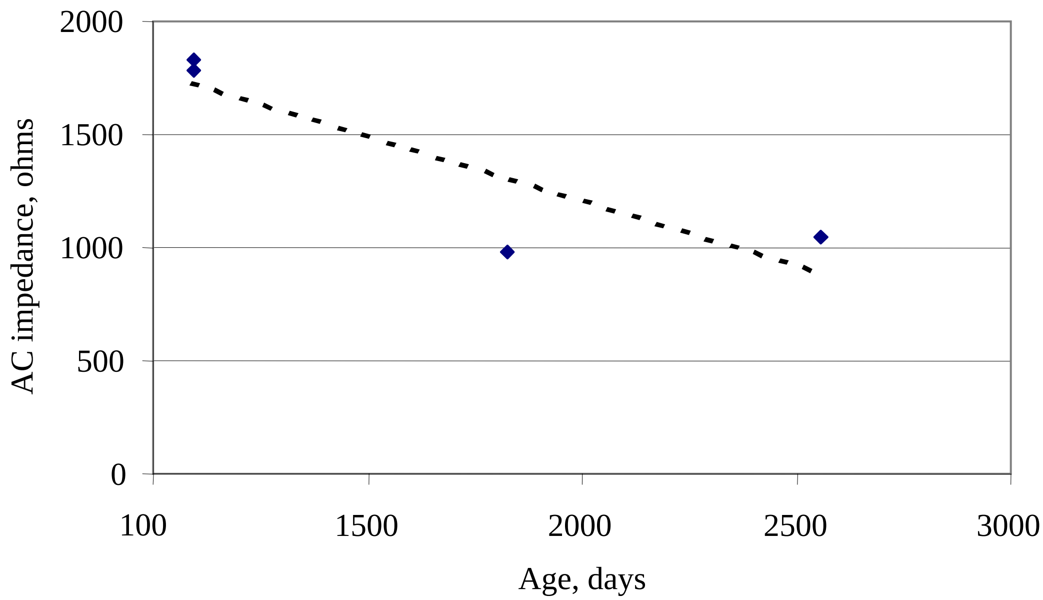


Figure 23. Graph. Plot of AC impedance versus age
for concrete cores (New York, limestone).

Figure 24 shows the plot of RCPT versus AC impedance for the 100-Hz test. The plot shows clearly that there is a nonlinear exponential relationship between the 100-Hz AC impedance test and the RCPT. A model developed for relating RCPT results to the 100-Hz AC impedance test results was as follows:

$$ACIMPH = 4953.8(e^{-0.001RCPT}) \quad (2)$$

Statistics:

N = 32; R^2 = 92.5 percent

where

ACIMPH = AC impedance measures at 100 Hz, ohms
ACIMPT = AC impedance measures at 1000 Hz, ohms
RCPT = RCPT results, coulombs

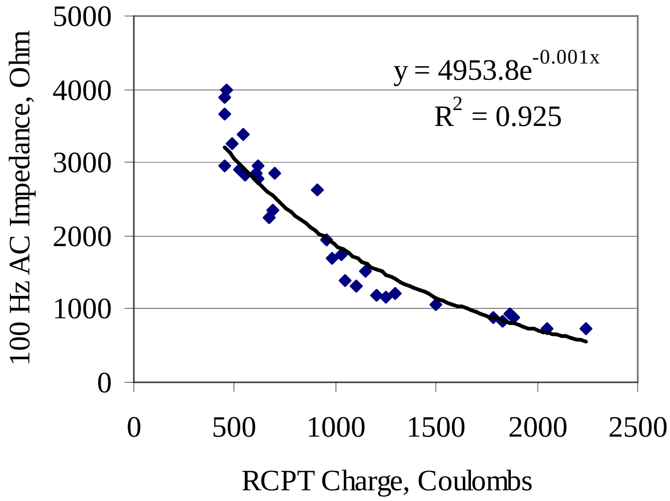


Figure 24. Graph. RCPT versus AC impedance for the 100-Hz test.

Using equation 2 and the ratings presented in table 10, the ratings presented in table 12 were developed for the 100-Hz AC impedance test. The presence of a functional correlation between the AC impedance measurements and the RCPT values is definitely encouraging. However, since the model shown in equation 2 was based on limited data, any attempt to use it beyond the scope of this work is not encouraged until further research is done.

Table 12. RCPT and 100-Hz AC impedance test ratings.

RCPT Results (coulombs)	**100-Hz AC Impedance (ohms)**	**Chloride Permeability**
> 4000	< 90	High
2000 to 4000	90 to 670	Moderate
1000 to 2000	670 to 1800	Low
100 to 1000	1800 to 4480	Very low
< 100	> 4480	Negligible

Analysis of Variance

The bivariate analysis was followed by a comprehensive ANOVA to determine their statistical significance. The hypothesis tested:

- Influence of early-age curing on core compressive strength and RCPT results.
- Influence of aggregate type on core compressive strength and RCPT results.
- Whether a significant difference existed between top and bottom RCPT results.

The results of ANOVA are summarized and discussed in the following sections.

Influence of Early-Age Curing on Core Compressive Strength and RCPT Results

For this analysis, the data were grouped according to location/climate (five treatments) and curing method (two treatments). The durability indicator variables used were core compressive strength and RCPT test result (charge in coulombs). All the data from the different age groups were grouped together for this analysis to ensure that a reasonable sample size for analysis was obtained. Tables 13 and 14 present summaries of ANOVA results (Duncan's multiple-range test of comparison of mean) for the effect of curing method on the durability indicator variable compressive strength and chloride permeability.

The information presented in tables 13 and 14 confirms the observed trends (see figures 1 through 5) that the curing method used had no significant impact on the long-term strength development or chloride permeability of the concrete. However, this observation does not imply that insulated curing is not useful for HES concrete. It is definitely useful for VES gain, as reported in the original SHRP study.[1]

Table 13. Summary of ANOVA results for the effect of curing method on strength.

Location	Variable	Class	Mean Value	Duncan Class*	Comments
Arkansas	Compressive strength, psi	None	8932	A	No significant difference
		Plastic sheeting (insulated)	9628	A	
Illinois	Compressive strength, psi	None	7443	A	No significant difference
		Plastic sheeting (insulated)	7598	A	
North Carolina	Compressive strength, psi	None	8223	A	No significant difference
		Plastic sheeting (insulated)	8519	A	
Nebraska	Compressive strength, psi	None	6330	A	No significant difference
		Plastic sheeting (insulated)	6542	A	
New York	Compressive strength, psi	None	—	—	Data available only for insulated sections
		Plastic sheeting (insulated)	—	—	

* Duncan's multiple range test; those with the same letter are not significantly different.

Table 14. Summary of analysis of variance results for the effect of curing method on concrete permeability.

Location	Variable	Class	Mean Value	Duncan Class*	Comments
Arkansas	Charge, coulombs	None	983	A	No significant difference
		Plastic sheeting (insulated)	1153	A	
Illinois	Charge, coulombs	None	2457	A	No significant difference
		Plastic sheeting (insulated0	2607	A	
North Carolina	Charge, coulombs	None	2509	A	No significant difference
		Plastic sheeting (insulated)	2652	A	
Nebraska	Charge, coulombs	None	2271	A	No significant difference
		Plastic sheeting (insulated)	2957	A	
New York	Charge, coulombs	None	—	—	Data available only for insulated sections
		Plastic sheeting (insulated)	—	—	

* Duncan's multiple-range test; those with the same letter are not significantly different.

Influence of Aggregate Type on Core Compressive Strength and RCPT Results

For this analysis, the data were grouped according to aggregate type. The strength indicator used was compressive strength of cores; the durability indicator used was the RCPT result (charge in coulombs), which is an indicator of concrete permeability. All the data from the different age groups were grouped together for this analysis. Tables 15 and 16 present summaries of the ANOVA (Duncan's multiple-range test of comparison of means).

Table 15. Summary of analysis of variance results for the effect of aggregate type on strength.

Variable	Class	Mean Value	Duncan Class*	Comments
Compressive strength, psi	Crushed granite	9372	A	Significant difference between crushed granite and others
	Marine marl	7486	B	
	Limestone	7877	B	

* Duncan's multiple-range test; those with the same letter are not significantly different.

Table 16. Summary of analysis of variance results for the effect of aggregate type on concrete permeability.

Variable	Class	Mean Value	Duncan Class*	Comments
Charge, coulombs	Crushed granite	2414	A	No significant difference
	Marine marl	2300	A	
	Limestone	1808	A	

* Duncan's multiple-range test; those with the same letter are not significantly different.

The information presented in tables 15 and 16 confirms the observed trends—the aggregate type has a significant impact on the long-term strength development, with the concrete mix made with crushed granite having the highest strength values. There was, however, no difference in the strengths of concrete made from marine marl or limestone. Aggregate type had no significant effect on concrete chloride permeability, as shown in table 16.

Evaluation of Top and Bottom RCPT Results

The RCPT results from measurements made on the top and bottom portions of the same core specimen were evaluated to determine if there was a significant difference in their values. Table 17 presents summaries of ANOVA results (test of significance (F-test) and Duncan's multiple range test of comparison of mean).

The information presented in table 17 confirms the observed trends seen in figures 16 through 18—there was no significant difference in chloride permeability for the top and bottom portions of the core specimens. This was also in agreement with the observed visual distress survey that showed no significant distress throughout the entire core specimen evaluated.

Table 17. Summary of analysis of variance results for evaluating specimen RCPT results.

Variable	Class	Mean Value	Duncan Class*	Comments
Charge, coulombs	Top of specimen	2944	A	No significant difference
	Bottom of specimen	2376	A	

* Duncan's multiple-range test; those with the same letter are not significantly different.

CHAPTER 4. CONCLUSIONS AND RECOMMENDATIONS

CONCLUSIONS

This study was intended to investigate the durability and integrity of HES concrete patches built in Arkansas, Illinois, Nebraska, New York, and North Carolina. Strength, modulus, resistance to chloride ion penetration, and visual condition of the patches were used as durability indicators. The results and recommendations presented in this report are based on a comprehensive study of the test data using statistical methods and engineering principles. Chapter 3 discussed in detail the effects of climate and material properties on the HES concrete durability. The following key observations were made from the data analysis:

- In most cases, the concrete continued to gain strength and modulus with time over the entire performance monitoring period.
- The resistance to chloride ion penetration increased only slightly with time in some instances and was relatively constant in other cases.
- Water reducer type had no significant effect on long-term durability.
- Curing method (insulated versus uninsulated) also had no significant effect on long-term compressive strength or the other durability indicators.
- Aggregate type used in the concrete mix had a significant effect on the magnitude of compressive strength but no effect on strength gain, chloride permeability, or AC impedance.
- Concrete mixtures made with crushed granite had the highest compressive strengths, typically 16 to 22 percent greater than that for limestone and marine marl.
- The RCPT test results had a good correlation (90 percent) with the 100-Hz AC impedance test results.
- There were insufficient data to determine the reliability of all three durability indicator variables.

RECOMMENDATIONS

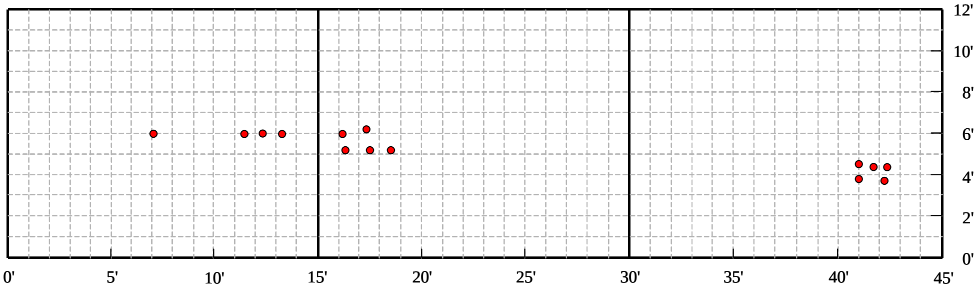
The study was successful to a large extent in that it demonstrated that HES concrete patches, when built using a variety of methods and materials and exposed to different climatic conditions, can perform adequately in the field with no extraordinary signs of durability-related distresses. However, since the study only spanned approximately 7 years into the life of the patches, additional data will be necessary to study in greater detail the effects of climate, construction, and site conditions that influence HES concrete durability.

It is therefore recommended that more data be collected as the patches age, and that these data be analyzed to make further observations of the long-term durability of these materials.

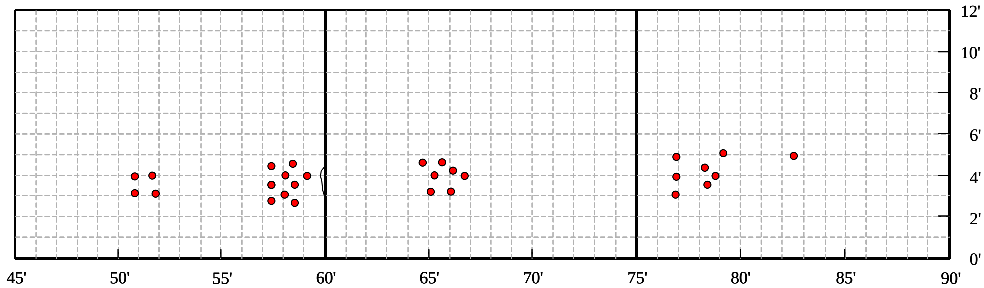
APPENDIX A. DISTRESS SURVEY MAPS

Distress survey maps representing the condition of the various patches are presented in figures 25 through 40 in this appendix. The circular dots on the maps indicate core locations. The patches in Arkansas, New York, and North Carolina did not show significant deterioration over time. Therefore, for these sections, distress maps are provided only for the years 1994 and 1998. On the other hand, sections in Illinois and Nebraska showed more pronounced deterioration over time. For these sites, distress maps are provided for all the years in which field surveys were conducted.

Section ID: NC 1
Inspection Date: 11/22/94



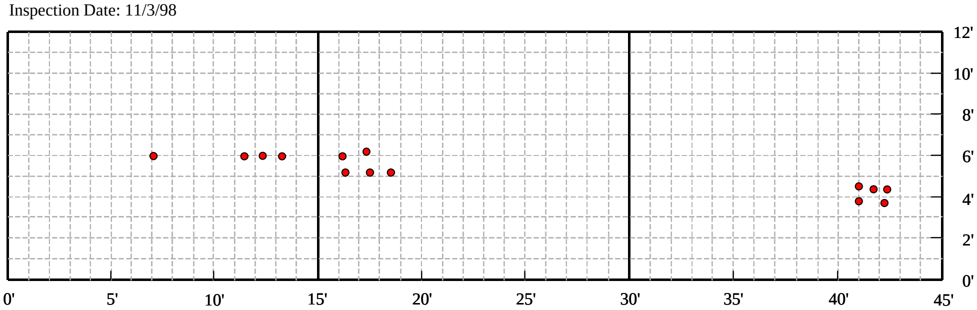
Comments: No signs of D-cracking or scaling.



Comments:

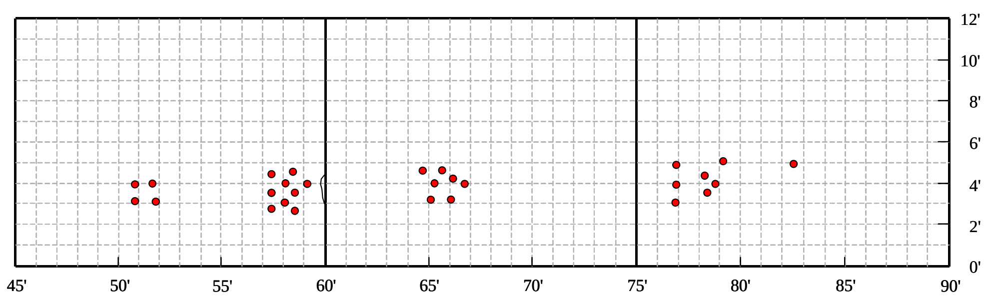
Figure 25. Graph. Distress map for patch NC-1 in North Carolina—1994.

Section ID: NC 1
Inspection Date: 11/3/98



Comments: No signs of D-cracking or scaling.

No change in condition (1995; 1996; 1997; 1998).



Comments:

Figure 26. Graph. Distress map for patch NC-1 in North Carolina—1998.

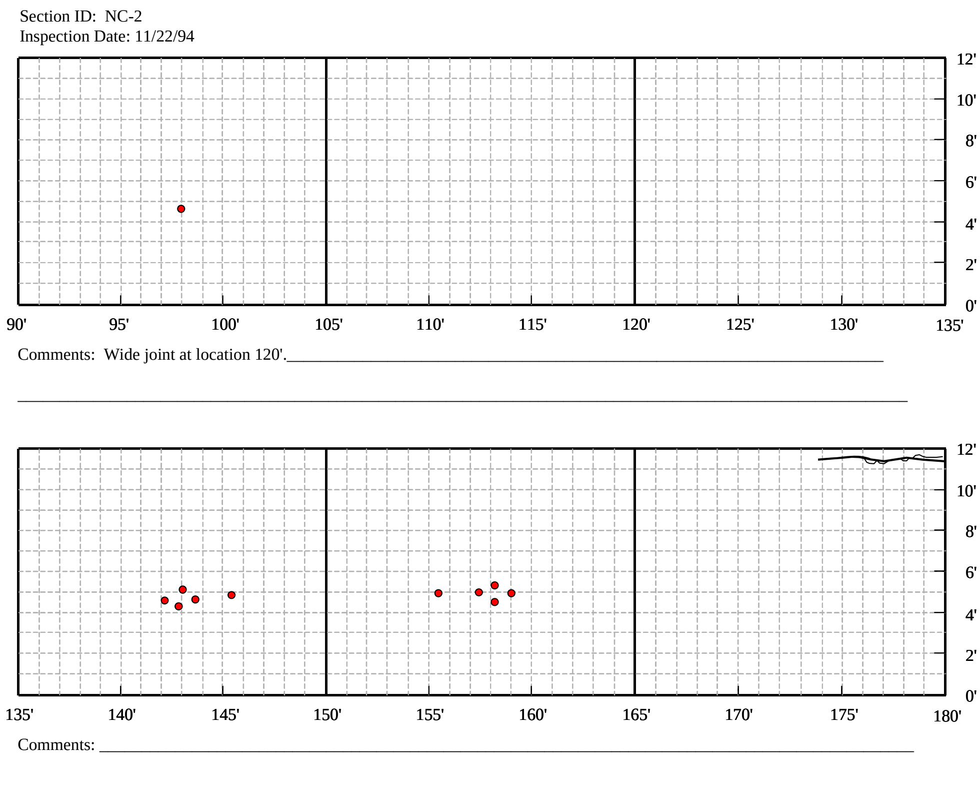


Figure 27. Graph. Distress map for patch NC-2 in North Carolina—1994.

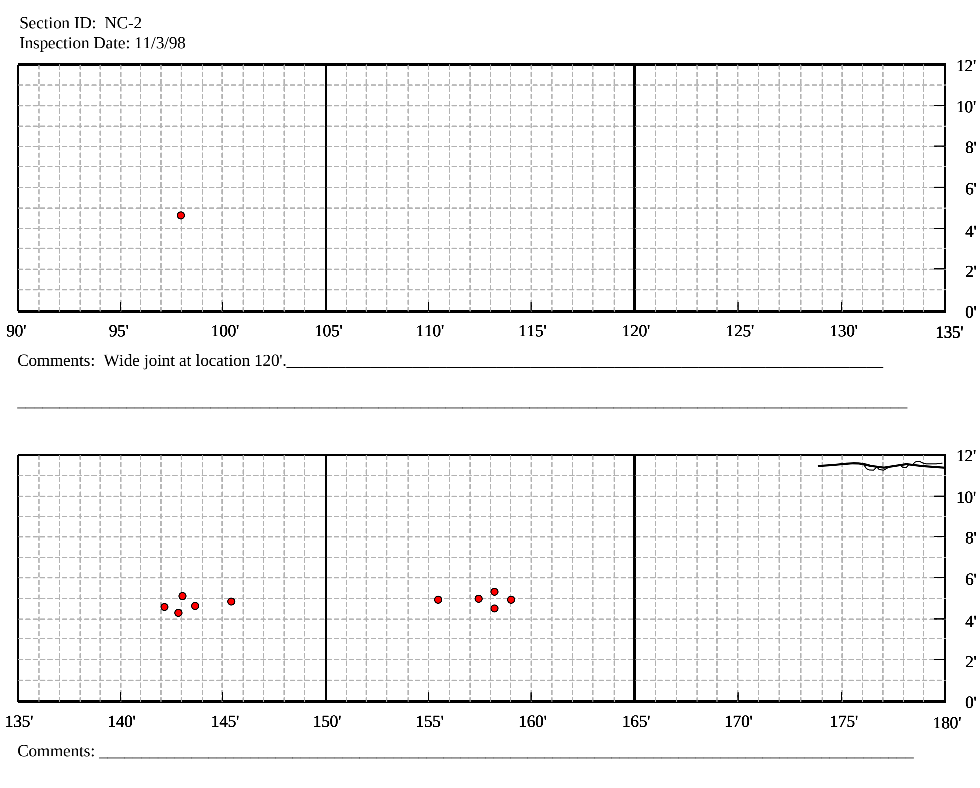
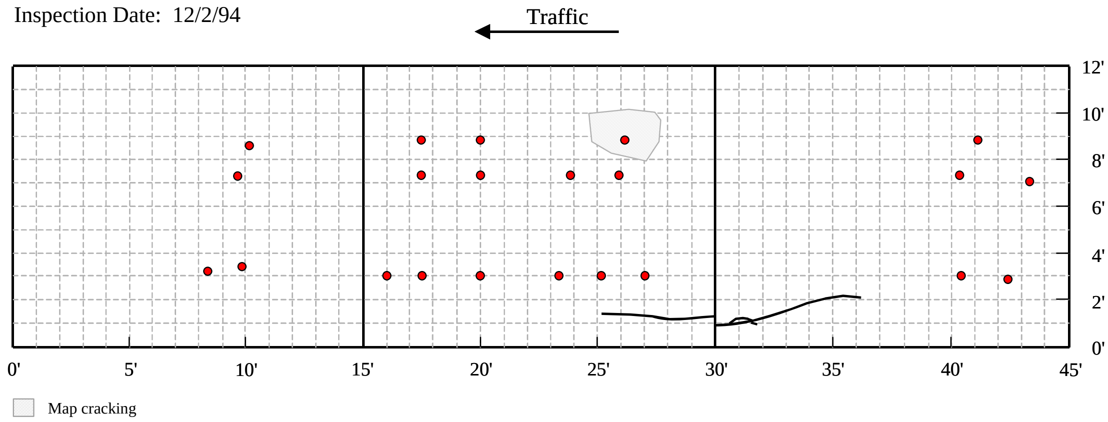


Figure 28. Graph. Distress map for patch NC-2 in North Carolina—1998.

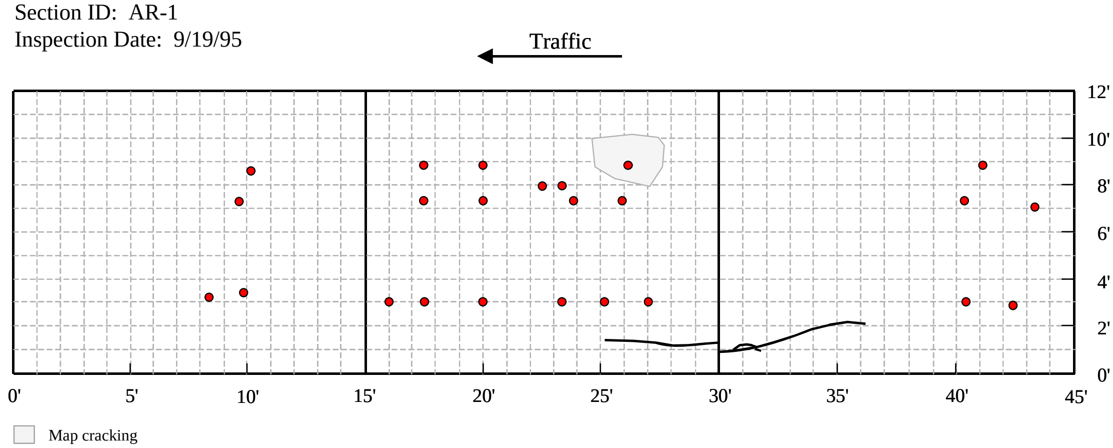
Section ID: AR-1
Inspection Date: 12/2/94



Comments: All joints are well sealed with HP sealant. No signs of scaling or D-cracking (12/2/94)

Figure 29. Graph. Distress map for patch AR-1 in Arkansas—1994.

Section ID: AR-1
Inspection Date: 9/19/95



Comments: All joints are well sealed with HP sealant. No signs of scaling or D-cracking (12/2/94)

Some surface crazing was noted on slab 2 (1995). Joint seals remain in good condition.

Section overlaid after 1995 inspection

Figure 30. Graph. Distress map for patch AR-1 in Arkansas—1995.

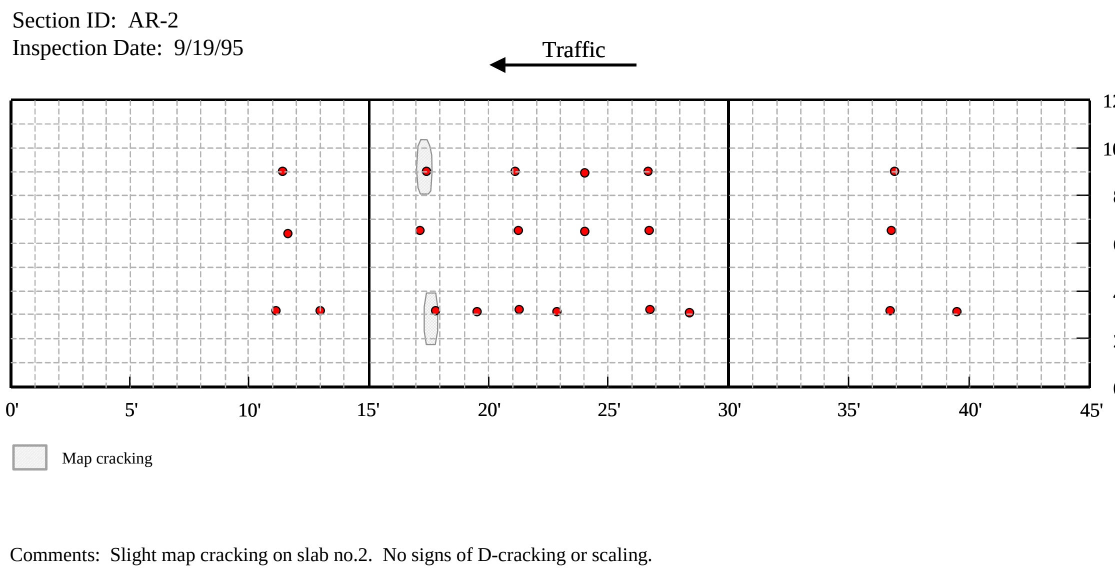


Figure 31. Graph. Distress map for patch AR-2 in Arkansas—1994.

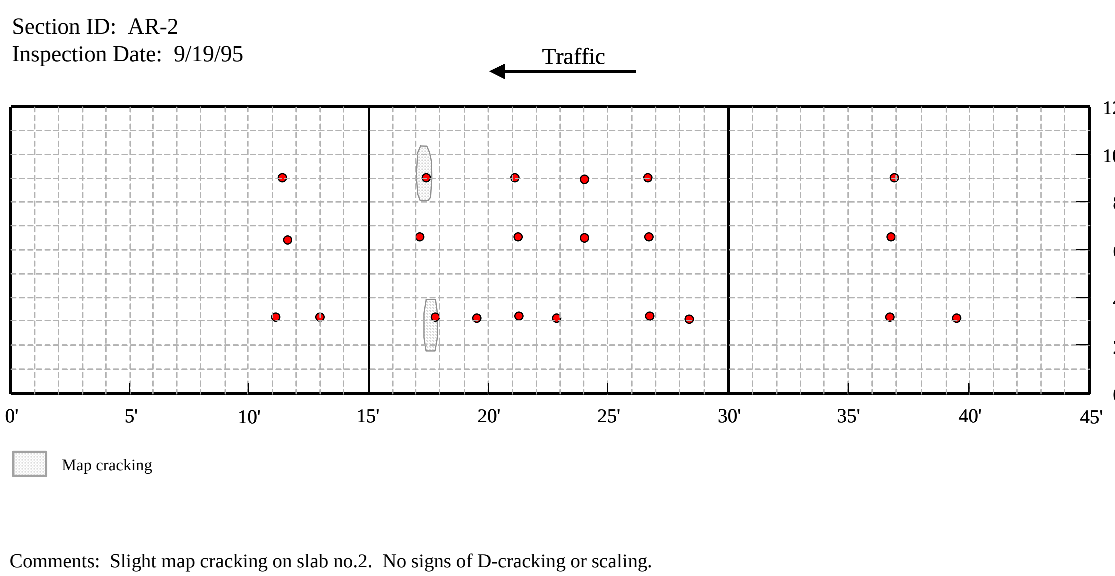


Figure 32. Graph. Distress map for patch AR-2 in Arkansas—1995.

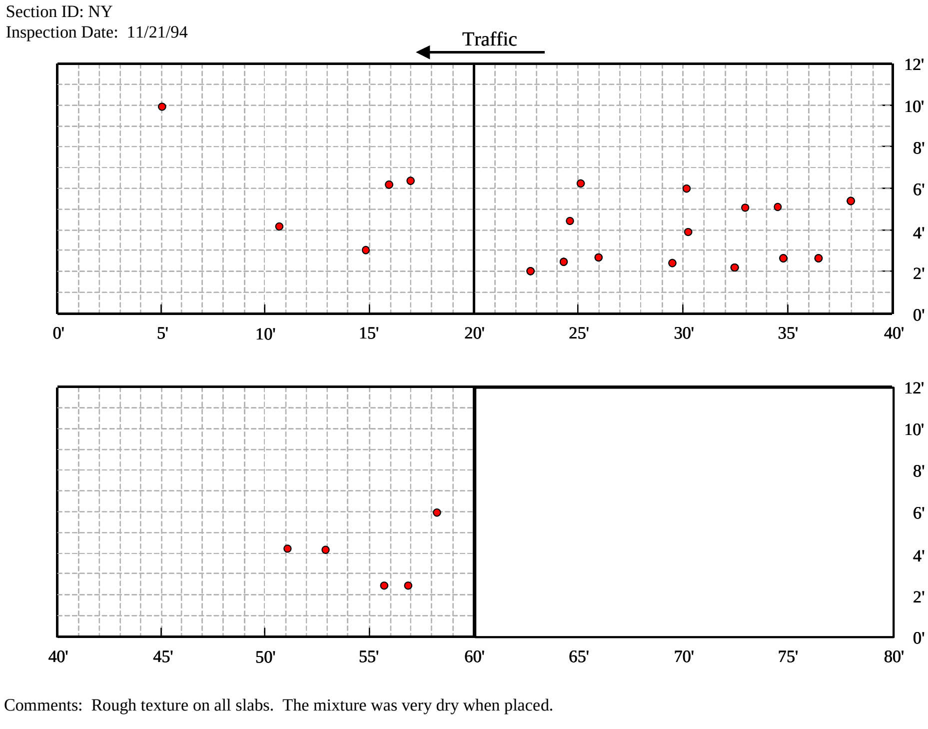


Figure 33. Graph. Distress map of section in New York—1994.

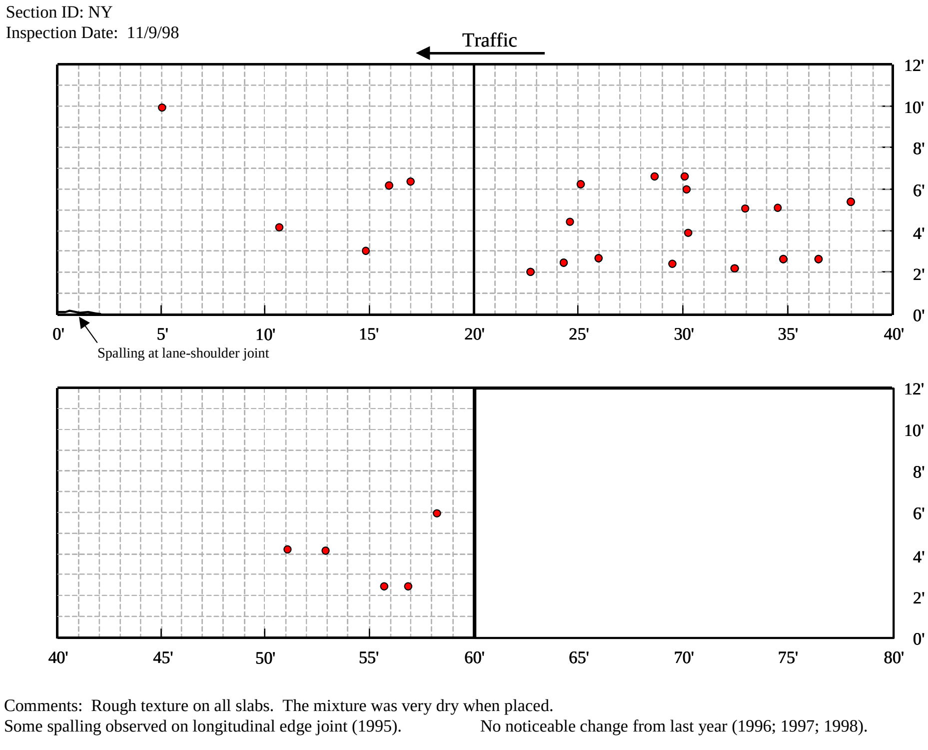
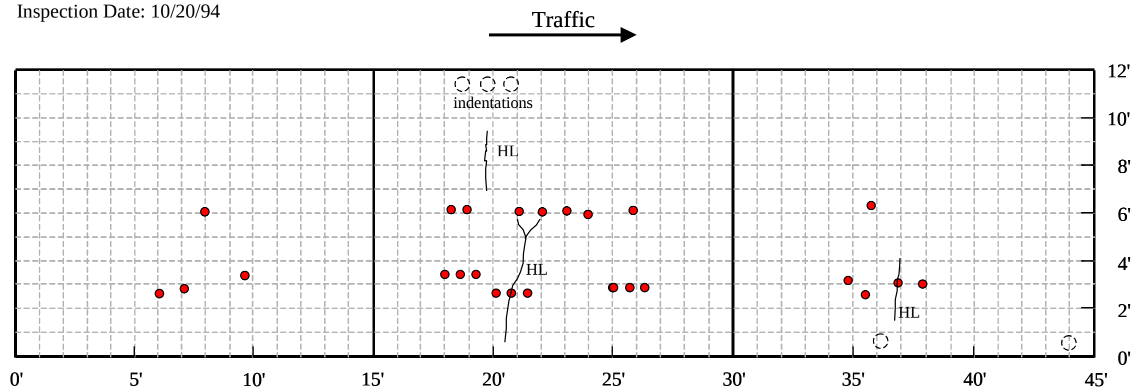


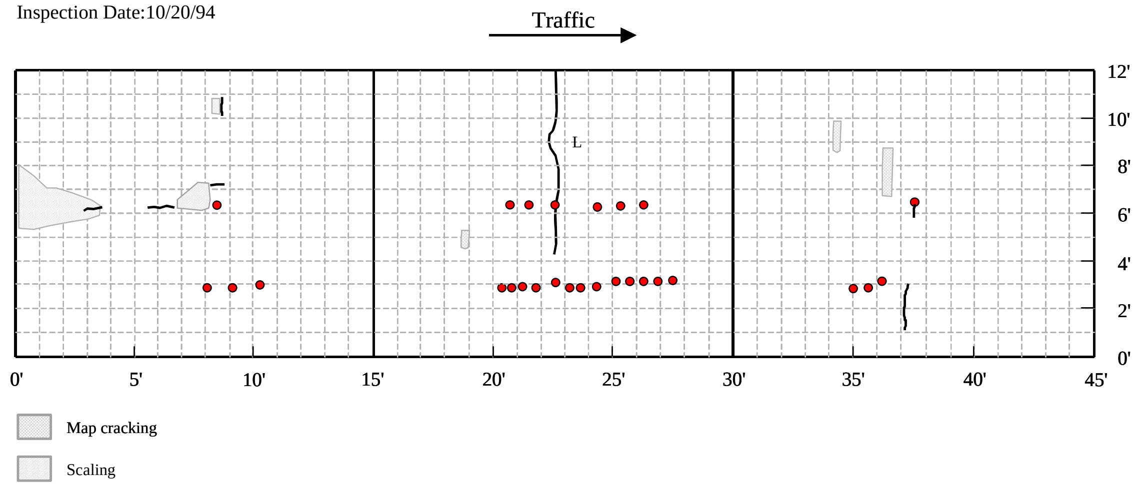
Figure 34. Graph. Distress map of section in New York—1998.

Section ID: IL-1
Inspection Date: 10/20/94



Comments: Insulated. Some scaling, which appear to be construction related. Much cleaner finish than IL-2.

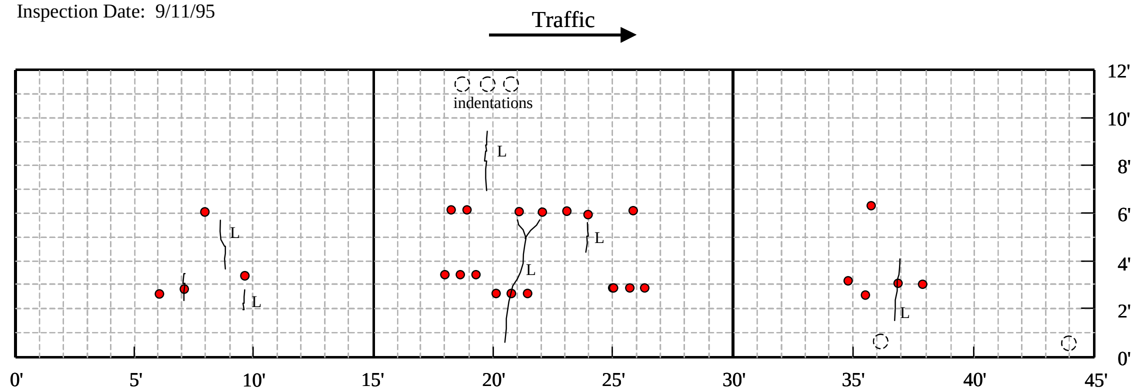
Section ID: IL-2
Inspection Date:10/20/94



Comments: Noninsulated. Shrinkage cracks, minor map cracking, and scaling. Surrounding pavement had been milled.

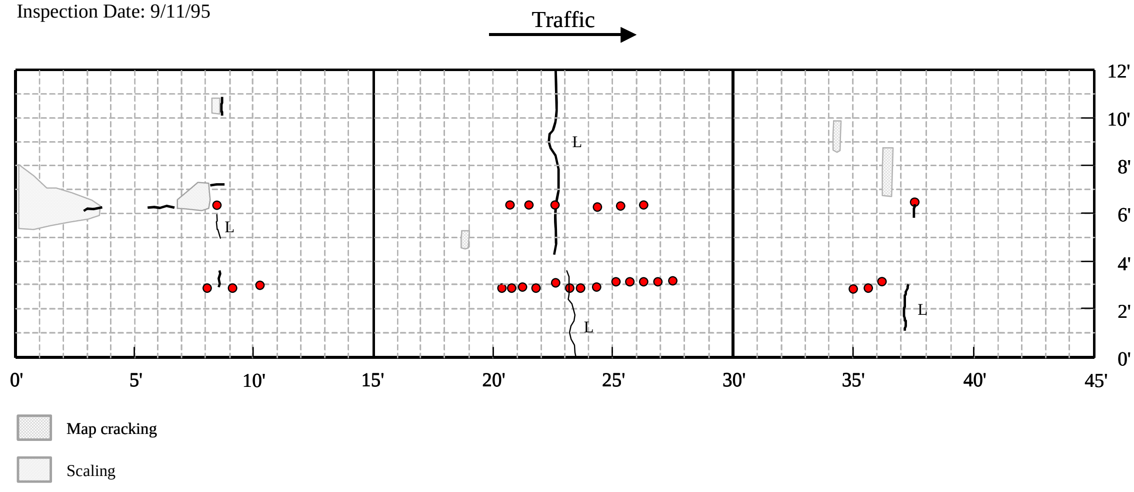
Figure 35. Graph. Distress maps for patches IL-1 and IL-2 in Illinois—1994.

Section ID: IL-1
Inspection Date: 9/11/95



Comments: Insulated. Some scaling, which appear to be construction related. Much cleaner finish than IL-2.

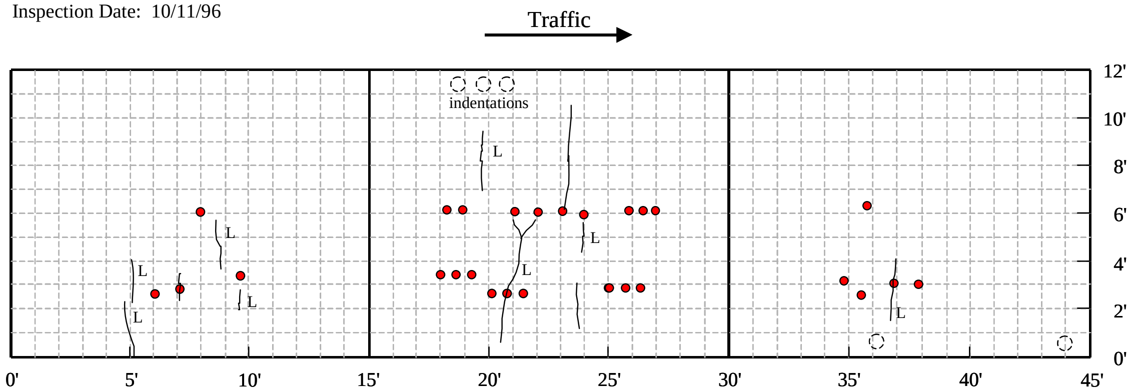
Section ID: IL-2
Inspection Date: 9/11/95



Comments: Noninsulated. Shrinkage cracks, minor map cracking, and scaling. Surrounding pavement had been milled.

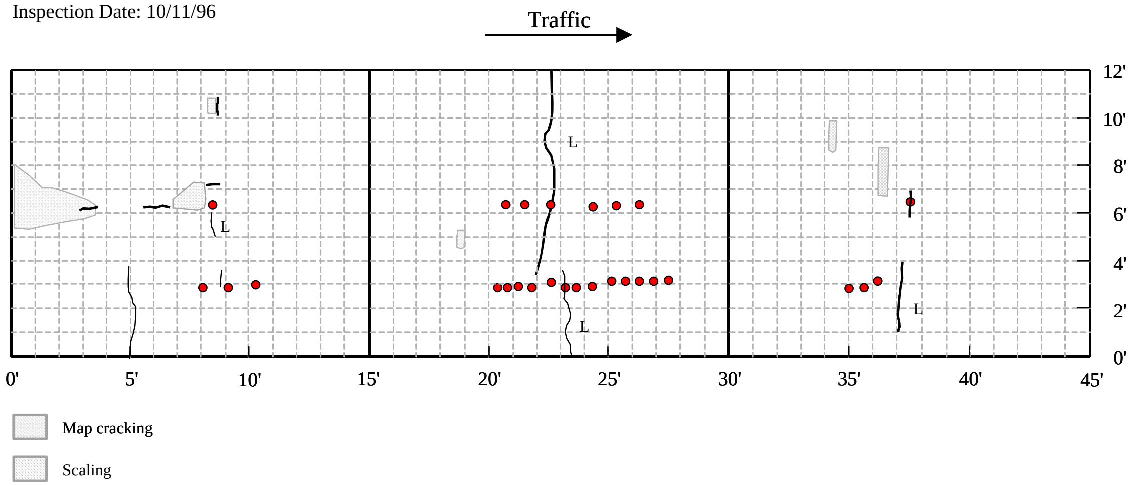
Figure 36. Graph. Distress maps for patches IL-1 and IL-2 in Illinois—1995.

Section ID: IL-1
Inspection Date: 10/11/96



Comments: Insulated. Some scaling, which appear to be construction related. Much cleaner finish than IL-2.

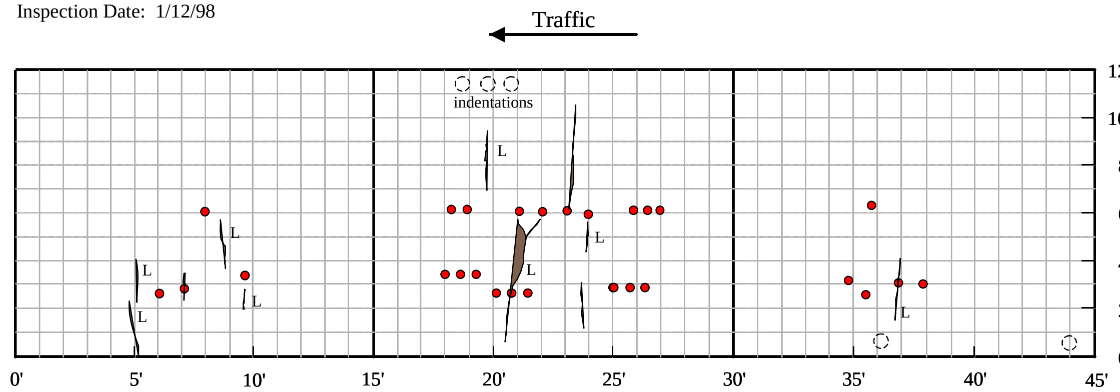
Section ID: IL-2
Inspection Date: 10/11/96



Comments: Noninsulated. Shrinkage cracks, minor map cracking, and scaling. Surrounding pavement had been milled.

Figure 37. Graph. Distress maps for patches IL-1 and IL-2 in Illinois—1996.

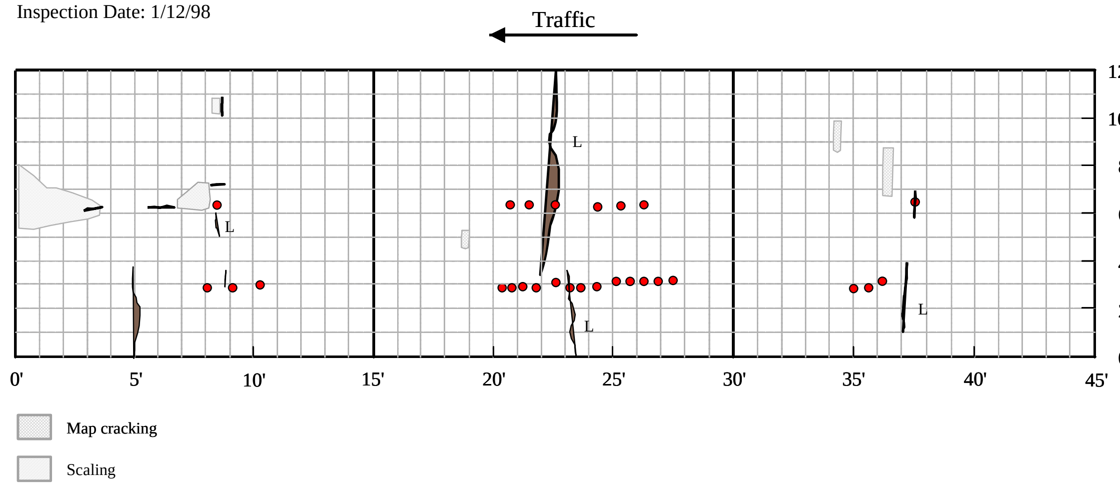
Section ID: IL-1
Inspection Date: 1/12/98



Comments: Insulated. Some scaling, which appear to be construction related. Much cleaner finish than IL-2.

_Slight increase in map cracking (1997)____

Section ID: IL-2
Inspection Date: 1/12/98

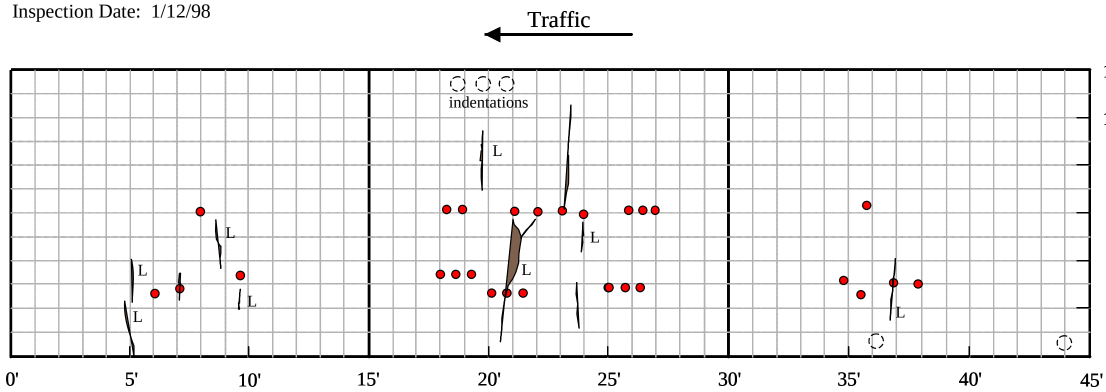


Comments: Noninsulated. Shrinkage cracks, minor map cracking, and scaling. Surrounding pavement had been milled.

_Slight increase in map cracking (1997)____

Figure 38. Graph. Distress maps for patches IL-1 and IL-2 in Illinois—1997.

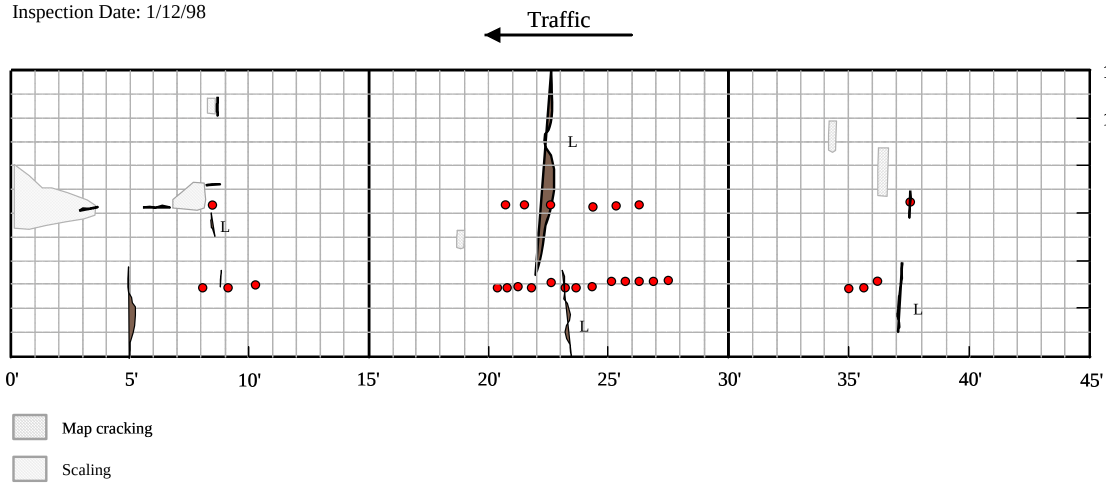
Section ID: IL-1
Inspection Date: 1/12/98



Comments: Insulated. Some scaling, which appear to be construction related. Much cleaner finish than IL-2.

_Slight increase in map cracking (1997)____

Section ID: IL-2
Inspection Date: 1/12/98

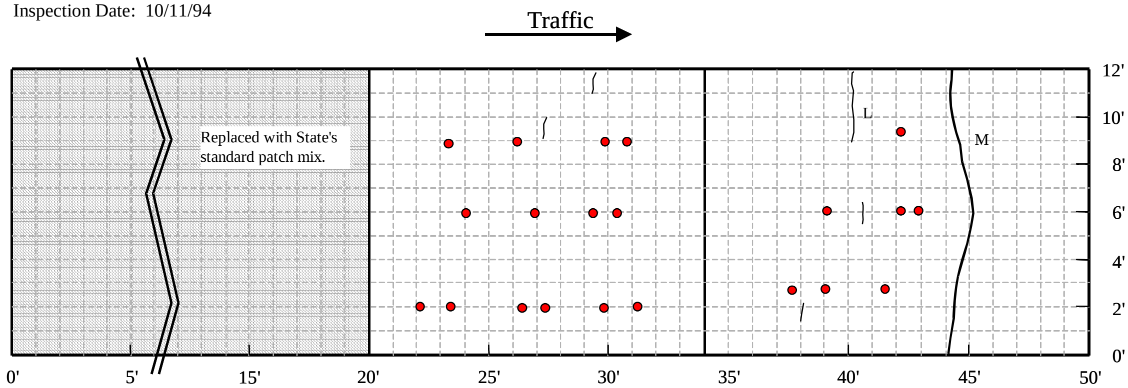


Comments: Noninsulated. Shrinkage cracks, minor map cracking, and scaling. Surrounding pavement had been milled.

_Slight increase in map cracking (1997)____

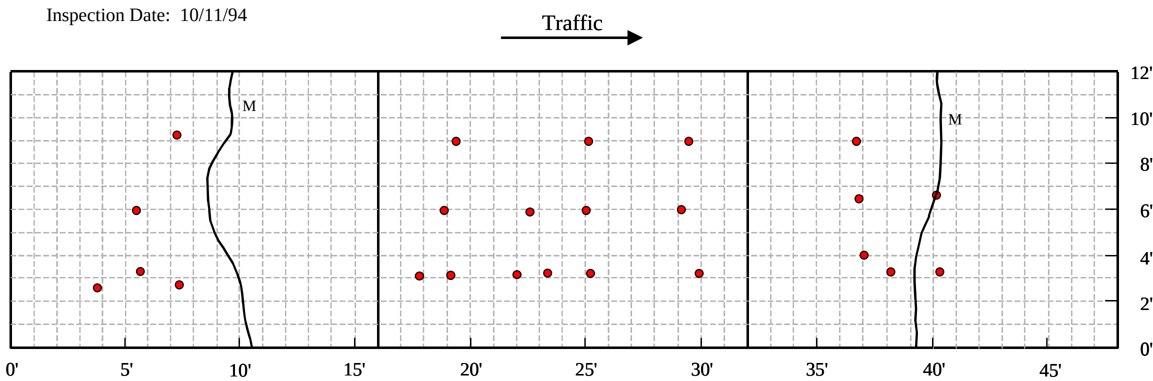
Figure 39. Graph. Distress maps for patches IL-1 and IL-2 in Illinois—1998.

Section ID: NE -1
Inspection Date: 10/11/94



Comments: Located ~ 700 ft west of Breslav Creek Bridge.

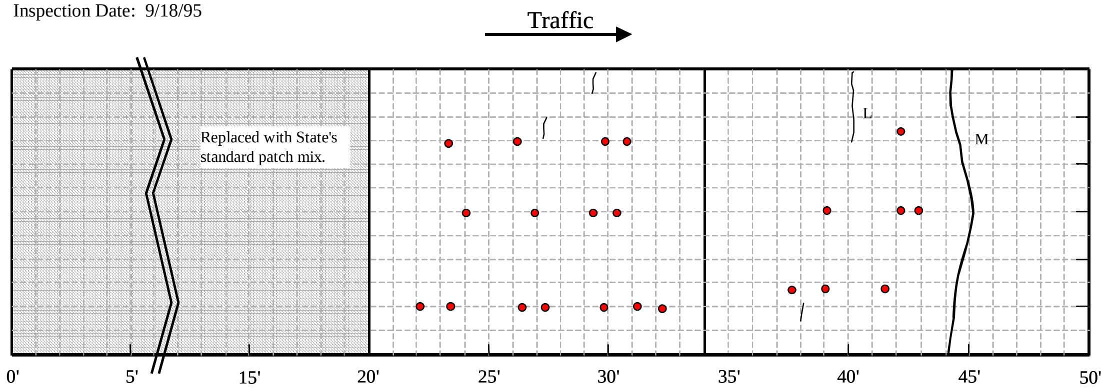
Section ID: NE-2
Inspection Date: 10/11/94



Comments: ______

Figure 40. Graph. Distress maps for patches NE-1 and NE-2 in Nebraska—1994.

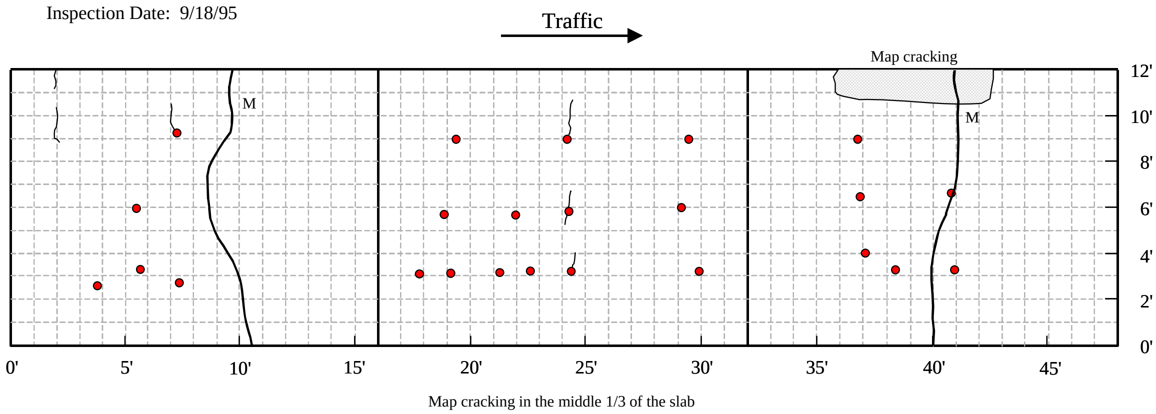
Section ID: NE -1
Inspection Date: 9/18/95



Comments: Located ~ 700 ft west of Breslav Creek Bridge.

_No significant change in condition (1995).________________________

Section ID: NE-2
Inspection Date: 9/18/95



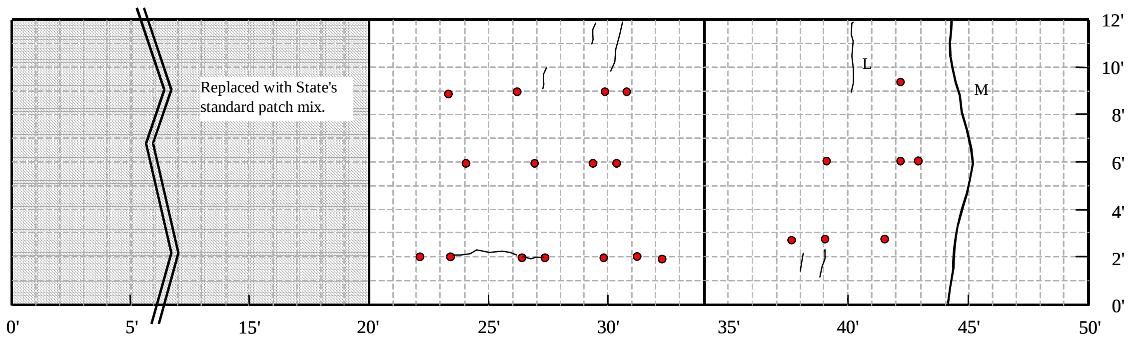
Comments: ____

____

Figure 41. Graph. Distress maps for patches NE-1 and NE-2 in Nebraska—1995.

Section ID: NE -1
Inspection Date: 9/26/96

Traffic



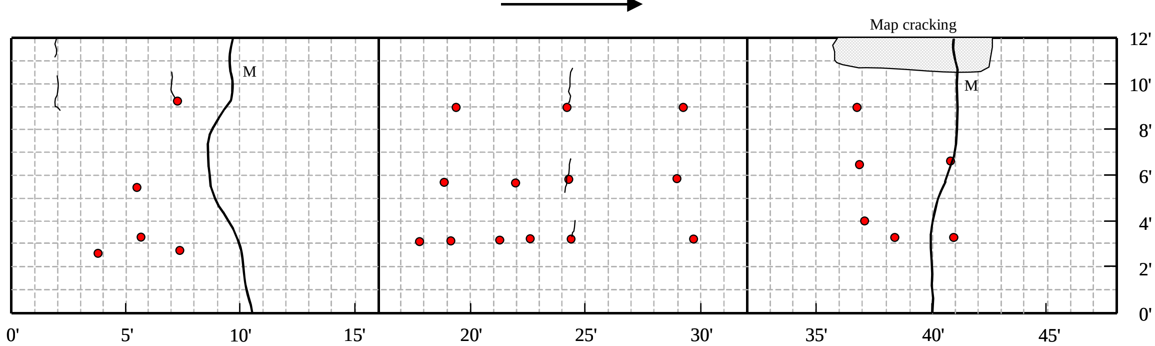
Comments: Located ~ 700 ft west of Breslav Creek Bridge.

_No significant change in condition (1995).____________________________

_No significant change in condition other than progress of surface cracks shown on the map (1996).________________________

Section ID: NE-2
Inspection Date: 9/26/96

Traffic



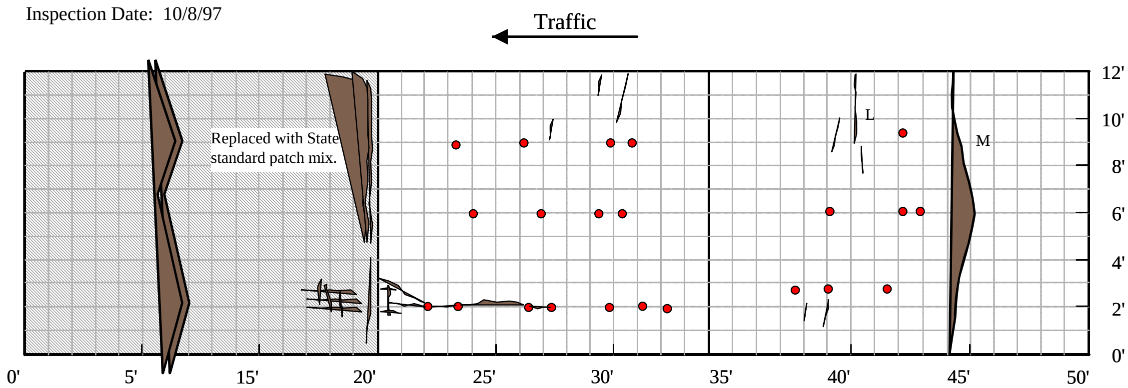
Map cracking in the middle 1/3 of the slab

Comments: ____

____

Figure 42. Graph. Distress maps for patches NE-1 and NE-2 in Nebraska—1996.

Section ID: NE-1
Inspection Date: 10/8/97



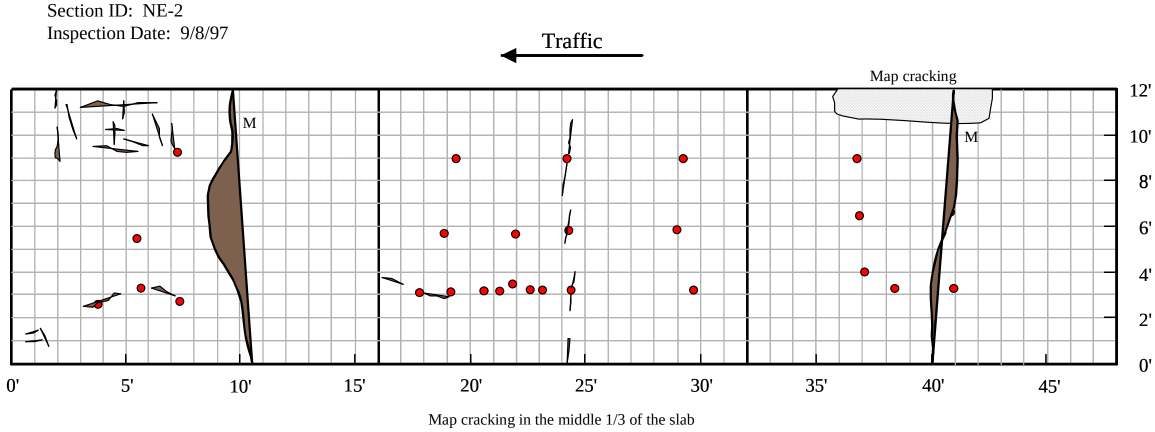
Comments: Located ~700 ft west of Breslav Creek Bridge.

No significant change in condition (1995).

No significant change in condition other than progress of surface cracks shown on the map (1996).

Increase in map cracking (1997).

Section ID: NE-2
Inspection Date: 9/8/97

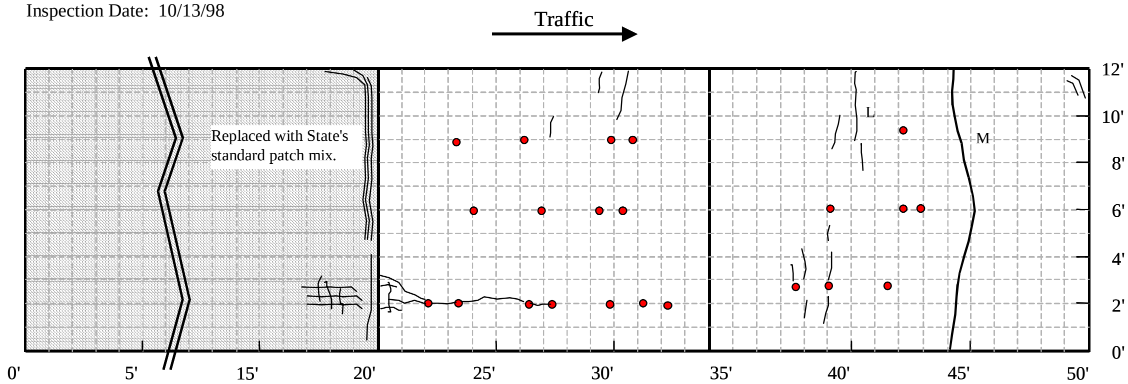


Comments: ____

____

Figure 43. Graph. Distress maps for patches NE-1 and NE-2 in Nebraska—1997.

Section ID: NE -1
Inspection Date: 10/13/98



Comments: Located ~ 700 ft west of Breslav Creek Bridge.

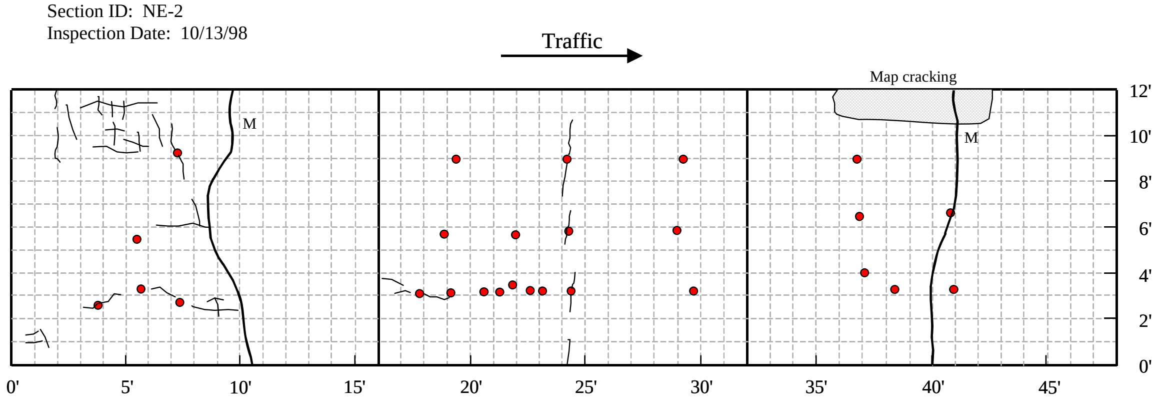
No significant change in condition (1995).

No significant change in condition other than progress of surface cracks shown on the map (1996).

Increase in map cracking (1997).

Significant increase in map cracking (1998).

Section ID: NE-2
Inspection Date: 10/13/98



Map cracking in the middle 1/3 of the slab

Comments: ______

Figure 44. Graph. Distress maps for patches NE-1 and NE-2 in Nebraska—1998.

REFERENCES

1. Zia, P., S.A. Ahmad, M.L. Leming, J.J. Schemmel, and R.P. Elliot, *Mechanical Behaviour of High Performance Concrete, Vol. 4: High Early Strength Concrete*. Strategic Highway Research Program, Washington, DC, 1993.

2. Mindess, S., and J.F. Young, *Concrete*. Prentice Hall, Englewood Cliffs, NJ, 1981.

3. Neter, J., W. Wasserman, and M.H. Kunter, *Applied Linear Statistical Models*. Richard D. Irwin, Inc., Homewood, IL, 1989.

4. Pfeifer, D.W., D.B. McDonald, and P.D. Krauss, “The Rapid Chloride Permeability Test and Its Correlation to 90-Day Chloride Ponding Test,” *Portland Cement Institute Journal*. January–February, 1994.