Long-Term Effects of Electrochemical Chloride Extraction on Laboratory Specimens and Concrete Bridge Components

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FOREWORD

Electrochemical chloride extraction (ECE) is a process that extracts or removes chloride ions from chloride-contaminated reinforced concrete structures. An electrical current is applied between the embedded steel and an external anode as a rehabilitation option to mitigate ongoing corrosion of the embedded steel.

The structures portion of the Strategic Highway Research Program (SHRP) evaluated ECE technology in detail and determined its feasibility through a laboratory study. Additionally, it performed four field validation studies from 1987 to 1992. The laboratory portion of the study affirmed the feasibility of ECE application, and three of the four field validation studies were successful. Although the SHRP study established the feasibility of extracting sufficient amounts of chloride ions from concrete bridge elements, it was not designed to ascertain the long-term effectiveness of the technology in mitigating corrosion. The primary goal of this study was to monitor 10 SHRP concrete laboratory specimens and 3 SHRP field validation sites for 5 years to determine the long-term effectiveness of ECE.

Jorge E. Pagán-Ortiz
Director, Office of Infrastructure
Research and Development

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16. Abstract
Electrochemical chloride extraction (ECE) is a process that extracts chloride ions from chloride-contaminated reinforced concrete structures by applying an electrical current between the embedded steel and an external anode. ECE is becoming increasingly popular as a rehabilitation option for chloride-contaminated reinforced concrete structures to mitigate ongoing corrosion of embedded steel.

In 1987, section 128 of the Surface Transportation and Uniform Relocation Assistance Act initiated the Strategic Highway Research Program (SHRP). The structures portion of SHRP evaluated ECE technology in detail and determined its feasibility through a laboratory study. Additionally, it performed four field validation studies from 1987 to 1992. The laboratory portion of the study affirmed the feasibility of ECE application, and three of the four field validation studies were successful. Although sufficient levels of chloride ions remained in the structure after the application of ECE, researchers found that the distribution of chloride ions in concrete and the production of hydroxyl ions at the concrete-steel interface could significantly delay the initiation of corrosion and provide an extension in service life. A long-term evaluation of ECE-treated laboratory concrete slabs was considered necessary to evaluate the remigration of chloride ions with time and to study the impact of the higher levels of hydroxyl ions at the concrete-steel interface in delaying the initiation of corrosion,. Although the SHRP study clearly established the feasibility of extracting sufficient amounts of chloride ions from concrete bridge elements, it was not designed to ascertain the long-term effectiveness of the technology in mitigating corrosion. The primary goal of this study was to monitor 10 SHRP concrete laboratory specimens and 3 SHRP field validation sites for 5 years to determine the long-term effectiveness of ECE.

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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
	NOTE: volumes greater than 1000 L shall be shown in m^3			
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact degrees)		
^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)

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EXECUTIVE SUMMARY

Electrochemical chloride extraction (ECE) is a process that extracts chloride ions from chloride-contaminated reinforced concrete structures. An electrical current is applied between the embedded steel and an external anode. The process was originally referred to as electrochemical chloride removal (ECR); however, due to the popularity of the abbreviation ECR to denote epoxy coated rebar, ECE was adopted, particularly in North America. In Europe, the process is often referred to as desalination or sometimes as electrochemical chloride migration (ECM). ECE is becoming popular as a rehabilitation option for chloride-contaminated reinforced concrete structures to mitigate ongoing corrosion of embedded steel.

In 1987, section 128 of the Surface Transportation and Uniform Relocation Assistance Act initiated the Strategic Highway Research Program (SHRP). The structures portion of SHRP evaluated ECE technology in more detail and determined its feasibility through a laboratory study. Additionally, it performed four field validation studies from 1987 to 1992. The laboratory portion of the study affirmed the feasibility of ECE application, and three of the four field validation studies were successful.

Although sufficient levels of chloride ions remained in the structure after the application of ECE, it was concluded that the distribution of chloride ions in concrete (lower concentrations around the reinforcing steel and higher concentrations away from the reinforcement) and the production of hydroxyl ions at the concrete-steel interface could significantly delay the initiation of corrosion and provide an extension in service life. A long-term evaluation of ECE-treated laboratory concrete slabs was necessary to evaluate the remigration of chloride ions with time and to study the impact of the higher levels of hydroxyl ions at the concrete-steel interface in delaying the initiation of corrosion,.

Four field validation trials were conducted between fall 1991 and fall 1992. Chloride removal was conducted using a variety of anode-electrolyte configurations on structures in four locations: a bridge deck in Ohio, marine bridge pilings in Florida, columns in New York, and an abutment in Ontario. Each field site was selected based on criteria established by the laboratory studies and the SHRP Expert Task Group (ETG). Active corrosion was occurring on a substantial portion of each selected component of each structure, and chloride contamination was well above the threshold level. To establish the effectiveness of lithium ion in controlling alkali-silica reaction (ASR), the abutment of the Ontario structure containing alkali reactive aggregate was selected.

Although the SHRP study established the feasibility of extracting sufficient amounts of chloride ions from concrete bridge elements, it was not designed to ascertain the long-term effectiveness of the technology in mitigating corrosion. The primary goal of this study was to monitor 10 SHRP concrete laboratory specimens and 3 SHRP field validation sites for 5 years to determine the long-term effectiveness of ECE.

The concrete laboratory specimens were exposed to Northern Virginia climate and were monitored once every quarter from 1995 to 1998. Previously, under the SHRP study, the specimens were monitored for 40 months; however, quarterly monitoring was terminated in 1998 due to noncommitment of funds to the contract. Data collected every quarter included

visual, delamination, half-cell potential surveys, macrocell current and driving volt data, alternating current (AC) resistance between the two mats, slab temperature, and corrosion rate measurements. At the conclusion of the study in 1999, one core was collected from each slab, and powdered concrete samples were collected from various depths to determine the distribution of chloride ions as a function of depth.

Of the four field validation sites, the marine piles in Florida were not included in this study due to concerns about the applicability of ECE in a marine environment. The Ontario site could not be monitored because it had been replaced with a new structure. The Federal Highway Administration (FHWA) had funded several piloted ECE treatments on bridge structures, two of which were selected for long-term evaluation in this study. One pilot application was performed on a bridge deck, and the other was performed on columns and cap beams. These structures are located in Arlington, VA, and Charlottesville, VA.

With the exception of the structure in Arlington, VA, three evaluations of treated elements and elements designated as controls on each bridge were performed during the 5-year monitoring period from 1994 to 1998. The Arlington, VA, structure was evaluated twice due to noncommitment of funds to the contract.

After 10.25 years of exposure, the control slab (not treated) had suffered significant damage due to corrosion. Two slabs treated at a lower total charge of 59.9 A-h/ft^2 (645 A-h/m^2) started to exhibit early signs of corrosion initiation. The other seven ECE-treated slabs had a total charge greater than 186 A-h/ft^2 (2,000 A-h/m^2) and did not exhibit any signs of corrosion initiation. The trends in data collected over 10.25 years suggest that the presently insignificant levels of corrosion activity in the slabs can be maintained for a minimum of an additional 10 years if the slabs are protected from future ingress of chloride ions from the environment.

Long-term evaluation of the field validation studies indicated that no corrosion activity had initiated in two of the four sites evaluated (Arlington, VA, and Charlottesville, VA). Sufficient time had not elapsed after the application of ECE to determine its effectiveness.

The presence or lack of corrosion could not be determined at the Lucas County, OH, site due to a thin epoxy overlay. Chloride data suggest that ECE was successful in extracting the chloride ions from the deck. Due to similarities in treatment and lower chloride content in the treated sections of the deck, it is expected that ECE treatment could effectively mitigate corrosion for a minimum of 10 years.

ECE treatment was ineffective on the structure located in Albany, NY. Corrosion was ongoing, and damage due to corrosion was observed. The failure of the treatment was attributed to the lack of quality of the repairs performed (i.e., cracking of patch material), nonrepair of cracks, no barrier to future ingress of chloride ions installed, and problems encountered during the application of ECE including containment of the electrolyte, nonuniform distribution of current, and lower pH values.

CHAPTER 1. INTRODUCTION

PROJECT BACKGROUND

ECE is a process that extracts chloride ions from chloride-contaminated reinforced concrete structures by applying an electrical current between the embedded steel and an external anode. The process was originally referred to as ECR. However, due to the popularity of the abbreviation ECR for epoxy coated rebar, ECE was adopted, particularly in North America. In Europe, the process is often referred to as desalination or sometimes as ECM. ECE is becoming popular as a rehabilitation option for chloride-contaminated reinforced concrete structures to mitigate ongoing corrosion of the embedded steel.

The technique of removing chloride ions from contaminated reinforced concrete by electrochemical means was first studied in the 1970s by the Kansas Department of Transportation (KDOT) and Battelle Columbus Laboratories.[1,2] Both studies were funded by FHWA, and some adverse effects were observed, including a reduction in rebar-to-concrete bonding and increased concrete permeability. Questions were raised with regard to the impact of the migration of ions other than chloride and the rate of remigration of the residual chloride ions.

In 1987, section 128 of the Surface Transportation and Uniform Relocation Assistance Act initiated SHRP. The structures portion of SHRP reevaluated ECE technology in detail and determined its feasibility through a laboratory study. Additionally, it performed four field validation studies from 1987 to 1992. The laboratory portion of the study affirmed the feasibility of ECE application, and three of the four field validation studies were successful.[3,4]

SHRP LABORATORY WORK

Laboratory work under the SHRP contract addressed several concerns with regard to the passage of large amounts of current through concrete. The effect of current and charge passed on concrete-steel bond strength was evaluated. A reduction of bond strength compared to controls containing salt was noted at a high current density 4.995 A/ft^2 (53.75 A/m^2) and/or high amount of total charge (200 A-h/ft^2 (2,150 A-h/m^2)) passed. Lower current density and/or lower charge were not found to have any adverse effects. Even at the highest current density and charge, concrete-steel bond strength was reduced only to the values equal to those of no-salt control specimens.

Lower current densities did not result in a reduction of concrete compressive strength, but a softening of the cement paste around the reinforcing steel was observed on specimens treated at high currents (2.00 A/ft^2 (21.5 A/m^2)) for long periods of time (500 A-h/ft^2 (5,375 A-h/m^2)). It was hypothesized that softening the paste could result in the reduction of concrete-steel bond strength.

Hydrogen embrittlement of conventional reinforcing steel was evaluated, and only a temporary loss of ductility was noted on smooth specimens. To control the generation of chlorine gas, several buffer solutions used to maintain the pH of the electrolyte were studied. A sodium borate buffer was identified to be the most effective and practical.

The acceleration of ASR by the generation of hydroxyl ions during the application of ECE was demonstrated. The use of lithium ion in the electrolyte was found to control ASR and prevent the resulting damage.

The SHRP study identified a safe upper current density limit of 0.50 A/ft^2 (5.4 A/m^2). Optimal chloride removal efficiencies were obtained with total charge accumulation ranging from 80 to 120 $A\text{-}h/ft^2$ (860 to 1,290 $A\text{-}h/m^2$). Additional charge accumulations resulted in diminishing chloride removal efficiency.

It was determined that the ECE treatment process could extract approximately 20 to 50 percent of the chloride ions from the concrete. The amount of chloride ions removed depended on several factors including the total amount and the distribution of chloride ions in the concrete, the reinforcing steel design, etc.

Although sufficient levels of chloride ions remained in the structure after the application of ECE, it was concluded that the distribution of chloride ions in concrete (i.e., lower concentrations around the reinforcing steel and higher concentrations away from the reinforcement) and the production of hydroxyl ions at the concrete-steel interface could significantly delay the initiation of corrosion and provide an extension in service life. A 40-month evaluation of ECE-treated laboratory concrete slabs was conducted to evaluate the remigration of chloride ions with time and to study the impact of the higher levels of hydroxyl ions at the concrete-steel interface in delaying the initiation of corrosion, At the end of the evaluation, it was found that ECE was effective in mitigating corrosion. To ascertain the length of time that ECE would be effective, a long-term evaluation was necessary.

SHRP FIELD STUDIES

Four field validation trials were conducted between fall 1991 and fall 1992. Chloride removal was conducted on structures at four locations: a bridge deck in Ohio, marine bridge pilings in Florida, columns in New York, and an abutment in Ontario. Each field site was selected based on criteria established by the laboratory studies and the SHRP ETG. There was active corrosion on a substantial portion of each selected component of each structure, and chloride contamination was well above the threshold level. To establish the effectiveness of lithium ion in controlling ASR, the abutment of the Ontario structure containing alkali reactive aggregate was selected.

Field worthiness of various ECE application systems was evaluated using a variety of anode-electrolyte configurations, and field application costs were determined. To ascertain the effectiveness of these field applications, long-term monitoring was recommended.

SCOPE AND PURPOSE

Although the SHRP study clearly established the feasibility of extracting sufficient amounts of chloride ions from concrete bridge elements, it was not designed to ascertain the long-term effectiveness of the technology in mitigating corrosion. The primary goal of this study was to monitor 10 SHRP concrete laboratory specimens and 3 SHRP field validation sites for 5 years to determine the long-term effectiveness of ECE. The secondary objective was to identify the

most appropriate laboratory and field test method(s) for evaluating and monitoring ECE performance.

The concrete laboratory specimens were exposed to Northern Virginia climate and monitored once every quarter from 1995 to 1998. Quarterly monitoring of the specimens was terminated in 1998 due to noncommitment of funds to the contract. Data collected every quarter included visual data, delamination, half-cell potential surveys, macrocell current and driving volt data, AC resistance between the two mats, slab temperature, and corrosion rate measurements. At the conclusion of the study in 1999, one core was collected from each slab, and powdered concrete samples were collected from various depths to determine the distribution of chloride ions as a function of depth.

Of the four field validation sites, the marine piles in Florida were not included in this study due to concerns about the applicability of ECE in a marine environment. The Ontario site could not be monitored because it had been replaced with a new structure. FHWA had funded several piloted ECE treatments on bridge structures, two of which were selected for long-term evaluation in this study. One pilot application had been performed on a bridge deck, and the other was performed on columns and cap beams. These structures are located in Arlington, VA, and Charlottesville, VA.

With the exception of the structure in Arlington, VA, three evaluations of treated elements and elements designated as controls on each bridge were performed during the 5-year monitoring period from1994 to 1998. The Arlington, VA, structure was evaluated twice due to noncommitment of funds to the contract.

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CHAPTER 2. LONG-TERM EVALUATION OF LABORATORY SLABS

The primary purpose of the laboratory slab study was to evaluate the long-term performance of ECE in an environment where the impact of other influential variables could be controlled or documented for a more accurate analysis. SHRP Contract C102-A included a laboratory evaluation of ECE. Twelve test slabs were fabricated in July and August 1988 and treated in May, June, and October 1989 using varying current densities and electrolytes. After treatment was complete, the slabs were exposed to Northern Virginia climate and monitored for 35–40 months. This study was completed in 1992, and results and findings were published in 1993.[(1)]

From 1989 to 1999, the slabs were continuously exposed to the Northern Virginia climate. During the interim period between the end of the SHRP study in 1992 and the start of this study in 1995, the slabs were briefly monitored for several months in 1993 by another entity. Since the start of this study, the slabs have been monitored on a quarterly basis up to the second quarter of 1998, and the final chloride data were collected in December 1999.

This 10-year exposure and monitoring provided significant insight into the ability of ECE technology to protect against corrosion of steel embedded in concrete. The findings and results are discussed below.

SLAB CONSTRUCTION

The test slabs were fabricated using concrete confirming to the Ohio Department of Transportation's class C concrete. The test slabs were 24 by 24 by 7.9 inches (610 by 610 by 200 mm) thick and contained two mats of reinforcing steel. The top mat comprised of two rebars with diameters of 7.5 and 5.1 inches (19 and 13 mm). The 7.5-inch (19-mm) bars were placed on top of the 5.1-inch (13-mm) bars, and the clear concrete cover over the 7.5-inch (19-mm) bars was 1.9 inches (48.26 mm). The bottom mat consisted of 6.2-inch (16-mm) crossing rebars. The clear concrete cover over the bottom mat of steel was 1 inch (25.4 mm). All bars were located on 7.1-inch (180-mm) centers.

The test slabs were fabricated using three lifts of concrete, and different amounts of chloride ions were added to each lift. No chloride ions were added to the first lift (3.1 inches (78.7 mm) thick); however, they were added to the second (3.1 inches (78.7 mm) thick) and third lifts (1.6 inches (40.4 mm) thick) at a rate of 1,722 and 3,875 ppm, respectively.

Both ends of all rebars protruded out of the slab. The steel in each mat was made continuous by welding a bar across one end of the rebars in each direction and connecting the external bars together. The two mats of the rebars were made electrically continuous through an external resistor and a switch. The resistor allowed a measurement of macrocell current between the top and the bottom mat, and the switch allowed disconnection of the mats for AC resistance and driving volt measurements.

During the study, numerous cores were collected from these slabs. Reinforcing steel was extracted in many of these cores. To ensure that the collection of the cores did not impact the study, continuity disrupted by coring was reestablished, and sides of the core holes were epoxy

coated. Many of the continuity connections between bars in core holes and the ends of bars with insufficient epoxy in core holes corroded with time. A section of the rebars that was 0.50 inches (12.7 mm) long projecting out of the concrete on the sides of the slabs was especially prone to corrosion as the epoxy coating weathered. This section of the rebar is termed the "external section" of the rebar for the purpose of this report.

ECE APPLICATION

Twelve slabs were fabricated for this study. One was designated as a spare (slab 6, which was not measured), and the remaining 11 were used. Of those 11 slabs, slab 1 was used to determine the maximum current density limit. The remaining 10 slabs were included in the long-term evaluation. The variables associated with each slab including the anodes and electrolytes used, applied current density, and total charge passed for each slab are listed in table 1.

Table 1. ECE application parameters for laboratory slabs.

Slab Number	Current Density (mA/m^2)	Total Charge ($A\text{-}h/m^2$)	Anode	Electrolyte
2 (Control slab)	N/A	N/A	N/A	N/A
3	600	645	Inert anode coating on titanium substrate	0.1 moles (M) sodium hydroxide (NaOH)
4	600	645	Inert anode coating on titanium substrate	0.1 M NaOH
5	600	3,226	Inert anode coating on titanium substrate	0.1 M NaOH
10	600	3,226	Inert anode coating on titanium substrate	0.1 M NaOH
11	200	3,226	Inert anode coating on titanium substrate	0.1 M NaOH
12	200	3,226	Inert anode coating on titanium substrate	Limewater
7	93	2,108	Concrete reinforcing steel mesh	Limewater and cellulose fiber
8	93	2,108	Concrete reinforcing steel mesh	Limewater
9	200	3,226	Inert anode coating on titanium substrate	0.1 M NaOH

1 mA/m^2 = 0.093 mA/ft^2
1 $A\text{-}h/m^2$ = 0.093 $A\text{-}h/ft^2$
N/A indicates data are not available.

The pH of the electrolyte was maintained at a fixed level during application of ECE on slabs 3–5 and 10–12 (ph-ECE slabs), whereas no control was exercised for slabs 7–9 (n-ECE slabs). After the chloride removal process was complete, the top surface of each slab was lightly sandblasted, and a siloxane-based sealer was applied to the surface.

DATA ANALYSIS

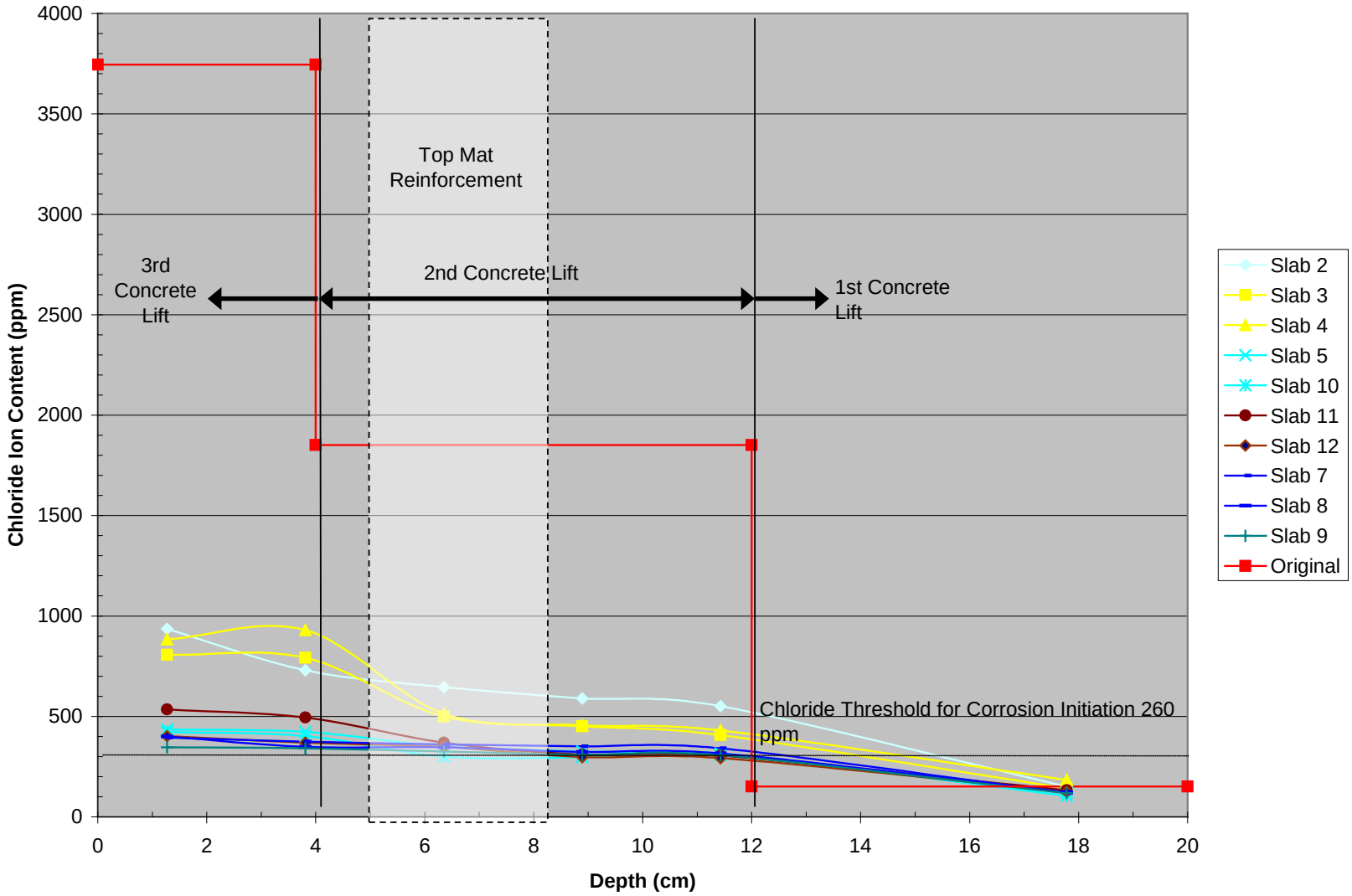
To obtain a complete analysis of performance of ECE in mitigating corrosion, data obtained during the SHRP study interim monitoring in 1993 and data collected in this study were combined.[1,2] With the exception of depths from which powdered samples were collected for chloride ion content analyses, all other data were collected using the same procedures and equipment utilized in SHRP and interim monitoring.

Chloride Ion Content Analysis

At the end of the study in December 1999 (10.25 years after ECE treatment), one core was collected from each slab, and chloride ion content analyses were performed on powdered samples from the cores. Concrete samples were collected from five nominal depths of 0.50, 1.50, 2.50, 3.50, 4.50, and 7.00 inches (12.7, 38.1, 63.5, 88.9, 114.3, and 177.8 mm). Concrete from 0.50 inches (12.7 mm) above and 0.50 inches (12.7 mm) below each nominal depth was powdered and mixed to prepare a sample representing that nominal depth. The collected powdered samples were analyzed for total chloride ion content, and the results are presented in figure 1.

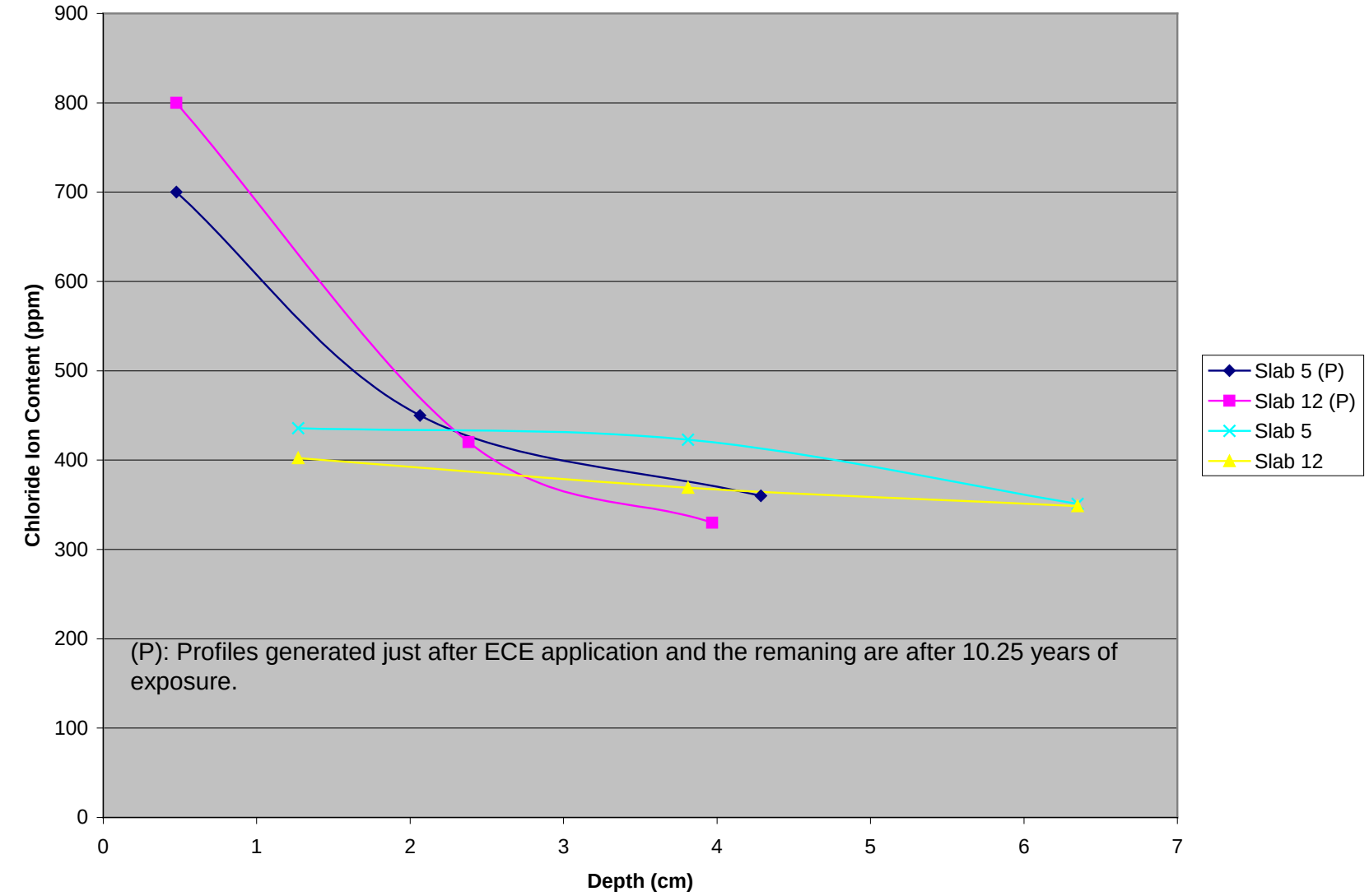
Prior to the application of ECE, the average chloride ion content measured in the bottom 3.1 inches (78.7 mm) of the slabs (first lift) was 151 ppm. Additionally, the average chloride ion measured in the middle 3.1 inches (78.7 mm) of the slabs (second lift) was 1,851 ppm, while the average chloride ion content measured in the top 1.6 inches (40.6 mm) of the slabs (third lift) was 3,475 ppm (see figure 1 for comparison purposes). Comparison of the chloride profile for slab 2 to the chloride profile for original chloride ion content shows the effect of diffusion at the end of 10.25 years. Over time, the chloride ions in slab 2 were diffused to reach a uniform distribution at all depths. The total quantity of chloride ions in slab 2 was calculated using the profile of chloride distribution assuming that the chlorides were uniformly distributed in the horizontal plane at each depth. Based on this calculation, the total quantity of chloride ions present in the slab after 10.25 years was approximately 3.5 oz (100 g). The calculated quantity (based on initial quantities of chlorides in each lift) of chloride ions added to the slab at construction was 9.38 oz (268 g), and the removal of cores from the slab accounted for 2.38 oz (68 g) of chloride ions. The difference of 3.5 oz (100 g) can be explained by the leaching of chloride ions from cracks and the delaminations along the top mat rebar and in the core holes.

All ECE slabs showed a distribution of chloride ions similar to that of slab 2, albeit at a lower but somewhat uniform concentration. Slabs 3 and 4, which received the least amount of ECE treatment (59.9 A-h/ft^2 (645 A-h/m^2) compared to over 186 A-h/ft^2 (2,000 A-h/m^2)) for other slabs), had a higher level of chloride ion concentration than the other ECE slabs at all depths in the second and the third lifts of concrete. To ascertain if this distribution of chloride ions in the ECE slabs resulted from diffusion of chloride ions and/or the application of ECE, chloride profiles from various time periods needed to be compared for each ECE slab. Limited profile data were available. Immediately after ECE application, chloride ion content from several depths of one core from slabs 5 and 12 were analyzed. Comparison of those profiles with profiles after 10.25 years of exposure suggests that some amount of diffusion occurred in the third lift of concrete for these two slabs (see figure 2).



1 inch = 2.54 cm

Figure 1. Graph. Chloride ion content versus depth.



1 inch = 2.54 cm

Figure 2. Graph. Comparison of chloride profiles.

Several times during the SHRP study, chloride analyses were performed on concrete samples collected from the top and bottom trace of crossing and single rebars of the top mat. This analysis was performed to study the migration of chloride ions in the vicinity of the reinforcement. Based on the data obtained, it was concluded that diffusion of chloride ions in the control slab (slab 2) was occurring as predicted by Fick's laws of diffusion, while no diffusion was occurring in the ECE treated slabs. By combining the top trace data for crossing bars from the SHRP study with data from the nominal depth of 1.5 inches (38.1 mm) in this study, figure 3 was developed. Although chloride content at the nominal depth of 1.5 inches (38.1 mm) was not from the trace of the top crossing bar, it represents chloride content of 1 inch (25.4 mm) of concrete above the top rebar and agrees reasonably well with the chloride ion content from the top crossing bar traces.

Figure 3 shows the diffusion of chloride ions with time from the top trace of the top mat rebar of slab 2. A similar but much lower magnitude trend is discernable in the ECE slabs, which is in agreement with data presented in figure 2. The chloride ion content above the top rebar trace in slabs 3 and 4 was higher than the other ECE slabs.

The chloride ion content at the top mat of reinforcing steel in all slabs was over the threshold required to initiate corrosion (260 ppm), and the chloride ion content at the bottom mat of reinforcing steel was below the threshold. Thus, all slabs had sufficient chloride ion concentrations at the top mat reinforcing steel to initiate corrosion and a sufficient difference in chloride ion concentration between the top and bottom mats to set up a macrocell corrosion cell between the two mats.

Visual and Delamination Surveys

Several visual and delamination surveys were performed during this study. Photographic documentation of observations was made when necessary. Because the slabs were small and thin, conducting delamination surveys using sounding techniques was difficult. Sand was placed on the surface of the slabs, and the movement of the sand upon striking a hammer on the surface was used to detect delaminations.

Due to degradation of the epoxy, rust stains on the external sections of the rebars were noted at the start of this study. All bars exhibiting rust stains were cleaned, and new epoxy was applied. Many of the resistors and switches connecting the top to the bottom mat reinforcing steel were repaired and replaced as necessary.

Slab 2 (Control)

The SHRP study reported that the corrosion of reinforcing steel remained active throughout the 40-month study, and a crack had formed along a top mat rebar. This crack grew over time, and by May 1998, the crack tracked down the side of the slab and terminated on top of the rebar. The condition of the crack in August 1994 can be seen in figure 4 and figure 5. Figure 6 shows the crack's termination on the rebar in 1998.

From August 1994 to the last survey in May 1998, additional cracks were observed on the top surface and the sides of the slab. A few hairline cracks were observed in 1995 on the top surface of the slab, which were not associated with any reinforcing steel.

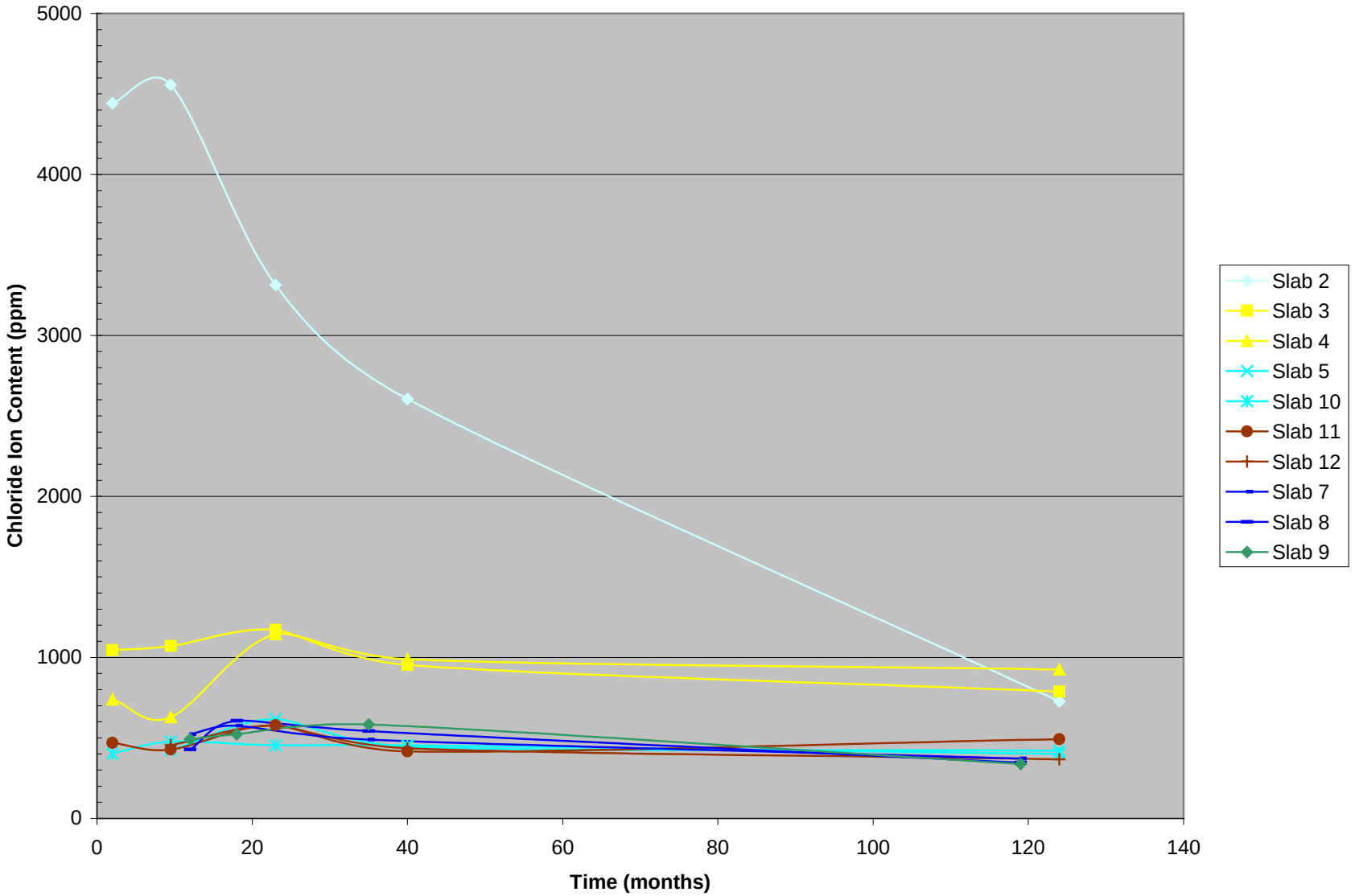


Figure 3. Graph. Chloride content versus time.

Figure 4. Photo. Condition of crack on the top surface of slab 2 in August 1994.

Figure 5. Photo. Termination of the top crack and other cracks on the front side of slab 2 in August 1994.

Figure 6. Photo. Cracks and corrosion products on the rear side of slab 2 in August 1998.

Cracks were also noted on the inside of core holes, and these cracks exhibited rust staining (see figure 7). Cracks in the core holes were determined to be at the level of the reinforcing steel. Corrosion of the ends of the rebars exposed in core holes was also noted (see figure 8). The cracks on the sides of the slabs continued to form and grew wider over time. They primarily originated at the external section of the rebars. On one side of the slab, a crack either connected or just passed over all four rebars protruding out of the concrete (see figure 9). Rust staining was observed on some of these cracks.

Delaminations detected on the surface of the slab increased, and by the end of the study, approximately 66 percent of the slab surface was delaminated. The presence of the crack along the rebar, cracks in the core holes, and cracks on the sides of the slab correlated with the delaminations that were detected. An autopsy of the single core collected from the slab in December 1999 indicated ongoing corrosion in localized areas of the extracted rebar. Expansive corrosion is shown figure 10.

Figure 7. Photo. Cracking and rust staining at the level of reinforcing steel in core holes of slab 2.

Figure 8. Photo. Corrosion on the ends of rebars exposed in core holes of slab 2.

Figure 9. Photo. Crack on the right side of slab 2.

Figure 10. Photo. Autopsy of core extracted from slab 2 in December 1999.

Slabs 3 and 4

In early 1995, slabs 3 and 4 started showing cracks on the sides, which widened and increased over time. Similar to the cracks on slab 2, many of these cracks originated at the protruding rebars and either connected or passed over them. Corrosion of the rebars just inside the concrete on the side of the slab resulted in a spall on slab 3 by September 1997 (see figure 11). Rust stains accompanied by cracks were observed on the sides of slab 4 as well (see figure 12). Corrosion of the rebar just inside the concrete also resulted in a delamination on slab 3 (see figure 13).

Figure 11. Photo. Corrosion and spalling on one rebar at the concrete-air interface on the side of slab 3 in September 1997.

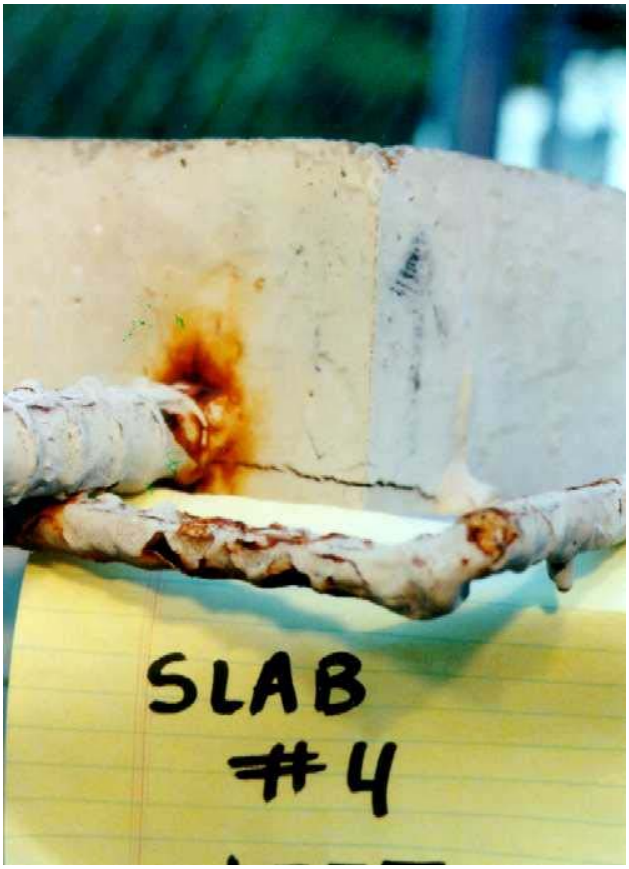


Figure 12. Photo. Rust staining and cracking on the side of slab 4.

Figure 13. Photo. Delamination (marked in yellow) found on the top surface of slab 3.

These slabs exhibited hairline cracking on the top surface, and the number of hairline cracks on the top surface increased with time. On both slabs, one crack connected two core holes on the opposite sides of the slab. Many of the hairline cracks initiated at core holes. Rust stains were noted on the top surface of both slabs (see figure 14). Corrosion of ends of rebars and continuity wires in core holes were observed. No ongoing corrosion was detected on the extracted rebar during autopsy of the core collected in December 1999 (see figure 15).

Figure 14. Photo. Rust staining on the top surface of slabs 3 and 4.

Figure 15. Photo. Autopsy of core extracted from slab 4 in December 1999.

Slabs 5 and 10

By spring 1997, cracks on the top surface of slab 5 appeared adjacent to core holes. Two vertical cracks were also noted on one side along the wall formed by a core hole. A few rust stains were observed on the surface, but they were not accompanied by cracks (see figure 16). By late 1998, a delamination was noted along the edge of the slab and adjacent to a core hole with vertical cracks. By January 1998, concrete failure occurred adjacent to the core hole between the vertical cracks in the delaminations, and the delaminations had grown to the adjacent core hole (see figure 17). The number of cracks on the sides of the slab increased with time, and there were rust stains in core holes adjacent to ends of exposed rebars.

At the inception of this study, only one crack was observed on the side of slab 10. By January 1998, some rust stains appeared on the top surface of the slab, but they were not accompanied by cracks (see figure 18). The number of cracks on the sides of this slab increased with time, and there were rust stains in core holes adjacent to the ends of rebars. In May 1998, a pop-out (i.e., a small piece of pavement broken loose from the surface) adjacent to a core hole was observed.

Figure 16. Photo. Rust stains on the top surface of slab 5.

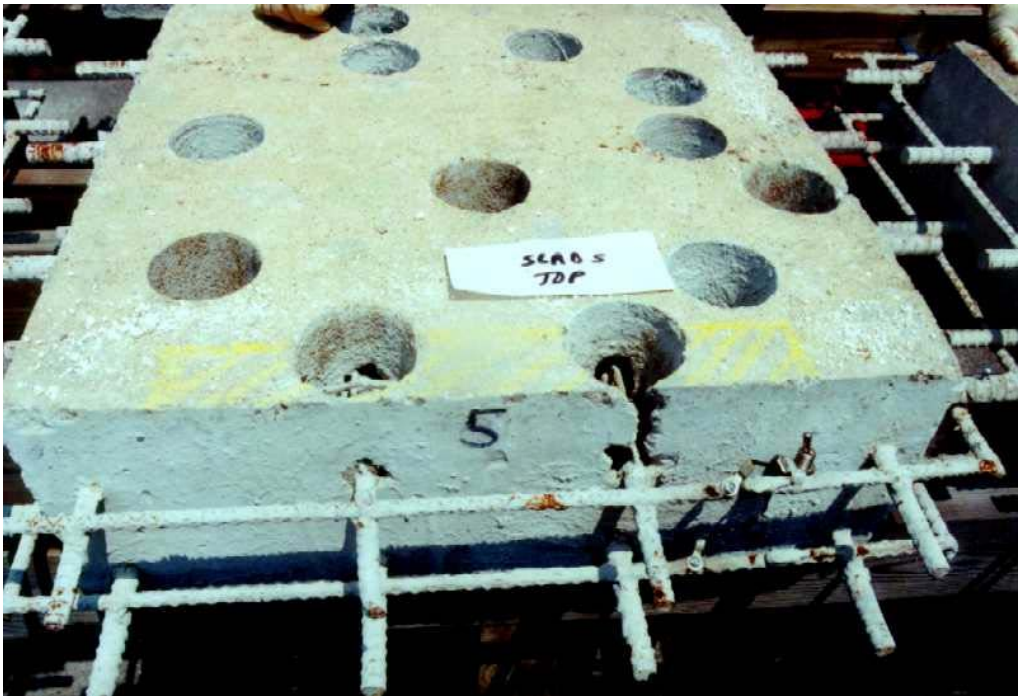


Figure 17. Photo. Delaminations (marked in yellow) and concrete failure near a core hole on slab 5.

Figure 18. Photo. Rust stains on the top surface of slab 10.

Slabs 11 and 12

No signs of deterioration were observed until spring 1997. Noncorrosion-related failure of the corner adjacent to a core hole on slab 11 (see figure 19) and failure of a core hole wall on slab 12 were observed. A rust stain appeared on the surface of slab 11, and rust stains were found at external sections of the rebars. By the end of the study, more cracks appeared on the sides of the slabs, and rust stains in core holes increased. An autopsy of the rebar extracted from a core in December 1999 showed no evidence of any corrosion (see figure 20).

Figure 19. Photo. Noncorrosion-related concrete failure adjacent to core holes on one corner of slab 11 in spring 1997.

Figure 20. Photo. Autopsy of core extracted from slab 12 in December 1999.

Slabs 7 and 8

At the beginning of the study, two fine cracks were observed on the top surface of slab 7, one of which was along a rebar. Two hairline cracks along rebars were also noted on the top surface of slab 8. No rust staining was associated with any of these cracks. By early 1997, a few cracks on the sides of the slabs appeared and were accompanied by rust stains at the external sections of rebars. In January 1998, a new crack and scattered minor rust stains were observed on the top surface of slab 8. By the end of the study, cracking on the sides of these slabs had increased, and there were rust stains inside the core holes. Rust stains created during the application of ECE by the use of steel mesh were not included in the analysis and are not included in this report.

Slab 9

No corrosion-induced damage was observed throughout the study on slab 9, with the exception of two rust stains that appeared in late 1997. A concrete spall in a core hole was noted in spring 1998, and some chipped concrete was observed adjacent to core holes in mid 1998.

Macrocell and Driving Voltages

To reduce the impact of temperature during measurement, macrocell currents were adjusted to 69.8 °F (21 °C) using a procedure developed by an FHWA study.[3] The impact of standardizing the macrocell currents to one temperature was minor in the analysis of the data.

The macrocell currents and the driving voltages of the slabs showed the difference between the control slab and the ECE slabs (see figure 21 and figure 22). The control slab's driving voltage and the macrocell currents were much higher throughout the study than that of the ECE slabs and correlated well with the observed damage.

The ECE slabs had much lower macrocell current and driving volts and exhibited lower levels of damage. The driving volts and macrocell currents for the ECE slabs varied within a narrow range, and no specific trend was observed over time. Analysis of variance of the data indicated that the driving volts and macrocell currents for slabs 3 and 4 were statistically different than those of the other ECE slabs. Figure 22 indicated that the driving volts for slabs 3 and 4 were somewhat higher than the volts for the other ECE slabs, especially after 40 months of monitoring. Such a difference was not noticeable in the macrocell data in figure 21. The higher driving volts and macrocell currents for these slabs correlated well with the amount of chlorides at the top mat steel level and the observed damage. The macrocell currents and driving volts for slabs 3 and 4 increased closer to the end of this study.

The macrocell currents and driving voltages of all ECE slabs were significantly lower than those of the control slab, and they indicated the level of reduction in corrosion of the rebars that had resulted from application of ECE. If the time-averaged macrocell current was calculated for each slab from the data in figure 21, the time-average macrocell current for the control slab was a minimum of 13 times larger than the time-average macrocell current of the individual ECE slabs.

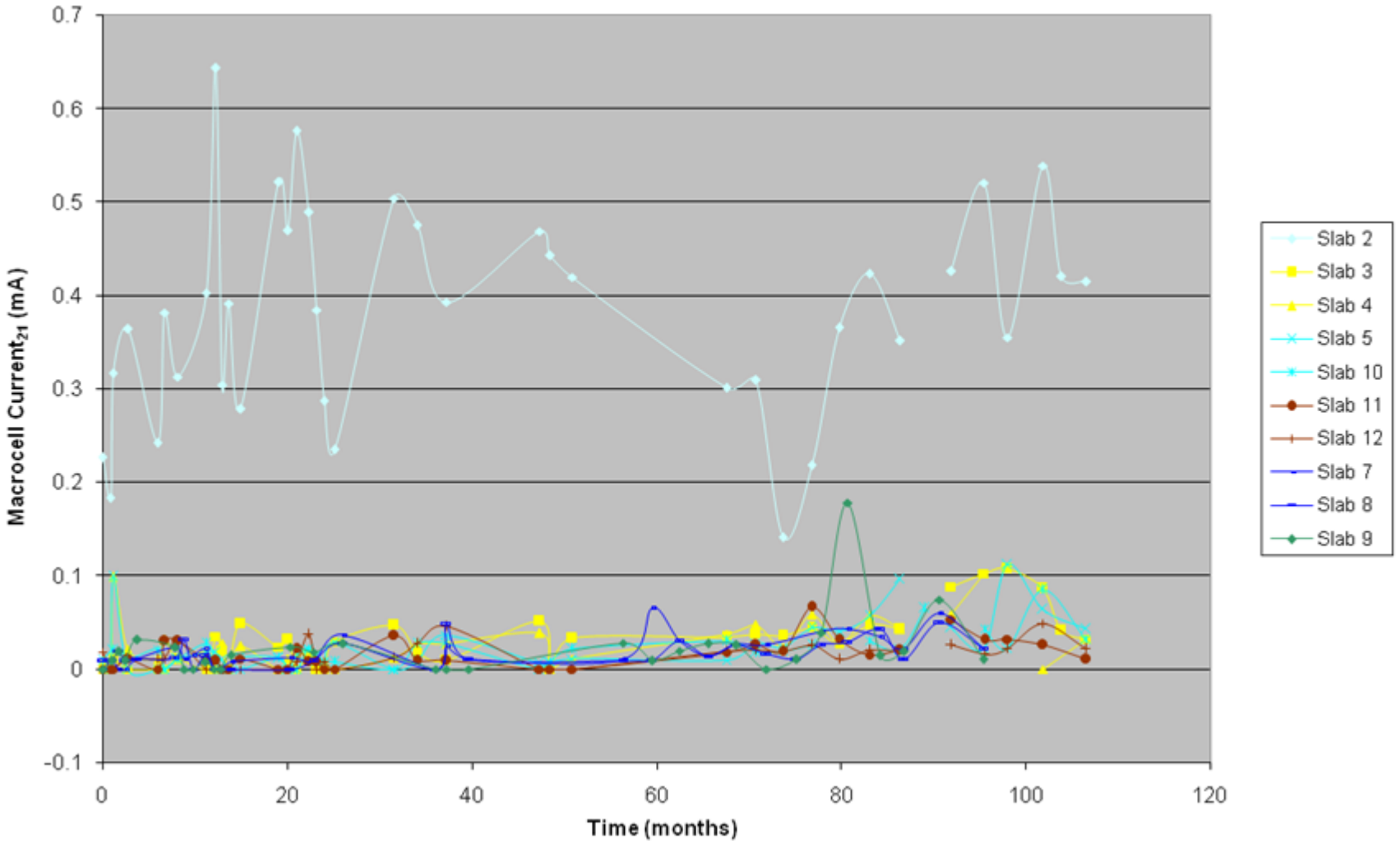


Figure 21. Graph. Macrocell current at 69.8 °F (21 °C) versus time.

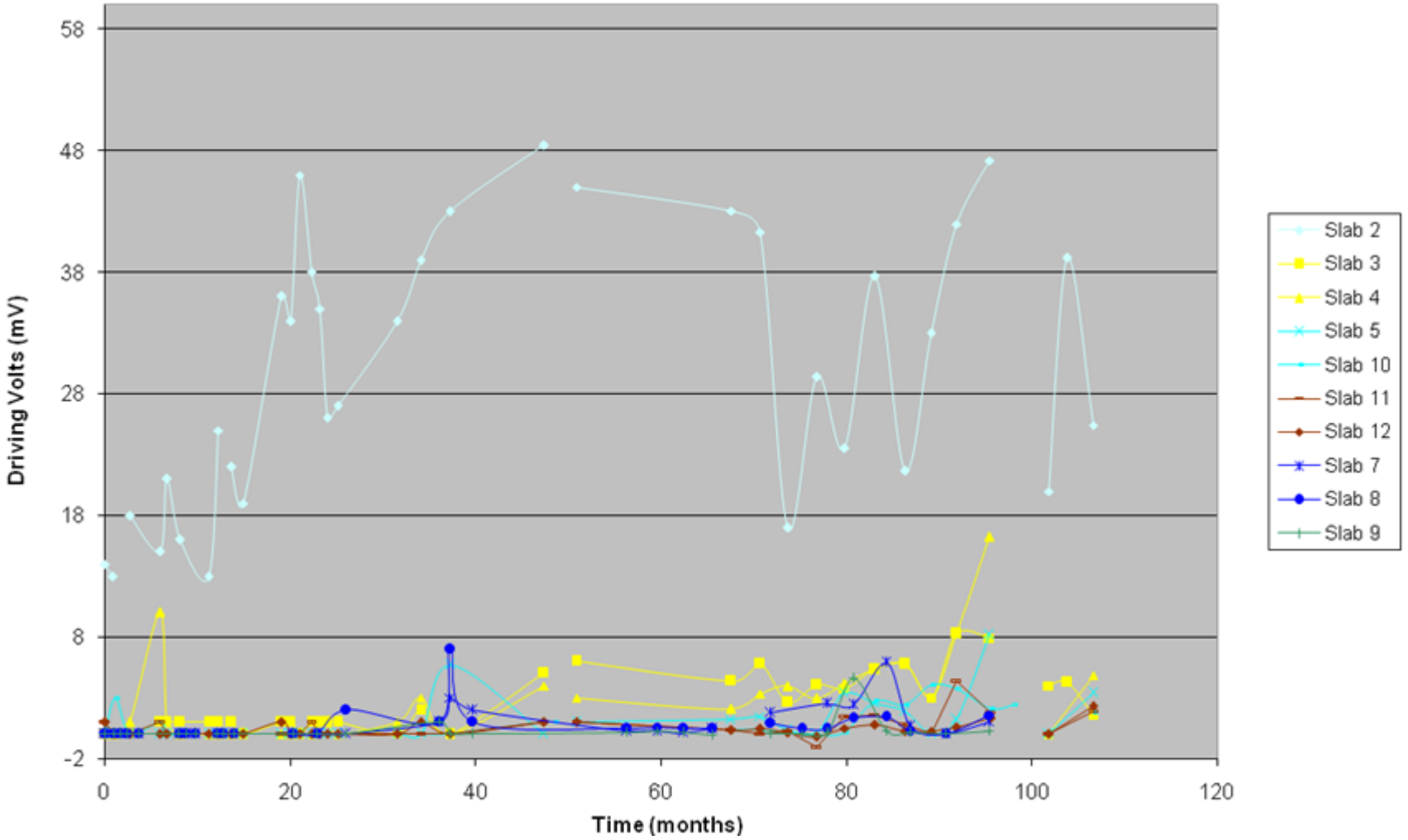


Figure 22. Graph. Driving volts versus time.

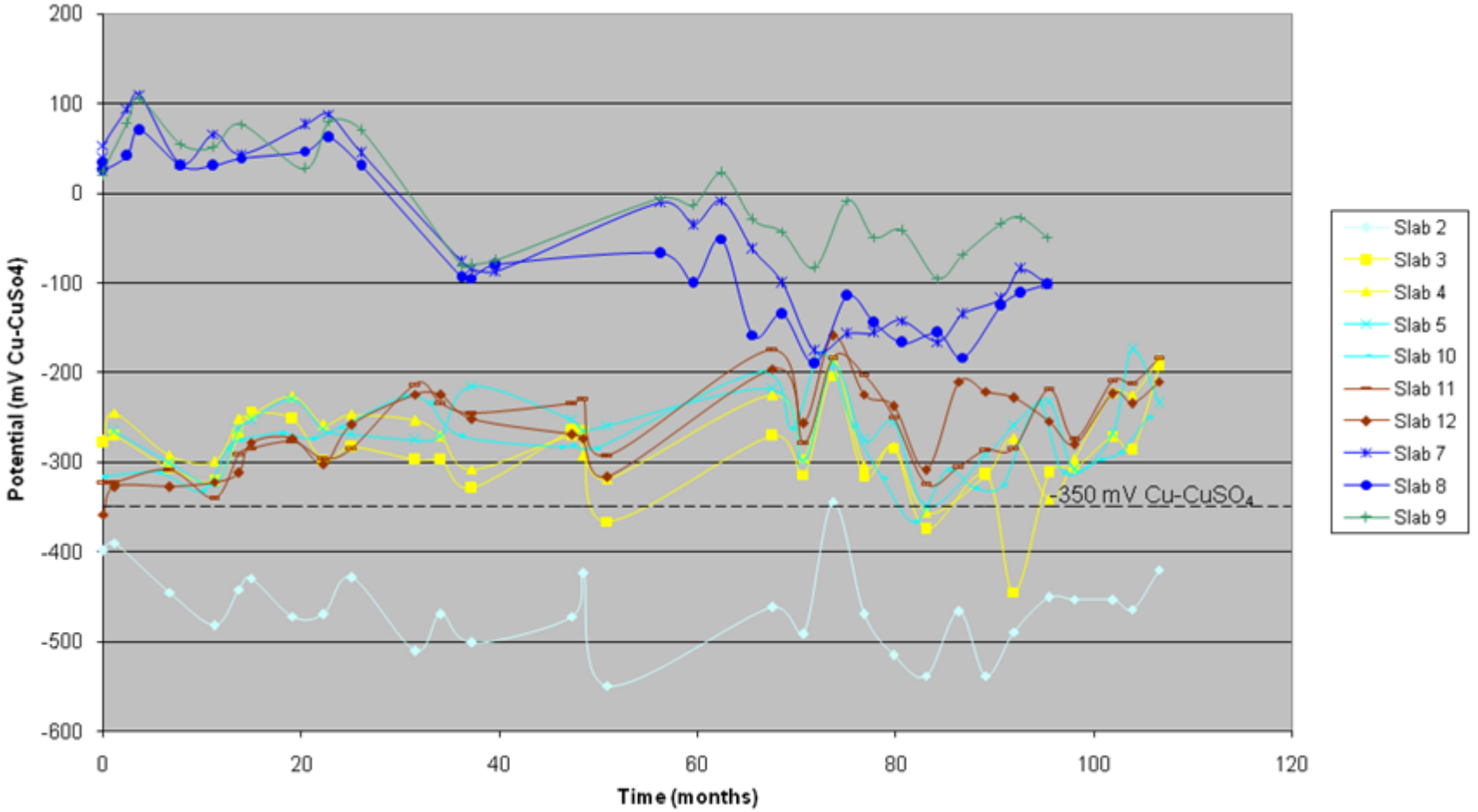


Figure 23. Graph. Average half-cell potential versus time.

Half-Cell Potential Survey

At the start of the SHRP study, 40 half-cell measurements were made per slab with respect to the copper-copper sulfate reference electrode. With the collection of the cores, the number of half-cell measurements reduced with time. In this study, 12 measurements were made on each slab during each evaluation. The potentials on the n-ECE slabs were positive during the SHRP study. Valid half-cell potentials must be negative with respect to the copper-copper sulfate reference electrode. It was concluded that the reason for the unexpected positive readings of the n-ECE slabs was the formation of junction potentials because the pH of the top layer of concrete was not maintained at the normal alkaline levels of concrete.

The average half-cell potential variation for each slab in figure 23 suggests that the slabs can be subdivided into three categories: the first category contains the control slab, the second category contains pH-ECE slabs, and the third category contains n-ECE slabs. As expected, the control slab exhibited active (more negative than -350 mV) potentials, which correlated well with the damage observed, macrocell currents, and driving voltages for the slab.[4] The pH-ECE slabs exhibited average potentials in the uncertain range (-200 to -350 mV) with the exception of slabs 3, 4, and 10. On three occasions for slab 3 and on one occasion for slabs 4 and 10, there were average potentials in the active range.[4] The average potential of the n-ECE slabs steadily decreased (become more negative), and after 35 months of exposure, they exhibited negative values. After 70 months of exposure, slabs 7 and 8 stabilized in the -100 to -200 mV range, whereas slab 9 stabilized in the zero to -100 mV range. This trend can be explained by the migration of ions that resulted in the junction potentials created.

Figure 24 shows the distribution of active potentials measured on each slab. The control slab remained active throughout the study, and 100 percent of its potentials were active. Slabs 3 and 4 showed an increase in active potentials several times during the study, and at one time, all potentials on slab 3 were active. Slabs 5 and 10 showed an increase in active potentials once during the SHRP study and several times after 80 months of exposure. Slabs 11 and 12 exhibited active potentials once during the SHRP study. Macrocell currents and driving volts also showed a corresponding increase in activity for slabs 3–5 and 10.

Corrosion Rate Measurements

SHRP researchers performed two to six corrosion rate measurements on each slab, depending on the type of evaluation. In this study, three measurements were performed on each slab during each evaluation. The linear polarization corrosion rate device (3LP-NBS) was used for all corrosion rate measurements. The average corrosion rate measurements for each slab as a function of time are presented in figure 25.

The average corrosion rate measurements for the n-ECE slabs were in the expected range and did not vary significantly over time. The macrocell currents, driving volts, and visual observations supported the corrosion rates for these slabs. The corrosion rate measurements on the control slab were in the expected range and were supported by the amount of damage observed on the slab. The variation in corrosion rate with time was somewhat in unison with the macrocell currents.

In contrast, the corrosion rates for the pH-ECE slabs were unexpectedly higher than the n-ECE slabs and were similar to the rates for the control slab. The corrosion rate data for these slabs were not supported by other data such as the macrocell currents, driving volts, and the physical condition of the slabs.

Corrosion rate measurements were influenced by many variables including concrete temperature, relative humidity inside the concrete, concrete resistance, etc. After the application of ECE, low corrosion rates are normally expected. As a result, the high corrosion rates observed for pH-ECE slabs contradicted that expectation. Either ECE was not effective or the rate measurements did not accurately reflect the true values. The effectiveness of the ECE process was demonstrated by all other data. Thus, the accuracy of the measurement was suspect. The difference in the corrosion rates measured between pH-ECE slabs and n-ECE slabs suggests that the control of pH impacts the ability to accurately measure corrosion rates. No experimentation was performed either in the SHRP study or in this study to ascertain the impact of pH control during the application of ECE on the measurement of corrosion rate.

The 3LP-NBS device does not have any control over the length of the rebar over which the measurement was made. For the purpose of calculating the corrosion rate, the length and size of the rebar under the probe of the device was used. Generally, the device makes the measurement over a much larger length of the rebar than the length that used in the calculation. The length of the rebar effected by the measurement was dependent on the depth of the rebar and the resistance of the concrete between the probe and the steel. An estimate of resistivity between the probe and steel can be obtained by looking at the resistance between the two mats of steel. Although the resistivity of the top layer of concrete and the layer of concrete between the two mats may not be the same, the change in resistivity probably would be in unison. Resistance between the two mats normalized to 69.8 °F (21 °C) is presented in figure 26, which shows that the resistance of the slabs increased over time. As increase in resistance reduced the error in the measurement of the rate and the actual corrosion currents also reduced. Thus, the increase in resistance should be accompanied by a reduction in the corrosion rate measurements. Figure 24 shows the reduction in corrosion rate with time as resistance increased. The variation in the corrosion rates with time is partially explained by the increase in resistance.

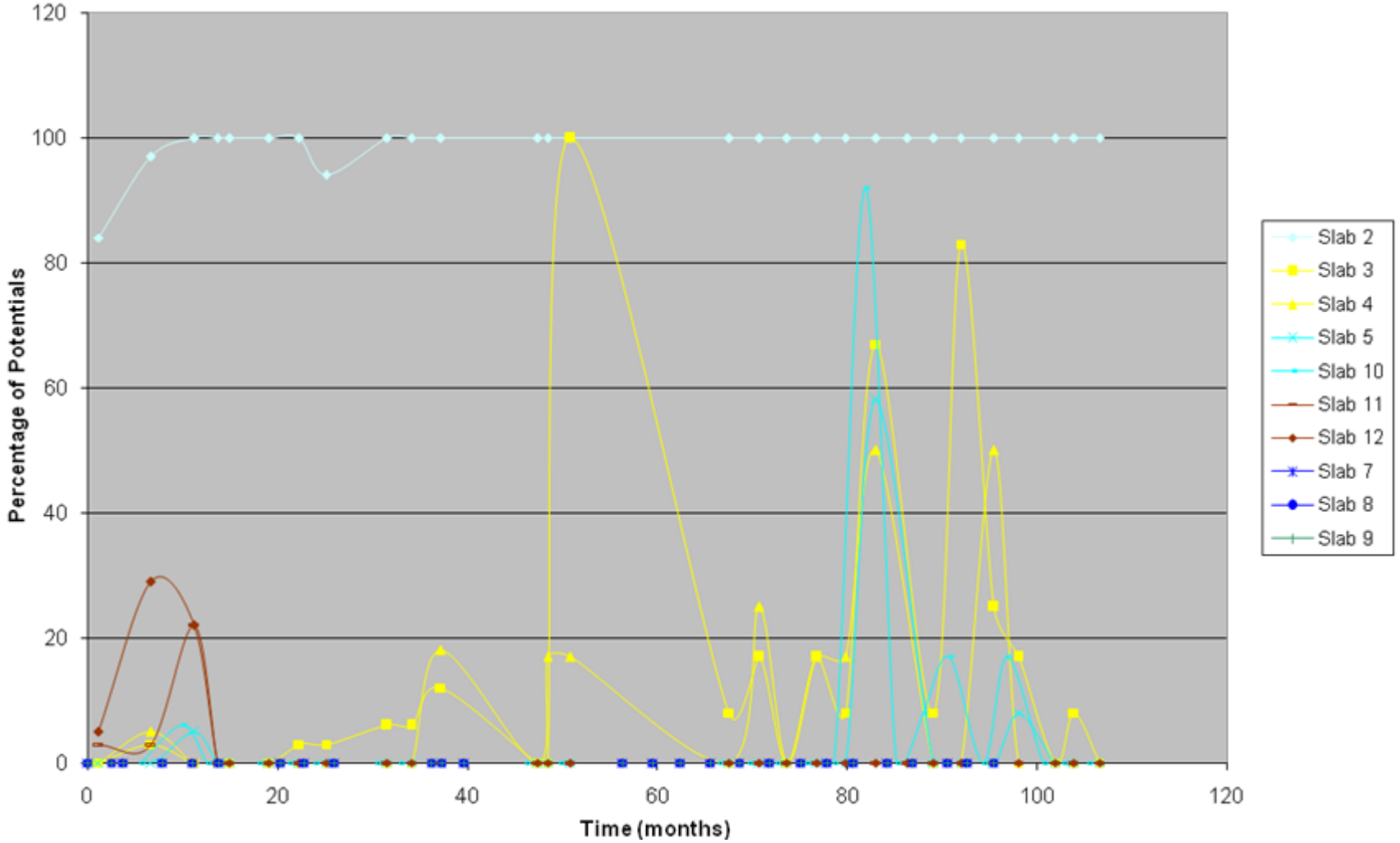
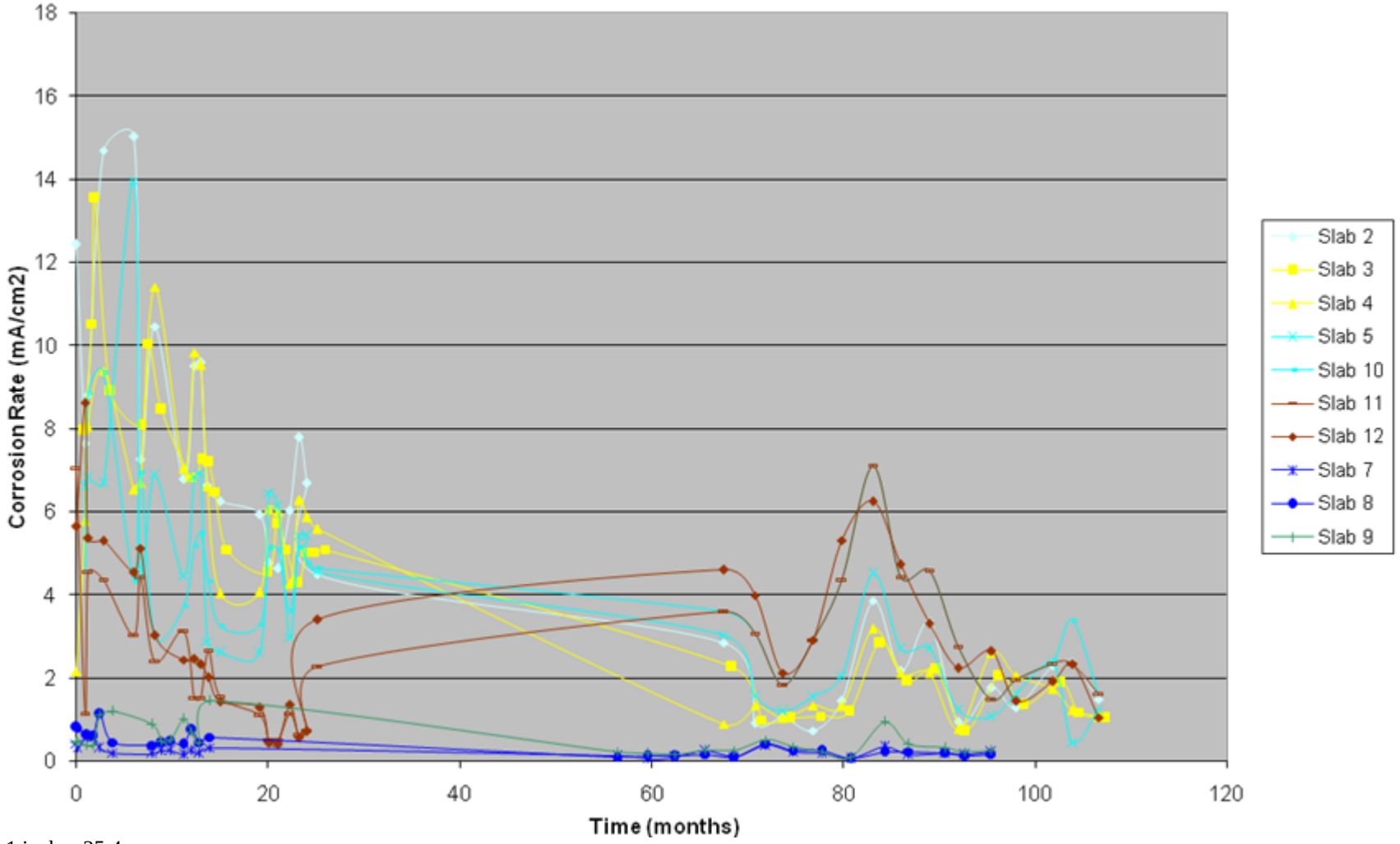


Figure 24. Graph. Percentage of active potentials versus time.



1 inch = 25.4 mm

Figure 25. Graph. Corrosion rate versus time.

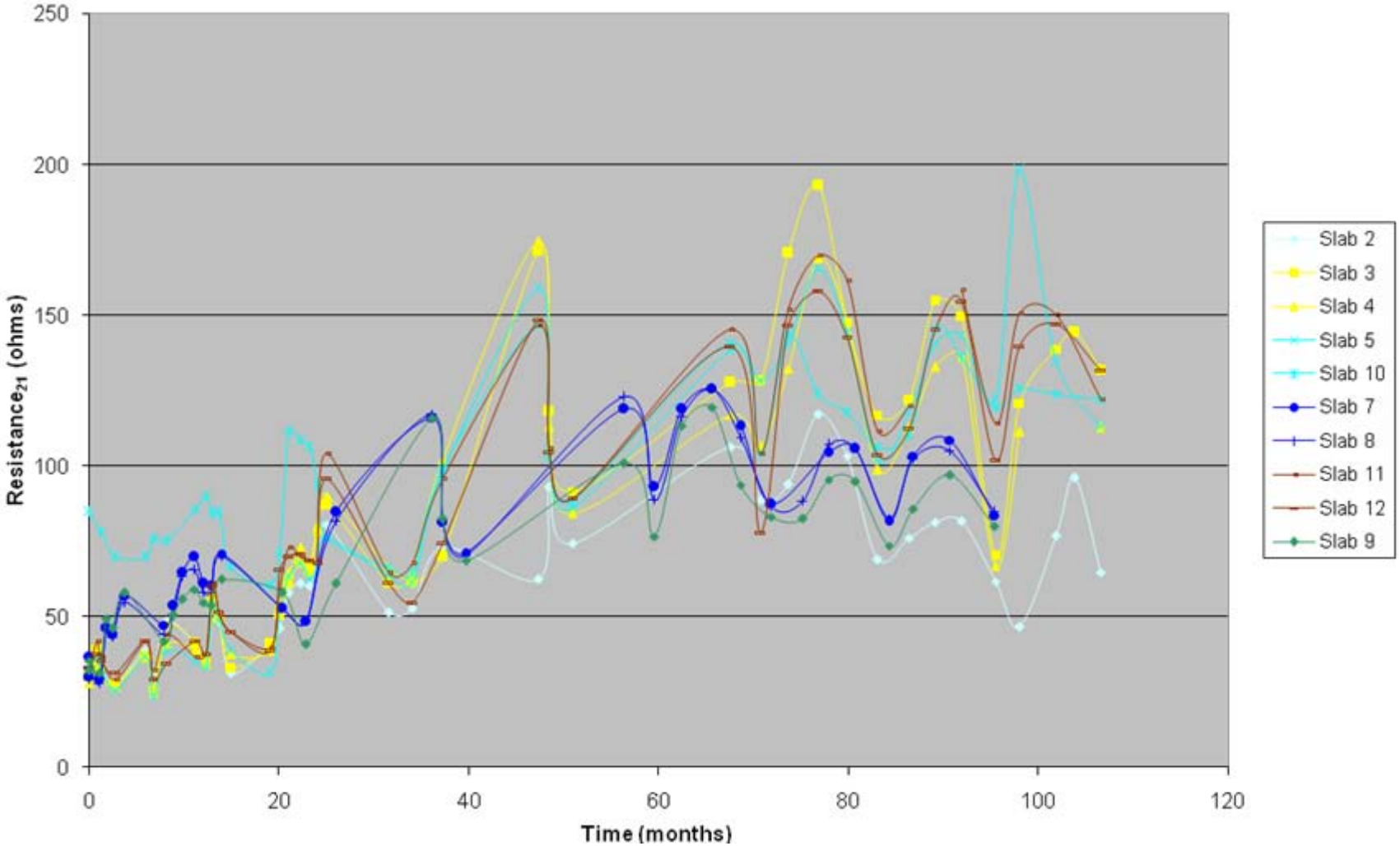


Figure 26. Graph. Resistance rate at 69.8 °F (21 °C) versus time.

CONCLUSIONS

The controlling parameter for initiation of corrosion of steel embedded in concrete is the ratio of chloride ions to hydroxyl ions at the steel-concrete interface. In untreated concrete, the concentration of hydroxyl ions at the steel-concrete interface varied within a narrow range. Because measuring the concentration of the hydroxyl ion at the steel-concrete was difficult, if not impossible, the chloride ion concentration (required to reach the critical chloride/hydroxyl ion ratio at the normally occurring concentration of hydroxyl ions in concrete) was used as the threshold designator. In ECE treated concrete, the hydroxyl concentration at the steel-concrete interface was much higher, and the generally accepted threshold value of chloride ion concentration for corrosion initiation was not applicable.

During ECE treatment, chloride ions were extracted from the concrete, producing a concentration gradient of chloride ions with depth. Simultaneously, the hydroxyl ion concentration at the steel-concrete interface was increased. Typically, the chloride ion concentration at the steel-concrete interface is lowered to levels below the generally accepted threshold of untreated concrete and increases with distance from the rebars in all directions, especially toward the surface where the temporary anode was installed. If the concentration of chloride ions increased and/or the concentration of hydroxyl ions was reduced at the steel-concrete interface, the critical chloride/hydroxyl ion ratio would be exceeded. Thus, the effectiveness of ECE was dependent on remigration and/or ingress of chloride ions and concentration of hydroxyl ions generated during ECE and their dissipation.

Because it was difficult to measure the hydroxyl ion content at the steel-concrete interface, the chloride ion concentration and detection of corrosion on the reinforcing steel must be used to ascertain the effectiveness of ECE. The presence of ongoing corrosion was detected by visual observation of tell tale signs of ongoing corrosion, detection of corrosion induced damage by delamination survey, and measurement of macrocell current, driving volts, half-cell potentials, and corrosion rates.

The chloride data for the ECE slabs indicated that the change in chloride ion concentration at the steel-concrete interface over 10.25 years was minimal (see figure 3). The difference in concentration of chloride ions between slabs 3 and 4 and the remaining slabs was attributed to the lower level of treatment (charge passed) for these two slabs. Figure 2 suggests that some remigration of chloride ions occurred in the third lift of concrete, although it was small in comparison to the control slab. At the end of the study, the concentration of chloride ions in all ECE slabs (except slabs 3 and 4) in the second and third lifts of concrete was uniform, and a sufficient gradient did not exist for significant further diffusion to occur. It should be noted that data in figure 1 and figure 2 came from only one core per slab which did not account for variations from one location to another in a slab. Although the data are limited, they are definite with explainable and expected trends.

A visual and delamination survey differentiated the control slab from the ECE slabs. The control slab suffered significant corrosion-induced damage, and the increase in damage was supported by macrocell, driving volt, half-cell, and corrosion rate data.

Although, cracking was observed on the sides of almost all slabs, it was not necessarily a reflection on the effectiveness of the ECE treatment. It was concluded that corrosion of the rebar at the concrete-air interface was not chloride induced. Instead, it was induced by exposure of the rebar to the environment due to weathering of the epoxy coating. This resulted in the formation of cracks at and near protruding rebars on the sides of the slab. Also, rebar ends exposed in core holes that were epoxy coated corroded and caused some cracking inside them.

Among ECE slabs, slabs 3 and 4 exhibited the highest level of cracking on the top surface. The macrocell, driving volt, and half-cell data indicated that corrosion activity was initiated in these slabs. If the cracking on the sides of slabs was eliminated from the analysis, only a few hairline cracks and rust stains were observed on the top surface of the ECE slabs. Evaluation of the origin of the rust stains on the surface of the ECE slabs indicated that they were not associated with corrosion of the embedded reinforcing steel. The pattern of distribution and the shallow penetration of the stains suggested that they were generated by corrosion of steel filings embedded in small cracks in the surface. The steel filings were produced during the extraction of steel in many of the cores.

The following conclusions were made:

- The chloride ion concentration in the third lift of concrete in the ECE slabs after treatment was significantly lower than the original amount, and diffusion of chlorides had further reduced the concentration gradient to present levels.

- Early signs of corrosion initiation were observed on slabs 3 and 4. The chloride content at the top mat reinforcement was higher in these slabs. Increased cracking on the top surface, slightly higher levels of macrocell current and driving volts, and half-cell potential activity suggest early stages of corrosion initiation.

- With the exception of some early signs of corrosion initiation on slabs 3 and 4, no evidence of corrosion initiation was detected in other ECE slabs. It can be concluded that the concentration of hydroxyl ions remained sufficiently high at the steel-concrete interface to prevent the initiation of corrosion in these slabs. ECE has been effective in mitigating corrosion in these slabs for 10.25 years.

- As minimal future diffusion of the chloride ions is expected, initiation of corrosion in these slabs was dependent on the dissipation of hydroxyl ions. Considering the trend in the macrocell current, driving volt, half-cell potential data, and especially the resistance between the two mats, it is possible that the present levels of corrosion activity in the ECE slabs can be maintained for another 10 years.

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CHAPTER 3. ECE APPLICATION ON BRIDGE DECKS

Two bridge decks, one located in Lucas County, OH, and the other located in Arlington, VA, were monitored to evaluate the effectiveness of ECE on bridge decks. There are some similarities between the two structures. Both are small with low average daily traffic. As a result, it was not an insurmountable problem to close one lane of the bridges for several weeks while the decks were undergoing ECE treatment. However, the exposure conditions for the two structures vary, and different anode-electrolyte configurations were used in these two structures. Additionally, different technologies (i.e., thin epoxy overlay versus silica fume concrete overlay) were used on the structures after the application of ECE to prevent future ingress of chloride ions. The use of an epoxy overlay on the Ohio bridge deck prevented the performance of electrochemical tests such as half-cell potential survey and corrosion rate measurements.

Post-ECE evaluations for both bridges were performed soon after the completion of the treatment. No other evaluations were performed in the interim between the post-ECE evaluation and the start of this study in 1994. When available and appropriate, the post-evaluation data collected by other researchers were included in the analysis for comparison. All structure background information and ECE application information has been obtained from appropriate reports and bridge information provided by local bridge owners.

NEAPOLIS/WATERVILLE BRIDGE, LUCAS COUNTY, OH

Structure Background

A two-lane bridge for SR-295 over Blue Creek near the intersection of SR-295 and Waterville-Neopolis Road in Lucas County, OH, was constructed in 1965 (see figure 27). The bridge is aligned in the north-south direction and is comprised of a continuously reinforced concrete deck supported by two piers. The 30-degree skewed deck is 72.98 ft (22.25 m) long and 35.98 ft (10.97 m) wide, carries two 12.99-ft (3.96-m)-wide traffic lanes, and has a 4.99-ft (1.52-m)-wide median on each side. The total surface area of the deck is 2,635.2 ft^2 (244 m^2).

ECE Application

A corrosion condition evaluation of the deck conducted in March 1991 documented a delaminated area of 148.81 $inches^2$ (93,005.58 mm^2).[(1)] The average chloride content varied from 5,820 ppm at a nominal depth of 0.55 inches (14 mm) to 860 ppm at a nominal depth of 2.54 inches (65 mm). Corrosion potential mapping identified 18 percent of the deck area to have a 90 percent probability of active corrosion. Corrosion rate measurements taken at four locations on the deck were in the high range (6.90 to 11.4 $\mu A/inches^2$ (1.07 to 1.78 $\mu A/cm^2$)), which are considered to result in concrete damage within 2 to 10 years.[(1)] Petrographic analysis determined that the deck concrete was of excellent quality considering materials, consolidation, air entrainment, water-cement ratio, and cement maturity.

In September 1991, an ECE treatment was applied to the eastern half of the bridge deck covering 1,317.6 ft^2 (122 m^2). The western half was not treated and was designated as a control.

Figure 27. Photo. Deck surface of bridge carrying SR-295 over Blue Creek in Lucas County, OH.

For the application of ECE, a catalyzed titanium mesh was used as an anode, and 0.08 M sodium borate solution was used as an electrolyte. The electrolyte was confined to the treated areas in a pond built on the surface, and the anode was submerged in the electrolyte. At the beginning of the application, the solution in the pond was exposed to the environment. To insulate and maintain a slightly higher electrolyte temperature, a plastic cover was installed over the pond. The cover was also expected to protect the electrolyte from dilution by rain. Windy conditions at the site made it difficult to maintain the cover, and it was removed after several weeks. An adjustment of the pH of the electrolyte was made once during the application. The startup current density of 0.07 A/ft^2 (0.7 A/m^2) was slightly lower than expected, and it reduced to 0.03 A/ft^2 (0.3 A/m^2) over time (see table 2). The lower current densities were attributed to an increase in concrete resistance due to colder temperatures during application.

Table 2. ECE application parameters in Lucas County, OH.

Description	North Zone	South Zone
Applied voltage (volts)	48–50	48–50
Initial current density (A/m^2)	0.7	0.7
Final current density (A/m^2)	0.3	0.3
Length of application (weeks)	9	9
Total charge (A-h/m^2)	640	610
Efficiency (percent)	20	20

1 A/m^2 = 0.093 A/ft^2
1 A-h/m^2 = 0.093 A-h/ft^2

The application was intended to accumulate a total charge of 93 A-h/ft^2 (1,000 A-h/m^2). Due to the difficulties encountered and the impending colder weather, the application was terminated when total charge accumulated exceeded 56 A-h/ft^2 (600 A-h/m^2). During summer 1992, a 0.25-inch (6.35-mm)-thick epoxy overlay was applied to the deck surface to prevent further ingress of chlorides into the concrete.

Field Evaluation

The following three field evaluations were performed in this study:

- First evaluation: April 17, 1995 (approximately 4 years after treatment).
- Second evaluation: March 27, 1997 (approximately 6 years after treatment).
- Third evaluation: April 21, 1998 (approximately 7 years after treatment).

Test Results

Visual Survey

No damage was observed on the deck surface during the three evaluations. However, cracking, rust staining, and spalling were found during the third evaluation of the soffit of the bridge deck (see figure 28).

Delamination Survey

A complete delamination survey was conducted on the entire surface of the bridge deck during each evaluation, and no delaminations were detected. Additionally, a delamination survey was performed on assessable sections of the soffit adjacent to the abutments during the third evaluation. Delaminated area totaling 23.11 ft^2 (2.14 m^2) was detected on the soffit of the treated section of the deck and on 6.05 ft^2 (0.56 m^2) on the soffit of the control section of the deck (see figure 29).

Figure 28. Photo. Cracking, rust staining, and spalling on the soffit of the bridge.

Figure 29. Photo. Delaminated areas (marked in red) on the soffit of the bridge.

Clear Concrete Cover Survey

A clear concrete cover survey was performed during the first evaluation using a cover meter. The results of the cover meter were verified with actual cover measurements obtained from cores extracted with embedded steel. The clear concrete cover ranged from 2.4 to 3.2 inches (61 to 81.3 mm) and averaged 2.7 inches (68.6 mm).

Chloride Ion Content Analysis

Prior to the application of ECE, one 3-inch (76.2-mm)-diameter core was collected and analyzed for chloride ion content at regular intervals of depth. After application of ECE, four cores were collected and similarly analyzed.[1] For the purpose of this report, chloride data collected by SHRP researchers after the application of ECE were post-treatment data.

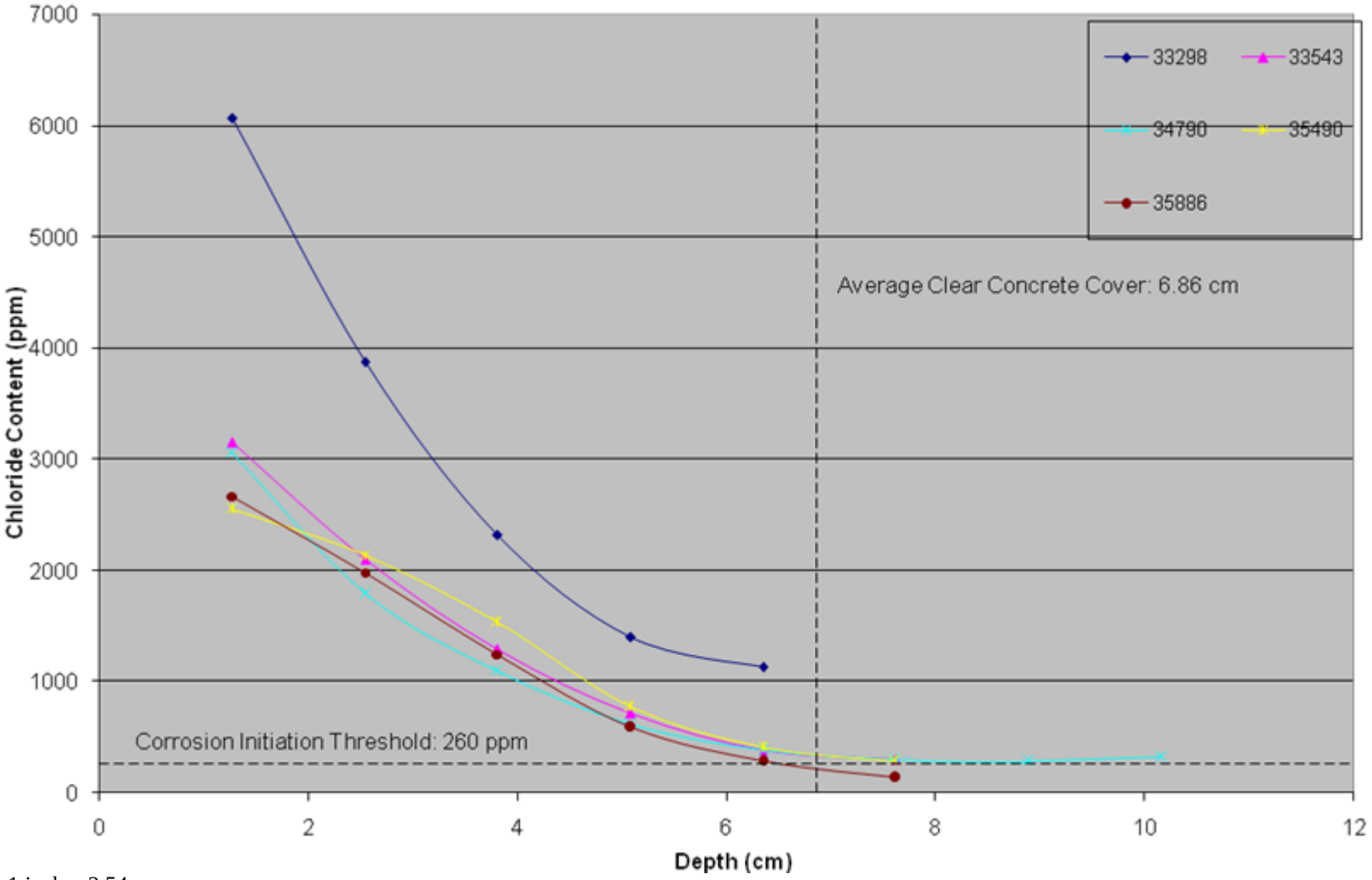
During each of the first 2 evaluations, 11 3-inch (76.2-mm)-diameter cores were collected, 6 from treated areas and 5 from areas designated as control. During the third evaluation, 12 2-inch (50.8-mm)-diameter cores were collected, 6 from treated areas and 6 from the control areas. Powdered concrete samples were collected from all cores at regular intervals of depth and analyzed for chloride ion content.

As powdered concrete samples were not collected from the same nominal depths from each of the post-treatment cores, the chloride content data were extrapolated to fixed nominal depths to allow averaging of the results of the four cores. The average chloride profiles from pre- and post-treatment cores and cores from the three evaluations are presented in figure 30 and figure 31.

Figure 30 shows that the chloride content at the same depths after ECE was lower, and some diffusion of chloride ions may have occurred in the top 0.8 inches (20.3 mm) of the concrete. In general, the average chloride content profiles of the post-treatment cores and the cores collected in the three evaluations were similar.

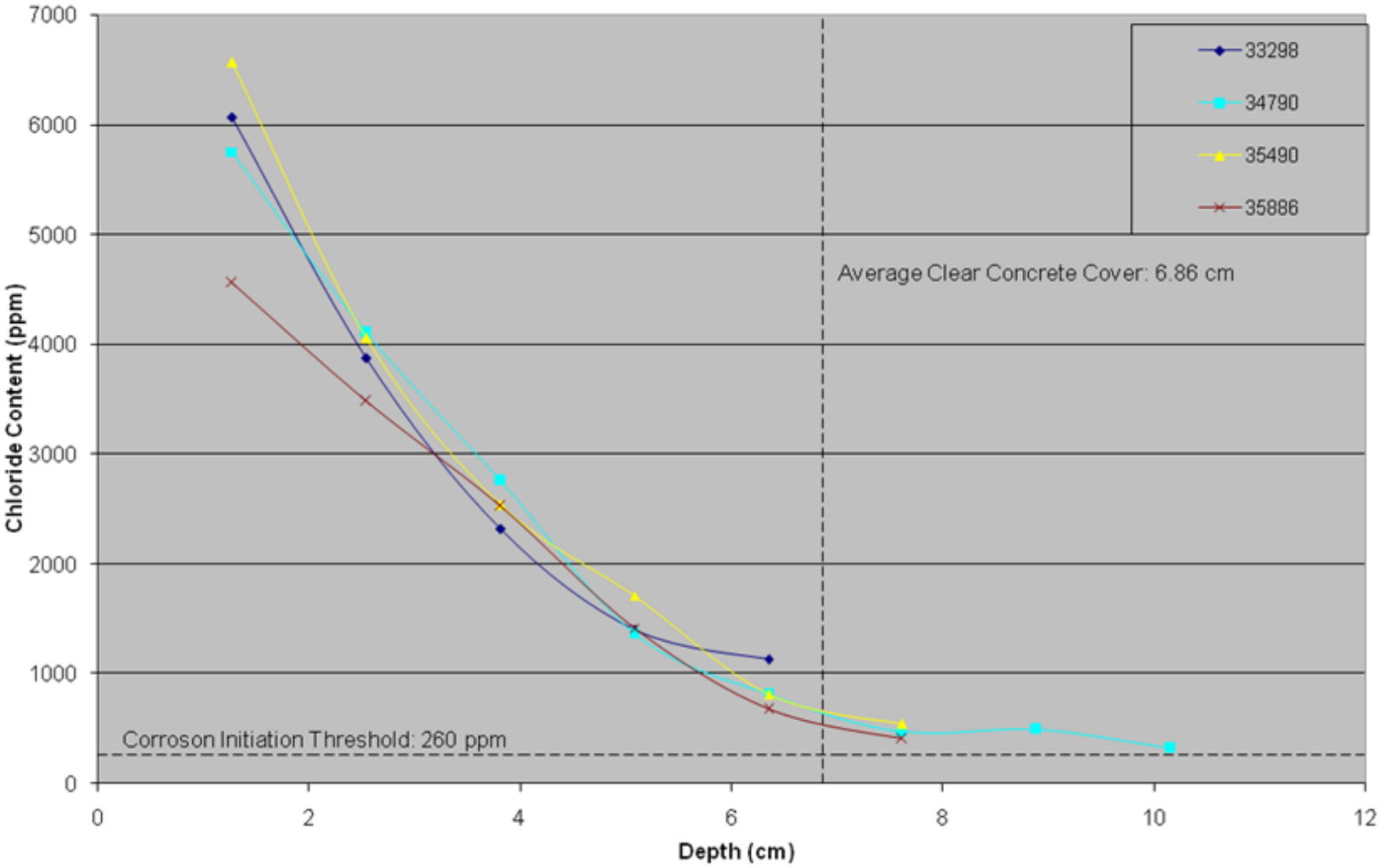
In figure 31, the pre-treatment core and the cores collected from the untreated section of the bridge deck showed similar chloride content profiles, suggesting that some diffusion of chloride ions may have occurred in the top 0.8 inches (20.3 mm) of the concrete.

For approximately 7 years after the application of ECE, chloride content at steel depth has remained above the threshold for corrosion initiation by 260 ppm in the treated and the control areas. The chloride ion content at the steel depth in the control area was higher than that of the treated area.



1 inch = 2.54 cm

Figure 30. Graph. Chloride profiles of ECE treated area versus time.



1 inch = 2.54 cm

Figure 31. Graph. Chloride profiles of control area versus time.

Conclusions for Lucas County, OH

The chloride ion content data indicate that the ECE process successfully removed a significant amount of chloride ions from the concrete above the top mat reinforcement and lowered the chloride ion content at the steel depth compared to the untreated control area.

The presence of the thin epoxy overlay had several impacts on the evaluation of this site. The overlay prevented electrochemical measurements such as half-cell potentials and corrosion rate measurements that could provide the information on the difference in the corrosion state of the two halves of the deck. It also made it difficult to detect delaminations under the overlay. The presence of the overlay impacted the ongoing corrosion process in the deck either by slowing it down or not allowing it to accelerate through keeping the needed moisture out of the concrete. Thus, it was not possible to determine the effectiveness of the ECE process in mitigating corrosion. Much longer term monitoring may show the difference between the control area and the treated section of the deck.

The treatment on this site can be compared to that of slabs 3 and 4 of the laboratory study. Slabs 3 and 4 had undergone 59.9 A-h/ft^2 (645 A-h/m^2) of ECE treatment compared to an average of 58.1 A-h/ft^2 (625 A-h/m^2) for this site. The anode used in both treatments was the same (inert anode coating on titanium substrate). Although the electrolytes that were used were different (0.1 M sodium hydroxide for slabs 3 and 4 and 0.08 M sodium borate for this site), the pH in both treatments were maintained in the alkaline range. Corrosion rate data collected after the application of ECE in the treated areas were higher than expected, but not as high as that of slabs 3 and 4. The chloride ion content at the top mat of reinforcement in the treated area of this site was much lower than that of slabs 3 and 4 (less than 400 ppm for the site and over 700 ppm for the slabs). Based on the performance of the slabs, the corrosion activity in the treated area can be expected to remain very low for more than 10.25 years after ECE treatment.

34TH STREET BRIDGE, ARLINGTON, VA

Structure Background

The 34th Street bridge in Arlington, VA, was constructed in 1968 to carry east- and westbound traffic over I-395. The 361.92-ft (110.34-m)-long and 57.99-ft (17.68-m)-wide deck consists of five simple spans supported by steel beams. It has a curb-to-curb width of 47.99 ft (14.63 m) and a deck surface area of 17,431.2 ft^2 (1,614 m^2). Although the bridge is wide enough to carry four lanes of traffic, it only carries one lane in each direction.

ECE Application

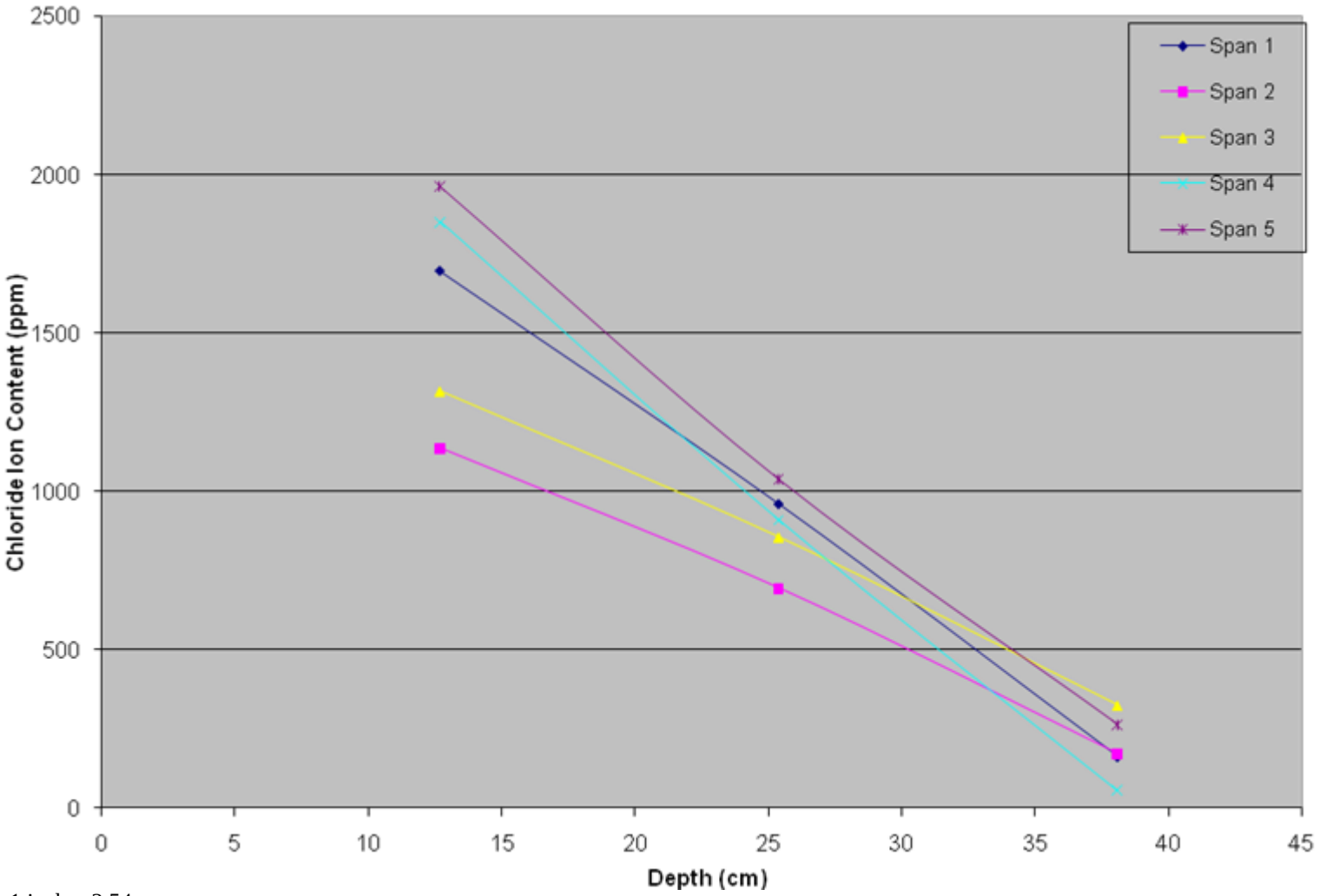
The Virginia Research Council and FHWA considered using the 34th Street bridge for a pilot application of ECE. The FHWA Demonstration Project #84 Team was requested to conduct a corrosion and concrete condition evaluation to ascertain the suitability of the bridge deck for ECE application. A thorough evaluation was performed in September 1993. Based on the results of the evaluation, the team recommended the application of ECE on the deck.[(2)]

Visual observation of the deck surface detected the presence of concrete scaling, corrosion-induced and shrinkage cracks, rust staining, and spalls with corroded reinforcing steel. The

delamination survey documented damage varying from 2 to 29 percent of the deck surface area in the five spans. In general, spans 4 and 5 exhibited the least amount of corrosion-induced damage (5 and 2 percent delamination, respectively, and lower amounts of spalling and rust staining), and spans 1 and 3 exhibited the most corrosion-induced damage (22 and 29 percent delamination, respectively, and a larger amount of spalling and rust staining). Span 2 exhibited a relatively larger amount of rust staining but a lower amount of delamination (7 percent).

The average clear concrete cover varied from 0.75 to 1.8 inches (19 to 45 mm) and averaged 1.3 inches (33 mm). The average chloride content extrapolated to the average steel depth of 1.3 inches (33 mm) was 971 ppm. Corrosion potential mapping indicated that 78 percent of the deck had a low probability of corrosion activity, and 3 percent had a high probability of corrosion activity. The corrosion status of the remaining 19 percent was uncertain. In total, 48 corrosion rate measurements were made, with 68 percent of these measurements in the passive range and 32 percent in the low to moderate range.

Rehabilitation of the bridge deck was performed between March and September 1995. All unsound concrete was removed and patched with a cementitious material. Budgetary constraints did not allow application of ECE to the entire bridge, and the treatment was limited to spans 4 and 5, totaling 7,776 ft^2 (720 m^2) of deck surface area. Spans 1–3 were designated as controls. Although spans 4 and 5 had lower levels of corrosion-induced damage, the chloride content interpolated to the average steel depth of 1.3 inches (33 mm) was 649 and 929 ppm compared to 819, 654, and 912 ppm for spans 1–3, respectively. Also, chloride content at lower depths in spans 4 and 5 were generally higher than the other spans (see figure 32). The lower levels of damage on spans 4 and 5 made them more advantageous for ECE application.



1 inch = 2.54 cm

Figure 32. Graph. Pretreatment chloride profiles.

To maintain traffic flow over the structure, one lane was treated at a time.[3] The eastbound lane (northern half of the deck) was treated first (see figure 33). Several treatment areas were created, two on the north half of the bridge, one in each span, and two on the south half of the deck of each span. A catalyzed titanium mesh sandwiched between two layers of synthetic felt was used. The anode was placed in a wooden dam built to contain the electrolyte on the surface of the deck. A plastic sheet was used to cover and protect the anode and the electrolyte. Pumps and hoses were used to circulate the electrolyte. Three different electrolytes were used, including lithium borate on the northern half of span 4 and calcium hydroxide on the southern half of both spans. On the northern half of span 5, tetraphenol phosphonium chloride solution ($(C_6H_5)_4P^+Cl^-$), which is a corrosion inhibitor, was applied on the deck surface prior to the placement of the anode system and lithium borate solution. Post-treatment analysis of powdered concrete samples collected from a depth of 0.75 to 1.25 inches (19 to 32 mm) indicated the presence of lithium borate in concentrations as high as 265 ppm and corrosion inhibitor in concentrations lower than the detectable limit of 25 ppm. Approximately 2 weeks after the ECE treatment, a 1.99-inch (51-mm)-thick silica fume overlay was placed on the entire deck.

The initial current densities for the treatment areas varied from 0.07 to 0.09 A/ft^2 (0.7 to 0.9 A/m^2), and the final current densities varied from 0.04 to 0.07 A/ft^2 (0.4 to 0.7 A/m^2). The total charge accumulated for each treatment area varied from 68.9 to 100.1 A-h/ft^2 (741 to 1,077 A-h/m^2), and chloride removal efficiency varied from 11.2 to 15.0 percent (see table 3).[4]

Figure 33. Photo. Northern half of the eastbound lane of 34th Street bridge in Arlington, VA, during ECE treatment.

Table 3. ECE application parameters in Arlington, VA.

Description	Span 4 North	Span 5 North	Span 4 South	Span 5 South
Size (m^2)	174	183	180	183
Applied voltage (volts)	30	33	29	37
Initial current density (A/m^2)	0.93	0.91	0.85	0.73
Final current density (A/m^2)	0.66	0.59	0.54	0.43
Length of application (weeks)	8.2	8.2	8.2	8.2
Total charge ($A\text{-}h/m^2$)	1,077	1,033	1,019	741
Efficiency (percent)	11.2	15.0	11.7	14.2

1 m^2 = 10.8 ft^2
1 A/m^2 = 0.093 A/ft^2
1 $A\text{-}h/m^2$ = 0.093 $A\text{-}h/ft^2$

Field Evaluation

Two field evaluations were performed, and spans 3–5 were evaluated. The field evaluations were performed on the following dates:

- First evaluation: May 21, 1996 and June 5, 1996 (approximately 1 year after treatment).
- Second evaluation: August 27–28, 1997 (approximately 2 years after treatment).

Test Results

Visual Survey and Delamination Surveys

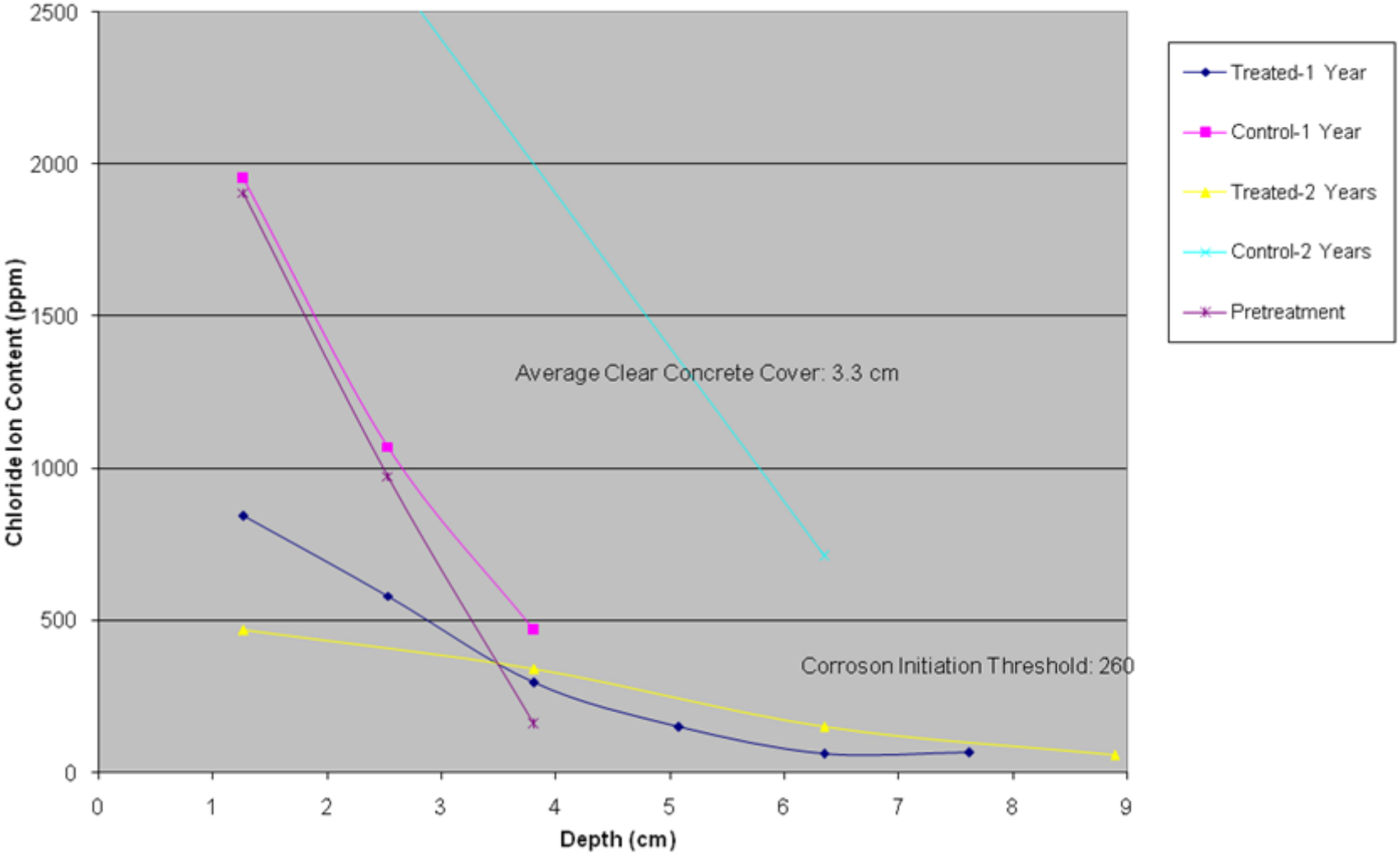
No visually observable damage or delaminated areas were noted in any span during either of the evaluations.

Chloride Ion Content Analysis

During the first evaluation, 12 cores were collected for chloride content analysis, 2 from span 3 and 5 each from spans 4 and 5. In the second evaluation, 14 cores were collected, 6 from span 4 and 4 each from spans 3 and 5.

Span 3 suffered the most damage (29 percent delamination in 1993) and thus had the largest amount of repairs. Some of the cores (one from the first evaluation and the remaining from the second) collected from span 3 contained patch repair material. The chloride profile data for these cores was not representative of the actual chloride content of the original deck concrete. Data from cores containing patch material were excluded from the analysis.

Chloride profile data presented in figure 34 indicate that the chloride ion content in span 3 was similar to conditions before 1995, while chloride content in the treated areas was much lower and diffusion of chloride ions occurred over time. The average chloride ion content at the average steel depth of 1.29 inches (33 mm) was in excess of the threshold of 260 ppm required to initiate corrosion.



1 inch = 2.54 cm

Figure 34. Graph. Chloride profiles versus time.

Corrosion Potential Survey

Half-cell potential data were collected on a 5-ft (1.53-m) grid in accordance with ASTM C-876-91, and the results from pre- and post-treatment evaluations are presented in figure 35.[(5)] This data show that after rehabilitation, the potential of the reinforcing steel shifted to the passive range. Shift in potential towards the passive range indicated a reduction in the propensity for corrosion. This was not only true for the ECE-treated areas but also the untreated deck of span 3.

Corrosion Rate Survey

Gecor 6 was used to measure the corrosion rate at 8–10 locations per span. Pretreatment average corrosion rate measurements and the two evaluation trip average corrosion rate measurements are presented table 4. With the exception of one average in the passive range (less than 0.6 μA/inches^2 (0.1 μA/cm^2), all other averages were in the low range (0.6 to 3 μA/inches^2 (0.1 to 0.5 μA/cm^2)). In terms of corrosion rate measurements, there was no change from the pre-treatment status.

Table 4. Average corrosion rates.

Span	1993	1996	1997
Span 3	0.12	0.25	0.27
Span 4	0.08	0.26	0.16
Span 5	0.12	0.33	0.22

Conclusions for Arlington, VA

Sufficient length of time had not passed since the application of ECE to determine its effectiveness. The chloride ion distribution data suggest that a significant amount of chloride ions were removed from the concrete above the top mat reinforcement, and the chloride ions diffused over time. Long-term monitoring of this site is recommended to document its performance.

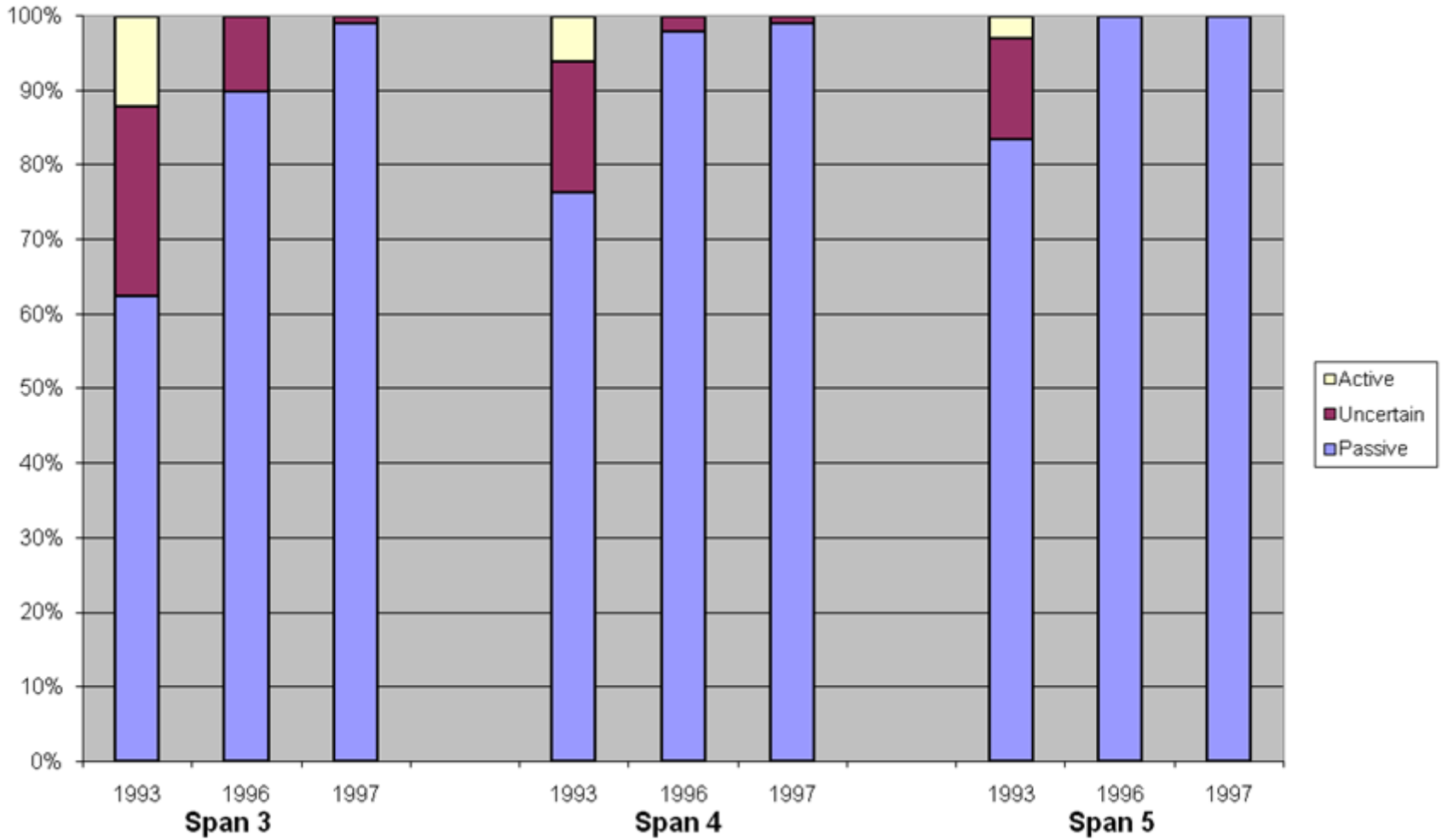


Figure 35. Graph. Half-cell potential distribution versus time.

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CHAPTER 4. ECE APPLICATION ON BRIDGE SUBSTRUCTURE ELEMENTS

CAMPUS LOOP BRIDGE, ALBANY, NY

Structure Background

The Campus Loop Bridge was constructed in Albany, NY, in 1962 and carries traffic from Hawkins Street in the east-west direction over NY-85. This slab-on-stringer bridge has two 18.04-ft (5.5-m)-wide travel lanes and is approximately 390.32 ft (119 m) long. Two abutments and four piers support five spans.

The piers are numbered from east to west, with pier 2 located in the median of NY-85. Each pier consists of two column bents, one on each side of the centerline of the deck (see figure 36). Each bent has three 42.1-inch (1,069.3-mm)-diameter columns, which are approximately 14.1 ft (4.3 m) high and vary in height from one end of the pier to the other. The columns are numbered 1 through 6 starting from the north and are supported by a raft foundation. The reinforcement in the columns is comprised of 13–17 vertical bars with a diameter of 1.1 inches (27.9 mm) and spiral ties with a diameter of 0.5 inches (12.7 mm).

ECE Application

In August 1991, a corrosion condition evaluation of the six columns of pier 2 was performed.[(1)] Fifteen percent of the column surface area was delaminated. The average chloride ion content was determined to be 1,967 ppm in the top 2 inches (50.8 mm) of concrete, and 74 percent of the corrosion potentials that were measured were found to be active. Linear polarization corrosion rate measurements suggested that damage in sound areas could be expected to occur within 2 to 10 years.

In May 1992, all delaminated concrete from subject columns was removed to the rebar level and repaired by patching with a cementitious material. In June 1992, electrochemical chloride extraction treatment developed under the SHRP program was applied to the six columns on pier 2. The pier cap beams were not treated.

Each column was wrapped by a 10.99-ft (3.35-m)-long anode/blanket composite, which was covered by a plastic film, and a 0.3 M sodium borate buffered electrolyte was used. The anode/blanket composite contained three layers of blanket material and the anode. The first layer was designed to retain the electrolyte and provide contact. A titanium mesh anode was sandwiched between the remaining two layers of blankets. The electrolyte was pumped to the top of the anode/blanket composite and allowed to flow down the column by gravity and collect in a sump. The containment of the electrolyte within the anode/blanket assembly was paramount to the success of the treatment. On columns 1 and 4, the containment could not be accomplished. Because these columns could not be treated, they were designated as untreated controls. For the application of ECE, two electrical zones were established, each containing two columns.[(1)] Zone 1 included columns 2 and 3, and zone 2 contained columns 5 and 6.

Figure 36. Photo. Pier 2 of the Campus Loop Bridge in Albany, NY.

Current densities in the two electrical zones ranged from 0.10 to 0.30 A/ft^2 (1.1 to 3.2 A/m^2), and they accumulated total charges of 80 and 93 $A\text{-}h/ft^2$ (860 and 1,000 $A\text{-}h/m^2$). Chloride ion removal for the two zones was estimated to be 0.32 to 0.52 oz/ft^2 (97 and 161 g/m^2), and current efficiencies were calculated to be 7 and 13 percent, respectively.

In addition to the containment problems, dissolution of the titanium anode at random locations suggesting high-localized current densities, cracking of the patches, and lowering of electrolyte pH to 3 were also observed.

Field Evaluation

The following three field evaluations were performed:

- First evaluation: July 13 and 14, 1995 (approximately 3 years after treatment).
- Second evaluation: August 15 and 16, 1996 (approximately 4 years after treatment).
- Third evaluation: May 2 to 4, 1998 (approximately 6 years after treatment).

Test Results

Visual Survey

During the first visit, patched areas on all columns were identified, and cracks were observed in all patched areas (see figure 37). A majority of cracks in patched areas were considered to be shrinkage cracks and did not exhibit signs of ongoing corrosion. During the second trip, a significant increase in cracking was observed, and many of the cracks inside and outside patched areas exhibited rust staining (see figure 38 through figure 40). Further increases in cracking and rust staining were observed during the third visit. Total cumulative crack length for cracks outside patched areas is presented in table 5.

Figure 37. Photo. Cracking on the patched areas on one of the columns of the Campus Loop Bridge in Albany, NY, during the first evaluation in 1995.

Figure 38. Photo. First view of cracks and rust staining in patched and nonpatched areas of the columns of the Campus Loop Bridge in Albany, NY, during the second visit in 1996.

Figure 39. Photo. Second view of cracks and rust staining in patched and nonpatched areas of the columns of the Campus Loop Bridge in Albany, NY, during the second visit in 1996.

Figure 40. Photo. Cracks and rust staining in a nonpatched area of a column of the Campus Loop Bridge in Albany, NY, during the second visit in 1996.

Table 5. Cumulative length of cracks observed outside patched areas.

Column	1995 (inches)	1996 (inches)	1998 (inches)
Column 1 (Control)		119	119
Column 2		43	93
Column 3		42	118
Column 4 (Control)		84	106
Column 5		16	22
Column 6	50	325	444

1 inch = 25.4 mm
Note: Blank cells indicate that no measurement is available.

Prior to the application of ECE, cracks were observed on all columns, with column 5 exhibiting the least amount of cracking and column 6 exhibiting the most. There is no indication in the SHRP reports that these cracks were repaired prior to or after the application of ECE. The data presented in the table do not suggest any difference between treated and untreated columns.

Joints in the bridge deck were leaking, and water was flowing down the pier caps and the columns (see figure 41). The water flowing through the leaking joints in winter was probably contaminated with deicing salts. It can be concluded that chloride ions from the contaminated water were introduced into the column concrete. Shrinkage cracks were observed in the patched areas immediately after the completion of the ECE treatment. It was suspected that chloride contaminated electrolyte may have entered the patched areas through these cracks and deposited chloride ions.

Figure 41. Photo. Water flowing down the pier caps and columns through leaking joints.

Delamination Survey

A delamination survey was performed on each column during each evaluation. By 1995, all patches on all columns were completely delaminated (see figure 42 through figure 44). In 1996, delaminations extended from the patches into the original concrete. Delaminations not associated with any patches were also detected. Cumulative percent delaminations as a function of time are presented in table 6.

Figure 42. Photo. First view of a completely delaminated patch (marked in yellow) on a column of the Campus Loop Bridge in Albany, NY.

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Figure 43. Photo. Second view of a completely delaminated patch (marked in yellow) on a column of the Campus Loop Bridge in Albany, NY.

Figure 44. Photo. Third view of a completely delaminated patch (marked in yellow) on a column of the Campus Loop Bridge in Albany, NY.

Table 6. Cumulative percentage of delaminations.

Column	1995 (ft^2)	1996 (ft^2)	1998 (ft^2)
Column 1 (Control)	20	30	30
Column 2	5	6	6
Column 3	6	13	13
Column 4 (Control)	16	23	23
Column 5	5	9	10
Column 6	8	20	20

1 m^2 = 10.8 ft^2

From 1995 to 1996, delaminations significantly increased in four of the six columns. A majority of the increase in the delaminations was attributed to an extension of delaminations from patches to the surrounding concrete and delaminations along cracks. Considering that the patches had suffered shrinkage cracking at or before ECE application and chloride contaminated electrolyte may have been absorbed into the patches, it can be hypothesized that excessive chlorides introduced into the patches in this manner were the primary reason for their failure. Also, chloride ions from the contaminated runoff water were entering cracks and resulted in corrosion-induced delaminations. It should be noted that no form of barrier to chloride ions was installed on the columns after the application of ECE.

Clear Concrete Cover Survey

Clear concrete cover over reinforcing steel was documented when the steel was exposed for electrical continuity testing. Average clear cover over reinforcing steel was 2.4 inches (61 mm). Average cover for each column ranged from 2.2 to 2.8 inches (55.9 to 71.1 mm).

Chloride Ion Content Analysis

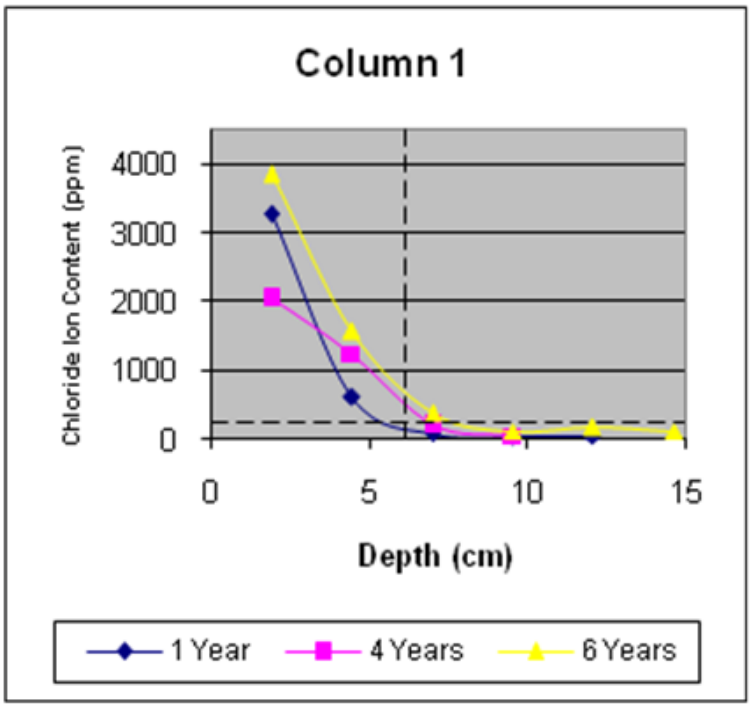
During the first evaluation in 1995, one 3-inch (76.2-mm)-diameter core was collected from a height of 2.98 to 4.30 ft (0.91 to 1.31 m) on the south side of each column. In the later two evaluations, two 3-inch (76.2-mm)-diameter cores were collected, one from the north side and one from the south side of the columns. These cores were also collected from heights of 2.98 to 4.00 ft (0.91 to 1.22 m).

Powdered concrete samples were collected from each core at varying depths. The powered concrete samples were analyzed in accordance with procedures prescribed in AASHTO T-260 standard practice.[(2)]

Figure 45 through figure 50 present the average variation of chloride ion content as a function of depth for each column. The data suggest that chloride ion content generally increased with time for all columns. This was expected as columns were exposed to chloride contaminated water runoff and splash. The chloride ions continued to migrate into the column concrete.

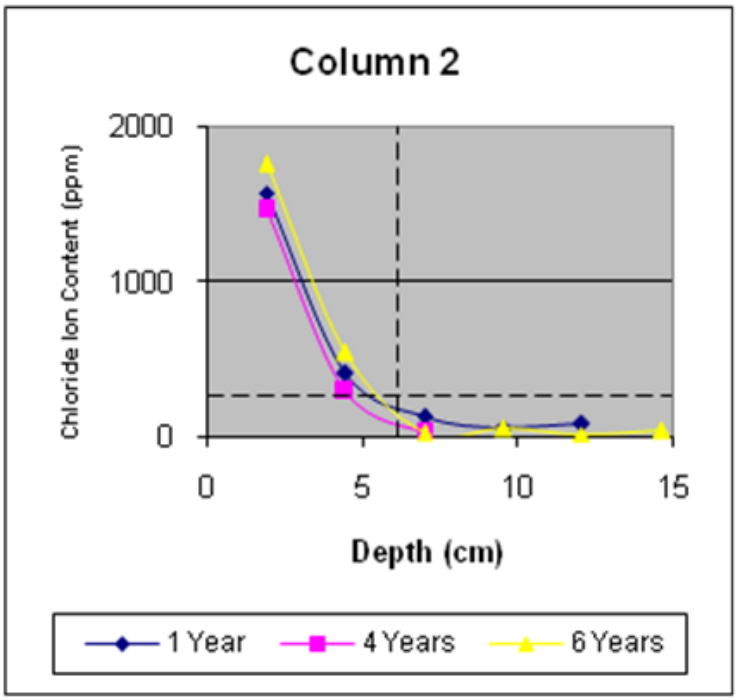
The average chloride ion content at the average steel depth was below the corrosion threshold for columns 2 and 4–6 and was above the threshold for the remaining columns. This explains the reduction in rate of increase of delaminations with time. Corrosion-induced damage primarily

occurred in patched areas with cracks and occurred adjacent to cracks in other areas where chloride ions had direct access to the steel.



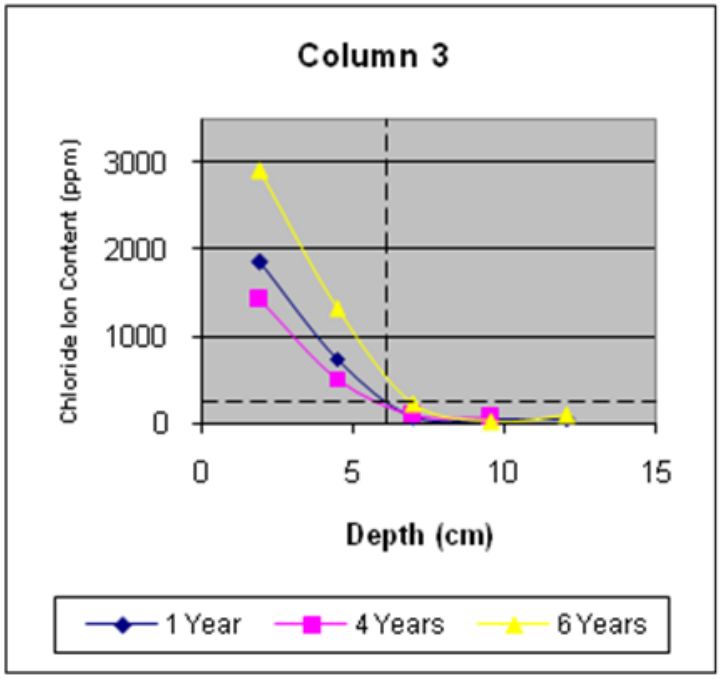
1 inch = 2.54 cm

Figure 45. Graph. Average chloride profile for column 1.



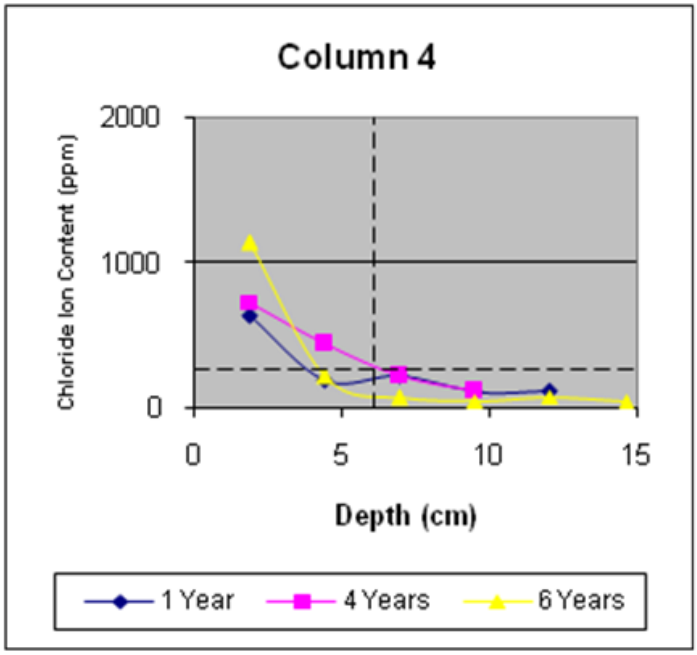
1 inch = 2.54 cm

Figure 46. Graph. Average chloride profile for column 2.



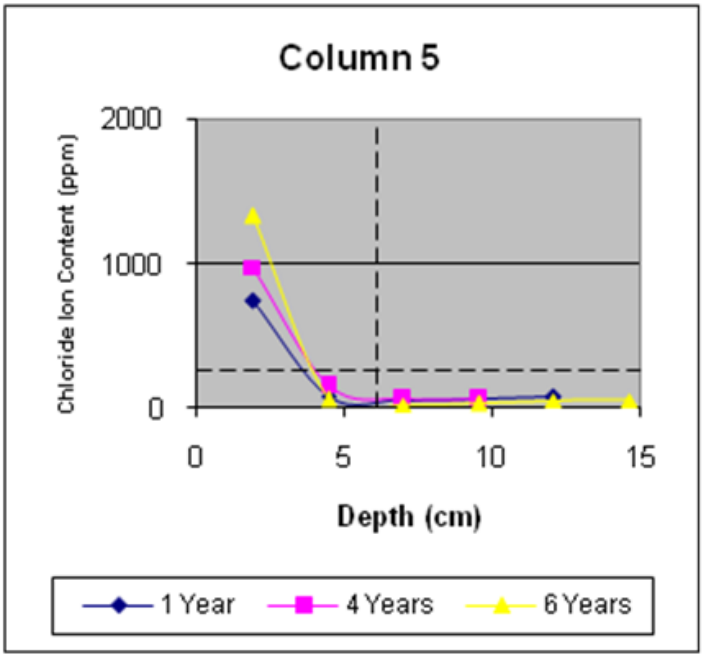
1 inch = 2.54 cm

Figure 47. Graph. Average chloride profile for column 3.



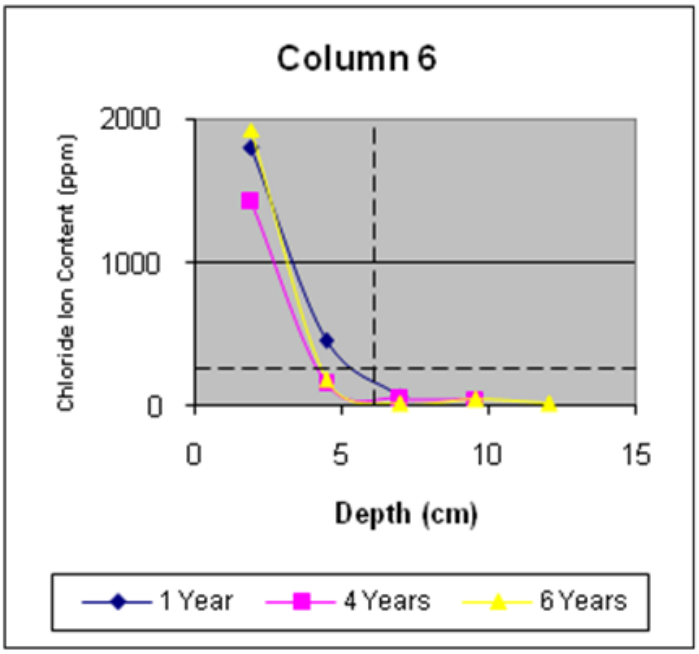
1 inch = 2.54 cm

Figure 48. Graph. Average chloride profile for column 4.



1 inch = 2.54 cm

Figure 49. Graph. Average chloride profile for column 5.



1 inch = 2.54 cm

Figure 50. Graph. Average chloride profile for column 6.

Corrosion Potential Survey

A half-cell potential survey was performed on a 2.00-ft (0.61-m) grid on each column with a copper-copper sulfate half-cell during each evaluation. In five of the six columns, the percentage of active potentials increased with time, and it remained constant in column 3 (see figure 51).

The average potential of column 1 remained more negative than -350 mV during the three evaluations (-365, -371, and -416 mV). Additionally, the average potential of column 6 became more negative than -350 mV since the second evaluation (-325, -496, and -424 mV). The increase in percentage of potentials in the active range correlated with the increase in chloride ion content and damage.

Corrosion Rate Survey

Corrosion rate measurements were made using the 3LP-NBS corrosion rate device at four locations on each column during each trip. Although corrosion rates fluctuated with time, they were generally high (i.e., in a range that suggests damage would occur in the next 2 to10 years) (see table 7).

Table 7. Average corrosion rate measurements (mA/m^2).

Year After Treatment	Column 1	Column 2	Column 3	Column 4	Column 5	Column 6
Pretreatment	1.04	1.46	1.38	0.28	1.08	1.79
3 years	4.55	3.53	0.86	2.29	1.97	0.74
4 years	0.21	2.08	0.74	3.82	0.99	0.26
6 years	2.02	2.45	0.35	3.81	3.6	1.82

1 mA/m^2 = 0.093 mA/ft^2

Conclusions

Although the application of ECE may remove a significant amount of chloride ions, the long-term effectiveness of ECE is dependent on the remigration of existing chlorides, ingress of new chlorides, the amount of hydroxyl ions generated at the steel-concrete interface, and the dissipation of the hydroxyl ions with time. At this test site, the requirement of extracting chloride ions from the concrete was met to a certain degree; however, other requirements for the long-term effectiveness of ECE were not met.

SHRP researchers suspected that chloride ions might have been inadvertently introduced into the columns through cracks in the patched areas during the application of ECE. In addition, the columns were continually exposed to chloride contaminated runoff. The patterns of damage observed on the columns suggest that the cracks in the patched areas and in other areas were the primary channels of chloride ingress, resulting in corrosion-induced damage.

The variation of chloride distribution in the column concrete over time, half-cell potential data, and corrosion rate measurements all suggest an increasing propensity for corrosion. Based on the data, it can be concluded that the application of ECE at this site was ineffective. The failure of ECE at this site can be attributed to the following:

- The quality of the repairs performed (i.e., cracking of patch material).
- Nonrepair of cracks.
- No barrier to future ingress of chloride ions installed.

- Problems encountered during the application of ECE including containment of the electrolyte, nonuniform distribution of current, and lowering of pH.

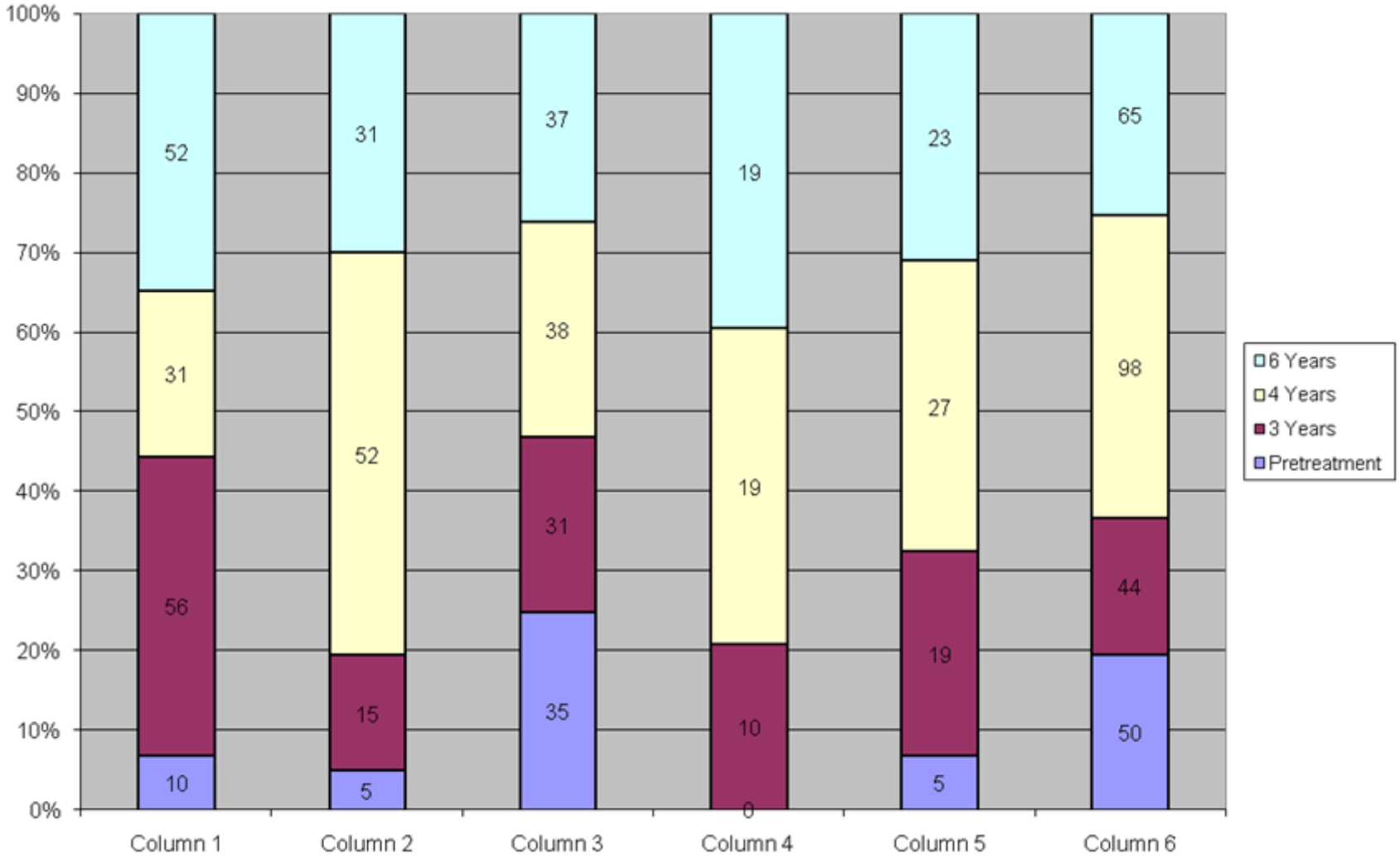


Figure 51. Graph. Distribution of active potentials versus time.

ROUTE 631 IN CHARLOTTESVILLE, VA

Structure Background

A four span slab-on-stringer bridge was constructed in 1969 to carry two northbound and two southbound lanes of Route 631 over I-64. Two abutments and three piers support the 334.56-ft (102-m)-long and 82-ft (25-m)-wide bridge (see figure 52). Each pier is comprised of two column bents, one on each side of the bridge centerline (see figure 53). The number of columns in a bent varies from three to four, although the total number of columns in each pier is seven. The piers are numbered from south to north. The western bents of piers 1 and 2 and the eastern bent of pier 3 are four-column bents, whereas the eastern bents of piers 1 and 2 and the western bent of pier 3 are three-column bents. The columns are numbered from west to east.

The length of the caps on the bents varies from 45.92 to 52.48 ft (14 to 16 m). The caps have a rectangular cross section of 4.3 ft (1.3 m) by 4.9 ft (1.5 m). The circular columns have a diameter of 3.6 ft (1.1 m) with a height of 18.4 to 21.3 ft (5.6 to 6.5 m).

ECE Application

This structure was selected for application of ECE in the Pilot Trial Program funded by FHWA and performed by the Virginia Transportation Research Council. Prior to the application of ECE, two sets of repairs were performed. In 1992, repairs were performed on the bottom of the pier caps and on the top of the columns. Just prior to the application of ECE in 1995, an additional 17.28 ft^2 (1.6 m^2) of delaminated concrete on the bottom of the pier caps and on the top of the columns was repaired.

A commercially available ECE treatment was applied on columns and cap beams of the western bents. The columns and cap beams of the eastern bents were designated as controls. A total surface area of 5,270.4 ft^2 (488 m^2) was treated on 3 cap beams and 11 columns.[(3)] The surface area for the control cap beams and columns totaled 4,654.8 ft^2 (431 m^2). A silane sealer was applied on all treated surfaces.

For the application of ECE, two different types of anode materials were used. On the flat faces of the pier caps, steel mesh was used as an anode. On the curved sections of the pier caps and columns, a commercially available titanium mesh anode was used. The titanium anode was used on curved surfaces because the steel mesh was too stiff. Cellulose fibers soaked in calcium hydroxide solution were sprayed to form a 2-inch (50.8-mm)-thick layer that encased the concrete surface and the anodes. The entire assembly was wrapped in plastic sheets and secured to minimize and control electrolyte evaporation from the cellulose fibers and protect the system from the environment.

After the installation of the plastic sheets, a short was discovered in pier 3. A section of the titanium anode applied on the subject pier had to be disconnected from the system, and that section did not receive ECE treatment.

Figure 52. Photo. Route 631 over I-64 bridge in Charlottesville, VA.

Figure 53. Photo. Soffit, bents, and columns of Route 631 over I-64 bridge in Charlottesville, VA.

The treatment for the piers varied from 72 to 77 days, the initial current densities varied from 0.03 to 0.041 A/ft^2 (0.3 to 0.45 A/m^2), and the final current densities varied from 0.011 to 0.014 A/ft^2 (0.12 to 0.15 A/m^2). The total charge passed varied from 23.1 to 35.5 A-h/ft^2 (249 to 382 A-h/m^2), and the current efficiencies were estimated to range from 8.6 to 27 percent (see table 8).

Table 8. ECE application parameters in Charlottesville, VA.

Description	Pier 1 Columns 1–4	Pier 2 Columns 1–4	Pier 3 Columns 1–3
Size (m^2)	177	176	134
Applied voltage (volts)	39	40	41
Initial current density (A/m^2)	0.30	0.45	0.37
Final current density (A/m^2)	0.12	0.15	0.12
Length of application (weeks)	11.0	10.6	10.3
Total charge (A-h/m^2)	276	382	249
Efficiency (percent)	12.3	27.0	8.6

1 m^2 = 10.8 ft^2
1 A/m^2 = 0.093 A/ft^2
1 A-h/m^2 = 0.093 A-h/ft^2

Field Evaluation

The following three field evaluations were performed:

- First evaluation: May 23 and 24, 1996 (approximately 1 year after treatment).
- Second evaluation: November 3 to 5, 1997 (approximately 2 years after treatment).
- Third evaluation: August 19 to 21, 1998 (approximately 3 years after treatment).

Test Results

Visual Survey

During each trip, visual inspection of the pier caps and the columns was performed. During the first two trips, no observable damage was noted. During the third trip, columns in pier 1 exhibited some deterioration. Noncorrosion-induced spalling was found on columns 1 and 6, rust staining was found on the bottom of columns 2 and 6, and cracking was found in a patch on column 6. A few rust stains were observed on the top of many columns. It was concluded that these stains resulted from water run-off carrying rust from other corroding metals on the bridge superstructure.

Sounding Survey

No delaminations were detected during the first two trips. During the third trip, two delaminations were detected on column 3, and one small delamination was found on column 2 of pier 1. Both delaminations on column 3 were located in repaired (i.e., patched) areas.

Clear Concrete Cover Survey

All cores collected for chloride ion content analysis during the first evaluation and some cores collected during the second evaluation contained reinforcing steel. Concrete cover over the embedded reinforcing steel was measured. Concrete cover varied from 1.3 to 3 inches (33.0 to 76.2 mm) for the columns and averaged 2.2 inches (55.9 mm), whereas concrete cover for the pier caps ranged from 3.5 to 4.3 inches (88.9 to 109.2 mm) and averaged 3.9 inches (99.1 mm).

Chloride Ion Content Analysis

During each evaluation, 12 2-inch (50.8-mm) cores were collected from the columns for chloride content analysis. To ensure that the chloride content was comparable from one evaluation to another, cores in subsequent trips were collected from areas adjacent to sampling sites in the previous evaluation. With the exception of one core, all other cores were collected from the lower half of the columns. Due to difficulties in collecting cores from the cap beams, only three cores were collected during the first evaluation.

From each core, powdered concrete samples were collected from regular intervals of 0.5 to 1 inch (12.7 to 25.4 mm) from the top surface of the core. The powdered samples were analyzed for total chloride ion content. The chloride ion contents of the samples collected from the cores were low. Only one core from a cap bean had chloride ions in excess of 500 ppm in the top 1.2 inches (30.5 mm) of concrete. Of the remaining 35 cores, only a couple of cores had chloride ion content in excess of 400 ppm in the top 1.2 inches (30.5 mm) of concrete. These cores were collected from treated and untreated columns. The highest average chloride ion content in the top 0.8 inches (20.3 mm) of concrete prior to the treatment was 848 ppm. The chloride ion content at the steel depth in all columns was below the threshold required to initiate corrosion.

Corrosion Potential Survey

Half-cell potentials were measured on a 0.98-ft (0.3-m) grid on the surface of the columns. All measurements were made after removing the silane sealer from the test location. Half-cell potentials measured in the first evaluation were not valid and are not reported. In general, most of the potentials measured were in the passive range. By the third evaluation, there was a slight shift in the potentials of the control columns toward the uncertain range.

Corrosion Rate Survey

Four corrosion rate measurements were made (one on each face) on columns 1–6 of each pier during the second and the third evaluations with the exception of three columns during the third evaluation. Two measurements were made on columns 1–3 of pier 1 during the third evaluation. Corrosion rates ranged from passive to low. Corrosion rate measurements suggest that no corrosion activity occurred.

Conclusions

The primary source of chloride ions in this structure was contaminated water run-off from the deck primarily from the leaking joints. Thus, the distribution of chloride ions was expected to be quite nonuniform. The ECE application report by Virginia Transportation Research Council documents the effectiveness of the treatment in removing chloride ions from the top 1.6 inches (40.6 mm) of concrete. The present corrosion status of the treated areas and the controls is benign, and damage only in localized areas is expected in the controls. Also, the application of ECE is recent on the structure, and it is too early to judge its long-term effectiveness at this site.

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2. AASHTO-260. (1997). *Standard Method of Test for Sampling and Testing for Chloride Ion in Concrete and Concrete Raw Materials*, American Association of State Highway and Transportation Officials, Washington, DC.

3. Clemena, G.G. and Jackson, D.R. (1996). *Pilot Applications of Electrochemical Chloride Extraction on Concrete Piers in Virginia*, Report No. VTRC 96-IR4, Virginia Transportation Research Council, Charlottesville, VA.

APPENDIX A. STATE-OF-THE-ART REPORT

ECE HISTORY AND CURRENT STATUS

ECE is a process where chloride ions are extracted from chloride-contaminated reinforced concrete structures by applying an electrical current between the embedded steel and an external anode. The process was originally referred to as ECR. However, due to confusion with the more popular abbreviation for epoxy coated rebar, ECE was adopted, particularly in North America. In Europe, the process is often referred to as desalination or sometimes as ECM. ECE is becoming popular as a rehabilitation option for chloride contaminated reinforced concrete structures to stop/mitigate ongoing corrosion of embedded steel.

The technique of removing chloride ions from contaminated reinforced concrete by electrochemical means was first studied in the mid 1970s by KDOT and Battelle Columbus Laboratories.[1,2] Both studies were funded by FHWA. Some adverse effects were observed in these studies possibly because of the high current densities used during treatment. These included some loss in rebar-to-concrete bonding, increased permeability in the concrete, and some uncertainties in the effects on the concrete of the migration of ions other than chloride and in the rate of remigration of the residual chloride in the concrete after the treatment. Consequently, further research on ECE was not pursued in the United States until the mid to late 1980s.

From 1988 to 1993, SHRP sponsored additional research on the ECE process. There were extensive laboratory investigations, and limited field trials were also conducted.[3,4] Independent studies in Europe during the mid 1980s resulted in a Norwegian patent for ECE in 1985 known as the Norcure process, which became commercially available in 1988.[5] A patent on some ECE process parameters was also granted in the United Kingdom in 1994.[6]

SHRP Laboratory Work

Laboratory work under the SHRP contract addressed several concerns that can arise as a result of the passage of large amounts of current through concrete. Reinforcing steel-concrete bond strength was measured over the full range of current and charge experienced for both chloride removal and cathodic protection. The application of a high current density (464 A/ft^2 (5,000 A/m^2)) and/or a high amount of total charge (186 A-h/ft^2 (2,000 A-h/m^2)) resulted in a reduction of bond strength when compared to controls containing salt. However, the use of either lower current density or lower charge had no adverse effect. Even at the highest current density and charge, bond strength was reduced only to the values equal to those of no-salt control specimens.

Concrete compressive strength was not reduced at lower current densities, but concrete treated at high current (1.9 A/ft^2 (20 A/m^2)) for long periods of time (464 A-h/ft^2 (5,000 A-h/m^2)) experienced a softening of the cement paste around the reinforcing steel. This softening is probably also responsible for the loss of bond strength of severely treated specimens. This strong treatment also caused one slab to crack and delaminate. As a result, the current regime used in previous studies (up to 19 A/ft^2 (200 A/m^2)) was judged to be excessive, and more modest treatment conditions were used for field trials.

The possible hydrogen embrittlement of conventional reinforcing steel was also studied. Although a slight temporary loss of ductility was noted on smooth specimens, this loss was not structurally significant. The generation of chlorine gas from the anode, which could present a safety hazard, was also studied. It was decided that the electrolyte should be maintained at a basic pH to prevent the generation of chlorine gas. Several buffers were studied for this purpose, and a sodium borate buffer was found to be the most effective and practical. Control of the electrolyte pH in this way also prevented any etching or acid attack of the concrete surface. Other studies have shown that this electrochemical treatment of concrete causes an increase in the alkali cation concentration in the vicinity of the reinforcing steel. This study confirmed these results and demonstrated that serious damage could result if the chloride removal process was used on concrete containing alkali-reactive aggregate. It was also found that the presence of lithium ion in the electrolyte could be used to mitigate this problem. Where alkali-sensitive aggregate is present, the use of lithium borate buffer is recommended.

Based on the laboratory and test yard results, a guideline was developed, and a chloride removal treatment process was defined, resulting in the effective removal of chloride without any damage to the concrete or reinforcing steel.[7] Treatment current density was limited to less than 0.5 A/ft^2 (5 A/m^2) of concrete. System voltage was also limited by the Occupational Safety and Health Administration to less than 50 V for safety reasons. Under these conditions, treatment time for chloride removal can be expected to be 2 to 4 weeks. Typical applied charge will be 74 to 112 A-h/ft^2 (800 to 1,200 A-h/m^2). Treatment times and charges greater than these will probably yield little additional benefit in terms of chloride removed or corrosion prevented.

The treatment process described above will remove about 20 to 50 percent of the chloride present in the concrete. The amount of chloride removed depends on several factors including the amount of chloride present, the distribution of chloride in the concrete, and the reinforcing steel design. Typically, after treatment is complete, sufficient chlorides remain in the structure to reinitiate corrosion. However, the remaining chloride is usually distributed away from the steel, and sufficient time is required for redistribution to take place. The return to corrosive conditions is further delayed by the buildup of alkalinity that occurs at the steel surface.

Corrosion may also be caused by factors other than chloride such as carbonation of the concrete cover due to reaction with atmospheric carbon dioxide or damage due to other acidic gases. The chloride extraction process is not designed for such cases. However, there is a similar patented process called realkalization that utilizes sodium carbonate as the electrolyte.[8] In such an application, the objective is to restore the alkalinity of the concrete surrounding the reinforcing

SHRP Field Studies

Four field validation trials were conducted between fall 1991 and fall 1992. Chloride removal was conducted on an Ohio bridge deck and on bridge substructures in Florida, New York, and Ontario. Based on the laboratory and test yard studies, a current less than 0.5 A/ft^2 (5 A/m^2) and a voltage less than 50 V was used. The treatment was applied until a total charge of 56 to 125 A-h/ft^2 (600 to 1,350 A-h/m^2) of concrete was accumulated. The pH of the electrolyte was maintained neutral or basic to prevent etching of the concrete surface and the evolution of chlorine gas. Each field site was selected based on criteria established by the laboratory studies and the SHRP ETG. Active corrosion was occurring on a substantial portion of each selected

structure, and chloride contamination was well above the threshold levels. The absence of alkali reactive aggregate was a factor until the final trial when a structure with alkali reactive aggregate was selected as a test case.

The first field trial was conducted on an Ohio bridge deck in fall 1991. No physical deterioration of the deck was evident. The treatment was conducted by constructing a pond on the deck and placing an inert catalyzed titanium anode in the pond with a sodium borate buffer electrolyte. Current density for this trial was low because of cold temperatures and resistive concrete; therefore, the treatment time was 61 days. Chloride analyses of the ponded electrolyte indicated that about 0.36 oz/ft^2 (110 g/m^2) of chloride of concrete was removed during the treatment at a current efficiency of about 20 percent. Problems encountered included vandalism and overflow of the pond due to excessive rainfall.

The second field trial was conducted in spring 1992 on pilings underneath the B.B. McCormick Bridge near Jacksonville, FL. Prefabricated anode/blanket composites were strapped on each pile, and seawater was used as the electrolyte. The system operated for 18 days at an average current density of 0.31 A/ft^2 (3.3 A/m^2) and accumulated a total charge of 125 A-h/ft^2 (1,350 A-h/m^2). The success of the trial was difficult to judge since the electrolyte could not be analyzed for an increase in chloride concentration. Although steel in the treatment area was strongly polarized and chloride was removed, the efficiency and amount of chloride removed could not be determined.

The third field trial was conducted in June 1992 on the substructure of a bridge in Albany, NY. Prefabricated anode/blanket composites were strapped on the columns, and a sodium borate electrolyte was continuously circulated through a closed system. Localized high current densities were reported, which was probably a result of the inhomogeneous nature of the structure due to patching. Current densities ranged from 0.09 to 0.3 A/ft^2 (1 to 3 A/m^2), and two zones accumulated 74 and 86 A-h/ft^2 (800 and 930 A-h/m^2) in 17 and 24 days, respectively. Based on concrete analyses, chloride removed was 0.29 to 0.49 oz/ft^2 (90 to 150 g/m^2) at a current efficiency of 7 to 13 percent. Problems at this site included difficulty in preventing electrolyte leakage through small vertical cracks that extended below the removal zone on two columns, and electrolyte dilutions due to rainwater runoff.

The last chloride removal field trial was conducted in August 1992 on abutments of a bridge over the Montreal River in Latchford, Ontario. This trial was especially important since the structure contained alkali reactive aggregate and was suffering from a combination of ASR and corrosion-induced damage. The laboratory studies found that unless lithium was in the electrolyte, ASR damage would be aggravated by the chloride removal process. Consequently, a lithium borate buffered solution was used. An anode/blanket composite was installed on each abutment corner, and electrolytes were continuously circulated through the system. The system operated for 23 days at an average current density of 0.15 A/ft^2 (1.6 A/m^2), accumulating 78 and 83 A-h/ft^2 (840 and 890 A-h/m^2) of charge in two zones. Post-treatment analyses showed that 0.74 and 0.49 oz (21 and 14 g) of chloride per square foot had been removed from the two zones at current efficiencies of 19 and 12 percent, respectively. Petrographic analysis of the concrete showed that the treatment did not aggravate the alkali-silica reaction occurring in the structure. The main problem was electrolyte dilutions due to excessive rainwater runoff.

The SHRP study identified the following required and desired criteria for ECE:

Required criteria:

- Chloride-induced corrosion.
- No prestressed steel.
- No alkali-reactive aggregate.
- Reasonable steel continuity.
- Reasonable concrete resistance.

Desired criteria:

- Minimum concrete damage.
- No coatings or overlay.
- Minimum exposed steel.
- Minimum disruption of traffic.
- Available AC.
- Excellent continuity of steel.
- Low concrete resistance.
- Simple geometry.
- Chloride-induced corrosion.

A candidate structure for ECE should undergo active corrosion caused by the presence of chloride ions at the level of the steel reinforcement. Although it is possible to conduct chloride extraction before corrosion begins, the benefit of such treatment is questionable.

Subsequent to SHRP effort, research and field work on ECE has continued in North America and Europe to address some of the unresolved issues as well as to develop and refine criteria for commercial application of the ECE process. (See references 9–16.)

Commercial Application of ECE

The Norcure System, which includes treatment processes for ECE and realkalization, was acquired by Fosroc International 1994 and is the only commercial process available in Europe and North America. It is operated through licensed applicators. American Concrete Technologies

in Boston, MA, also owns licensing rights for the Norcure System but is in the process of being relinquished.

Since the late 1980s and early 1990s, hundreds of structures worldwide have been treated with ECE and realkalization. Table 9 provides a list of the ECE projects completed to date in the United States and Canada, and table 10 summarizes the worldwide ECE and realkalization projects from 1987 to 1999. Table 10 does not contain a complete listing of all projects completed to date; however, the information is accurate up to 1996. It is estimated that the total concrete surface treated by ECE and realkalization at the present time exceeds 4 million ft^2 (372,000 m^2).

Table 9. ECE Projects completed in the United States and Canada.

No.	Location	Structure	Component	Year	Area Treated	
					m^2	ft^2
1	Burlington, Ontario, Canada	Burlington Bay Skyway	Pillar	1989	Demo	Demo
2	Winnipeg, Manitoba, Canada	Lakeview parking garage	Slab	1990	Demo	Demo
3	Toronto, Ontario, Canada	Credit view overpass	Columns	1990	Demo	Demo
4	Winnipeg, Manitoba, Canada	Bridge underpass	Wall	1990	30	323
5	Winnipeg, Manitoba, Canada	Rt-90 underpass	Wall	1991	Demo	Demo
6	Lucas County, OH	Neapolis/Waterville Road	Deck	1991	61	660
7	Albany, NY	Hawkins Street over NY-85	Columns	1992	57	616
8	New York, NY	Watch tower	Columns	1993	300	3,228
9	Ottawa, Ontario, Canada	Bell Canada parking garage	Slab	1993	Demo	Demo
10	Saskatoon, Saskatchewan, Canada	Hwy-16/Hwy-11 overpass	Columns	1994	150	1,614
11	Regina, Saskatchewan, Canada	Hwy-1/Hwy-6 overpass	Columns	1995	370	3,981
12	Morinville, Alberta, Canada	Grandin Avenue overpass	Columns	1995	55	592
13	Regina, Saskatchewan, Canada	Hwy-6/Hwy-11 overpass	Columns	1995	180	1,937
14	Arlington, VA	34th Street over 1-395 bridge	Deck	1995	733	7,887
15	Charlottesville, VA	Rt-631 over I-64 bridge	Columns	1995	488	5,251
16	Sioux City, SD	1-29 overpass	Columns	1995	150	1,614
17	Dover, DE	Bridge deck	Deck	1997	1,550	16,678
18	Burlington, Ontario, Canada	Burlington Bay Skyway	Columns	1997	268	2,884
19	Toronto, Ontario, Canada	Bridge substructure	Chambers	1997	180	1,937
20	Syracuse, NY	Parking garage	Slab	1997	100	1,076
21	Minneapolis, MN	Bridge substructure	Piers	1997	225	2,421
22	Winnipeg, Manitoba, Canada	Bridge deck	Deck	1997	270	2,905
23	Peoria, IL	Bridge substructure	Piers	1998	462	4,971
24	Omaha, NE	Bridge substructure	Piers	1998	1,525	16,409
25	Council Bluffs, LA	Bridge substructure	Piers	1998	463	4,982
26	Winnipeg, Manitoba, Canada	Bridge substructure	Piers	1998	220	2,367
27	St. Adolphe, Manitoba, Canada	Bridge deck	Deck	1998	1,115	11,997
28	Burlington, Ontario, Canada	Burlington Bay Skyway	Piers	1999	1,533	16,495
29	Omaha, NE	Bridge substructure	Piers	1999	1,400	15,064
30	Jackson, MI	Bridge substructure	Piers	1999	109	1,173
31	Minot, ND	Bridge substructure	Piers	1999	100	1,076
32	Washington, DC	Bridge substructure	Piers	1999	220	2,367
Total					**12,314**	**132,505**

Table 10. Summary of worldwide ECE and realkalization projects.

Year	ECE Projects			Realkalization Projects			Combined ECE/ Realkalization Projects		
	Number of Projects	Total Area Treated		Number of Projects	Total Area Treated		Number of Projects	Total Area Treated	
		m^2	ft^2		m^2	ft^2		m^2	ft^2
1987	0	0	0	1	400	4,304	1	400	4,304
1988	4	90	968	8	1,560	16,786	12	1,650	17,754
1989	18	1,165	11,890	9	1,161	12,492	27	4,826	51,282
1990	7	2,550	27,438	7	20,542	221,032	14	23,092	248,470
1991	2	300	3,228	23	19,647	211,402	25	19,947	214,630
1992	7	1,040	11,190	20	23,719	255,216	27	24,759	266,407
1993	11	2,076	22,338	37	30,239	325,372	48	32,315	347,709
1994	17	8,740	94,042	41	26,642	286,668	58	34,182	367,789
1995	26	9,938	106,933	46	36,452	392,224	72	46,390	499,156
1996	24	11,094	119,371	53	43,463	467,662	77	51,715	556,453
1997	6	2,593	27,901	0	0	0	6	2,593	27,901
1998	5	3,785	40,727	1	110	1,184	6	3,895	41,910
1999	5	3,362	36,175	1	288	3,099	6	3,650	39,274
Total	**132**	**46,733**	**502,201**	**247**	**204,223**	**2,197,439**	**379**	**249,414**	**2,683,049**

ECE Specifications

Currently, there are no specifications for ECE either in the United States or Europe. The projects that have been completed in North America have followed guidelines developed by SHRP with additional criteria introduced by the client organization. However, the ECE ETG set up by FHWA has been discussing the issue of producing a specification since 1995 and is on the verge of producing a state-of-the-art report which will eventually be converted to specifications. The National Association of Corrosion Engineers has also produced a draft state-of-the-art report on ECE.[17] Similarly, the European Corrosion Society is working on ECE specifications as part of their activities for producing standards for electrochemical treatments for reinforced concrete structures.[18]

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