Long-Term Performance of Corrosion Inhibitors Used in Repair of Reinforced Concrete Bridge Components

Publication No. FHWA-RD-01-097

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Federal Highway Administration
Research and Development
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FOREWORD

In 1987, the Strategic Highway Research Program (SHRP) launched a research effort to evaluate the effectiveness of using corrosion inhibitors as a means for mitigating corrosion in reinforced concrete bridge components. That project, completed in 1993, involved a laboratory study and field validation, and concluded that corrosion inhibitors could be applied successfully with field repair and rehabilitation techniques.

Although the SHRP study established the effectiveness of using corrosion inhibitors on concrete bridge components, it was not designed to ascertain the long-term effectiveness of the technology in mitigating corrosion. This follow-on study of the SHRP effort was initiated by the Federal Highway Administration (FHWA) in August 1994 and ended in July 1999. The primary goal of this study was to monitor the SHRP field sites for 5 years to determine the long-term effectiveness of corrosion inhibitors. An analysis of the results concluded that neither of the corrosion inhibitors evaluated in this study, using the specified repairs and exposed to the specific environments, provided any corrosion-inhibiting benefit.

This report will be of interest to engineers involved in bridge design, bridge performance evaluation and prediction, and bridge maintenance and rehabilitation.

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NOTICE

SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
NOTE: volumes greater than 1000 L shall be shown in m^3				
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact degrees)		
°F	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	°C
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact degrees)		
°C	Celsius	1.8C+32	Fahrenheit	°F
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

*SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380. (Revised March 2002)

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EXECUTIVE SUMMARY

In 1987, the Strategic Highway Research Program (SHRP), mandated by the United States Congress under section 128 of the Surface Transportation and Uniform Relocation Assistance Act, launched multiple research efforts to study all aspects of reinforced concrete deterioration. One of the projects (SHRP C-103) under the Structures portion of SHRP evaluated the effectiveness of using corrosion inhibitors to mitigate corrosion in reinforced concrete bridge components. This project, which concluded in 1993, involved a laboratory study and field validation.

Under the field validation program, several field sites were established to evaluate the effectiveness of two of the corrosion-inhibitor systems identified in the laboratory study on mitigating corrosion of reinforced concrete bridge components. The two systems were spray-on applications of Postrite and/or DCI® admixture (calcium nitrite-based inorganic inhibitors), and spray-on MCI®2020 and/or MCI®2000 admixture (amine-based organic inhibitors). Two bridge structures were selected for deck trials and four bridges were selected for substructure trials. However, only five of the six structures were included in the project. The Maryland site was not treated with inhibitors as planned because of lack of funds and was excluded from the SHRP study. The field validation study concluded that corrosion inhibitors could be successfully applied with field repair and rehabilitation techniques.

A follow-on study of the SHRP effort was initiated by the Federal Highway Administration (FHWA) in August 1994. The primary objective of this multitask FHWA project, which ended in July 1999, was to determine the effectiveness of cathodic protection, electrochemical chloride extraction, and corrosion-inhibitor treatment systems installed during the SHRP effort. This was to be achieved through long-term evaluation of 32 field test sites in the United States and one Canadian Province, as well as a number of laboratory concrete slab specimens.

One task the FHWA program required was monitoring the long-term performance of corrosion-inhibitor treatments on selected components of five bridges that were treated and evaluated under the SHRP C-103 project. These bridges were located in:

- Saint Paul, MN
- Buffalo, NY
- Wilkes Barre, PA
- Christiansburg, VA
- Port Gamble, WA

The structure in Virginia was eliminated from this study after the first evaluation because the design of the test areas would not allow a fair assessment of the inhibitor performance.

Three evaluations over a period of 5 years were conducted on structures in Minnesota, New York, and Pennsylvania; two evaluations were performed on the structure in Washington State.

On each structure, three similar test areas were delineated. Repairs were performed in these test areas using the same materials and procedures, with the exception of the inclusion of corrosion inhibitors in two of the three areas. The third test area was designated a control area. Postrite and/or the DCI admixture system was used in the repairs on one test area and MCI 2020 and/or MCI 2000 admixture was used in the repairs on the other test area.

An analysis of the results of visual and delamination surveys, half-cell potential surveys, corrosion rate measurements, and total chloride ion content determination concluded that neither of the corrosion inhibitors evaluated in this study, using the specified repairs and exposed to the specific environments, provided any corrosion-inhibiting benefit.

With the exception of the Port Gamble test site, shrinkage cracking plagued repairs in all other sites. The concrete surrounding the patched areas was contaminated with chloride ions to varying degrees. In some sites, shrinkage cracking allowed faster ingress of chloride ions into the repair patches. In all four sites, the results of the visual and delamination surveys and corrosion rate measurements showed no difference between patches containing corrosion inhibitors and those that did not.

1.0. INTRODUCTION

1.1. Project Background

In 1987, the Strategic Highway Research Program (SHRP), mandated by the U.S. Congress under section 128 of the Surface Transportation and Uniform Relocation Assistance Act, launched multiple research efforts to study all aspects of reinforced concrete deterioration. One of the projects (SHRP C-103) under the Structures portion of SHRP evaluated the effectiveness of using corrosion inhibitors to mitigate corrosion in reinforced concrete bridge components. This project, which concluded in 1993, involved a laboratory study and field validation.

1.2. SHRP Laboratory Work

The laboratory portion of the SHRP study evaluated 17 corrosion-inhibiting systems for reinforced concrete bridge components. Based on the initial evaluation, five corrosion inhibitors were selected for further testing. These were Alox 901 (organic surface-applied), Cortec VCI-1337 [MCI-2020] (organic surface-applied), Cortec VCI-1609 [MCI-2000] (organic admixture), DCI (inorganic admixture), and sodium tetraborate (inorganic surface-applied).(1) Nine small-scale slabs were constructed representing different treatment conditions. The three surface-applied corrosion inhibitors were also tested on salvaged portions of a deck slab from a bridge replacement project on I-80 in Pennsylvania. It was determined from the evaluation of these slabs that the use of Alox and Cortec on the deck slab resulted in a reduction of corrosion activity regardless of the pretreatment corrosion rate. DCI was found to be effective in reducing corrosion activity for specimens with low pretreatment corrosion rates. The benefits from the use of sodium borate were not as evident when compared to the control slabs.

1.3. SHRP Field Studies

Under the field validation program, several field sites were established to evaluate the effectiveness of two of the corrosion-inhibitor systems identified in the laboratory study on mitigating corrosion of reinforced concrete bridge components. The two systems were spray-on applications of Postrite and/or DCI admixture (calcium-nitrite based inorganic inhibitors), and spray-on MCI 2020 and/or MCI 2000 admixture (amine-based organic inhibitors). Two bridge structures were selected for deck trials and four for substructure trials. However, only five of the six structures were included in the project. The Maryland site was not treated with inhibitors as planned because of lack of funds and thus was excluded from the SHRP study. The field validation study concluded that corrosion inhibitors could be successfully applied with field repair and rehabilitation techniques. Although Postrite/DCI showed promising results in some cases, long-term corrosion assessment data were needed to draw any firm conclusions on the effectiveness of inhibitor-modified concrete systems.(2)

1.4. FHWA Follow-On Study

A follow-on study of the SHRP effort was initiated by the Federal Highway Administration (FHWA) in August 1994. The primary objective of this multitask FHWA project, which ended in July 1999, was to determine the effectiveness of cathodic protection, electrochemical chloride extraction, and corrosion inhibitor treatment systems installed during the SHRP effort. This was to be achieved through long-term evaluation of 32 field test sites in the United States and one Canadian Province, as well as a number of laboratory concrete slab specimens. The secondary objective of this research was to identify the most appropriate laboratory and field test method(s) for evaluating and monitoring the performance of the corrosion-control techniques and procedures involved in the project.

One task the FHWA program required was monitoring the long-term performance of corrosion-inhibitor treatments on selected components of five bridges that were previously treated with inhibitor and evaluated under the SHRP Contract C-103. These bridges are in the States of Minnesota, New York, Pennsylvania, Virginia, and Washington.

1.5. Scope and Purpose

Although the SHRP study established the effectiveness of using corrosion inhibitors on concrete bridge components, it was not designed to ascertain the long-term effectiveness of the technology in mitigating corrosion. As mentioned above, the primary goal of this study was to monitor the five field sites for 5 years to determine the long-term effectiveness of corrosion inhibitors.

Fifteen field evaluations were planned, three visits to each site (years 1, 3, and 5).

1.5.1. Evaluation of Field Sites

The details for each site, along with the monitoring results, are discussed individually later in this report. The following work was conducted during the site visits:

1. Review of past reports.
2. Visual inspection.
3. Sounding (delamination) survey.
4. Concrete cover measurements.
5. Corrosion potential measurements.
6. Corrosion rate measurements.
7. Chloride content analysis.

To assess the long-term performance of the field inhibitor sites, it was not considered necessary to perform all of the above tasks during each of the three visits. Thus, cover depth measurements were performed only during the first visit and delamination surveys were performed during the first and third visits.

The Christiansburg, VA, site was dropped from the program after the first visit for the following reasons:

- As suggested by the limited delamination and chloride ion concentration data, the control and treated areas on the deck may not have been in the same corrosion condition at the start of the study.

- The size of the control areas on the deck and substructure were inadequate.

- The control area on the deck was bordered by an expansion joint. Generally, on bridge decks, corrosion-induced damage is more prominent at the expansion joints.

- Some sections of the control area on the column were not exposed to the same environment as the treated areas. The treated areas were exposed to contaminated water run-off, whereas some regions of the control area were not exposed to any contaminated water run-off.

- The region of the control area above and below the plugged and patched areas on the column may have been exposed to different amounts of contaminated water run-off.

A separate report for one evaluation of the Christiansburg site was issued in July 1995; the results of that evaluation are not included in this study. Three visits each were made to sites at St. Paul, MN; Buffalo, NY; and Wilkes Barre, PA. Only two visits were made to the Port Gamble, WA, site because the last installment of contract funds was not available.

2.0. ELMWOOD AVENUE BRIDGE OVER NY ROUTE 198, BUFFALO, NY

2.1. Structure Background

The Elmwood Avenue bridge over NY Route 198, Buffalo, NY, carries two northbound and two southbound lanes of traffic. The bridge has four spans and is supported by three piers with four circular columns each. Figure 2-1 shows a general view of the Elmwood Avenue bridge. Rehabilitation of the bridge (deck, approaches, sidewalks, piers, and abutments) started in March 1992, and was completed in October 1992. The substructure repair work was performed in stages between these dates as weather permitted.

Personnel from the New York State Department of Transportation (NYSDOT) sounded the substructure components prior to the rehabilitation work and determined the location and extent of the hollow-sounding areas to be patched. Sawcuts, 19 mm deep, were cut along the borders of the identified hollow-sounding areas and the concrete was removed to a depth of 19 mm below the reinforcing steel with pneumatic hammers. The patch cavities were backfilled with concrete. The exposed reinforcing steel was severely corroded and was not cleaned before the concrete was placed. Sound chloride-contaminated concrete adjacent to the patch areas was left in place.

Two corrosion-inhibitor admixtures were used, Cortec 2000 and DCI-S. The columns and pier caps for each pier were patched. The middle pier, columns, and pier cap were patched with concrete containing Cortec 2000 corrosion inhibitor admixture at a dosage rate of 1.2 kilograms per cubic meter (kg/m^3). The north pier was patched with concrete containing DCI-S corrosion-inhibitor admixture at a dosage rate of 29.9 liters per cubic meter (L/m^3). The columns and pier cap of the south pier were patched with portland cement concrete and were designated as the control.

Corrosion performance evaluations of the inhibitor admixtures had been limited to the south faces of the three pier caps. A sounding survey conducted prior to rehabilitation indicated a significant difference between the south and north faces of the three pier caps. The south faces were more severely and uniformly delaminated than the north face of the pier caps. The percentages of hollow-sounding areas in the south face for the control (south pier), Cortec (middle pier), and DCI (north pier) pier caps were 51, 59, and 56, respectively. The percentages of hollow-sounding areas in the north face for the control, Cortec, and DCI pier caps were 17, 40, and 22, respectively. To assess the effectiveness of the corrosion-inhibitor treatments, corrosion performance evaluations were limited to equivalent areas on the west sections of the south face of the three pier caps. Figures 2-2 through 2-5 show sections of the south pier, middle pier, and north pier that were evaluated.

Figure 2-1. General view of the Elmwood Avenue bridge over NY Route 198, Buffalo, NY.

2.2. Field Evaluation

Field evaluations were performed on the following dates:

First evaluation	October 17-19, 1994	~2 years after treatment
Second evaluation	May 7-9, 1997	~5 years after treatment
Third evaluation	June 17-23, 1998	~6 years after treatment

2.3. Test Results

The following sections describe the results of five standard surveys and the chloride ion content analysis.

2.3.1. Visual Survey

Visual surveys were conducted during all three visits. In October 1994, approximately 2 years after the repair of the piers, the west section of the south face of all three pier caps appeared sound and no spalls were observed. The patched areas, particularly those with inhibitor treatments, were severely cracked. The observed cracking pattern was mud-flat cracking, typical of drying shrinkage cracks. During the second and third visits (May 1997, and June 1998, respectively), the drying shrinkage cracks appeared to be about the same. However, minor spalling was observed in the three piers.

(a)

(b)

(c)

Figure 2-2. Views of test sections on: (a) south pier (control), (b) middle pier (Cortec 2000), and (c) north pier (DCI).

2.3.2. Delamination Survey

Delamination surveys were conducted during the first and third visits; results are presented in table 2-1. The first evaluation (October 1994) revealed hollow-sounding areas in patches of all three south face sections and in the original concrete within the south face sections of the control and Cortec-treated pier cap faces. It was not determined whether the hollow-sounding areas were the result of continued corrosion, drying shrinkage cracking, or disbondment of the patch material. It was noted in the SHRP C-103 Field Validation report (Publication No. SHRP-S-658) that difficulties were encountered in bonding the cast-in-place patch concrete to the original concrete. Researchers detected hollow-sounding areas in patch concrete within weeks after the repairs were completed. The researchers did not survey all of the patch areas and did not verify that all hollow-sounding areas were repaired.

In general, the hollow-sounding areas increased between the first and third evaluations. The increase in hollow-sounding areas with respect to the first evaluation was significant, especially on the original concrete areas of the control pier and on patched areas of the treated piers. The overall percentages of hollow-sounding areas for the control pier increased from 4.8 to 13.1; the middle pier (Cortec 2000-treated) increased from 4.3 to 5.1 percent; and the north pier (DCI-treated) increased from 6.0 to 13.6 percent.

An attempt was made to identify the cause(s) of the hollow-sounding areas. Several 7.6-centimeter cores were collected from sound and deteriorated areas located in patches or original concrete. Corrosion-induced delaminations were observed in some cores; bond failure between the patch and the original concrete was observed in others.

2.3.3. Clear Concrete Cover Survey

A clear concrete cover survey was performed during the first evaluation using a cover meter. Readings were taken in the patch concrete, at the interface, and in the original concrete. The results of the cover meter were verified at random locations with actual cover measurements at drill holes. The results are shown in table 2-2. The average clear concrete cover for the middle pier was greater than that for the south and the north piers.

Table 2-1. Delamination survey results.

Location	Survey Section, m	Survey Area, m^2	Approx Patch Area, m^2	Evaluation Date	Number of Hollow Sounding Areas		Hollow Sounding, m^2		Hollow Sounding, %		Overall Hollow Sounding Area, %
					Original	Patch	Original	Patch	Original	Patch	
South Pier (Control)	1.4 x 12.1	16.9	10.6	October, 1994	3	4	0.55	0.26	8.7	2.5	4.8
Middle Pier (Cortec)	1.4 x 12.1	16.9	3.9	October, 1994	3	2	0.18	0.55	1.4	14.1	4.3
North Pier (DCI)	1.4 x 12.1	16.9	9.9	October, 1994	0	3	0.00	1.01	0.0	10.2	6.0
South Pier (Control)	1.4 x 12.1	16.9	10.6	June, 1998	5	4	1.29	0.92	20.5	8.7	13.1
Middle Pier (Cortec)	1.4 x 12.1	16.9	3.9	June, 1998	2	2	0.26	0.60	2.0	15.4	5.1
North Pier (DCI)	1.4 x 12.1	16.9	9.9	June, 1998	1	10	0.09	2.20	1.3	22.2	13.6

Table 2-2. Cover depth survey results.

Test Area	Location	Average Cover Depth, cm	Combined Average, cm
South Pier (control)	Patch	4.27	4.57
	Interface	4.50	
	Original	4.93	
Middle Pier (Cortec)	Patch	5.66	6.43
	Interface	6.25	
	Original	7.39	
North Pier (DCI)	Patch	3.71	4.42
	Interface	4.83	
	Original	4.72	

2.3.4. Corrosion Potential Survey

Corrosion potential measurements were conducted as indicated by the American Society for Testing and Materials (ASTM) C-876 during all three visits. The test areas were approximately 1.4 x 12.1 m for the south pier, 1.4 x 5.5 m for the middle pier, and 1.4 x 9.2 m for the north pier. Potential measurements were taken on a 0.61-m grid on patched (P) areas, interface (I), and original (O) concrete. However, the majority of the measurements were located on patches and original concrete. According to ASTM C-876, rebars with corrosion potentials less than -350 mV (millivolts measured against a copper-copper sulfate electrode (CSE)) have a high probability of active corrosion. When the potentials are in the range of -200 mV to -350 mV, corrosion activity is uncertain. Rebars with corrosion potentials that are greater than -200 mV have a low probability of corrosion.

Summaries of corrosion potential measurements for the three visits are shown in tables 2-3 through 2-5. Corrosion potential data should be analyzed with caution from inhibitor treated areas, as the presence of inhibitor in the concrete can impact potential measurements due to the formation of junction cells.

Table 2-3. Corrosion potential summary (October 1994).

Inhibitor Treatment	Concrete Type*	No. of Measurements	Average Potential mV vs. CSE	Standard Deviation	% Corrosion Potentials, mV vs. CSE*		
					<-200	-200 to -350	>-350
Control	Original	12	-400	86	0	25	75
	Patch	21	-177	38	71	29	0
Cortec	Original	9	-201	48	56	44	0
	Patch	8	-138	62	87	13	0
DCI	Original	13	-229	31	15	85	0
	Patch	17	-198	62	59	41	0

* millivolts measured against a copper-copper sulfate electrode

Table 2-4. Corrosion potential summary (May 1997).

Inhibitor Treatment	Concrete Type*	No. of Measurements	Average Potential mV vs. CSE	Standard Deviation	% Corrosion Potentials, mV vs. CSE*		
					<-200	-200 to -350	>-350
Control	Original	14	-464	98	0	7	93
	Patch	21	-216	50	43	52	5
Cortec	Original	11	-281	51	11	89	0
	Patch	8	-240	27	0	100	0
DCI	Original	13	-269	46	0	83	17
	Patch	17	-230	91	53	35	12

* millivolts measured against a copper-copper sulfate electrode

Table 2-5. Corrosion potential summary (June 1998).

Inhibitor Treatment	Concrete Type*	No. of Measurements	Average Potential mV vs. CSE	Standard Deviation	% Corrosion Potentials, mV vs. CSE*		
					<-200	-200 to -350	>-350
Control	Original	14	-394	139	7	25	68
	Patch	21	-112	64	86	14	0
Cortec	Original	11	-235	67	27	73	0
	Patch	8	-236	37	25	75	0
DCI	Original	13	-254	45	0	86	14
	Patch	17	-259	81	31	56	13

* millivolts measured against a copper-copper sulfate electrode

In general terms, there was a slight increase in the active potentials for the control and DCI-treated piers. The potentials for the Cortec-treated pier remained in the passive to uncertain range.

2.3.5. Corrosion Rate Survey

Corrosion rate tests were conducted with an NBS-3LP corrosion rate device and measurements were made at 31 locations (11 in the control section and 10 each in the Cortec- and DCI-treated sections) during each of the three visits. Measurements were made on original concrete, on the patches, and at the interface. Table 2-6 shows average corrosion rate values determined in each of the test areas for all three visits.

Table 2-6. Corrosion rate results.

Location	Concrete Type	Average Corrosion Rate, mA/m^2 *		
		Oct. 1994	May 1997	June 1998
South Pier (control)	Original	0.103	0.229	0.713
	Interface	0.108	0.367	0.502
	Patch	0.097	0.166	0.237
Middle Pier (Cortec)	Original	0.170	0.284	0.417
	Interface	0.219	0.429	0.247
	Patch	0.554	0.795	0.658
North Pier (DCI)	Original	0.161	0.092	0.245
	Interface	0.189	0.206	0.426
	Patch	0.379	0.533	0.920

* milliamps per square meter
1 mA/m^2 = 10.753 mA/ft^2

The guidelines for interpreting corrosion rate data obtained with the NBS-3LP device are shown in table 2-7.

Table 2-7. Corrosion rate interpretation guidelines.

Guidelines for Interpreting 3LP Corrosion Rate Data	
Corrosion Current Density (Icorr), mA/m^2 *	**Predicted Time to Damage**
less than 0.019	none
0.019 to 0.093	10 to 15 years
0.093 to 0.930	2 to 10 years
greater than 0.930	less than 2 years

* milliamps per square meter
$1\ mA/m^2 = 10.753\ mA/ft^2$

As the corrosion rate varies significantly with temperature, the variation in data as a function of time and a comparison between test sections provides more information than the actual magnitude of the corrosion rate measurements. The highest rate of increase in corrosion rate with time in patched areas was observed on the north pier (DCI-treated). Similarly, the highest rate of change in original concrete and on the interface was observed on the south (control) pier. In general, the corrosion rate data do not differentiate between the treated areas and the control.

2.3.6. Chloride Ion Content Analysis

Total chloride ion content analysis as per the standard American Association of State Highway and Transportation Officials (AASHTO) T-260[(3)] method was conducted during the first and third visits. Powdered samples (at 1.27-cm intervals from the top surface to beyond the depth of the embedded steel) were obtained from cores collected from original concrete, patched areas, and the interface. The chloride content analysis results from the first and third visits are given in tables 2-8 and 2-9, respectively.

The chloride content in the various test areas has increased in a span of about 4 years, from the first evaluation in October 1994 to the third evaluation in June 1998. The chloride content at the average level of the reinforcing steel (5.1 cm) was above the corrosion threshold for all piers. The average chloride content at the level of the reinforcing steel in patched areas was approximately 50 percent higher on the treated piers than on the control pier. The average chloride content at the steel depth in patched and interface areas for the treated and control piers was approximately 3 to 4 times the chloride corrosion threshold.

2.4. Conclusions

All patched areas, particularly those treated with the inhibitor, had suffered shrinkage cracking. The cause of shrinkage cracking was not determined in this study. Immediately after the installation of the repairs, hollow-sounding areas were detected. The formation of hollow-sounding areas was attributed to lack of bonding between the patch material and the original concrete. Generally, bond failure occurs soon after the installation of new concrete and does not continue to occur with time. The size and number of hollow-sounding areas in patches continued to increase with time. This suggests that a process other than bond failure was responsible for the increase in hollow-sounding areas. Some cores extracted from patches in the control pier and the treated piers exhibited ongoing corrosion and consequent formation of delamination.

The average corrosion rate measurements in patched areas and original areas were of a magnitude that can result in corrosion-induced damage in 2 to 10 years. The variation in average corrosion rates with time suggests that corrosion continued unabated in the patches of the control area and the treated areas. Also, the corrosion rate in the patches of the control area and the area treated with the DCI inhibitor increased with time; it varied with time for the patches treated with the Cortec inhibitor. Corrosion was ongoing in the original concrete and was increasing with time. The corrosion rates at the interface between the patches and the original concrete in treated areas were similar to or higher than that for the control area.

Sufficient levels of chloride ions were present everywhere to ensure continuation of corrosion. The cracking of the patches probably aided the ingress of chloride ions (CI) into the patches and reached such high levels of concentration within 6 years of completion of the repairs.

Considering that corrosion has continued unabated in patches of inhibitor-treated areas at similar or higher rates than the control, it may be concluded that the inhibitors did not provide any protection against corrosion.

Table 2-8. Total chloride ion content analysis (first visit).

Location	x,y Coordinates, m	Concrete Type	Cl Sample Depth, cm	Cover Depth, cm	Cl Ion Content, ppm*
South Pier (control)	0.43, 0.46	Original	1.27 2.54 3.81 5.08	3.43	1226 766 664 409
	10.9, 0.38	Interface	1.27 2.54 3.81 5.08	4.06	1660 1762 1405 1201
	5.02, 0.40	Patch	1.27 2.54 3.81 5.08	4.32	766 255 281 281
Middle Pier (Cortec)	0.62, 0.43	Interface	1.27 2.54 3.81 5.08	5.33	639 741 894 817
	1.54, 0.40	Patch	1.27 2.54 3.81 5.08	4.83	792 409 358 332
North Pier (DCI)	6.95, 1.05	Interface	1.27 2.54 3.81 5.08	5.33	4802 2963 3448 2478
	2.58, 0.40	Patch	1.27 2.54 3.81 5.08	3.30	1149 741 383 332

* ppm = parts per million

Table 2-9. Total chloride ion content analysis (third visit).

Location	x,y Coordinates, m	Concrete Type	Cl Sample Depth, cm	Cover Depth, cm	Cl Ion Content, ppm
South Pier (control)	0.54, 0.46	Original	1.27 2.54 3.81 5.08 6.35 7.62	3.43	1541 1542 1162 834 601 421
	10.9, 0.46	Interface	1.27 2.54 3.81 5.08 6.35 7.62	4.06	1773 1542 1588 1505 926 1133
	5.08, 0.46	Patch	1.27 2.54 3.81 5.08 6.35 7.62	4.32	357 245 222 144 323 338
Middle Pier (Cortec)	0.77, 0.46	Interface	1.27 2.54 3.81 5.08 6.35 7.62	5.33	792 580 1320 865 990 788
	3.0, 0.24	Patch	1.27 2.54 3.81 5.08 6.35 7.62	4.83	1522 927 634 537 541 472
North Pier (DCI)	6.92, 1.08	Interface	1.27 2.54 3.81 5.08 6.35 7.62	5.33	3124 3542 2161 1881 1129 699
	2.77, 0.37	Patch	1.27 2.54 3.81 5.08 6.35 7.62	3.30	2942 1664 1047 604 465 435

3.0. STATE ROAD 2042 OVER I-81, WILKES BARRE, PA

3.1. Structure Background

The Pennsylvania State Road (SR) 2042 bridge consists of two three-span structures and carries traffic in an east-west direction over the north and southbound lanes of I-81. The west bridge carries SR 2042 traffic over southbound I-81, and the east bridge carries SR 2042 traffic over northbound I-81. Each bridge is approximately 43 m long and 12 m wide, consisting of two 3.66-m-wide traffic lanes and two 2.44-m-wide breakdown lanes. Piers are numbered 1 through 4 from east to west. Piers 1 and 2 are on the east bridge, and piers 3 and 4 are on the west bridge. Figures 3-1 (a) and (b) show general views of the west bridge and the east bridge, respectively.

Figure 3-1. General views of the SR 2042 bridge structures over I-81: (a) west bridge and (b) east bridge.

Piers 1, 2, and 3 were selected for SHRP C-103 field trial installation of shotcrete/corrosion-inhibitor repair systems. Areas of unsound concrete were delineated and removed to a depth of 19 millimeters (mm) below the reinforcing steel level with pneumatic hammers before repairs were begun. The exposed steel was sandblasted to near-white metal prior to application of the shotcrete and inhibitor treatment. In all patch repair areas, a wire mesh was tied to the reinforcing steel mat.

Pier 1 was used as the control. The repair cavities on this pier were backfilled with a standard shotcrete mix without any inhibitor treatment.

Pier 2 was repaired with shotcrete admixed with DCI inhibitor added at a rate of 1.9 liters (L) per bag of cement (approximately 4.99 L/m^3 of concrete). In addition, four spray applications of Postrite (a 15-percent calcium-nitrite solution) were applied to the repair cavities with exposed steel prior to backfilling with inhibitor-treated shotcrete. The surface of the repair cavity for each application was sprayed until it was saturated. The second application was done approximately 2 hours after the initial application, the third 8 hours after the initial application, and the final application was done immediately prior to the application of the shotcrete.

Pier 3 was repaired with shotcrete treated with Cortec MCI 2000 inhibitor. The MCI 2000 admixture was added at a rate of 0.18 L per bag of cement (approximately 1.2 L/m^3 of concrete).

All testing was limited to equal areas of 7.6 m^2 on one face of each of the three pier caps. The east face was chosen for piers 1 and 2, and the west face was chosen for pier 3.

Figures 3-2 (a-c) show the test areas on piers 1, 2, and 3, respectively, while figure 3-2 (d) shows testing in progress utilizing a snooper.

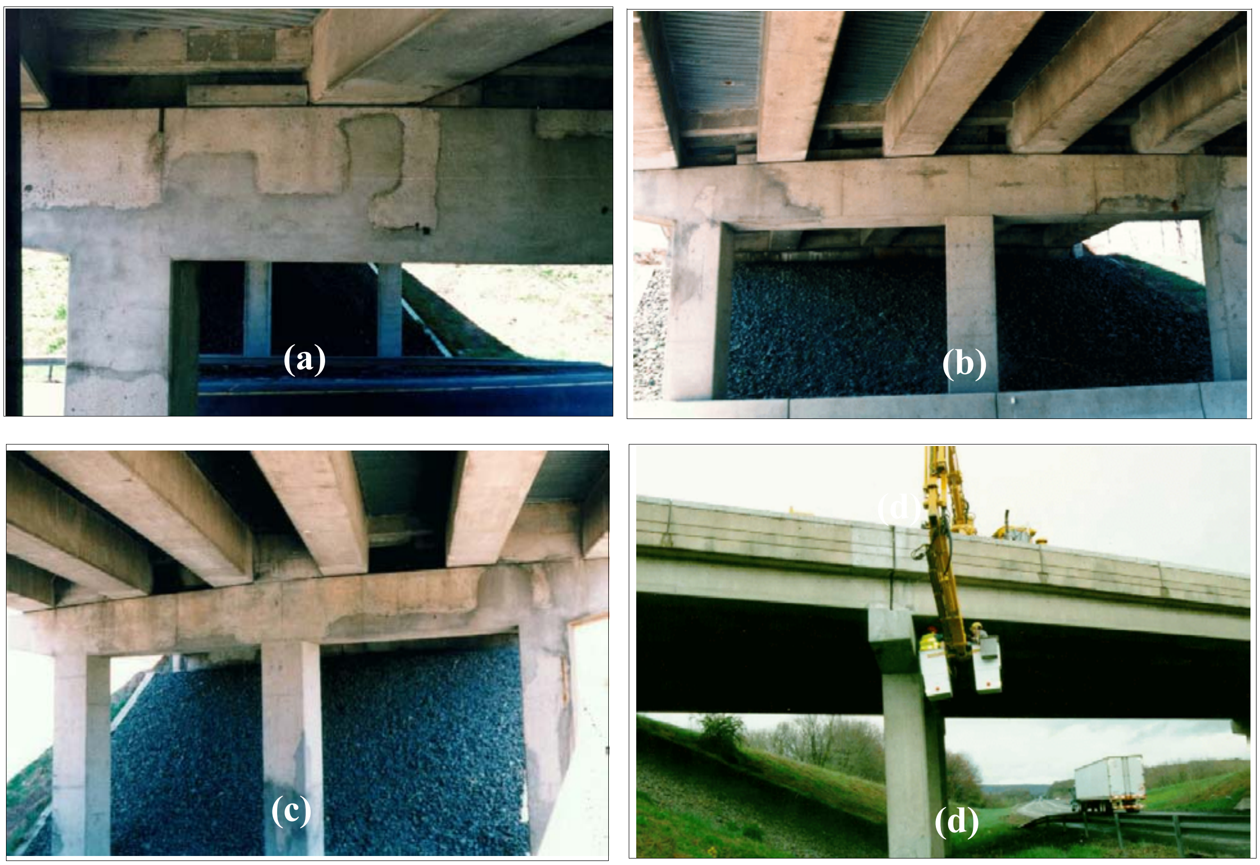

Figure 3-2. General views of piers:
(a) pier 1, (b) pier 2, (c) pier 3, and (d) field evaluation in progress.

3.2. Field Evaluation

A total of three field evaluations were performed in this study on the following dates:

First evaluation	November 16-18, 1994	~2 years after treatment
Second evaluation	May 5-6, 1997	~5years after treatment
Third evaluation	September 29-30, 1998	~6 years after treatment

3.3. Test Results

Six types of information were gathered. These results are discussed below.

3.3.1. Visual Survey

During the first visit, the concrete surface of the three pier caps did not have spalls. All patched areas on the three pier caps were cracked. Detailed mapping of the cracks was not performed. Cracks generally occurred along patch perimeters and the crack pattern within patched areas was typical of drying shrinkage cracking. Cracking was not observed in the original concrete areas. Visual observations of the three piers during the second visit did not show much change compared to the first evaluation. During the third visit, approximately 2.5 m of new cracks were documented in the patch areas of pier 3 (treated with Cortec 2000). These cracks did not appear to be caused by drying shrinkage. Outside the survey area of pier 3, corrosion products were visible in the original concrete. The shrinkage cracks on the other two piers remained the same. Spalls were not visible on any of the three pier caps.

3.3.2. Delamination Survey

During the first visit, hollow-sounding areas were detected in the patched areas and original concrete of pier 2 (treated with DCI Postrite). The hollow-sounding areas were 0.372 m^2 and 0.093 m^2 for the patched and original concrete, respectively. During the third visit, 0.093 m^2 of hollow-sounding area in the original concrete was found on pier 1 (control). The hollow-sounding area in the original concrete on pier 2 remained the same. However, the hollow-sounding area in the patched section on pier 2 was found to be somewhat less than that documented during the first visit. This discrepancy resulted from a confusion of the grids marked on the surface of the test area. Drawings of the delaminations suggest that the delaminations increased with time. No hollow-sounding sections were identified on pier 3 (treated with Cortec 2000). Table 3-1 summarizes the delamination survey results.

Table 3-1. Delamination survey results.

Location/Treatment	Survey Date	No. of Hollow-Sounding Areas		Hollow-Sounding Surface Area, m^2	
		Original	Patched	Original	Patched
Pier 1, East Face (control)	November 1994	0	0	0	0
Pier 2, East Face (Postrite, DCI)		2	4	0.093	0.372
Pier 3, West Face (MCI 2000)		0	0	0	0
Pier 1, East Face (control)	September 1998	1	0	0.093	0
Pier 2, East Face (Postrite, DCI)		3	4	0.093	0.186
Pier 3, West Face (MCI 2000)		0	0	0	0

Notes:
Survey section in each case is (1.07 x 7.41)m = 7.93 m^2
Original = original concrete
Patch = patch concrete placed in October 1992

3.3.3. Clear Concrete Cover Survey

A clear concrete cover survey was performed during the first evaluation using a cover meter. Readings were taken in the patch concrete, at the interface, and in the original concrete. The results of the cover meter were verified at random locations, with actual cover measurements at drill holes. The results are shown in table 3-2. The average clear concrete cover for the middle pier was greater than for the south and the north piers.

3.3.4. Corrosion Potential Survey

Corrosion potential measurements were conducted according to ASTM C-876[(4)] during all three visits. Potential measurements were taken on a 0.61-m grid on patched (P) areas, interface (I), and original (O) concrete. However, the bulk of the measurements were for patched and original concrete. Summaries of corrosion potential measurements for the three visits are shown in tables 3-3 through 3-5. Guidelines for interpretation of corrosion potentials were discussed earlier.

Table 3-2. Cover depth survey results.

Location/ Treatment	x, y Coordinates, m	Cover Depth, cm	Average Cover Depth, cm	Average Pre-Construction Depth, cm
Pier 1, East Face (control)	O (0.79, 0.61) O (7.47, 0.91) I (2.06, 0.91) I (4.64, 1.07)	6.35 5.33 2.67 4.95	4.83	5.18
Pier 2, East Face (Postrite, DCI)	O (3.78, 0.40) O (7.26, 0.61) I (2.10, 0.67) I (14.51, 0.76)	4.83 6.99 5.33 6.10	5.82	6.22
Pier 3, West Face (MCI 2000)	O (1.89, 0.31) O (7.53, 0.46) I (3.05, 0.46) I (4.79, 0.92)	6.60 6.73 4.57 4.95	5.72	5.77

Notes:
O = original concrete; I = interface of original and patch concrete
(0,0) coordinate is at the southern top corner of pier cap; x is measured in the northerly direction, and y measured in the southerly direction

Table 3-3. Corrosion potential summary (November 1994).

Location/ Treatment	Concrete Type	No. of Potential Measure-ments	Average Potential, mV vs. CSE	Standard Deviation	% Corrosion Potentials, mV vs. CSE*		
					<-200	-200 to -350	>-350
Pier 1, East Face (control)	O P	12 12	-249 -52	70 48	25 100	75 0	0 0
Pier 2, East Face (Postrite, DCI)	O P	13 10	-202 -218	127 86	62 60	23 20	15 20
Pier 3, West Face (MCI 2000)	O P	12 12	-209 -91	92 48	33 100	67 0	0 0

O = original concrete; P = patch concrete
* millivolts measured against a copper-copper sulfate electrode

Table 3-4. Corrosion potential summary (May 1997).

Location/ Treatment	Concrete Type	No. of Potential Measure-ments	Average Potential, mV vs. CSE	Standard Deviation	% Corrosion Potentials, mV vs. CSE*		
					<-200	-200 to -350	>-350
Pier 1, East Face (control)	Original Patch	14 12	-282 -74	83 55	21 100	57 0	21 0
Pier 2, East Face (Postrite, DCI)	Original Patch	13 10	-242 -246	126 83	31 40	46 30	23 30
Pier 3, West Face (MCI 2000)	Original Patch	11 12	-202 -98	98 64	45 92	55 8	0 0

* millivolts measured against a copper-copper sulfate electrode

Table 3-5. Corrosion potential summary (September 1998).

Location/ Treatment	Concrete Type	No. of Potential Measure-ments	Average Potential, mV vs. CSE	Standard Deviation	% Corrosion Potentials, mV vs. CSE*		
					<-200	-200 to -350	>-350
Pier 1, East Face (control)	Original Patch	20 16	-239 -60	70 53	25 94	75 6	0 0
Pier 2, East Face (Postrite, DCI)	Original Patch	23 11	-207 -242	98 108	48 55	39 18	13 27
Pier 3, West Face (MCI 2000)	Original Patch	21 15	-246 -70	91 54	24 93	66 7	10 0

* millivolts measured against a copper-copper sulfate electrode

3.3.5. Corrosion Rate Survey

Corrosion rate tests were conducted on all three visits. An NBS-3LP (linear polarization) corrosion rate device was used and measurements were made at a total of 18 locations (6 measurements per pier). On each pier, two measurements were made in the original concrete, at patch areas, and at the interface of the original and patch concrete. The results are presented in table 3-6. Corrosion rate measurements made on the original concrete taken prior to the rehabilitation in October 1992 are also included. However, it should be mentioned that the locations for the 1992 corrosion rate measurements were not the same as for the 1994-1998 measurements. The corrosion rate measurements obtained in the patch areas were not accurate because: (1) no reinforcing steel layout information was available, and (2) the calculated corrosion rate values did not take into account the polarized area of the mesh. The relative difference in the data can be used to study the impact of inhibitors in mitigating corrosion.

Table 3-6. Summary of corrosion rate measurements.

Location	Concrete Type	Corrosion Rate, mA/m^2			
		Oct. 1992	Nov. 1994	May 1997	Sept. 1998
Pier #1 (control)	O P I	0.248	0.166 0.077 0.160	0.178 0.064 0.315	0.084 0.120 0.154
Pier #2 (Postrite/DCI)	O P I	0.977	0.194 0.136 0.201	0.162 0.119 0.276	0.092 0.108 0.111
Pier #3 (MCI 2000)	O P I	1.107	0.390 0.099 0.130	0.286 0.066 0.363	0.777 0.118 0.428

1 mA/m^2 = 10.753 mA/ft^2
O = original concrete; P = patch concrete placed in 1992; I = interface of original and patch

It should be noted that the corrosion rates measured on the original concrete in October 1992, shortly after rehabilitation, were quite high. However, the locations were not the same as those used in the subsequent measurements from 1994 to 1998. Corrosion rates in patched areas had practically remained stable and were similar for all three piers, whereas at the patch/original concrete interface, corrosion rate data from pier 3 exhibited a significantly larger increase than that for the other two piers.

3.3.6. Chloride Ion Content Analysis

During the first visit in November 1994, powdered concrete samples were collected from the original concrete, patches, and interface areas from each of the three piers (a total of nine locations). At each location, 4 samples were collected at progressive increments of 13 mm down from the surface for a total of 36 samples. The chloride analyses data for the first visit are presented in table 3-7. Data collected prior to rehabilitation are also included in table 3-7. The three pier caps had high chloride contamination before rehabilitation. Chloride contents near the depth of the reinforcing steel were 6 to 13 times the corrosion threshold level (260 ppm). Two years after rehabilitation, chloride contents were very high at the interface areas, demonstrating that high chloride-contaminated concrete was left adjacent to patch areas. The patch concrete chloride content was low. The chloride content of pier 2 in the original concrete after 2 years was well above the corrosion threshold level.

Table 3-7. Total chloride ion content data (first visit).

Location/ Treatment	x, y Coordinates, m	Chloride Sample Depth, cm	Chloride Ion Content, ppm
Pier 1, East Face (control)	O (1.22, 0.61)*	1.27 2.54 3.81 5.08	1379 2810 2018 2043
	O (4.27, 0.61)*	1.27 2.54 3.81 5.08	1967 2401 3295 2810
	O (7.47, 0.92)	1.27 2.54 3.81 5.08	674 0 138 23
	I (4.64, 1.07)	1.27 2.54 3.81 5.08	3780 3826 2087 2304
	P (6.19, 0.92)	1.27 2.54 3.81 5.08	56 89 49 31
Pier 2, East Face (Postrite/DCI)	O (0.61, 0.61)*	1.27 2.54 3.81 5.08	3934 2273 3167 2427
	O (1.83, 0.61)*	1.27 2.54 3.81 5.08	3091 3372 3065 1890

Notes:
* Preconstruction data
O = original concrete; P = patch concrete placed in 1992; I = interface of original and patch concrete
(0,0) coordinate is at the southern top corner of pier cap; x measured in the northerly direction and y measured in the southerly direction

Table 3-7. Total chloride ion content data (first visit) (continued).

Location/ Treatment	x, y Coordinates, m	Chloride Sample Depth, cm	Chloride Ion Content, ppm
Pier 2, East Face (Postrite/DCI)	O (3.78, 0.40)	1.27 2.54 3.81 5.08	2248 3775 3484 3096
	I (2.10, 0.67)	1.27 2.54 3.81 5.08	1995 1829 628 411
	P (2.71, 0.61)	1.27 2.54 3.81 5.08	15 59 28 28
Pier 3, West Face (MCI 2000)	O (4.97, 0.92)*	1.27 2.54 3.81 5.08	5006 5722 4342 4036
	O (2.44, 0.92)*	1.27 2.54 3.81 5.08	4138 4521 3678 3014
	O (7.53, 0.46)	1.27 2.54 3.81 5.08	401 289 255 64
	I (4.97, 0.92)	1.27 2.54 3.81 5.08	2766 4950 2996 3428
	P (4.76, 0.85)	1.27 2.54 3.81 5.08	166 0 64 299

Notes:
* Preconstruction data
O = original concrete; P = patch concrete placed in 1992; I = interface of original and patch concrete
(0,0) coordinate is at the southern top corner of pier cap; x measured in the northerly direction and y measured in the southerly direction

During the third visit, chloride samples were collected from nine locations adjacent to the locations used on the first visit. At each location, 4 samples were collected at progressive increments of 13 mm down from the surface for a total of 36 samples. The chloride analyses data for the third visit are presented in table 3-8.

3.4. Conclusions

All patches on the control pier and the treated piers were experiencing shrinkage cracking. By the third evaluation, additional cracks had developed in the patches treated with Cortec inhibitor. Rust staining that originated in a crack at the patch concrete interface of a patch treated with the Cortec inhibitor was noted in the third evaluation. Hollow-sounding areas were observed on the patch of the pier treated with DCI inhibitor and they increased with time.

The chloride ion content at the steel depth in the patches was below the threshold required to initiate corrosion.

Corrosion rate measurements in patched areas were not accurate, because the correct surface area of the steel subjected to the measurement could not be determined. But the trends in the corrosion rate measurements indicated that corrosion rates in the patched areas remained relatively constant with time in the control pier and the treated piers, and the rates for the treated patches and the control patches were similar. Corrosion rates at the interface were generally higher than those in the patches.

The signs of ongoing corrosion in the patches of the treated areas and at the interfaces suggest that the inhibitors were not providing any protection.

Table 3-8. **Total chloride ion content data (third visit).**

Location/ Treatment	x, y Coordinates, m	Chloride Sample Depth, cm	Chloride Ion Content, ppm
Pier 1, East Face (control)	O	1.27 2.54 3.81 5.08	733 493 212 87
	I	1.27 2.54 3.81 5.08	1579 1551 1212 902
	P	1.27 2.54 3.81 5.08	54 45 93 212
Pier 2, East Face (Postrite/DCI)	O	1.27 2.54 3.81 5.08	3525 3666 3012 2820
	I	1.27 2.54 3.81 5.08	1692 2256 1500 1128
	P	1.27 2.54 3.81 5.08	87 62 0 42
Pier 3, West Face (MCI 2000)	O	1.27 2.54 3.81 5.08	240 129 53 70
	I	1.27 2.54 3.81 5.08	1410 4373 2816 3125
	P	1.27 2.54 3.81 5.08	2111 128 89 216

Notes:
O = original concrete; I = interface; P = patch

4.0. HOOD CANAL BRIDGE, PORT GAMBLE, WA

4.1. Structure Background

The Hood Canal bridge, located west of Seattle, WA, is a primary link to the Olympic Peninsula. The 2397-m-long pontoon bridge carries Washington Route 104 traffic across the Hood Canal, which contains brackish tidal water. Due to the extreme depth of the natural canal, greater than 92 m at mid-channel, the structure was designed as a floating bridge that opens at the center to allow passage of ship traffic. The roadway is elevated above the pontoon decks. The bridge substructure, columns, and crossbeams are supported by the pontoons. Figures 4-1 (a) and (b) show general views of the Hood Canal bridge.

Figure 4-1. General views of the Hood Canal bridge.

The floating bridge was constructed in 1961. In 1979, the western half of the bridge sank in a storm and was subsequently replaced. Corrosion of the concrete reinforcing steel in the pontoon decks and substructure members is evident throughout the older eastern half of the floating bridge. Previous repairs to the eastern half of the bridge included patching of the pontoon decks, columns, and crossbeams, and the coating of these surfaces with epoxy. Even though the pontoon decks are about 1.5 m above high tide, they are constantly exposed to wind-blown brackish water mist, especially on the southern side of the decks. Water tends to pond on the deck surface. As a result, corrosion has continued unabated in the original and repaired concrete areas.

In July 1992, the three southern pontoon deck cells (1D, 2D, and 3D) of pontoon S on the eastern half of the bridge, each measuring 3.66 m by 9.14 m, were selected as a SHRP C-103 corrosion-inhibitor-treatment field-trial site. Delaminated areas on these cells were identified and delineated. Hollow-sounding areas in cells 1D, 2D, and 3D were found to be 8.9, 25.6, and 30.4 percent, respectively. Deteriorated concrete from the delaminated areas was removed to a depth of about 19 mm below the top reinforcing steel and the cavities backfilled with non-air-entrained cementitious patching material. The exposed reinforcing steel in all three cells was sandblasted to near-white metal on the day of patching. Immediately prior to the placement of the repair concrete, patch cavities were painted with a neat cement slurry. Cell 1D had no inhibitor treatment and was designated as the control. Corrosion inhibitor treatment was applied to the patches in cells 2D and 3D. Cell 1D had 11 patches, cell 2D had 13 patches, and cell 3D had 9 patches. The size of the patches ranged from 0.072 m^2 to 2.6 m^2. Figures 4-2 (a), (b), and (c) show partial views of the test locations.

Figure 4-2. Partial views of test cells on the Hood Canal bridge: (a) cell 1D (control), (b) cell 2D (MCI 2020/2000), (c) cell 3D (Postrite/DCI).

Cortec MCI 2020/2000 was used as the corrosion-inhibitor system in cell 2D. Cell 3D was treated with the Postrite/DCI inhibitor system. Cortec 2020 was sprayed on the exposed reinforcing steel prior to the placement of the patch concrete, and Cortec 2000 was added to the patch concrete at a rate of 1.2 kg/m^3. Care was taken during the spraying of Cortec 2020 to avoid contact with substrate concrete, because Cortec 2020 has been reported to cause a detrimental reduction in bond strength. For the 3D cell repairs, Postrite was sprayed on the exposed reinforcing steel and substrate concrete prior to the placement of the repair concrete, which contained DCI at a dosage rate of 39.94 L/m^3.

4.2. Field Evaluation

Field evaluations were conducted under the FHWA contract and done on the following dates:

First evaluation	November 5 and 6, 1994	~2 years after treatment
Second evaluation	June 25 and 26, 1996	~4 years after treatment

A limited evaluation consisting of visual and delamination surveys was conducted under the SHRP program in October 1993. The observations from the 1993 survey are also included here.

4.3. Test Results

The six types of results collected are presented below.

4.3.1. Visual Survey

Visual inspections were performed in October 1993, November 1994, and June 1996, about 1.3 years, 2.4 years, and 4 years after the repairs were completed. During the October 1993 inspection, no cracking or spalling was observed in the control, Cortec, or Postrite/DCI cells. However, a 0.3-m-long crack was observed in patch # 2 in the Postrite/DCI cell during the November 1994 inspection. The crack appeared to be caused by drying shrinkage. No further cracks were documented during the 1996 evaluation.

4.3.2. Delamination Survey

Delamination surveys were conducted in October 1993 and November 1994. Due to inclement weather, the delamination survey could not be conducted during the 1996 evaluation. The results of the delamination survey are presented in table 4-1. Hollow-sounding areas were first detected 1.3 years after patching in the Cortec cell (cell 2D) and after 2.4 years in the control and Postrite/DCI cells (cells 1D and 3D, respectively). During the 1994 evaluation, hollow-sounding areas were detected in 2 of the 11 patches in cell 1D; 2 of the 13 patches in cell 2D; and 3 of the 9 patches in cell 3D. The corresponding delaminations were computed to be 2.2, 5.2, and 12.2 percent, respectively. The hollow-sounding areas in cell 2D increased in size between the first and second evaluations.

4.3.3. Clear Concrete Cover Survey

Table 4-2 presents the patch cover depths measured at about 3 days (July 1992) and 2.4 years (November 1994) after repair. The cover depths measured in July 1992 and November 1994 were in general agreement and showed that cover depths were shallow in all three cells. The average cover depths for cells 1D, 2D, and 3D were 3.35, 3.53, and 3.10 cm, respectively. The original concrete cover depths prior to repair were also shallow, with averages of 2.90, 2.87, and 3.10 cm for cells 1D, 2D, and 3D, respectively.

Table 4-1. Delamination survey results.

Cell	Patch No.	Patch Area, m^2	Hollow Sounding per Patch				Hollow Sounding for Each Cell			
			Oct. 1993		Nov. 1994		Oct. 1993		Nov. 1994	
			m^2	%	m^2	%	m^2	%	m^2	%
	1	0.07								
	2	0.29								
	3	0.14								
	4	0.20								
	5	0.42							0.07	2.2
1 D (control)	6	0.59								
	7	0.26								
	8	0.16								
	9	0.50								
	10	0.24								
	11	0.48			0.07	15.3				
	1	0.38								
	2	0.28								
	3	0.33								
	4	0.27	0.011	4.1	0.15	57.4				
	5	0.11								
2 D (Cortec MCI 2020/2000)	6	0.20					0.041	0.5	0.408	5.2
	7	0.36	0.030	8.2	0.25	70.7				
	8	0.35								
	9	0.25								
	10	2.31								
	11	1.30								
	12	1.46								
	13	0.28								
	1	0.40								
	2	0.60								
	3	0.69								
	4	0.74								
3 D (Postrite/ DCI)	5	0.99			0.31	31.1			0.995	12.2
	6	0.28								
	7	2.59								
	8	0.62			0.19	30.0				
	9	1.23			0.50	40.7				

Table 4-2. Clear concrete cover measurements.

Cell	Patch No.	Cover Depth, cm	
		July 1992	Nov. 1994
1 D (control)	1	3.96	4.45
	2	2.34	–
	3	3.35	–
	4	3.35	–
	5	2.74	2.92
	6	2.74	–
	7	4.06	–
	8	2.84	2.54
	9	3.15	2.79
	10	4.98	–
	11	4.47	4.06
	Average	***3.45***	***3.35***
	Std. Dev.	***0.81***	***0.84***
2 D (Cortec MCI 2020/2000)	1	4.98	–
	2	3.25	–
	3	3.76	3.43
	4	4.67	–
	5	–	–
	6	4.06	4.32
	7	2.84	–
	8	3.56	3.18
	9	3.15	3.56
	10	4.06	–
	11	–	3.15
	12	3.66	–
	13	3.76	–
	Average	***3.78***	***3.53***
	Std. Dev.	***0.64***	***0.48***
3 D (Postrite/DCI)	1	4.17	2.67
	2	5.99	–
	3	3.56	3.56
	4	5.69	–
	5	2.74	4.32
	6	3.35	–
	7	2.64	2.54
	8	4.17	–
	9	2.84	2.41
	Average	***3.91***	***3.10***
	Std. Dev.	***1.22***	***0.81***

4.3.4. Corrosion Potential Survey

Potential measurements were made at the center of each patch in the respective cells. Table 4-3 presents a summary of the corrosion potential distribution in the three cells measured just prior to repair and about 1.3 years, 2.4 years, and 4 years after repair.

Corrosion potentials were mostly in the active range before the repairs were performed, but were less active with time after the repairs. However, as apparent from table 4-3, a significant percentage of the potentials remained in the active range for the inhibitor-treated patches as compared to the control patches.

Table 4-3. Summary of corrosion potential measurements.

Cell No.	Date	No. of Potential Measure-ments	Average Potential, mV vs. CSE	Standard Deviation	% Corrosion Potentials, mV vs. CSE**		
					<-200	-200 to -350	>-350
1 D (control)	July 1992*	11	-440	104	0	27	73
	Oct. 1993	11	-260	48	0	91	9
	Nov. 1994	11	-266	36	9	91	0
	June 1996	11	-270	97	18	73	9
2 D (Cortec MCI 2020/2000)	July 1992*	13	-474	91	0	15	85
	Oct. 1993	12	-307	97	25	25	50
	Nov. 1994	13	-353	104	8	46	46
	June 1996	13	-352	123	8	54	38
3 D (Postrite/DCI)	July 1992*	9	-436	69	0	22	78
	Oct. 1993	9	-298	70	11	56	33
	Nov. 1994	9	-285	58	0	78	22
	June 1996	9	-260	83	33	44	22

* Pretreatment data
** millivolts measured against a copper-copper sulfate electrode

4.3.5. Corrosion Rate Survey

The corrosion rate measurements were made with an NBS-3LP device at the center of selected patches; results are presented in table 4-4. The average corrosion rates in all three test cells had progressively increased from the pre-repair values.

Table 4-4. Corrosion rate measurements.

Cell	Patch No.	Corrosion Rate, mA/m^2			
		July 1992	**Oct. 1993**	**Nov. 1994**	**June 1996**
1 D (control)	1		0.299	0.384	0.619
	2				
	3				
	4				
	5	0.042	0.254	0.196	0.220
	6				
	7				
	8		0.265	0.183	0.279
	9	0.149	0.323	0.310	0.465
	10				
	11		0.320	0.521	0.955
	Average	***0.095***	***0.292***	***0.319***	***0.508***
2 D (Cortec MCI 2020/2000)	1				
	2				
	3	0.379	0.369	0.248	0.333
	4				
	5				
	6		0.379	0.589	0.992
	7				
	8		0.410	0.777	0.718
	9		0.246	0.356	0.540
	10				
	11	0.443	0.264	0.304	0.303
	12				
	13		0.358		
	Average	***0.411***	***0.338***	***0.455***	***0.578***
3 D (Postrite/DCI)	1		0.370	0.407	0.512
	2				
	3	0.347	0.443	0.950	1.143
	4				
	5	0.135	0.277	0.432	0.990
	6				
	7	0.247	0.283	0.697	0.443
	8				
	9		0.246	0.640	0.998
	Average	***0.243***	***0.324***	***0.625***	***0.817***

* Pretreatment data

1 mA/m^2 = 10.753 mA/ft^2

Corrosion rates were higher for the inhibitor-treated patches than for the control. The Postrite/DCI-treated patches had the highest average corrosion rates and the largest increase with time.

4.3.6. Chloride Ion Content Analysis

The chloride content of the test cells was not determined before the November 1994 field evaluation, and no chloride data were collected in 1996. Data collected in 1994 are presented in table 4-5. The rate of chloride ingress into the patched areas appeared to be fairly rapid. The chloride content near the steel depth in the patches, as well as in the original concrete, was greater than the minimum threshold value of 260 ppm required to initiate corrosion.

4.4. Conclusions

With the exception of one crack observed during the first evaluation in a patch treated with Cortec inhibitor, no other cracking was noted in the patches. Delaminations were first detected in patches treated by the Cortec inhibitor and, with time, delaminations were noted in the control patches and patches treated with the DCI inhibitor.

The chloride ion content at the steel depth in patches generally exceeded the threshold required to initiate corrosion.

Average corrosion rates in patches increased with time for the patches in the control area and the treated patches, and were higher in treated patches than in the control patches. Also, the increase in corrosion rate with time in the treated patches was higher than that for the control patches.

Clear evidence of ongoing corrosion in inhibitor-treated patches indicated that the inhibitors were not providing any protection.

Table 4-5. Total chloride ion content data.

Cell	Patch No.	Steel Depth, cm	Sample Depth, cm	Cl Content, ppm
1 D (control)	Patch #5	2.92	1.27 2.54 3.81	1775 445 450
	Patch #11	4.06	1.27 2.54 3.81	2036 1130 368
	Near Patch #11*		1.27 2.54 3.81	2665 2220 1353
2 D (Cortec MCI 2020/2000)	Patch #3	3.43	1.27 2.54 3.81	1217 379 176
	Patch #8	3.18	1.27 2.54 3.81	2151 1435 565
	Near Patch #8*		1.27 2.54 3.81	3223 3350 2309
3 D (Postrite/DCI)	Patch #3	3.56	1.27 2.54 3.81	1013 499 266
	Patch #9	2.41	1.27 2.54 3.81	719 366 258
	Near Patch #3*		1.27 2.54 3.81	2409 1524 1072

* Original concrete

5.0. TRUNK HIGHWAY (TH)-3 OVER SOUTHVIEW BOULEVARD IN ST. PAUL, MN

5.1. Structure Background

The TH-3 bridge over Southview Boulevard is located in south St. Paul, MN. TH-3 (recently renamed TH-52) is a main northbound arterial carrying southern traffic into St. Paul. The bridge was built in 1973 and is approximately 50 m long and 14 m wide, with one breakdown lane and two traffic lanes. The breakdown and right traffic lanes are approximately 3.35 m and 3.65 m wide, respectively. There is also a 0.92-m-wide shoulder adjoining the left traffic lane. Spans 1, 2, and 3 are 13.5 m, 23.0 m, and 12.5 m long, respectively. The deck, parapets, and substructure are cast-in-place reinforced concrete. The superstructure consists of precast, prestressed I-beams. Figures 5-1 (a) and (b) show general views of the structure and deck, respectively.

Figure 5-1. TH-3 over the Southview Boulevard bridge: (a) structure and (b) deck.

The deck was rehabilitated and overlaid with a 5.08-cm Low-Slump Dense Concrete (LSDC) mix in May-June 1992. About 3.81 cm of the original chloride-contaminated concrete was milled off to the top layer of reinforcing steel and unsound areas were removed with a pneumatic hammer. The entire deck surface was sandblasted to remove laitances prior to placement of the overlay.

Two corrosion-inhibitor treatment methods were used in rehabilitation of the deck. Cortec MCI 2000 (an organic amine) was used in span 1 and was added to the overlay concrete at a rate of 1.2 L/m^3. Span 2 was treated with DCI (calcium nitrite), which was added to the overlay concrete at a rate of 19.96 L/m^3. In addition, span 2 received three spray-on applications of Postrite (a 15-percent solution of calcium nitrite) prior to the placement of the DCI-treated overlay. The application rate for Postrite was about 3.75 L/m^3. Span 3 was designated as the control span and was overlaid with standard Minnesota Department of Transportation (MNDOT) LSDC mix.

5.2. Field Evaluation

Field evaluations were performed on the following dates:

First evaluation	October 5-7, 1994	~2.4 years after treatment
Second evaluation	August 1-2, 1996	~4.3 years after treatment
Third evaluation	May 12-13, 1998	~6.0 years after treatment

Some data collected in May 1992 (prior to the rehabilitation) and in May 1994 (about 2 years after the inhibitor treatment) are also included in this report for comparison.

5.3. Test Results

The results of six types of data collection are presented below.

5.3.1. Visual Survey

The visual survey results (crack-length measurements) for all three evaluations are presented in table 5-1.

During the first evaluation, the concrete surface of the right traffic and breakdown lanes appeared to be sound. However, some longitudinal cracking was observed on the deck surface. Span 1 (Cortec 2000 treatment) exhibited larger cracks than span 2 (Postrite/DCI treatment) and span 3 (control). The crack frequencies for spans 1, 2, and 3 were 0.265 m/m^2, 0.004 m/m^2, and 0.053 m/m^2, respectively. All the cracks appeared to be caused by drying shrinkage. It should be mentioned that no cracking was observed in August 1992, about 3 months after placement of the overlay. Crack surveys were not performed between August 1992 and October 1994.

During the second evaluation, a visual survey of the right traffic lanes and the breakdown lanes was performed. Crack lengths in the right traffic lane were found to increase significantly since the first evaluation in October 1994. The DCI-treated span (span 2) exhibited significantly less cracking than the other two spans. Crack frequencies for the right lane of spans 1, 2, and 3 were 0.309 m/m^2, 0.018 m/m^2, and 0.280 m/m^2, respectively. Crack frequencies for the breakdown lane of spans 1, 2, and 3 were 0.034 m/m^2, 0.020 m/m^2, and 0.051 m/m^2, respectively.

During the third evaluation, the right traffic and breakdown lanes were found to be in good condition, with the exception of small areas that showed light scaling. A significant increase in crack length was evident in the breakdown lane of span 1 and span 3. Overall, the DCI- treated span exhibited less cracking than the other two spans, though there was an increase in the crack frequency when compared to the second evaluation. Crack frequencies for the right traffic lane were 0.309 m/m^2, 0.069 m/m^2, and 0.280 m/m^2 for spans 1, 2, and 3, respectively, and for the breakdown lane of spans 1, 2, and 3 were 0.131 m/m^2, 0.024 m/m^2, and 0.211 m/m^2, respectively.

Table 5-1. Crack survey results.

Date	Span	Location	Inhibitor Treatment	Area Surveyed, m^2	Total Crack Length, m	Crack Frequency, m/m^2
Oct. 1994	1	Right lane	Cortec	48.36	12.80	0.265
	2	Right lane	DCI	84.44	0.30	0.004
	3	Right lane	None	45.76	2.44	0.053
Aug. 1996	1 1	Right lane Breakdown lane	Cortec Cortec	48.36 44.36	14.94 1.52	0.309 0.034
	2 2	Right lane Breakdown lane	DCI DCI	84.44 77.38	1.52 1.52	0.018 0.020
	3 3	Right lane Breakdown lane	None None	45.76 41.94	12.80 2.13	0.280 0.051
May 1998	1 1	Right lane Breakdown lane	Cortec Cortec	48.36 44.36	14.94 5.79	0.309 0.131
	2 2	Right lane Breakdown lane	DCI DCI	84.44 77.38	5.79 1.83	0.069 0.024
	3 3	Right lane Breakdown lane	None None	45.76 41.94	12.80 8.84	0.280 0.211

5.3.2. Delamination Survey

No hollow-sounding areas were detected during any of the three evaluations.

5.3.3. Clear Concrete Cover Survey

The concrete cover depth survey results are shown in table 5-2. The mean cover depths for the Cortec (span 1), Postrite/DCI (span 2), and control (span 3) locations were 7.16, 6.50, and 7.95 cm, respectively. The variability of cover depths was the least for the Postrite/DCI span, with a coefficient of variation (CV) of 5.8 percent, and the greatest for the control span with a CV of 14.0 percent. The control span cover depth variability was considered to be typical.

Table 5-2. Cover survey results.

Date	Span	Inhibitor Treatment	No. of Measurements	Mean Depth, cm	Standard Deviation, cm	CV,%
Oct. 1994	1	Cortec	35	7.16	0.66	9.2
	2	DCI	35	6.50	0.38	5.8
	3	None	35	7.95	1.12	14.0

CV = Coefficient of variation

5.3.4. Corrosion Potential Survey

Corrosion potential measurements were conducted on a 0.61-m grid following ASTM C-876 during all three evaluations; the summary results are shown in table 5-3.

Table 5-3. TH-3 over Southview Boulevard bridge—corrosion potential summary

Date	Span	Inhibitor Treatment	No.**	Average Potential mV vs. CSE	Standard Deviation	% Potentials, mV vs. CSE***		
						<-200	-200 to -350	>-350
May 1992*	1	None	456	-145	77	91	9	0
	2	None	779	-120	71	97	3	0
	3	None	448	-137	75	93	7	0
Oct. 1994	1	Cortec	196	-142	35	94	6	0
	2	DCI	193	-89	30	100	0	0
	3	None	181	-150	50	88	12	0
Aug. 1996	1	Cortec	245	-162	44	82	18	0
	2	DCI	369	-132	56	88	12	0
	3	None	223	-174	57	75	25	0
May 1998	1	Cortec	244	-172	74	62	37	1
	2	DCI	413	-147	79	86	10	4
	3	None	215	-173	69	76	21	3

* = Pretreatment data
** = Total number of measurements
*** millivolts measured against a copper-copper sulfate electrode

The corrosion potential data interpretation guidelines have been discussed previously. It can be seen from Table 5-3 that the corrosion potentials were mostly in the passive range before the rehabilitation. About 2 years after the rehabilitation (first evaluation), the corrosion potential distribution was approximately the same. During the second evaluation, a shift was noticed in the corrosion potential distribution toward the uncertain range for all three spans; however, no active potentials were encountered. During the third evaluation, some active potentials were found that amounted to 1 percent, 4 percent, and 3 percent for spans 1, 2, and 3, respectively. Overall, the corrosion activity on this structure can be categorized as low.

5.3.5. Corrosion Rate Survey

Corrosion rate tests were conducted during all three evaluations with an NBS-3LP corrosion rate device and measurements were made at 32 locations (8 in span 1, 16 in span 2, and 8 in span 3). The results are shown in table 5-4 which also includes some data obtained in May 1992 before rehabilitation. The guidelines for interpreting corrosion rate data obtained with the 3LP device were discussed in an earlier section of this report.

The average corrosion rate in all three spans before rehabilitation was quite high (in the range of 5.70 to 7.27 $\mu A/cm^2$). The rates decreased significantly after about 2 years, as reflected by the data collected during the first evaluation in October 1994. However, there was no significant difference in the corrosion rates in the inhibitor-treated and untreated spans. Most corrosion rate measurements obtained for spans 1 and 2 during the second evaluation were comparable to measurements obtained during the first evaluation. The corrosion rate measurements obtained for span 3 were suspect, probably due to a measurement error; thus, they are not reported here. Data obtained during the third evaluation showed an increase in the average rates for span 3 (control), while the average rates for spans 1 and 2 remained about the same.

The corrosion rate measurements may be considered as moderately high as per the interpretation guidelines. However, they were not consistent with the largely passive corrosion potentials measured on the deck surface and the low levels of chloride measured at the depth of the reinforcing steel (see section below).

5.3.6. Chloride Ion Content Analysis

Powdered concrete samples were collected with a hammer drill from 15 different locations (5 on each span) during the first evalutation. At four locations on each span, sampling was at the approximate average steel depth, while at one location in each case, samples were collected at nominal 1.27-cm increments down to the average steel depth. The powdered samples were analyzed for total chloride ion content using the standard AASHTO T-260 procedure. The results are shown in table 5-5. As mentioned earlier, the average cover depths for the Cortec (span 1), DCI (span 2), and control (span 3) locations were 7.16, 6.50, and 7.95 cm, respectively.

Table 5-4. Corrosion rates.

Location	x, y Coordinates, m	Corrosion Rate, mA/m²			
		June 1992	Oct. 1994	Aug. 1996	May 1998
Span 1 (Cortec 2000)	0.92, 2.44	0.325	0.200	0.140	0.106
	4.58, 5.49	0.418	0.110	0.052	0.051
	2.75, 6.71	0.468	0.176	0.163	0.129
	0.92, 9.76	0.736	0.249	0.171	0.144
	6.41, 9.76	0.703	0.215	0.163	0.092
	5.80, 14.03	0.804	0.153	0.076	0.152
	2.14, 12.20	–	0.179	0.175	0.243
	5.19, 12.20	–	0.160	0.055	0.088
	Average	***0.576***	***0.180***	***0.125***	***0.126***
Span 2 (DCI)	0.92, 15.25	0.359	0.164	0.177	0.005
	2.75, 17.08	0.471	0.219	0.265	0.003
	4.27, 18.30	–	0.125	0.148	0.012
	5.19, 18.91	0.351	0.115	0.136	0.014
	1.53, 22.57	0.399	0.216	0.292	0.304
	6.10, 22.57	–	0.170	0.136	0.203
	3.97, 25.01	0.614	0.160	0.169	0.220
	2.44, 27.45	–	0.311	0.257	0.305
	4.27, 30.50	–	0.153	0.195	0.208
	5.19, 30.50	–	0.139	0.148	0.140
	6.10, 30.50	–	0.176	0.174	0.259
	0.31, 30.50	–	0.100	0.195	0.267
	2.14, 32.33	0.557	0.265	0.250	0.299
	5.80, 33.55	0.468	0.182	0.166	0.198
	3.97, 35.99	0.404	0.147	0.166	0.238
	6.41, 27.45	0.556	0.226	0.204	0.242
	Average	***0.497***	***0.173***	***0.187***	***0.232***
Span 3 (control)	0.92, 37.21	0.325	0.220	–	0.490
	3.36, 39.04	0.418	0.218	–	0.485
	4.27, 42.09	–	0.154	–	0.280
	6.41, 43.92	0.469	0.240	–	0.424
	3.36, 45.14	0.736	0.224	–	0.509
	0.92, 45.75	0.703	0.249	–	0.709
	4.27, 47.58	–	0.168	–	0.288
	6.10, 47.58	–	0.241	–	0.487
	Average	***0.530***	***0.215***	–	***0.459***

Notes:
1 mA/m^2 = 10.753 mA/ft^2
(0,0) coordinate is at the southeast corner of the structure; x is measured in a westerly direction and y is measured in a northerly direction

Table 5-5. TH-3 over Southview Boulevard bridge—chloride ion content.

Span No.	Location (x, y Coordinates, m)	Cl Sample Depth, cm	Total Cl Ion Content, ppm
1 (Cortec)	Right traffic lane (6.41, 9.76)	1.27	1573
		2.54	274
		3.81	61
		5.08	72
		6.35	276
		7.62	138
	Breakdown lane (0.92, 2.44)	7.62	481
	Right traffic lane (2.75, 8.54)	7.62	28
	Right traffic lane (4.58, 5.49)	7.62	49
	Right traffic lane (5.80, 14.03)	7.62	92
2 (DCI)	Right traffic lane (5.80, 33.55)	1.27	558
		2.54	61
		3.81	79
		5.08	417
		6.35	350
		7.62	92
	Right traffic lane (6.41, 27.45)	6.35	87
	Right traffic lane (3.97, 35.99)	6.35	253
	Breakdown lane (0.31, 30.50)	6.35	3
	Breakdown lane (2.14, 32.33)	6.35	161
3 (control)	Right traffic lane (6.41, 43.92)	1.27	1197
		2.54	269
		3.81	74
		5.08	31
		6.35	69
		7.62	110
		8.89	41
	Right traffic lane (3.36, 39.04)	8.89	33
	Right traffic lane (3.36, 45.14)	8.89	28
	Breakdown lane (0.92, 37.21)	8.89	143
	Breakdown lane (0.92, 45.75)	8.89	59

Note:
The average chloride content at the average steel depth prior to rehabilitation was below the threshold level of 260 ppm.

The average steel depths for spans 1 and 3 were approximately 7.62 cm; span 2 was approximately 6.35 cm. Total chloride ion content at the average steel depth in span 1 (except for one location) was below the minimum threshold value of the 260 ppm required to initiate corrosion of steel embedded in concrete. A similar situation was evidenced in span 2. In span 3 (control), the chloride ion content was much below the threshold value. Limited chloride analysis was also carried out during the third evaluation and showed the same trends. The results are not reported here.

The chloride content at the average steel depth in the right traffic lane before rehabilitation of the bridge deck in May-June 1992 was below the minimum threshold value of 260 ppm for all three spans.

5.4. Conclusions

Cracking of the overlay was prevalent in all spans and increased with time. The span treated with the Cortec inhibitor exhibited the highest crack density, followed by the control, and the DCI-treated span exhibited the least amount of cracking. No delaminations were detected in any evaluations.

The average corrosion rates were in a range that could result in damage in 2 to 10 years and any corrosion would increase with time. The average corrosion rates in the inhibitor-treated spans were higher than the control span. The corrosion rates were probably exaggerated due to deep cover over the reinforcing steel.

The chloride ion content at the steel depth prior to the installation of the repairs was below the threshold and the corrosivity of the environment was mild. It was difficult to judge the performance of the inhibitors at this site.

6.0. CONCLUSIONS

Based on the findings of the long-term evaluations, the following conclusions were reached:

1. In three out of the four structures evaluated in this study, the control areas and the treated areas experienced shrinkage cracking.

2. Signs of ongoing corrosion in patched areas treated with corrosion inhibitors were noted in three out of the four structures.

3. The corrosion inhibitors did not provide any protection against corrosion in the environments in which they were evaluated.

7.0. REFERENCES

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