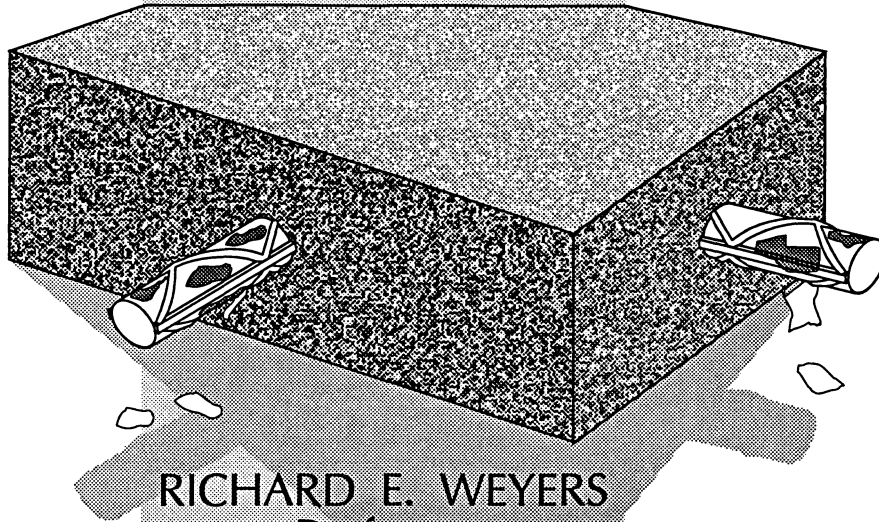


FINAL REPORT

**FIELD INVESTIGATION
OF THE CORROSION PROTECTION
PERFORMANCE OF BRIDGE DECKS AND PILES
CONSTRUCTED WITH EPOXY-COATED
REINFORCING STEEL IN VIRGINIA**



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VIRGINIA TRANSPORTATION RESEARCH COUNCIL

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Supplementary Notes				
<p>Abstract</p> <p>The corrosion protection performance of epoxy-coated reinforcing steel (ECR) was assessed in three bridge decks and the piles in three marine structures in Virginia in 1996. The decks were 17 years old, two of the marine structures were 8 years old, and the other marine structure was 7 years old at the time of the investigation. The deck investigations included visually surveying surface cracks in the right traffic lane and drilling 12 cores randomly located in the lowest 12th percentile cover depth. The pile investigations included removing 1 core at an elevation between high and low tides from each of 30 piles. The evaluation of the concrete in each core included visually inspecting and measuring moisture content, absorption, percent saturation, carbonation depth, and effective chloride diffusion constant. The evaluation of the ECR from each core included visually inspecting and measuring physical damage, coating thickness, adhesion loss and corrosion at damaged sites, and undercoating corrosion at adhesion test sites. The chloride content of the concrete and the carbonation of the ECR trace were also determined for each core.</p> <p>In the majority of bars examined, the epoxy coating has debonded or is debonding from the reinforcing bar. This occurs without the presence of chloride, and its rate is related to concrete moisture conditions, temperature, coating defects, and other bar and coating properties. Based on the results of this field study, epoxy coatings can be expected to debond from reinforcing steel in Virginia's marine environments in about 6 years and from bridge decks in about 15 years.</p> <p>The authors recommend that additional bridge decks be evaluated to confirm these results.</p>				

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(The opinions, findings, and conclusions expressed in this
report are those of the authors and not necessarily
those of the sponsoring agencies.)

Virginia Transportation Research Council
(A Cooperative Organization Sponsored Jointly by the
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ABSTRACT

The corrosion protection performance of epoxy-coated reinforcing steel (ECR) was assessed in three bridge decks and the piles in three marine structures in Virginia in 1996. The decks were 17 years old, two of the marine structures were 8 years old, and the other marine structure was 7 years old at the time of the investigation. The deck investigations included visually surveying surface cracks in the right traffic lane and drilling 12 cores randomly located in the lowest 12th percentile cover depth. The pile investigations included removing 1 core at an elevation between high and low tides from each of 30 piles. The evaluation of the concrete in each core included visually inspecting and measuring moisture content, absorption, percent saturation, carbonation depth, and effective chloride diffusion constant. The evaluation of the ECR in each core included visually inspecting and measuring physical damage, coating thickness, adhesion loss and corrosion at damaged sites, and undercoating corrosion at adhesion test sites. The chloride content of the concrete and the carbonation of the ECR trace were also determined for each core.

In the majority of the bars examined, the epoxy coating has debonded and is debonding from the reinforcing bar. This occurs without the presence of chloride, and its rate is related to concrete moisture conditions, temperature, coating defects, and other bar and coating properties. Based on the results of this field study, epoxy coatings can be expected to debond from reinforcing steel in Virginia's marine environments in about 6 years and from bridge decks in about 15 years.

The authors recommend that additional bridge decks be evaluated to confirm these results.

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INTRODUCTION

The extent of the rapid deterioration of reinforced concrete bridges from chloride ion-induced corrosion is well known, and a multitude of corrosion abatement techniques have been developed for existing and newly constructed bridges to address this problem. The use of epoxy-coated reinforcing steel (ECR) was one of the techniques developed to extend the service life of newly constructed concrete bridge components. ECR was first used in the construction of a bridge deck in Pennsylvania in 1973 under the Federal Highway Administration's (FHWA) National Experimental and Evaluation Program (NEEP) Project No. 16.¹ By 1976, 40 bridge decks had been constructed with ECR in 18 states and the District of Columbia under this program. Currently, ECR is the most used corrosion protection method for concrete bridges in the United States.

Until 1986, when Florida reported that the Long Key Bridge showed signs of corrosion only 6 years after construction, the corrosion protection effectiveness of ECR remained unquestioned.² Since then, 12 other field studies have been conducted.³ Conclusions have been mixed, from satisfactory performance for bridge decks to poor performance for substructures, with predictions that ECR will not provide long-term (50 years) corrosion protection for either.³ The reasons for the conflicting findings include limited or inappropriate evaluation methods that always accompany a lack of knowledge of the cause(s) of failure and subjectively defined failure criteria. Thus, the Virginia Department of Transportation (VDOT) initiated a field investigation to determine the performance of ECR in Virginia.

PURPOSE AND SCOPE

The purpose of this field investigation was to determine the performance of ECR in Virginia. The investigation included a review of the literature and the evaluation of three bridge decks in a deicing salt environment and the piles in three substructures in a marine environment.

METHODS

The investigation followed the guidelines of the NCHRP 10-37B protocol, which presents the measurable performance indicators, investigative procedures and plans, and methods of analysis.³

Bridge Decks

Selection and Description

The three bridge decks were selected from the Salem District in Southwest Virginia, which uses approximately 25 salt applications per winter maintenance season. The oldest bridge decks constructed with ECR were identified: 10 with final posting dates in 1979. Upon visual inspection, none of the 10 bridges showed any deterioration as spalls or patched spalled areas. The criteria used to select the bridges for this study were as follows: a multispan structure, a minimum of two lanes in a single traffic direction, and moderate to high average daily traffic. Three decks were selected following an initial visual inspection: structure number (SN) 1026, 220 Northbound, Botetourt County; SN 1029, 220 Northbound, Franklin County; and SN 8003, Prices' Fork Road, Northbound, Town of Blacksburg. The decks are within an 81-km radius of Roanoke, Virginia.

The three bridge decks were constructed with ECR in the top mat only and were about 17 years old at the time of the field investigations in April-May 1996. SN 1026 is on an upgrade and is a three-span simply supported steel girder creek crossing structure with stay-in-place forms. SN 1029 is on a downgrade and is a three-span simply supported steel girder creek crossing structure with conventional forms. SN 8003 is on a downgrade curve and is a two-span continuous overpass structure with stay-in-place forms.

Field Survey

The field investigation was limited to the right lane only for reasons of worker safety and because typically the right lane of bridge decks is the first lane to deteriorate. The field

investigations consisted of mapping the visual cracks and drilling 12 cores 102 mm in diameter through a reinforcing bar 16 mm in diameter. A sample size of 12 to 15 was chosen as a reasonable batch size for not wrongly accepting measured parameters.³ The sample locations were chosen randomly from the 12th percentile of the lowest cover meter depth readings, and no effort was made either to include or exclude areas with cracks above the reinforcing steel. A minimum of 40 cover depth readings were taken on each span, and three cores were taken from each span or one-third deck section. The 12th percentile cover depth sample criterion represents the amount of deck deterioration defined as the end of functional service in the worst deteriorated lane.⁴ In addition, powdered chloride samples were taken at depths of 13, 25, 38, and 51 mm adjacent to each core location using a carbide bit 28 mm in diameter and the sampling equipment presented in AASHTO T 260, Section 16. Figures 1, 2, and 3 show the deck investigation area, visual cracks, and core and chloride sample locations for SN 1026, 1029, and 8003, respectively.

The drilled cores were allowed to surface dry at the bridge site before being numbered and wrapped with a 0.10-mm sheet of polyethylene, aluminum foil, a 0.10-mm sheet of polyethylene, and duct tape to maintain the in-place moisture content of the cores.

Laboratory Evaluation

Each core was removed from the plastic-covered transport container and unwrapped immediately before the laboratory evaluation. The laboratory evaluation consisted of visual inspection of the core, measurement of the cover depth, and measurement of the combined AC impedance of the cover concrete and ECR. If a crack was observed in the surface above the ECR, a 102-mm cold chisel was placed over the crack and the core split along the crack plane to remove the ECR. If no crack was observed, the core was wrapped with duct tape and cut with a water-cooled saw over and parallel with the ECR to a depth of 13 mm from the top and bottom of the ECR and then split with cold chisel to remove the ECR.

Upon removal of the ECR, the depth of carbonation was determined and carbonation was tested in the ECR trace. The acid-soluble chloride content of the concrete was determined in the ECR trace from the drilled powdered samples. If the concrete was cracked to or partially to the depth of the ECR, the chloride content was also determined at the bar depth but generally about 50 mm from the ECR trace. A section of concrete at the depth of the ECR was broken from the split cores, and the moisture content and absorption were determined in accordance with ASTM C 642, Standard Test Method for Specific Gravity, Absorption and Voids in Hardened Concrete.

The ECR was visually inspected for coating defects (cracks, mashed areas, holes, and areas peeled from the bar and left in the concrete trace). The visual corroded area was measured, and the number of holidays and dry knife adhesion were determined. Twelve measurements were taken between and on the cross and longitudinal ribs. Thus, a total of 36 coating thickness measurements were made with an electromagnetic gage, Minitest 500 made by Elektro-Phisik, Germany. Coating adhesion was measured on the top and bottom of the ECR at three locations.

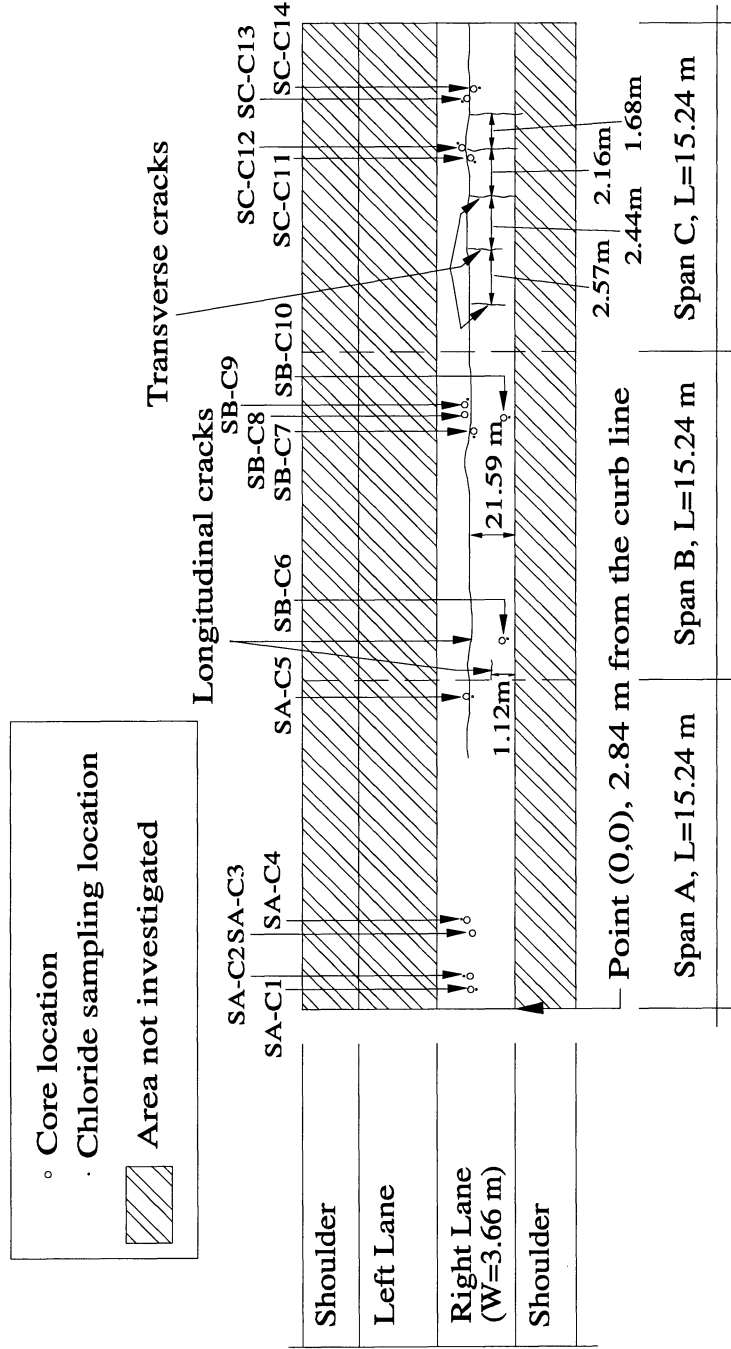


Figure 1. Bridge Deck Investigation Area, Sample Location and Cracking of SN 1026, Botatourt County

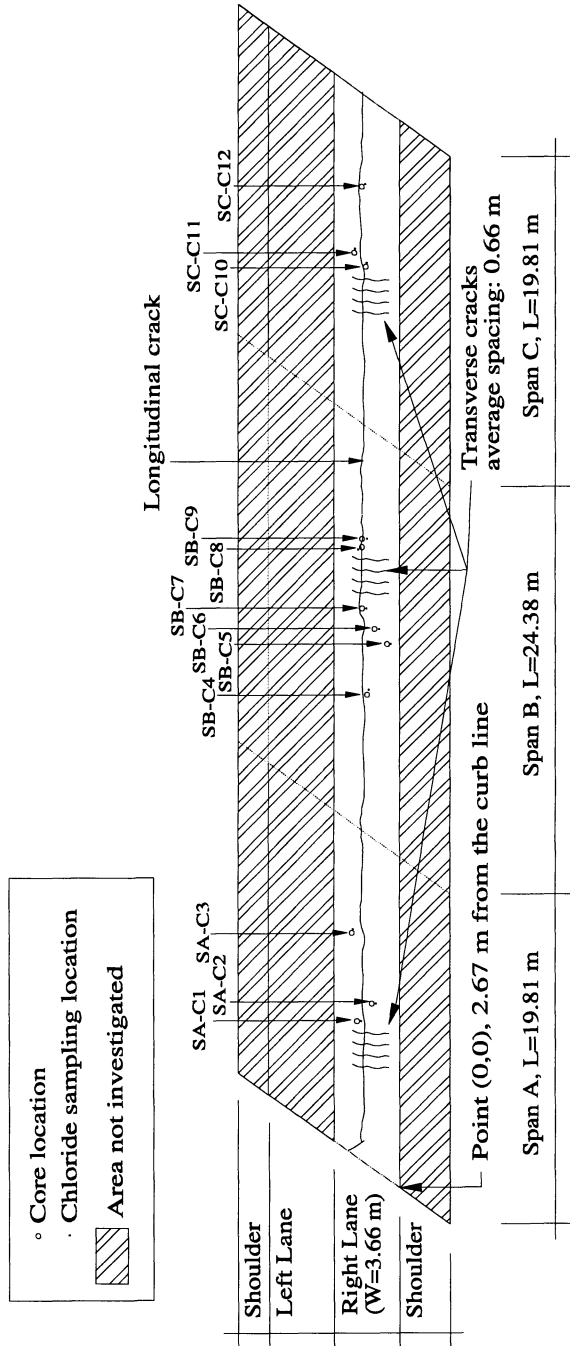


Figure 2. Bridge Deck Investigation Area, Sample Location and Cracking of SN 1029, Franklin County

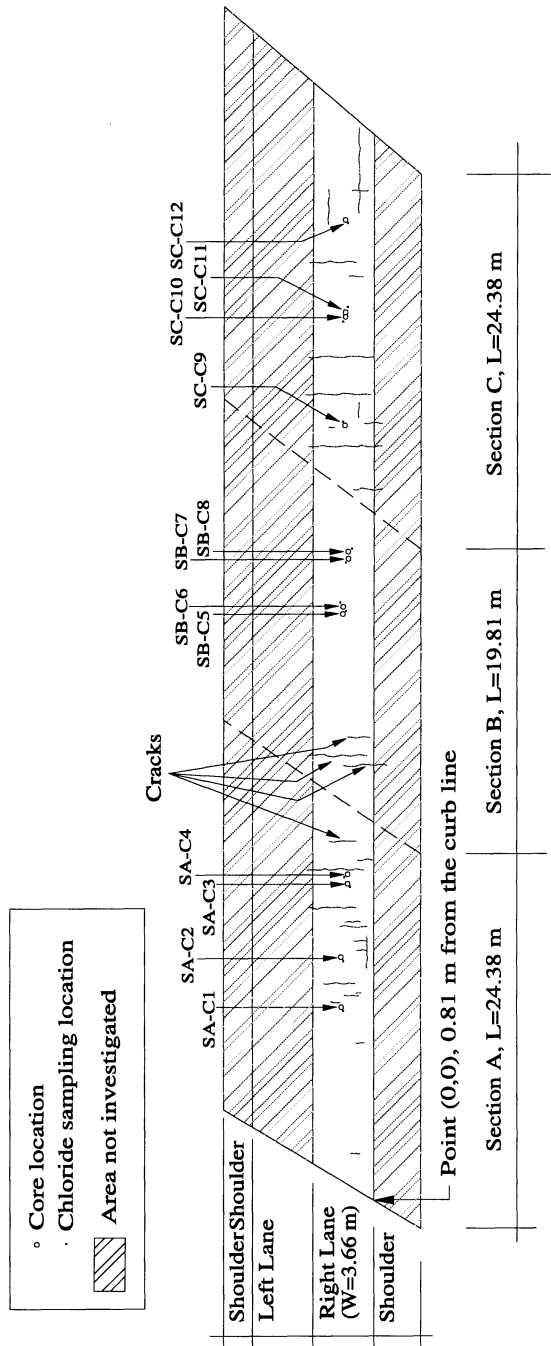


Figure 3. Bridge Deck Investigation Area, Sample Location and Cracking of SN 8003, Blacksburg

If the ECR was oriented in the concrete such that a longitudinal rib was on the top, the adhesion was measured on the two sides of the ECR.

The acid-soluble chloride content was also determined from the field-drilled powdered samples. The effective chloride diffusion constant for each bridge was determined by a minimum of the sum of squared errors procedure, which is presented elsewhere.⁴

Piles

Selection and Description

The oldest substructures built in a marine environment with ECR were identified, and three were selected following an initial visual inspection. All three substructures were constructed with prestressed driven piles about 610 mm square. The tie bars are smooth ECR 9.5 mm in diameter, and the main reinforcing is uncoated prestressing cable. The three structures are SN 1965, Route 17 over Chuckatuck Creek; SN 1812, Route 258 over Mill Creek; and SN 1008, Route 182 over the Machipongo River. These substructures were selected to represent the brackest water conditions on Virginia's coastal highways. The structures were built in 1988, 1989, and 1988, and are in Virginia's Suffolk District. At the time of the field investigation, two of the structures were 8 years old and one was 7 years old. The piles are exposed to brackish water with a pH and percent chloride content of 6.8, 0.87 percent; 7.6, 1.06 percent; and 7.7, 1.54 percent for SN 1965, 1812, and 1008, respectively.

Field Survey

The field survey was limited to drilling one core from a pile through the ECR within the tidal zone. The cores were drilled in the approximate center of the piles on a straight section of the ECR. A total of 7, 11, and 12 cores 53 mm in diameter were taken from SN 1965, 1812, and 1008, respectively. Because of the difficulty in locating the small ECRs with relatively deep covers, the ECR was not always centered or near centered in the core and the core did not always break off at a depth below the ECR.

The drilled cores were allowed to surface dry at the bridge site before being numbered and wrapped with a 0.10-mm sheet of polyethylene, aluminum foil, a 0.10-mm sheet of polyethylene, and duct tape to maintain the in-place moisture content of the cores. It is Virginia's practice to coat the concrete of all prestressed concrete piles with two layers of epoxy coating. Thus, for piles in marine environments, Virginia uses a dual corrosion protection system: epoxy coating of the concrete surface and ECR.

Laboratory Evaluation

Each core was removed from the plastic-covered transport container and unwrapped immediately before the laboratory evaluation. Laboratory evaluation consisted of visual inspection of the core, measurement of the cover depth, and measurement of the combined AC impedance of the cover concrete and ECR. A 13-mm top section was sawn off the wrapped core just before the AC impedance measurement because of the anticipated high impedance of the concrete epoxy coating.

The wrapped cores were cut with a water-cooled saw over and parallel with the ECR to a depth near the ECR from the top and bottom of the core and then split with a cold chisel to remove the ECR. The top 13-mm section was used to determine the acid-soluble chloride content and the depth of carbonation. Upon removal of the ECR, the coating was visually inspected for defects, the coating thickness was measured at six locations on the top and bottom of the ECR, the number of holidays was determined, the dry knife adhesion of the epoxy coating was measured, and the corrosion state of the steel under the coating at the adhesion test sites was noted. The adhesion was measured at three locations on the top and bottom of the ECR.

A section of concrete at the depth of the ECR was broken from the split cores, and the moisture content and absorption were determined for each core included in the laboratory investigation. The acid-soluble chloride content of the concrete was also determined in the ECR trace from the drilled powdered samples. Only those cores with the ECR in the near center of the core and encased in concrete were included in the laboratory evaluation. A total of 23 cores were included: 6 from SN 1965, 9 from SN 1812, and 8 from SN 1008.

RESULTS

Decks

Visual Deck Cracks

Figures 1, 2, and 3 show the visually observed cracks in SN 1026, 1029, and 8003, respectively. Cracking occurred in both the transverse and longitudinal directions. In general, transverse cracks occurred over the ECR, and the longitudinal cracks over girders. The crack frequency for the inspected area was 0.510, 0.273, and 0.429 m/m² for SN 1026, 1029, and 8003, respectively. The cracking appeared to be related to the flexibility of the structure and the quality of the concrete. SN 1029 has the shortest spans and the highest quality concrete based on visual appearance, percent absorption, and chloride diffusion constants. SN 8003 has the poorest quality concrete and is a continuous structure. SN 1026 has the longest simply supported span, and the quality of the concrete in this deck is between that of SN 1029 and 8003.

Concrete Cores

All the cores from the three decks had the same visual appearance. The coarse aggregate used was a crushed limestone with a maximum size of 19 mm, and the fine aggregate was a natural sand. The aggregates were well graded and uniformly distributed, the cement matrix was gray with a normal amount of entrapped and entrained air, and the concrete was well consolidated.

As shown in Table 1, the number of cores with surface cracks was 6, 3, and 6 for SN 1026, 1029, and 8003, respectively. Surface crack widths in general were either 0.127 or 0.254 mm on the top surface. The carbonation depth was greater for those cores with surface cracks. Cores SA-C4, SB-C5, and SC-C9 from SN 8003 were carbonated to the depth of the ECR, but none of the ECR traces for the three decks was carbonated. The average cover depths for the 12 cores were 71, 58, and 54 mm for SN 1029, 1026, and 8003, respectively.

Table 2 presents the moisture content, absorption, percent saturation, effective chloride diffusion constants, and the linear relationships between the diffusion constant and the moisture content for the three decks. The statistical parameters were based on 12 samples for each deck. As shown, the percent saturation at the ECR was the same for all three decks, which confirms the previously presented results for bridge decks in Pennsylvania.⁵ However, the moisture content and absorption varied from the lowest to the highest for SN 1029, 1026, and 8003. The coefficients of variation showed that the moisture content, absorption, and saturation were relatively uniform throughout the decks. The percent absorption and effective diffusion constants showed that the quality of the concrete varied from highest to lowest for SN 1029, 1026, and 8003. There was a strong linear relationship between the effective chloride diffusion constant and the moisture content (slope = 0.025, correlation coefficient = 0.997), which shows that the greater the number of water-filled voids, the higher the effective diffusion constant. The annual temperature variations and aggregates for the bridge decks were almost identical. Thus, the quality and permeability of the in-place concrete are a function of the quality of the cement paste.

Table 2 also shows the times to reach the chloride corrosion threshold limit of 0.71 kg/m^3 for exposed steel for the lowest 12th percentile cover depth. The times determined from the diffusion analysis, which are presented elsewhere, were 70, 22, and 7 years for SN 1029, 1026, and 8003, respectively.^{4,6} The average service life of bridge decks in Virginia built with bare steel is 36 years, and the standard deviation of the normal distribution is 13 years.⁴ Thus, the times to initiate corrosion of 7 to 22 years represent the lower 3rd and 25th percentile, respectively, and 70 years represents the upper 1st percentile of the bridge decks in Virginia, considering that it takes 5 years from corrosion initiation to cracking of the cover concrete.

TABLE 1. CORE CRACKING, CARBONATION, AND COVER CONCRETE

Bridge SN Core No.	Surface Crack Width (mm)	Carbonation Depth (mm)	Trace	Concrete Cover (mm)
1026				
SA-C1	0.127	38	No	61
SA-C2	0.127	6	No	66
SA-C3	0.254	51	No	66
SB-C4	0.254	32	No	46
SB-C5	None	6	No	56
SB-C6	0.254	51	No	56
SB-C7	None	6	No	66
SB-C8	None	6	No	56
SB-C9	None	6	No	66
SC-C10	None	19	No	48
SC-C11	0.254	25	No	46
SC-C12	None	13	No	58
				$\bar{x} = 58$
1029				
SA-C1	None	None	No	66
SA-C2	None	None	No	76
SA-C4	None	2	No	66
SA-C5	0.504	None	No	69
SB-C6	None	3	No	81
SB-C7	None	3	No	64
SB-C8	None	3	No	66
SB-C10	0.254	3	No	64
SC-C11	None	None	No	74
SC-C12	None	None	No	76
SC-C13	None	3	No	76
SC-C14	0.05	3	No	76
				$\bar{x} = 71$
8003				
SA-C1	0.254	25	No	51
SA-C2	None	13	No	51
SA-C3	None	13	No	51
SA-C4	0.254	56	No	56
SB-C5	0.178	58	No	58
SB-C6	None	10	No	56
SB-C7	0.178	6	No	66
SB-C8	None	6	No	58
SC-C9	0.178	56	No	56
SC-C10	None	13	No	53
SC-C11	0.127	25	No	46
SC-C12	None	6	No	51
				$\bar{x} = 54$

TABLE 2. MOISTURE CONTENT, ABSORPTION, SATURATION, AND EFFECTIVE CHLORIDE DIFFUSION CONTENTS

Parameter	Structure Number		
	1026	1029	8003
Moisture content (%)			
Mean	4.64	4.11	5.80
Standard deviation	0.32	0.53	0.74
Coefficient of variation	6.8	13	13
Absorption (%)			
Mean	5.56	4.89	6.98
Standard deviation	0.31	0.55	0.81
Coefficient of variation	5.6	11	12
Saturation			
Mean	83.5	84.0	83.2
Standard deviation	3.2	4.4	3.6
Coefficient of variation	3.8	5.2	4.3
Diffusion constant (mm ² /yr)	30	14	80
Time to reach 0.71 kg Cl/m ³ for 12th percentile steel cover depth (yr)	22	70	7.0

Epoxy-Coated Reinforcing Steel

Table 3 presents the average (12 readings for each position) coating thickness between the ribs, on the cross ribs, and on the longitudinal ribs. The sum coating thickness is the average of the 36 readings. As shown, the coating thickness on the ribs was always greater than between the ribs. The coating thickness specification for these ECRs was 127 to 229 μm . The average thickness between the ribs was less than the specified minimum thickness only on SN 1029, and only 4 of the 12 bars were less than the minimum specified.

The average thickness between the ribs was greater than the specified thickness of 229 μm on one bar on SN 1026 and three bars on SN 8003. The coefficient of variation of the coating thickness between the ribs ranged from 15 to 29 percent, 6 to 21 percent, and 13 to 25 percent for SN 1026, 1029, and 8003, respectively. The average coefficients of variation were 20, 14, and 17 percent, respectively. Thus, the coating thickness between the ribs was relatively uniform. The sum coating thickness was more variable, with coefficient of variation ranges of 23 to 30 percent, 23 to 35 percent, and 23 to 34 percent and averages of 26, 30, and 26 percent for SN 1026, 1029, and 8003, respectively.

Table 3 also presents the percent visual damage (mashed areas, holes, and peeled areas) and the chloride concentration in and adjacent to the ECR trace (approximately 50 mm from the ECR at the ECR depth). As shown, only two samples from SN 8003 had a total coating damage of greater than 2 percent. However, corrosion was visible at the damaged areas on 19 of the 36 ECR samples. Of these 19, the chloride content in the ECR trace was greater than the threshold value of 0.71 kg/m³ on 11. Some of the corrosion may have taken place prior to the placing of

TABLE 3. COATING THICKNESS AND DAMAGE AND CONCRETE CHLORIDE IN AND ADJACENT TO ECR TRACE

Bridge SN Core No.	Coating Thickness (μm)				Damage (%)			Chloride (kg/m^3)	
	Between	Cross	Long	Sum	No Corr.	Corr.	Sum	Trace	Adj.
1026									
SA-C1	194	311	308	271	0.36	0.34	0.70	0.25	--
SA-C2	199	308	262	256	0.00	0.00	0.00	0.14	--
SA-C3	177	324	288	263	0.00	0.00	0.00	0.76	0.81
SB-C4	181	335	306	274	0.10	0.70	0.80	0.53	0.58
SB-C5	177	326	284	263	0.00	0.00	0.00	0.21	--
SB-C6	167	296	310	258	0.00	0.00	0.00	0.74	0.31
SB-C7	207	329	280	272	0.00	0.00	0.00	0.23	--
SB-C8	308	514	442	421	0.10	0.10	0.00	0.41	--
SB-C9	178	325	277	260	0.00	0.00	0.00	0.41	--
SC-C10	159	303	276	246	0.25	0.55	0.80	2.14	--
SC-C11	133	235	218	196	0.10	0.00	0.10	2.55	1.19
SC-C12	141	265	226	210	0.00	0.02	0.02	1.70	--
1029									
SA-C1	111	209	224	181	0.00	0.00	0.00	0.29	--
SA-C2	209	355	335	300	0.00	0.00	0.00	0.30	--
SA-C4	139	287	255	227	0.00	0.00	0.00	0.31	--
SA-C5	115	226	194	178	0.00	0.10	0.10	0.45	0.80
SB-C6	163	334	316	271	0.50	0.10	0.60	0.25	--
SB-C7	132	323	264	240	0.00	0.00	0.00	0.20	--
SB-C8	135	248	238	207	0.00	0.02	0.02	0.22	--
SB-C10	125	264	176	188	0.00	1.30	1.30	1.09	0.89
SC-C11	196	378	372	315	0.00	0.00	0.00	0.28	--
SC-C12	193	367	362	307	0.00	0.00	0.00	0.21	--
SC-C13	140	291	253	228	0.10	0.70	0.80	0.25	--
SC-C14	113	231	242	195	0.00	0.90	0.90	0.24	--
8003									
SA-C1	280	321	391	333	1.40	0.60	2.00	4.95	4.85
SA-C2	159	235	245	213	0.00	4.60	4.60	6.56	--
SA-C3	239	331	377	316	0.15	1.25	1.40	4.28	--
SA-C4	158	245	204	202	0.00	0.10	0.10	5.11	3.04
SB-C5	206	397	367	323	0.00	0.10	0.10	1.93	1.11
SB-C6	183	261	321	257	0.00	0.10	0.10	2.09	--
SB-C7	179	322	332	278	0.00	0.50	0.50	1.12	1.04
SB-C8	212	339	350	300	0.00	0.00	0.00	1.30	--
SC-C9	243	404	346	331	0.00	6.80	6.80	2.23	2.60
SC-C10	188	369	276	278	0.30	0.00	0.30	3.71	--
SC-C11	229	377	329	312	0.20	0.00	0.20	3.80	3.88
SC-C12	215	382	333	310	0.30	0.00	0.30	3.12	--

the concrete. Of interest is that four ECR samples with damage and no visible corrosion had chloride contents in the trace above the corrosion threshold level of $0.71 \text{ kg}/\text{m}^3$. However, the

amount of damage was low, less than 0.3 percent. Also, three ECR samples with no damage and no visible corrosion had chloride contents above 0.71 kg/m^3 .

With respect to the chloride content in the ECR trace, 5 on SN 1026, 1 on SN 1029, and all 12 on SN 8003 were above the corrosion threshold level of 0.71 kg/m^3 . The core locations with trace and adjacent chloride contents had visible surface cracks. However, the surface cracking did not appear to have significantly influenced the chloride content in the ECR trace because the chloride contents in and adjacent to the trace were approximately equal, except for cores SC-C11/SN 1026 and SA-C4/SN 8003.

Table 4 presents the combined AC impedance of the cover concrete and epoxy coating, percent of holes, number of holidays (*contin.* represents the condition where the holiday detector continuously beeped along the ECR sample), and the residual dry knife adhesion of the coating to the bar. An adhesion rating of 1 represents the adhesion characteristics of an ECR prior to being placed in concrete, and 5 represents no adhesion.

As shown, there does not appear to be a relationship between the impedance and any single parameter as holes, holidays, adhesion rating, and corrosion under the coating. However, the bridge deck with the lowest impedance values, SN 8003, had the highest chloride content in the ECR trace, the greatest number of holes in the coating, the greatest number of holidays, the most corrosion under the coating, and a large amount of adhesion loss. It is most likely that impedance is a complex relationship involving these parameters and the moisture content of the concrete. However, the adhesion rating and moisture content may be interrelated because the decks with the highest moisture content, SN 1026 and 8003, had the highest adhesion rating and thus the greatest adhesion loss.

As shown in Table 4, the percent bare area (holes) was very low. Only one specimen, SA-C2/SN 8003, exceeded 2 percent. With the exception of specimen SB-C10/SN 1029, which had 1.4 percent bare area, the other specimens had less than 1 percent. However, the holiday count was very high: 24 of the 36 specimens had continuous holidays, and the remaining 12 had 3 or more holidays. The holiday counts exceeded the specification limit of 2 per 305 mm of bar (the specimen bar length was 102 mm). The high holiday count was most likely related to the saturation of the epoxy coating, coating debondment, and/or corrosion of the steel under the coating rather than the as-construction condition.

Of the 72 average adhesion rates presented in Table 4, 4 were 1, 18 were 2, 11 were 3, 13 were 4, and 26 were 5. The highest adhesion loss was associated with the highest concrete moisture content (see Tables 2 and 4). Of the 18 cores with chloride contents greater than 0.71 kg/m^3 , corrosion was observed under the coating at adhesion test sights on 10 specimens. Corrosion was also observed under the coating of one specimen where the chloride content did not exceed 0.71 kg/m^3 . For the 8 specimens where corrosion was not observed under the coating, the chloride content in the trace ranged from 3.80 to 0.74 and the adhesion values ranged from 2 to 5.

TABLE 4. IMPEDANCE, HOLES, HOLIDAYS, AND DRY KNIFE ADHESION OF ECR SAMPLES

Bridge SN Core No.	Impedance (ohm x 10 ³)	Holes (%)	Holidays (No.)	Adhesion Rating		Corr. Under Coating	
				Top	Bottom	Top	Bottom
1026							
SA-C1	32.0	0.2	Contin.	5	5	None	Some
SA-C2	27.0	0.0	5	5	1	None	None
SA-C3	29.0	0.0	13	5	4	+Very little	None
SB-C4	9.3	0.1	Contin.	5	4	None	None
SB-C5	34.0	0.0	4	*5	5	None	None
SB-C6	6.8	0.0	10	*5	5	+None	None
SB-C7	--	0.0	5	5	3	None	None
SB-C8	30.0	0.1	10	5	2	None	None
SB-C9	11.5	0.0	Contin.	5	4	None	None
SC-C10	4.5	0.5	Contin.	5	4	+Some	Some
SC-C11	8.1	0.0	Contin.	4	2	+None	None
SC-C12	17.0	0.0	Contin.	3	5	+None	Some
1029							
SA-C1	2.3	0.0	Contin.	2	2	None	None
SA-C2	17.0	0.0	16	4	2	None	None
SA-C4	12.0	0.0	22	1	1	None	None
SA-C5	5.9	0.0	Contin.	2	2	None	None
SB-C6	9.5	0.0	Contin.	2	3	None	None
SB-C7	14.0	0.0	Contin.	2	2	None	None
SB-C8	9.8	0.0	Contin.	2	2	None	None
SB-C10	7.2	1.4	Contin.	2	2	+None	None
SC-C11	26.0	0.0	9	3	1	None	None
SC-C12	14.0	0.0	3	3	2	None	None
SC-C13	19.0	0.0	Contin.	3	2	None	None
SC-C14	19.0	0.9	Contin.	2	2	None	None
8003							
SA-C1	1.6	0.2	Contin.	*5	5	*+All	All
SA-C2	1.7	3.1	Contin.	5	4	+Some	Very little
SA-C3	1.6	0.3	Contin.	5	5	+All	Some
SA-C4	2.6	0.0	Contin.	5	5	+Very little	Some
SB-C5	6.1	0.0	Contin.	5	3	+ Very little	Some
SB-C6	3.6	0.1	Contin.	5	5	+None	Some
SB-C7	4.9	0.0	Contin.	3	3	+None	None
SB-C8	3.8	0.0	8	3	3	+None	None
SC-C9	2.6	0.3	Contin.	4	4	+Some	Some
SC-C10	5.5	0.0	Contin.	4	4	+None	None
SC-C11	3.5	0.0	Contin.	4	4	+None	None
SC-C12	4.0	0.0	3	5	5	+None	None

*Side 1, Side 2; +chloride content in trace greater than 0.71 kg/m³.

The epoxy coating is debonding from the steel bar surface in bridge decks in Virginia, and corrosion is occurring under the epoxy coating. Figure 4 presents Core SA-C1 SN 8003 and



Figure 4. Corrosion of ECR in Core SA-C1 from SN 8003, Blacksburg

shows corrosion in the ECR trace and on the ECR. Corrosion has occurred on the ribs, cross and longitudinal, and between the ribs. Core SA-C1 and C2 fractured at the depth of the ECR during removal and had corrosion products in the traces and entrapped air voids adjacent to the ECR. Fracture of these cores at the ECR depth occurred with very little resistance. The coring depth was at least 51 mm below the bottom of the ECR, and these two cores were the only cores that fractured at the depth of the ECR with such ease. It was our opinion, at the time of coring, that the corrosion of these ECRs had initiated a delamination crack at the depth of the ECR resulting in the fracturing of the concrete at the depth of ECR during removal. Subsequent sounding with a 1.36-kg hammer confirmed the delaminations at core locations SA-C1 and C2 from SN 8003. The delaminated areas at core locations SA-C1 and C2 were 0.60 and 0.14 m², respectively. The sounding at the remaining 10 core locations on SN 8003 indicated sound cover concrete.

Figures 5 and 6 present dry knife adhesion values of 1 and 3, respectively. An adhesion value of 1 represents a condition where very little to no coating could be removed by the knife prying action. As shown in Figure 6, some coating could be removed from the shiny metallic surface with no undercoating corrosion. Figures 7 and 8 represent an adhesion rating of 5, and Figure 7 shows white and black corrosion products. The initial corrosion state without the presence of oxygen is shown in Figure 7, and the advanced corrosion state with undercoating red oxide corrosion products is shown in Figure 8. These undercoating corrosion states were observed on the same ECR.

Piles

Concrete Cores

On these cores, the coarse aggregate was a crushed siliceous gravel with a maximum size of 19 mm, the fine aggregate was a natural sand, and a portion of the cement binder was a slag cement. The aggregates were well graded and uniformly distributed, the cement matrix was dense with a normal amount of entrapped and entrained air, and the concrete was well consolidated. All the cores with the exception of MC-8 had a well-adhered epoxy surface coating. The concrete surface of the pile from which core MC-8 was taken had not been coated with epoxy.

As shown in Table 5, there were no surface cracks on any of the cores and the carbonation depth was 0 for all practical purposes. The mean core cover depths for SN 1965, 1812, and 1008 were 63, 54, and 53 mm, respectively. The average cover depth for all three structures was 56 mm, and the standard deviation 7 mm. The cover depths ranged from 41 to 66 mm.

Table 6 presents the moisture content, absorption, and percent saturation for the three structures. The variability of the moisture content and absorption was greater for the piles than for the bridge decks. However, the moisture content, absorption, and percent saturation of the piles were within the range of those of the bridge decks except for structure SN 1965, which had a percent saturation of 89.5. The mean percent saturation of the remaining bridge deck and pile were between 82.6 and 84.0.

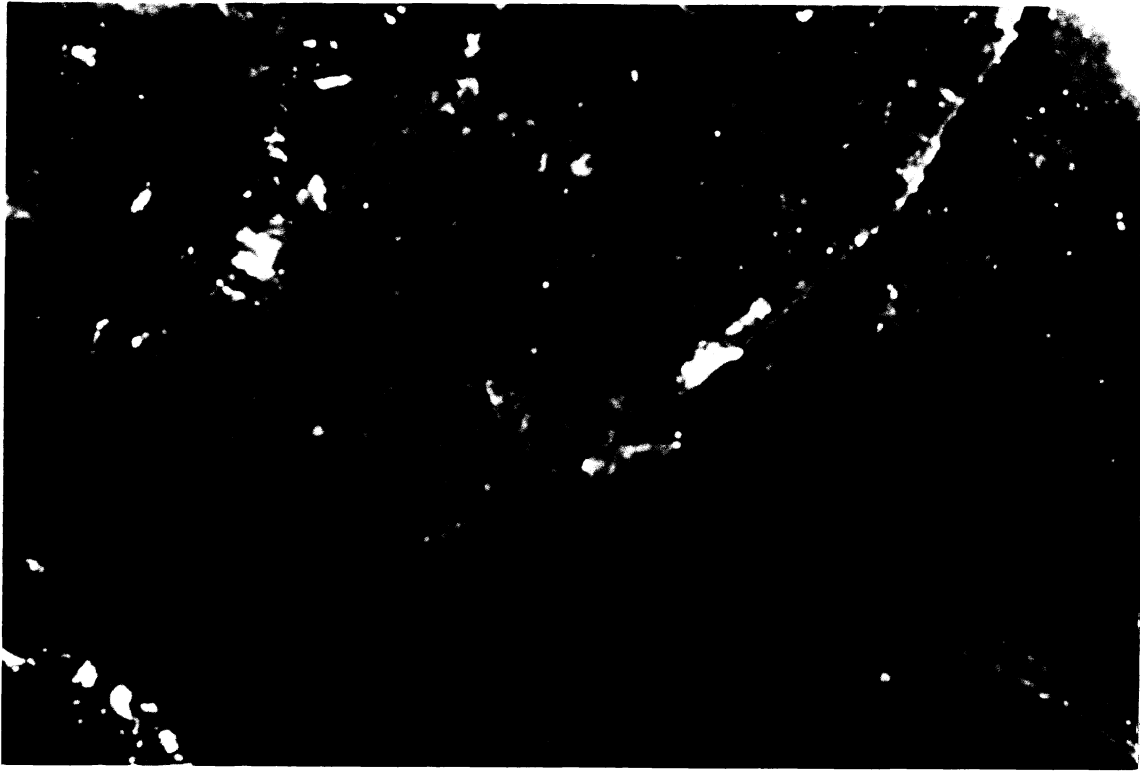


Figure 5. Adhesion Rating 1, Core SB-C7 from SN 8003, Blacksburg



Figure 6. Adhesion Rating 3, Core SB-C7 from SN 8003, Blacksburg

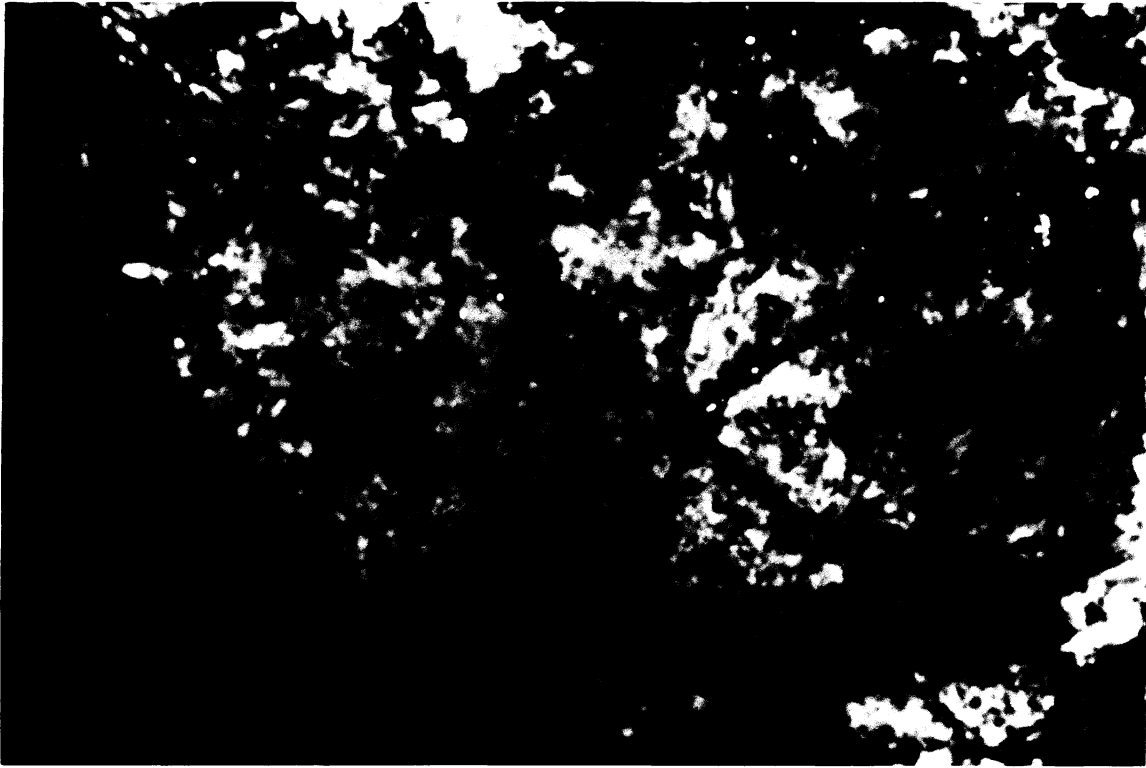


Figure 7. Adhesion Rating 5 and Initial Corrosion Stage Undercoating, Core SA-C1 from SN 8003, Blacksburg



Figure 8. Adhesion Rating 5 and Final Corrosion Stage Undercoating, Core SA-C1 from SN 8003, Blacksburg

TABLE 5. CORE CRACKING, CARBONATION, AND COVER DEPTH

Bridge SN Core No.	Surface Crack Width (mm)	Carbonation Depth (mm)	Concrete Cover (mm)
1965			
C-1	None	0	53
C-3	None	0	61
C-4	None	0	66
C-5	None	0	66
C-6	None	0	66
C-7	None	0	66
			$\bar{x} = 65$
1812			
MC-1	None	0	51
MC-2	None	0	51
MC-3	None	0	61
MC-4	None	0	48
MC-5	None	0	64
MC-6	None	0	51
MC-7	None	0	56
MC-8	None	0	48
MC-9	None	0	61
			$\bar{x} = 54$
1008			
MR-1	None	0	56
MR-2	None	0	41
MR-6	None	0	51
MR-8	None	0	51
MR-9	None	0	56
MR-10	None	0	51
MR-11	None	0	56
MR-12	None	0	58
			$\bar{x} = 53$

TABLE 6. MOISTURE CONTENT, ABSORPTION, AND SATURATION

Parameter	Structure Number		
	1965	1812	1008
Moisture content (%)			
Mean	4.05	5.06	4.14
Standard deviation	1.48	0.82	0.36
Coefficient of variation	36.6	16.3	8.8
Absorption (%)			
Mean	4.51	6.10	5.03
Standard deviation	1.28	0.89	0.49
Coefficient of variation	28.3	14.5	9.8
Saturation (%)			
Mean	89.5	82.8	82.6
Standard deviation	3.02	3.84	3.10
Coefficient of variation	3.4	4.6	3.8

Epoxy-Coated Reinforcing Steel

Table 7 presents the coating thickness, damage, and chloride content in the top 13-mm section and in the ECR trace. The coating thickness specifications for these smooth 9.5-mm-diameter tie bars was 127 to 229 μm . As shown in the sum column of Table 7, which is the average of the 12 readings, only 2 were less than 127 μm , 6 were greater than 229 μm , and 15 were within the range of 127 to 229 μm .

Only two ECRs showed damage: MR-2 and MR-6. Both exceeded the damage specification limits because of incomplete coating of the bars. Corrosion was visually evident in the holes created by the incomplete coating of MR-2. The corrosion most likely occurred prior to the casting of the pile because the chloride content in the ECR trace was significantly less than the chloride threshold level of 0.71 kg/m^3 . The chloride content in the trace of cores MR-2 and MR-6 was 0.20 and 0.26 kg/m^3 , respectively.

Table 8 presents the impedance, holidays, and dry knife adhesion of the epoxy coating. There was no clear relationship between the impedance and the damaged/holiday condition of the coating except, in general, when the holidays were continuous the impedance was low. It is most likely that impedance is a complex relationship between the conductivity of the concrete and the epoxy coating.

Because the cores were 53 mm in diameter, an ECR of 53 mm in length with 1 holiday would exceed the specification limit of 2 per 305 mm. Thus, 8 of the 23 ECR samples exceeded the holiday specification. However, 5 of the 8 had a continuous holiday signal, and 3 of these 5 had no visual evidence of coating imperfections. These three continuous holiday signals may be related to the saturating of the epoxy coating.

Of the three structures, the adhesion rating of SN 1812 was significantly better than that of SN 1965 and 1008. SN 1965 and 1008 had an adhesion rating of 5, complete coating debondment; SN 1812 ECR had blue epoxy coating; and SN 1965 and 1008 had a green epoxy coating. SN 1812 had three adhesion ratings of 5, three of 4, two of 3, four of 2, and five of 1.

TABLE 7. COATING THICKNESS AND DAMAGE AND CONCRETE CHLORIDE CONTENT

Bridge SN Core No.	Coating Thickness (μm)			Holes (%)			Chloride (kg/m^3)	
	Top	Bottom	Sum	No. Corr.	Corr.	Sum	Surface	Trace
1965								
C-1	66	165	116	0	0	0	0.82	0.47
C-3	258	228	241	0	0	0	0.38	0.38
C-4	165	122	144	0	0	0	0.15	0.38
C-5	313	301	307	0	0	0	0.30	0.38
C-6	171	213	192	0	0	0	0.29	0.21
C-7	165	148	157	0	0	0	0.41	0.17
1812								
MC-1	240	162	201	0	0	0	1.99	1.19
MC-2	167	177	172	0	0	0	1.52	0.63
MC-3	180	177	178	0	0	0	0.49	0.42
MC-4	89	71	80	0	0	0	0.16	0.11
MC-5	238	184	211	0	0	0	0.24	0.24
MC-6	261	179	220	0	0	0	0.33	0.15
MC-7	194	190	192	0	0	0	0.28	0.25
MC-8	239	186	213	0	0	0	6.86	2.18
MC-9	189	167	178	0	0	0	0.57	0.43
1008								
MR-1	321	174	247	0	0	0	0.34	0.22
MR-2	486	88	287	0.5	24.6	25.1	0.18	0.20
MR-6	148	110	129	20.1	0	20.1	0.30	0.26
MR-8	130	251	191	0	0	0	0.32	0.05
MR-9	428	530	479	0	0	0	0.33	0.35
MR-10	162	247	205	0	0	0	0.32	0.14
MR-11	273	180	226	0	0	0	0.23	0.15
MR-12	708	316	512	0	0	0	0.22	0.29

TABLE 8. IMPEDANCE, HOLIDAYS AND DRY KNIFE ADHESION OF ECR

Bridge SN Core No.	Impedance (ohm × 10 ³)	Holidays (No.)	Adhesion Rating		Corr. Under Coating	
			Top	Bottom	Top	Bottom
1965						
C-1	20	Contin.	5	5	None	None
C-3	460	0	5	5	Some	Very little
C-4	46	Contin.	5	5	Some	Very little
C-5	270	0	5	5	None	None
C-6	35	1	5	5	Very little	None
C-7	370	Contin.	5	5	None	None
1812						
MC-1	190	0	1	4	None	Oxide layer
MC-2	340	0	1	1	None	None
MC-3	1,100	0	2	1	None	None
MC-4	150	2	1	1	None	None
MC-5	--	1	4	3	Oxide layer	None
MC-6	210	0	5	3	None	None
MC-7	99	0	5	5	Oxide layer	Oxide layer
MC-8	35	0	4	2	Oxide layer	Oxide layer
MC-9	41	0	2	2	None	None
1008						
MR-1	160	0	5	5	None	None
MR-2	35	Contin.	5	5	All	Some
MR-6	47	Contin.	5	5	Some	None
MR-8	13	0	5	5	None	None
MR-9	1,100	0	5	5	None	None
MR-10	1,100	0	5	5	None	None
MR-11	1,100	0	5	5	None	None
MR-12	1,100	0	5	5	None	Some

An adhesion rating of 1 represents a newly coated bar prior to being placed in concrete. Figures 9, 10, and 11 present adhesion ratings of 1, 2, and 4, respectively, for SN 1812. Figure 12 presents an adhesion rating of 5 for SN 1965. The results of the adhesion tests clearly demonstrated that the epoxy coating was debonding prior to the arrival of the chlorides.

Of the 48 adhesion test locations, 15 had visual evidence of an oxide layer, or very little, some, or all the area under the test location was corroded (see Table 8). As shown in Table 7, only 2 of the 23 ECR traces had a chloride content greater than the corrosion threshold level and both of the ECRs had visual evidence of an oxide layer under the coating (see cores MC-1 and MC-8). The adhesion ratings for these 2 ECR samples ranged from 1 to 4 for MC-1 and 2 to 4 for MC-8.

It is interesting that the chloride content of core MC-8 was significantly higher than that of the other cores. MC-8 had no concrete epoxy surface coating, whereas the other cores were coated. This demonstrates the effectiveness of the concrete epoxy coating in excluding chlorides from the concrete.

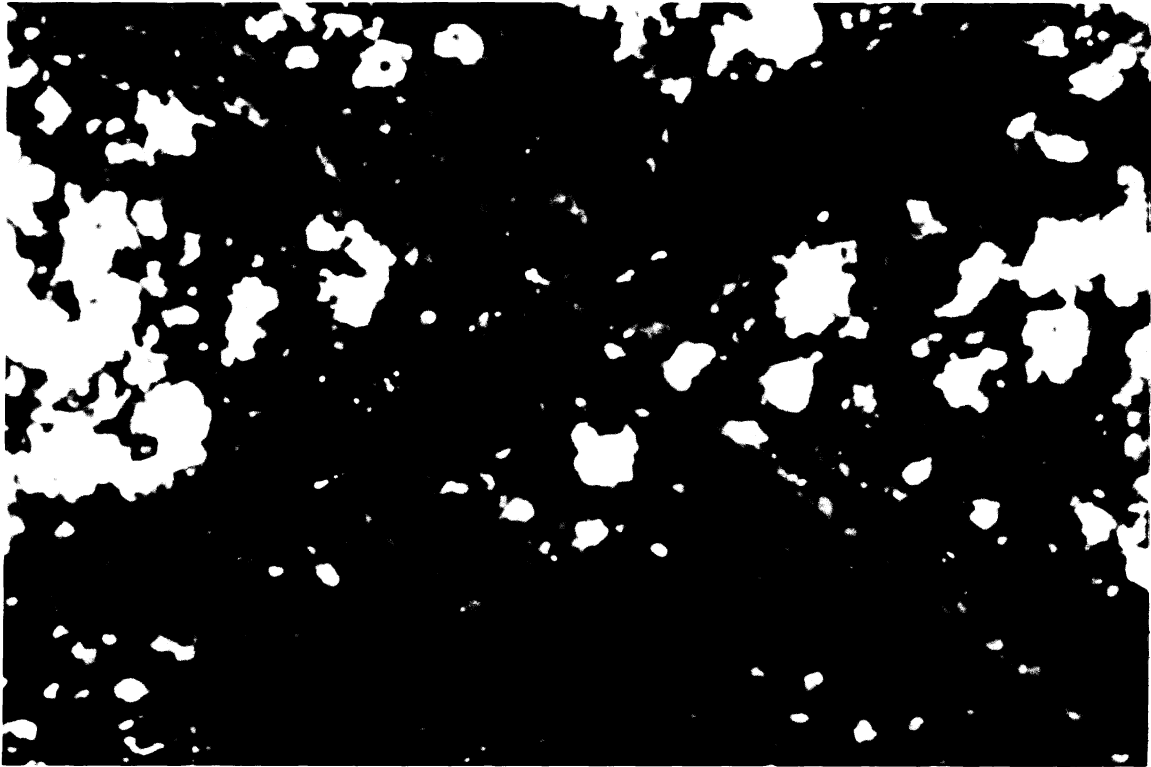


Figure 9. Adhesion Rating 1, SN 1812

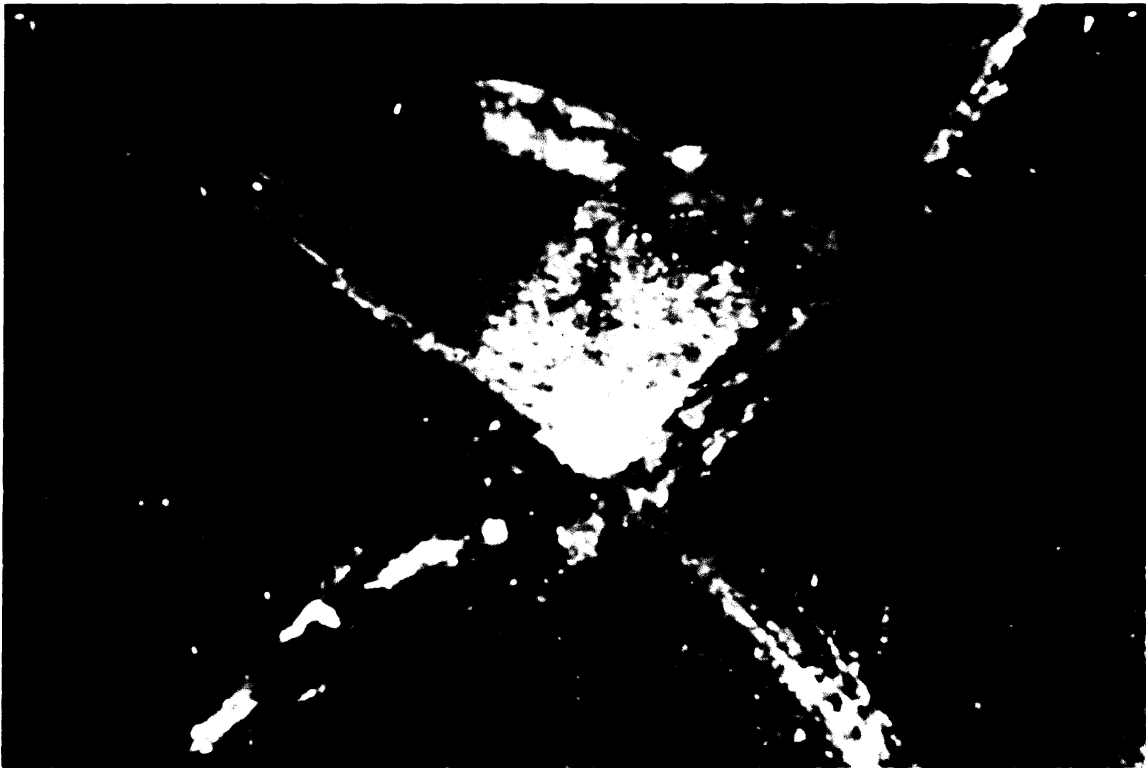


Figure 10. Adhesion Rating 2, SN 1812

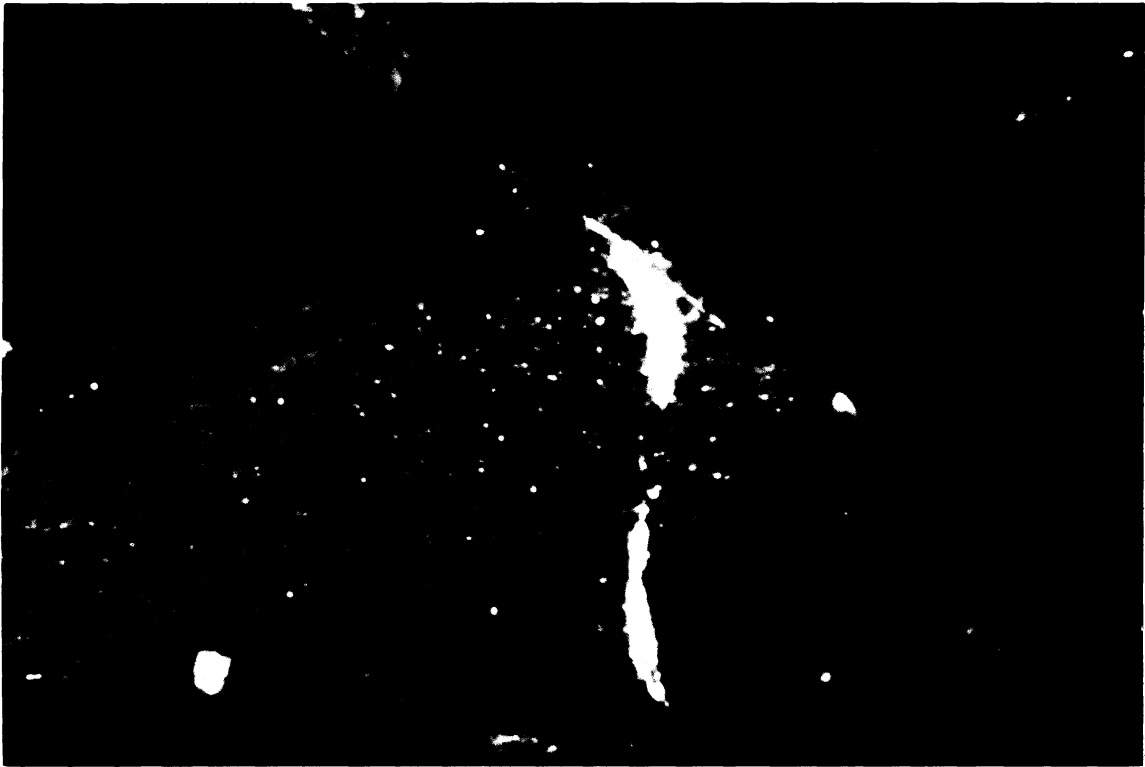


Figure 11. Adhesion Rating 4, SN 1812



Figure 12. Adhesion Rating 5, SN 1965

DISCUSSION

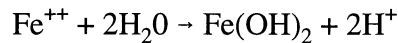
ECR Corrosion Mechanism

A review of the international literature of the laboratory and field performance of ECR revealed two corrosion protection theories for ECR³:

1. *Physical barrier theory.* The epoxy coating acts as a barrier, preventing chloride ions and other aggressive matter from coming in contact with the steel surface.
2. *Electrochemical barrier theory.* The epoxy coating acts as a high-resistance coating, reducing macro cell corrosion by increasing the electrical resistance between neighboring coated steel locations where the cathodic reaction (reduction of oxygen) can take place.

Regardless of which protection theory is applicable, the corrosion protection performance of ECR depends on adequate adherence when the chloride arrives at the steel depth and an adequate uniform coating thickness with a low number of defects.

Sagues showed that once corrosion of ECR begins when the coating has debonded, the corrosion propagation time period from initiation to cracking and spalling of the concrete is expected to be the same or less with ECR as for black steel.⁷ The undercoating corrosion initiates and proceeds in an oxygen-reduced environment by the hydrolysis of ferrous hydroxide,⁷



and the environment under the coating becomes acidic. Additional chlorides are drawn under the coating from the bulk concrete pore water by the accumulation of positively charged ions under the coating, and corrosion accelerates in an enriched hydrogen and chloride environment. The white ferrous hydroxide ($\text{Fe}(\text{OH})_2$) is converted to black magnetite, as shown in Figure 7. The black magnetite is converted to a green hydrated magnetite ($\text{Fe}_3\text{O}_4 \cdot \text{H}_2\text{O}$), which then oxidizes to form hydrate ferric oxide ($\text{Fe}_2\text{O}_3 \cdot \text{H}_2\text{O}$), red-brown rust as shown in Figure 8. Other investigations have verified this mechanism.^{8,9} Thus, neither ECR corrosion protection theory is applicable to the condition where the coating is debonded from the steel when the chloride arrives at the depth of the ECR. Thus, the effects of adhesive strength and the interrelation of coating defects on the rate of debondment are extremely important relative to the corrosion protection efficiency of ECR.

Debondment Potential

In 1869, Dupre presented the following relationship for the reversible thermodynamic work of adhesion, WA ¹⁰

$$WA = \gamma_a + \gamma_b - \gamma_{ab}$$

where γ_a and γ_b are the surface free energies of the polymer and metal oxide, respectively, and γ_{ab} is the polymer metal oxide interfacial free energy. A negative work of adhesion reflects the instability of the interface where the polymer and metal oxide layers dissociate spontaneously. A positive work of adhesion indicates that the interface is thermodynamically stable. For ECR, the

epoxy is bonded to a layer of iron oxide, the thickness of which depends on the time between the blasting to near white metal to the application of the coating.

In 1974, Gledhill and Kinlock reported the thermodynamic work of adhesion for the epoxy-ferric oxide interface for dry and wet environments as 291 and -255 mJ/m², respectively.¹¹ The change from positive to negative work provides the driving potential for the displacement of the epoxy from the ferric oxide surface by water. Also, below the epoxy glass transition point, *T_g*, of 85°C, the activation energy, *E_a*, for the displacement of the epoxy by water was reported to be a constant of 32 kJ/mole. This is greater than the secondary bond energy of 10 to 26 kJ/mole that occurs in the adhesion of two surfaces. Thus, there is a potential for the wet debondment of the epoxy from the reinforcing bar, and the displacement energy will always be greater than the bonding energy.

The debondment of the epoxy from the reinforcing steel in moist and continuously wet concrete has been shown in this and other studies.³

Debondment Kinetics

The corrosion protection performance of ECR is, thus, a problem of the kinetics (rate) of the coating debondment and chloride ingress, that is, whether the coating will be adhered or debonded when the chloride arrives at the bar depth.

The rate of debonding of epoxy coatings from reinforcing steel is a function of the environmental exposure conditions as influenced by the concrete in which the ECR is embedded; the coating properties such as thickness, permeability, and number of adhesive bond sites; the amount and type of coating defects; and the surface properties of the bar, metallic composition, roughness, and cleanness. The rate of debondment of epoxy from ferric oxide surfaces increases significantly at relative humidities greater than 60 percent and temperatures above 20°C.¹² In areas with moderate rainfall such as Virginia, the relative humidity of the concrete at the depth of the ECR in bridge decks rarely decreases below 80 percent.¹³ For marine structures, the relative humidity of the concrete is continuously greater than 80 percent. At a concrete relative humidity of less than about 70 percent, the rate of corrosion of steel in concrete is almost zero. Thus, concrete in environments that contain sufficient moisture for corrosion also have sufficient moisture for coating debondment. The temperature of the concrete at the ECR depth in Virginia is less than -15 to 40°C. Concrete pore water contains significant quantities of calcium, sodium, potassium, and hydroxide ions, and it has been shown that sodium ions in concrete pore water may contribute to the debonding of the epoxy from the bar.¹⁴ Also, the surface of clean reinforcing steel contains significant amounts of carbon, copper, silicon, nitrogen, sulfur, and sodium, which the epoxy coating must first wet and then bond to.¹⁵

Leidhesier and Funke presented the following hypothesis for the debondment of continuous organic coatings from metal surfaces and then provided the supporting evidence.¹²

1. Water disbondment is a consequence of the formation of many molecular layers of water at the metal/coating interface.
2. Water moves through the coating by diffusion through the polymer or through capillaries or pores in the coating.

3. The driving force for directional water transport through the coating to the interface is diffusion under a concentration gradient.
4. Water accumulation at the interface is made possible by the presence of nonbonded areas of sufficient dimension for the formation of liquid water.
5. The local water volume grows laterally along the metal/polymer interface under a concentration gradient force.

For the ECR system, the liquid concrete pore water is separated from a ferric oxide layer by the epoxy coating and thus provides the concentration gradient or the diffusional driving force.

Sagues proposed the following steps for the corrosion of ECR with coating imperfections (holes, holidays, and thinned coating areas)¹⁶:

1. Coating damage occurs during shipping, storage, and handling at the job site.
2. Debondment increases at damaged sites during shipping and storage.
3. Additional damage occurs during concrete placement.
4. Adhesion loss increases from damaged sites in chloride-free concrete.
5. Chlorides arrive, and corrosion takes place under the coating at a rapid rate in an acidic environment.

The epoxy coating debondment identified in this study indicates that the epoxy debonded from the reinforcing steel in marine environments in about 4 to 8 years and in bridge decks in 12 to 15 years. Also, the rate of debondment is a function of the quality of the concrete. The rate of debondment is less in high-quality concretes where the moisture content is lower and the rate of diffusion of chloride ion is slower.

The rate of epoxy coating debondment, corrosion under the coating, and delamination of the cover concrete identified in this study is not an isolated case. For example, Krass, McDonald, and Sherman reported on four bridges built between 1973 and 1978 in Minnesota where the overall coating adhesion was considered poor.¹⁷ Thirty-four cores were taken, and the dry knife adhesion and chloride content at the depth of the ECR were measured in 31 ECR sections. Of these sections, 25 had chloride contents at the depth of the reinforcing steel less than 0.71 kg/m³. Of the 25 sections, the epoxy coating had debonded (adhesion value of 3 and greater) from 16 sections, or 64 percent, of the ECR with a chloride content less than 0.71 kg/m³ and an adhesion value of 3 or greater (four with 3 and 12 with 5).

Sagues reported on 30 substructures in Florida's marine environments.⁷ The lack of coating adhesion was widespread and affected virtually all the structures 4 years and older, 29 of 30 bridges. Except for the 5 bridges in the Florida Keys, there was no evidence of corrosion of the ECR at the time of the investigation.

These two studies show that coating debondment is occurring within the same time period as found in this study. Other field studies confirm the findings of this study. Summaries of the field performance of ECR have been presented by Weyers, Manning and Smith, and Virmani.^{3,18,19}

Corrosion Service Life Extension for ECR

Manning presented two scenarios for ECR in chloride-laden environments¹⁸:

1. For relatively poor quality concrete, the chloride ions penetrate the concrete quickly and corrosion takes place only at bare, holiday, and thin coating areas until the coating loses adhesion by water dissociation and undercoating corrosion takes place.
2. For relatively good quality concrete, the chloride ions arrive at the bar when the coating has debonded and corrosion takes place primarily as undercoating corrosion.

For SN 8003, the chlorides arrived at the ECR depth at core locations SA-C1 and SA-C2 at about 7 years (see Table 1). For bare bar, the time from corrosion initiation to cracking and delamination is about 5 years in Virginia.²⁰ These two locations were delaminated at the time of the investigation when the bridge was 17 years old. Thus, ECR provided about 5 years of service life extension at these deck locations. The deck was constructed with relatively poor-quality concrete. For structures such as SN 1026 and 1029 where the coating will be debonded when the chlorides arrive, the service life extension will be nil.

For substructures where the concrete is coated with epoxy, the chlorides will arrive when the ECR coating has debonded, and thus for these structures, the ECR service life extension will be nil.

Based on the three decks evaluated, epoxy coatings in Virginia will be debonded from the bar in about 15 years. The average service life of decks built in Virginia with bare bar is 36 years with a standard deviation of 13 years.⁴ Therefore, ECR may provide an increase in service life for only 5 percent of the bridge decks built in Virginia. ECR increases the in-place cost of bridge decks built in Virginia by about 5 percent.²¹ Thus, the use of ECR in 95 percent of the bridges in Virginia may not be cost-effective.

Expanding the Database

The results of this study are based on investigations of six bridges. The three decks evaluated are representative of the range of concrete qualities produced in 1979. Because of the potential significance of the findings, additional decks should be evaluated to confirm the results of this and other studies. A reasonable expansion of the database would include the evaluation of 16 bridges, 1 bridge constructed each year between 1979 and 1994. The evaluation of these decks would provide information on the time to coating debondment, effects of changes in the epoxy coatings on debondment, changes in concrete quality, and changes in concrete cover in bridges constructed over the 16-year period.

CONCLUSIONS

1. Corrosion mechanisms for ECR in Florida substructures are applicable to bridge decks and piles in Virginia.
2. Debondment of epoxy coating on reinforcing steel takes place at a faster rate in lower quality concrete with higher moisture contents and higher effective chloride diffusion constants.

3. Epoxy coating on the steel tie bars in prestressed concrete piles with an epoxy concrete surface coating debonds from the steel before the chloride reaches the ECR.
4. Based on the three bridges evaluated, epoxy coating on the steel tie bars in prestressed concrete piles debonds from the steel in 8 years or less in Virginia's marine environments.
5. Based on the three bridges evaluated, epoxy coating on reinforcing bars in bridge decks in Virginia debonds from the steel surface in about 15 years.
6. The deck of bridge SN 8003 shows that at high chloride diffusion rates, the chlorides reach the depth of the ECR before the epoxy coating on the steel debonds. Thus, ECR does provide additional service life for these structures.
7. Based on the six bridges evaluated, it is highly likely the epoxy coating debonds from the steel surface before the chlorides arrive. Thus, ECR does not provide any additional service life for these structures.
8. The in-place cost of ECR is about 33 percent greater than that of bare bar. ECR increases the cost of a typical bridge deck by about 5 percent as compared to bare bar. Based on this study, chlorides appear not to reach most bars before the coating debonds. This calls into question whether ECR will extend the service life of bridge decks in Virginia and whether the use of ECR in Virginia is cost-effective.
9. Epoxy coating on the concrete surface of piles in the splash surface zone significantly reduces the ingress of chlorides into the concrete for at least 8 years.
10. Epoxy coating on the concrete surface of piles in the splash zone is well adhered to the concrete surface after 8 years of exposure to the brackest water.
11. Virginia's present bridge deck cover depth specification appears to be providing the desired level of cover depth.

RECOMMENDATIONS

1. *Evaluate additional decks to confirm the results of this and other studies.* This is warranted because of the potential significance of the finding of this study that the additional service life provided by ECR is limited by the debondment of the epoxy coating prior to the arrival of chlorides. This calls into question the effectiveness of ECR in marine environments and in Virginia's bridge decks. Since cover depth studies in Virginia have been somewhat limited, a representative sample of bridge decks constructed in Virginia between 1979 and 1994 should be selected and the cover depth distributions measured to validate the inference that ECR coating debondment is occurring and the present specification for cover depth is providing the desired end results. The number of decks and cover deck measurements on a deck should be a statistically based sample size for a confidence level of at least 90 percent.
2. *For prestressed piles in marine environments, continue using the present specifications for concrete surface coatings of epoxy in tidal and splash zones, low-permeability concrete, a calcium nitrite corrosion inhibitor, and the specified clear concrete cover depth.*

3. *Determine the life cycle cost of corrosion protection systems presently being used in Virginia.* Thus, service life extensions of the various systems should be estimated for each system. Service life extensions should be estimated from the time to initiate corrosion using an effective chloride diffusion constant for Virginia and the time for cracking to take place after corrosion initiates. Life cycle costs should include the cost to rehabilitate the structural components because the protective method may significantly influence the rehabilitation alternatives and costs.

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REFERENCES

1. Kilareski, W.P. (1977). *Epoxy Coatings for Corrosion Protection of Reinforcement Steel, STP 629 Chloride Corrosion of Steel in Concrete*, American Society for Testing Materials.
2. Smith, L.L. (March 1993). *Corrosion of Epoxy-Coated Rebar in a Marine Environment*, TRB Circular 403, Transportation Research Board, Washington, D.C.
3. Weyers, R.E. (November 1995). *Protocol for In-Service Evaluation of Bridges with Epoxy-Coated Reinforcing Steel*, NCHRP 10-37B, Transportation Research Board, Washington, D.C.
4. Weyers, R.E. et al. (1994). *Concrete Bridge Protection and Rehabilitation: Chemical and Physical Techniques, Service Life Estimates*, SHRP-S-668, National Research Council, Washington, D.C.
5. Carrier, R.E., D.C. Pu, and P.D. Cady (1975). *Moisture Distribution in Concrete Bridge Decks and Pavements*, SP 47-8, American Concrete Institute, Detroit.
6. Weyers, R.E. et al. (1993). *Concrete Bridge Protection, Repair, and Rehabilitation Relative to Reinforcement Corrosion: A Methods Application Manual*, SHRP-S-360, National Research Council, Washington, D.C.
7. Sagues, A.A. et al. (1994). *Corrosion of Epoxy-Coated Rebar in Florida Bridges, Final Report to Florida DOT*, WPI No. 0510603, University of South Florida, Tampa.
8. Martin, J.W. et al. (1995). *Degradation of Powder Epoxy-Coated Panels Immersed in a Saturated Calcium Hydroxide Solution Containing Sodium Chloride*, FHWA-RD-94-174, Federal Highway Administration, Washington, D.C.

9. Pyc, W.A., R.E. Weyers, and M M. Sprinkel (1997). *Corrosion Protection Performance of Corrosion Inhibitors and Epoxy-Coated Reinforcing Steel in a Simulated Concrete Pore Water Solution*, Virginia Transportation Research Council, Charlottesville.
10. Zisman, W.A. (1977). Influence of Constitution of Adhesion, *Handbook of Adhesives*, 2nd ed., edited by I. Skeist, Van Nostrand Reinhold Co., New York.
11. Gledhill, R.A., and A.J. Kinlock (1974). Environmental Failure of Structural Joints, *Journal of Adhesion*, Vol. 6, pp. 315-330.
12. Leidheiser, H., and W. Funke (1987). Water Debondment and Wet Adhesion of Organic Coatings on Metals: A Review and Interpretation, *Journal of Oil Color Chemistry Association*, Vol. 70, pp. 122-132.
13. Stark, D. et al. (1993). *Eliminating or Minimizing Alkali-Silica Reactivity*, SHRP-C-343, National Research Council, Washington, D.C.
14. Sagues, A.A. (1991). *Mechanism of Corrosion of Epoxy Coated Reinforcing Steel in Concrete*, Final Report, FL/DOT/RMC/0543-3296, Florida Department of Transportation, Materials Office, Gainesville.
15. Dillard, J.G. et al. (1993). *Concrete Bridge Protection and Rehabilitation: Chemical and Physical Techniques, Feasibility Studies of New Rehabilitation Techniques*, SHRP-S-665, National Research Council, Washington, D.C.
16. Sagues, A.A. et al. (1994). *Corrosion of Epoxy Coated Rebar in Florida Bridges*, University of South Florida, College of Engineering, Tampa.
17. Krass, P.D., D.B. McDonald, and M.R. Sherman (YEAR?). *Corrosion Investigation of Four Bridges Built Between 1973 and 1978 Containing Epoxy-Coated Reinforcing Steel*, Final Report, MN/RC-96/25, Minnesota Department of Transportation, St. Paul.
18. Manning, D. G. (1995). Corrosion Performance of Epoxy-Coated Reinforcing Steel: North American Experience, *Construction and Building Materials*, Vol. 10, No. 5, pp. 349-365.
19. Smith, J.L., and V.P. Virmani (1996). *Performance of Epoxy Coated Rebars in Bridge Decks*, FHWA-RD-96-092, Federal Highway Administration, Washington, D.C.
20. Liu, Y. (1996). *Modeling the Time-to-Corrosion Cracking of the Cover Concrete in Chloride Contaminated Reinforced Concrete Structures*, Ph.D. Dissertation, Virginia Polytechnic Institute & State University, Blacksburg.
21. Dotson, F. (1996). *Cost Data for Construction Projects in Virginia for 1995 and 96*, Virginia Department of Transportation, Structure & Bridge Division, Richmond.