

FINAL REPORT

FIELD PERFORMANCE OF EPOXY-COATED REINFORCING STEEL IN VIRGINIA BRIDGE DECKS

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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.)

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ABSTRACT

In this study, the corrosion protection performance of epoxy-coated reinforcing steel (ECR) was evaluated using approximately 250 concrete cores from 18 bridge decks in Virginia. The decks were 2 to 20 years old at the time of the investigation. The deck field inspections included a crack survey and cover depth determination in the right traffic lane. A maximum of 12 cores with the top reinforcement randomly located in the lowest 12th percentile cover depth were taken from each bridge deck. Because of the safety concerns associated with taking cores from the lower steel mat, and to minimize damage to the bridge, a maximum of only 3 cores were taken through the truss bars. The laboratory evaluation of the concrete cores included a visual examination and a determination of the carbonation depth, moisture content, absorption, percent saturation, and chloride content at a 13-mm depth. The rapid chloride permeability test was also performed for the surface and base concrete on samples obtained from the cores taken through the truss bars to determine chloride permeability. The ECR inspection consisted of a visual examination, a damage evaluation, and a determination of coating thickness and adhesion. The condition of the steel underneath the epoxy coating was also evaluated.

Adhesion loss of the epoxy coating to the steel surface was detected in all but one deck that was 4 years old and older. The epoxy coatings were debonding from the reinforcing bars. Whereas a bonded coating can be expected to protect the steel, a debonded coating allows chlorides, moisture, and oxygen to reach the steel and initiate a rapid corrosion mechanism. Reinforcing bars in various stages of adhesion loss showed visible signs of a corrosion process underneath the coating, suggesting that ECR will provide little or no additional service life for concrete bridge decks in comparison to bare steel. Other systems that will provide longer protection against chloride-induced corrosion of the reinforcing steel with a higher degree of reliability should be considered.

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INTRODUCTION

The extent of the rapid deterioration of reinforced concrete bridges from corrosion induced by chloride ions 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 Project No. 16 (Kilareski, 1977). 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 (Pyć, 1998).

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 (Smith, 1993). Since then, several other field studies have investigated the performance of ECR (Weyers, 1995). Conclusions have been mixed, from satisfactory performance for bridge decks to poor performance for substructures, with predictions that ECR would not provide long-term (50 years) corrosion protection for either (Weyers, 1995). Thus, the Virginia Department of Transportation (VDOT) initiated a field investigation to determine the performance of ECR in Virginia.

In Phase I of this investigation, conducted in 1996, the researchers assessed the corrosion protection performance of ECR in three bridge decks and in the piles in three marine structures in Virginia (Weyers et al., 1997). At the time of the investigation, the decks were 17 years old, two of the marine structures were 8 years old, and the other marine structure was 7 years old. The deck investigations included visually surveying surface cracks in the right traffic lane and drilling 12 cores selected randomly from locations having cover depths in the lowest 12th percentile. 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 the 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 either had debonded or was debonding from the reinforcing bar. This occurred without the presence of chloride, and its rate was related to concrete moisture conditions, temperature, coating defects, and other bar and coating properties. Based on the results of this field study, the researchers estimated that the epoxy coatings could be expected to debond from reinforcing steel in Virginia's marine environments in about 6 years and from bridge decks in about 15 years. This conclusion called into question the effectiveness of ECR in marine environments and in Virginia's bridge decks. Because of the potential significance of the finding that the additional service life provided by ECR was limited by the debondment of the epoxy coating prior to the arrival of chlorides, the researchers initiated this Phase II study to confirm these results.

Several other studies were conducted as part of this investigation, including a historical performance review of ECR (Pyć, 1998; Zemajtis, Weyers, Sprinkel & McKeel, 1996); an investigation of the corrosion protection performance of corrosion inhibitors and ECR in a simulated concrete pore water solution (Pyć, Weyers & Sprinkel, 1998); a performance evaluation of corrosion inhibitors and galvanized steel in concrete exposure specimens (Zemajtis, Weyers & Sprinkel, 1999a); and an evaluation of the corrosion protection performance of low-permeability concretes in exposure specimens (Zemajtis, Weyers & Sprinkel, 1999b).

LITERATURE REVIEW

A detailed discussion of the literature in the following areas may be found in Pyć (1998):

- protection methods for reinforcing against chloride-induced corrosion
- epoxy coating as the most used protection method
- the laboratory and field performance of ECR

- corrosion inhibitors for reinforcing steel, low-permeability concrete, and combined corrosion protection systems for reinforced concrete structures
- methods for evaluating the protection offered by ECR.

The review offered here summarizes the most important points with regard to this study.

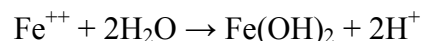
ECR Corrosion Mechanism

A review of the international literature on the laboratory and field performance of ECR revealed two corrosion protection theories for ECR (Weyers, 1995):

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 macrocell 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 of the epoxy coating to the steel bar when chloride arrives at the depth of the steel and an adequate, uniform coating thickness with a low number of defects.

Sagues et al. (1994) showed that once corrosion of ECR begins when the coating has debonded, the time from initiation of corrosion to cracking and spalling of the concrete is expected to be the same or less with ECR as with bare steel. The corrosion under the coating initiates and proceeds in an oxygen-reduced environment by the hydrolysis of ferrous hydroxide



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. 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. Other investigations have verified this mechanism (Martin et al., 1995; Pyć, Weyers & Sprinkel, 1997). 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 its interrelation with coating defects on the rate of debondment are extremely important relative to the corrosion protection efficiency of ECR.

Manning (1995) presented two scenarios for ECR in chloride-laden environments:

1. For relatively poor quality concrete, the chloride ions penetrate the concrete quickly and corrosion occurs only at areas with no coating or flawed or thin coating 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 after the coating has debonded and corrosion takes place primarily as undercoating corrosion.

Debondment Potential

In 1869, Dupre (cited in Zisman, 1977) presented the following relationship for the reversible thermodynamic work of adhesion, WA

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

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 and 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. 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 previously (Weyers, 1995; Weyers et al., 1997).

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, i.e., its metallic

composition, roughness, and cleanness. The rate of debondment of epoxy from ferric oxide surfaces increases significantly at a relative humidity greater than 60% and temperatures above 20 °C (Leidheiser & Funke, 1987). 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% (Stark et al., 1993).

For marine structures, the relative humidity of the concrete is continuously greater than 80%. At a concrete relative humidity of less than about 70%, 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 ranges from -15 °C 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 (Sagues, 1991). 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 (Dillard et al., 1993).

Leidheiser and Funke (1987) presented the following hypothesis for the debondment of continuous organic coatings from metal surfaces and then provided the supporting evidence.

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 et al. (1994) proposed the following steps for the corrosion of ECR with coating imperfections (holes, flaws, and thinned coating areas):

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.

PURPOSE AND SCOPE

The purpose of this research was to validate the findings of the Phase I study and determine the performance of ECR in additional concrete bridge decks. Parameters influencing the service life of ECR to be examined included coating damage, thickness, and adhesion to the steel surface; cover depth over the reinforcing bar; and the properties of the surrounding concrete. The initial and life-cycle costs of ECR were also to be estimated and compared with the costs of other protective systems typically used with concrete bridge decks. The fundamental parameter to be determined in this study was the approximate time for the epoxy coating to lose adhesion to the steel bar.

The analysis was based on data obtained for 18 bridge decks in Virginia built with ECR. Construction costs of concrete bridge decks with ECR or bare steel were estimated from current bid prices for reinforcing bars, concrete, and corrosion inhibitors.

METHODS AND MATERIALS

Eighteen bridge decks were selected for analysis to provide a sample that included two decks from each of VDOT's nine districts and, generally, two decks built in every other year from 1977 through 1995. None of the decks had overlays. The research consisted of four main tasks: a field investigation, laboratory testing of specimens from the field, a statistical analysis, and a life-cycle cost analysis for bridge decks built with ECR and other corrosion protection systems. Cores were taken from the top ECR and bottom ECR (truss bars) for analysis. Table 1 lists the structure number, year built, age at coring, and number of cores taken for each bridge.

A maximum of 12 cores with the top ECR and 3 cores with the bottom ECR were obtained from each bridge. Statistically, 12 samples is a sufficient number for this study (Weyers, 1995).

Field Survey

Cover Depths and Coring

Before the cores were drilled, span lengths were measured and coring locations for each span were determined. The skew angle was also calculated to indicate the direction of the main

Table 1. Bridge Decks Surveyed in Phase II

VDOT District	Structure No.	Year Built	Bridge Age (yr)	No. of Cores	
				Top Mat	Bottom Mat
Bristol	1136	1995	2	12	3
Bristol	6243	1995	2	12	2
Salem	6161	1987	10	12	3
Salem	1015	1987	10	10	3
Lynchburg	1020	1983	14	9	2
Lynchburg	1004	1983	14	12	3
Richmond	2022	1989	9	10	3
Richmond	6005	1989	9	12	3
Suffolk	2021	1981	16	12	3
Suffolk	1032	1980	17	12	3
Fredericksburg	1006	1993	4	12	3
Fredericksburg	1004	1993	4	12	3
Culpeper	1001	1992	5	12	3
Culpeper	1019	1990	7	9	2
Staunton	2068	1978	19	12	3
Staunton	1056	1977	20	12	2
Northern Virginia	2262	1985	12	12	3
Northern Virginia	1029	1986	11	12	3
Total				206	50

reinforcing bar. According to the present practice in Virginia, the transverse reinforcement should be parallel with the end of the slab on bridges having a skew of less than 20° and perpendicular to the beams on bridges having a skew of more than 20°. Determining the direction of the reinforcement in the deck, measuring the cover depth, and calculating the lowest 12th percentile cover depth for each span of the bridge were the main steps necessary in selecting the core locations. The cover depth was measured because as cover depth increases, the time required for chlorides to reach the reinforcement also increases.

A total of 40 non-biased cover depth measurements were made for each bridge span or one-third section using the reinforcing bar locator Profometer 3 produced by Proseq SA, Switzerland. It is generally accepted that the time to rehabilitate a deck is based on the time for corrosion-induced spalling to occur in the 12% of the deck with the lowest cover depth. Consequently, cores were taken from deck areas within the lowest 12% cover depth calculated, assuming a normal distribution, from cover depth measurements for each bridge deck.

The cores, 102 mm in diameter, were drilled through only the main reinforcing bar at a great enough distance from the beams to avoid cutting through the additional bars. The VDOT crews took precautions while drilling the “deep” cores containing truss bar specimens to avoid drilling through the full depth of the deck.

Visual Condition, Carbonation, and Delaminations

Each bridge deck was examined visually. Structure dimensions, the deck configuration, and the superstructure type were determined as well as the general condition of the bridge deck.

Based on the general observation that the right lane deteriorates first, the field survey was limited to this lane (Weyers, 1995).

Drilled cores were tested for depth of carbonation and allowed to surface dry. The cores were then numbered and wrapped in clear polyethylene wrap, aluminum foil, and duct tape to maintain the in-place moisture content. The bridge deck was then inspected for delaminations through use of a heavy metal rod to detect delaminations around the top reinforcing bar core locations. When the inspection was completed, the cores were transported to the laboratory and stored in plastic-covered containers until testing.

Laboratory Testing

Concrete

Visual Examination

A visual examination was performed on each concrete core immediately after it was unwrapped. The cover depth was measured and compared to cover depth measurements obtained in the field.

Rapid Permeability Testing

The top portions of cores taken through the bottom reinforcement truss bar were used for the permeability tests. Two concrete disks were cut from each core to allow for permeability testing of the surface and base concrete. The rapid chloride permeability test (ASTM C 1202-94) was performed on two or three cores from each bridge deck at the Virginia Transportation Research Council (Whiting, 1981). Two or three samples can provide a reasonable indication of the permeability of the concrete. The test is based on the measurement of the electrical conductance of concrete samples and its relation to concrete's resistance to penetration by chloride ions. An electrical current is passed through a concrete disk 51 mm thick and 102 mm in diameter for 6 hours. One end of the specimen is immersed in a 0.3 N sodium hydroxide solution, and the other in a 3% by mass sodium chloride solution. A potential difference of 60 V DC is applied to the specimen, and the total charge passed, in coulombs, is recorded. Resistance to chloride ion penetration is based on the measured total charge transfer (see Table 2). The charge passed is less for concrete with a high resistance to chloride ion penetration.

Table 2. Chloride Ion Permeability Based on Charge Passed

Charge Passed (coulombs)	Chloride Ion Penetration
>4,000	High
2,000 to 4,000	Moderate
1,000 to 2,000	Low
100 to 1,000	Very low
<100	Negligible

Chloride Content

Powdered concrete samples were collected from each core at a depth of 13 mm and tested for chloride content in accordance with ASTM C 114-97. The test procedure is based on a potentiometric titration of 10-g concrete samples with 0.05 N silver nitrate solution. The percent chloride was calculated along with the chloride equivalent in kilograms per cubic meter of concrete. Small disks containing the ECR were cut from each core using a water-cooled diamond saw.

Moisture Content, Absorption, and Saturation

The moisture content and absorption of the concrete at the top and bottom bar depths were determined in accordance with ASTM C 642-90. Two portions from each concrete core were obtained. The following weights were determined for each test sample: initial weight, oven-dry weight, and saturated weight after immersion. Moisture content, absorption, and saturation were calculated from the obtained weights. Moisture absorption and saturation can influence chloride migration, corrosion of the reinforcement, and adhesion of a coating to the reinforcement.

ECR

ECR specimens, approximately 102 mm long, obtained from each concrete core were evaluated for damage, holes, holidays (or flaws), thickness, adhesion, and corrosion. A total length of 0.9 to 1.2 m of ECR from the top mat and 0.2 to 0.3 m from the bottom mat was examined for each bridge deck. The tested epoxy coating was one of two colors: green or red-brown. The red-brown coating was found in four older structures: SN1056 built in 1977, SN2021 built in 1981, and SN1020 and SN1004 built in 1983.

Damage, Holes, Holidays, and Thickness

Each ECR specimen extracted from the concrete core was inspected for damage. A Tinker & Razor Model M/1 Holiday Detector was used in accordance with ASTM G 62 to locate any holidays in the coating not visible with the unaided eye. Coating thickness was measured in accordance with ASTM G 12 using the coating thickness gage Minitest 500 produced by Elektro-Phisik, Germany.

Adhesion

Adhesion of the epoxy coating to the reinforcing steel was tested in accordance with the knife-peel test (Ontario Ministry of Transportation, 1993). An X cut, two cuts approximately 9 mm in length, was made in the coating between bar deformations, and an area was exposed by inserting the blade of an X-ACTO knife underneath the coating. In this test, an adhesion number

Table 3. Adhesion Rating

Adhesion Number	Description of Tested Area
1	Unable to insert blade tip under coating
2	Total area of exposed steel < 2 mm ²
3	2 mm ² < total area of exposed steel < 4 mm ²
4	Total area of exposed steel > 4 mm ²
5	Blade tip slides easily under coating, levering action removes entire section (approximately 40 mm ²) of coating

from 1 through 5 is assigned to each sample (see Table 3). Six adhesion tests were performed on each ECR specimen, and the average adhesion rating was calculated for each specimen.

The steel surface under the coating was examined visually and with the scanning electron microscope (SEM). Energy diffraction analysis of X-rays (EDAX) and X-ray photoelectron spectroscopy (XPS) were used to evaluate the chemical composition of the exposed steel. The color of the steel surface under the coating (a proxy for the chemical composition) was compared later with determined adhesion ratings. SEM, EDAX, and XPS measurements were made of five selected specimens, which represented the range of the steel surface colors.

Corrosion

Electrochemical impedance spectroscopy (EIS) and linear polarization (LP) measurements were made of three ECR top mat specimens, one from each of three spans from each bridge deck. EIS is a technique used in the evaluation of coatings and the interface between a metal and a conductive solution and can provide an indication of the protection provided by the coating. Direct current (DC) potential and a small superimposed alternating current (AC) excitation are applied to a metal sample immersed in solution using a potentiostat. AC and AC potential are measured and converted into a complex impedance. LP, a DC technique, permits rapid determination of the instantaneous corrosion current density (corrosion rate). LP is capable of measuring very low corrosion rates (less than 0.1 mil per year). LP analysis is based on the observation that for potentials more noble or more active than the corrosion potential, within 10 mV, the applied current density is a linear function of the electrode potential (Clear, 1994).

Polarization resistance measurements were made in the range of -20 to +20 mV with respect to the E_{corr} using a scan rate of 0.1 mV/s. An EIS test was conducted after a delay of 15 minutes during which specimens were allowed to return to their rest potential from the polarized condition. EIS measurements were made in the frequency range between 5000 and 0.001 Hz.

Statistical Analysis

Statistical analysis was performed using the results of various testing procedures applied to the ECR specimens and concrete cores. The analysis consisted of two parts. The first part included an evaluation of the influence of various characteristics on the adhesion of the epoxy

coating to the steel surface. The second part concentrated on the relation between the impedance values measured using EIS and other properties of ECR specimens.

The statistical evaluation was performed using multiple linear regression. Average values were incorporated into the analysis to ensure the independence of all variables included in the model. All coating and concrete properties were introduced into the statistical model as independent variables.

Adhesion

Adhesion of the epoxy coating to the reinforcing steel surface is expected to influence the field performance of ECR. Coating thickness, damage, holidays, holes, the moisture and chloride concentration in the concrete, and the chemistry (as indicated by color) of the steel surface underneath the coating were expected to affect the adhesion.

Coating Impedance

The determination of the coating impedance was suggested as a method to evaluate the performance of the ECR. Impedance values of the epoxy coating obtained using EIS were compared with the coating damage, holidays, holes, thickness, and steel color underneath the coating.

Life-Cycle Cost Analysis

The service life extension of bridge decks with ECR in comparison with other corrosion protection systems presently used in the United States was estimated. Initial costs for bridge decks with ECR, bare steel, low-permeability Class A4 concrete, corrosion inhibitors, and their combinations were calculated. The present value of the life-cycle cost using a 5% interest rate for a 75-year design life was determined for the following systems:

1. ECR with bridge deck class (Class A4) concrete
2. ECR with low-permeability Class A4 concrete
3. bare steel with Class A4 concrete
4. bare steel with low-permeability Class A4 concrete
5. bare steel with Class A4 concrete and corrosion inhibitor (Darex corrosion inhibitor, retarded [DCI-S], 10 L/m³ of concrete)
6. bare steel with low-permeability Class A4 concrete and corrosion inhibitor (DCI-S, 10 L/m³ of concrete).

Data used for the life-cycle cost determination included cost, inflation, interest rate, and service life. Costs were estimated for 1 square foot of bridge deck surface area. The bridge deck was assumed to be 200 mm thick with two layers of reinforcing steel, top and bottom: No. 5 bars (15.8 mm) with 200 mm spacing and No. 4 bars (12.7 mm) with 300 mm spacing. Concrete and reinforcing steel costs were calculated based on bid information provided by VDOT.

The inflation rate for the evaluated systems, with the exception of low-permeability concrete, was assumed to be equal to the actual price change. A deflation in price was expected for low-permeability concrete such that the 1997 and 1996 prices of this product were anticipated to be equal. The true interest rate should be considered to be about 4% to 6% (Weyers, Prowell, Sprinkel & Vorster, 1993). Thus, an interest rate of 5% was used in the life-cycle cost analysis.

A design service life of 75 years was selected as the comparison period for the evaluated systems. Service life estimates were calculated based on the time-to-initiated corrosion and time-to-spalling estimated in previous field and laboratory studies (Clear, Hartt, McIntyre & Lee, 1995; Larsen, 1993; Liu, 1996; Liu & Weyers, 1998; Perregaux & Brewster, 1992; Weyers et al., 1993).

RESULTS AND DISCUSSION

Field Survey

Cover Depths

Cover depth measurements for the 21 bridge decks (Phase I and Phase II) were normally distributed, with a mean of 65 mm and a standard deviation of 9.1 mm. VDOT currently specifies a bridge deck cover depth of 65 to 75 mm. Of the 18 bridge decks studied in Phase II, 8 had an average cover depth less than 65 mm, 9 between 66 and 74 mm, and 1 greater than 75 mm.

The span on each bridge with the lowest cover depth was identified, and the lowest 12th percentile cover depth was determined. Of the 18 decks, 4 had a lowest 12th percentile cover depth of less than 50 mm, 13 had a cover depth between 51 and 65 mm, and 1 had a depth of 66 mm. In general, the reinforcement in the decks being evaluated would not be expected to be exposed to sufficient chloride to cause corrosion because of the good cover depths and high-quality concrete.

Visual Condition, Carbonation, and Delaminations

The visual condition of all the decks was good, with no spalling and very little cracking. The cracking in the right lane was longitudinal, parallel with main beams or girders. The deck concrete was not measurably carbonated, less than 1 mm. No delaminations were detected at

core locations. Delaminations were detected in only one bridge deck in Phase I of the project: SN8003 built in 1979 in Blacksburg.

Laboratory Evaluation

Concrete

All cores had a similar appearance. The coarse aggregate used was a crushed stone, angular in shape, with a maximum size of about 25 mm. The fine aggregate was manufactured sand. The only exception was the concrete from four bridge decks, SN1006 and SN1004 built in 1993, SN1001 built in 1992, and SN1019 built in 1990, which had crushed gravel as the coarse aggregate and natural sand as the fine aggregate. The aggregates were well graded and uniformly distributed. The cement matrix was gray in color with a normal amount of entrained and entrapped air. Concrete in all cores was well consolidated. Observed surface cracks were shallow, and the widths ranged from 0.007 to 0.025 mm. The cores indicate that the concrete in the decks should provide reasonable protection for the reinforcement.

Rapid Permeability Test

The average permeability of the top 50 mm of the cores ranged from very low to moderate (Figure 1). In Figure 1, as in the other figures showing averages with intervals, the bars indicate the average with a 95% confidence interval. The average permeability of the base concretes ranged from very low to high (Figure 2). No relationship between permeability and adhesion of the coating was seen.

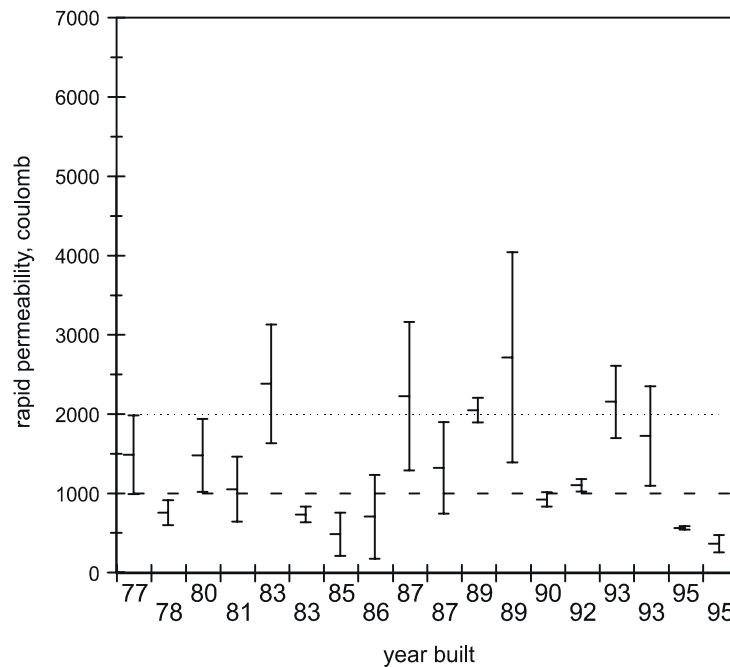


Figure 1. Average Rapid Chloride Permeability, Surface Specimens

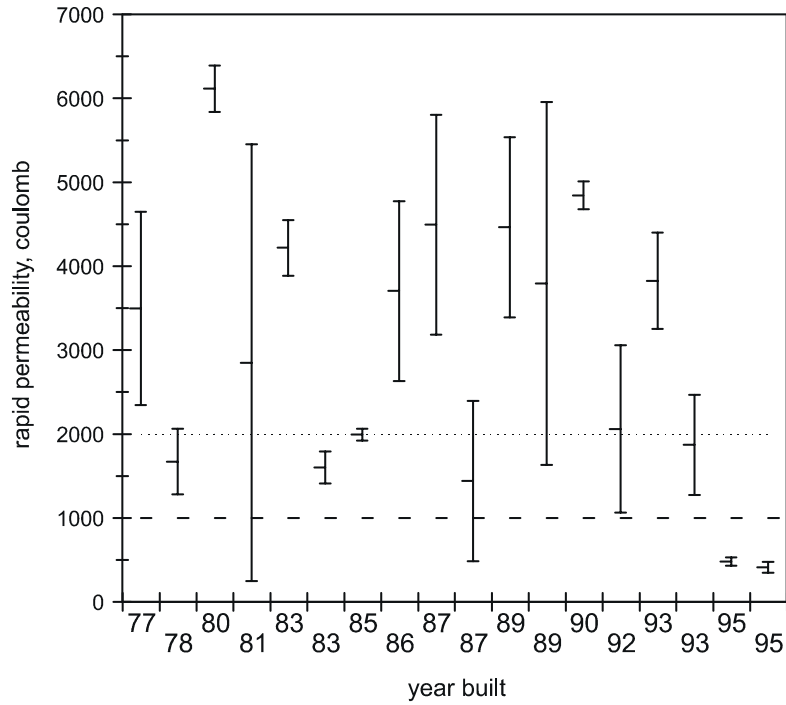


Figure 2. Average Rapid Chloride Permeability, Base Specimens

Chloride Content

Table 4 shows the average chloride ion content at the 13-mm depth. The chloride data indicated that the decks have been subjected to considerable chloride but because of the good

Table 4. Average Chloride Content at 13-mm Concrete Depth

Structure No.	Year Built	No. of Samples	Chloride Content (kg/m ³ of concrete)		
			Average	SD	Coefficient of Variation
1056	1977	12	3.97	1.18	30
2068	1978	11	5.01	1.30	26
1032	1980	12	1.32	0.42	31
2021	1981	12	1.09	0.69	63
1004	1983	12	4.46	0.77	17
1020	1983	9	2.36	1.57	67
2262	1985	12	2.16	0.77	36
1029	1986	12	1.32	0.87	66
1015	1987	10	5.77	2.31	40
6161	1987	12	1.59	0.62	39
6005	1989	11	0.74	0.35	47
2022	1989	9	0.89	0.40	45
1019	1990	10	1.70	0.81	48
1001	1992	12	2.54	0.81	32
1004	1993	12	0.84	0.32	38
1006	1993	12	0.86	0.55	64
6243	1995	13	1.17	0.66	56
1136	1995	12	1.40	0.87	42

quality of the concrete and cover over the reinforcement, much of the chloride has not likely reached the level of the reinforcement.

Moisture Content, Absorption, and Saturation

The percent moisture and absorption were normally distributed. Figure 3 shows the average moisture content and absorption for the concrete around the top mat of reinforcement for each bridge deck. Figure 4 shows the data for the bottom truss bars. In general, the concrete moisture and absorption contents at the top and bottom reinforcing mats were similar.

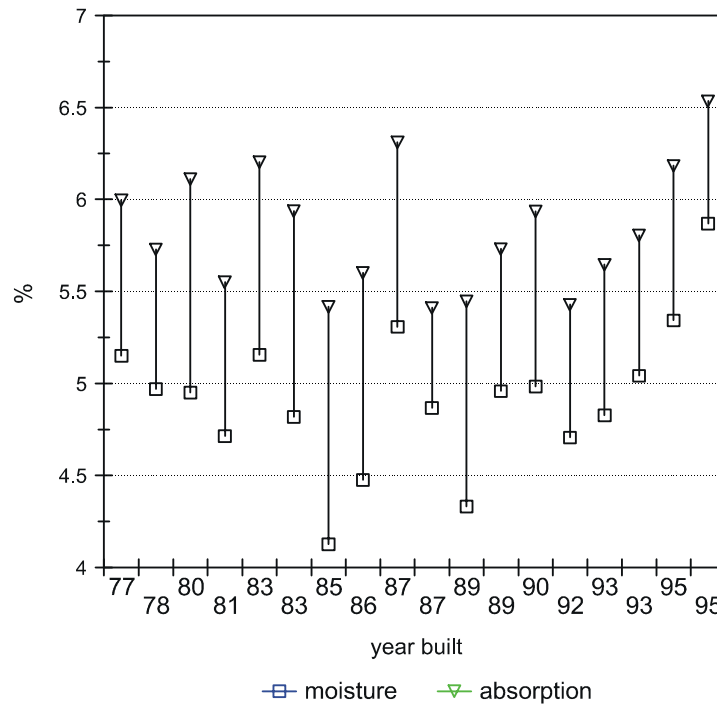


Figure 3. Average Moisture and Absorption, Top Bars

The average saturation of the concrete around the top and truss bars was between 72% and 92% (Figure 5). This saturation level provides sufficient moisture near the reinforcement to facilitate adhesion loss between the epoxy and reinforcement and to enhance corrosion.

ECR

Damage

The ECR specimens were examined visually, and the percent damaged area was calculated, with 1 mm² accuracy. Mashed, dented, and scraped spots, as well as cracks and blisters, were seen. The results of the visual inspection are presented in Figures 6 and 7. The damage on the top and bottom bars was below the specification limit of a maximum 1% in each

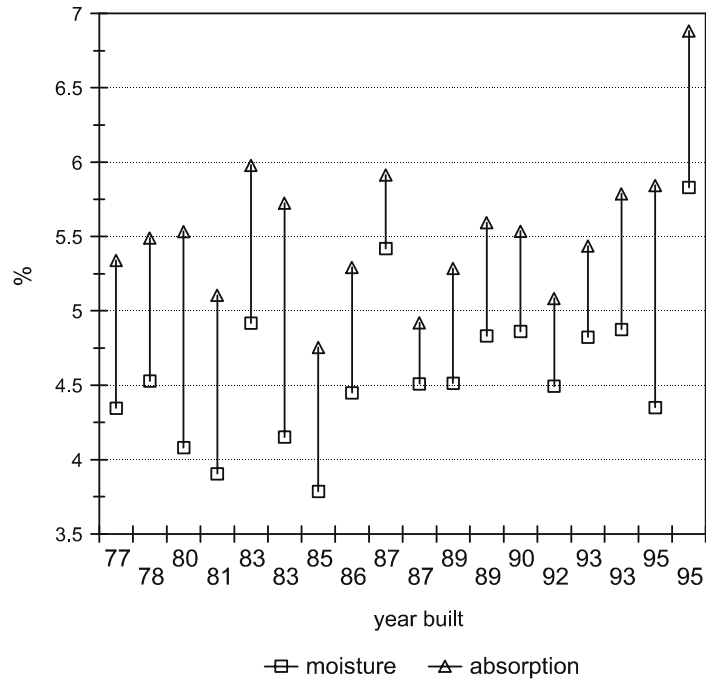


Figure 4. Average Moisture and Absorption, Truss Bars

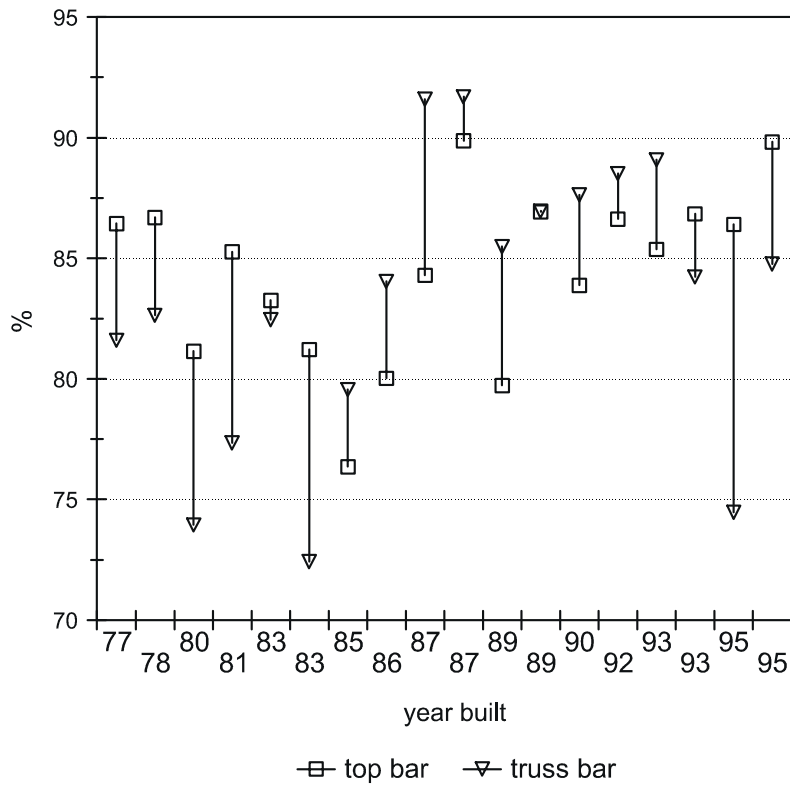


Figure 5. Average Saturation, Top and Truss Bars

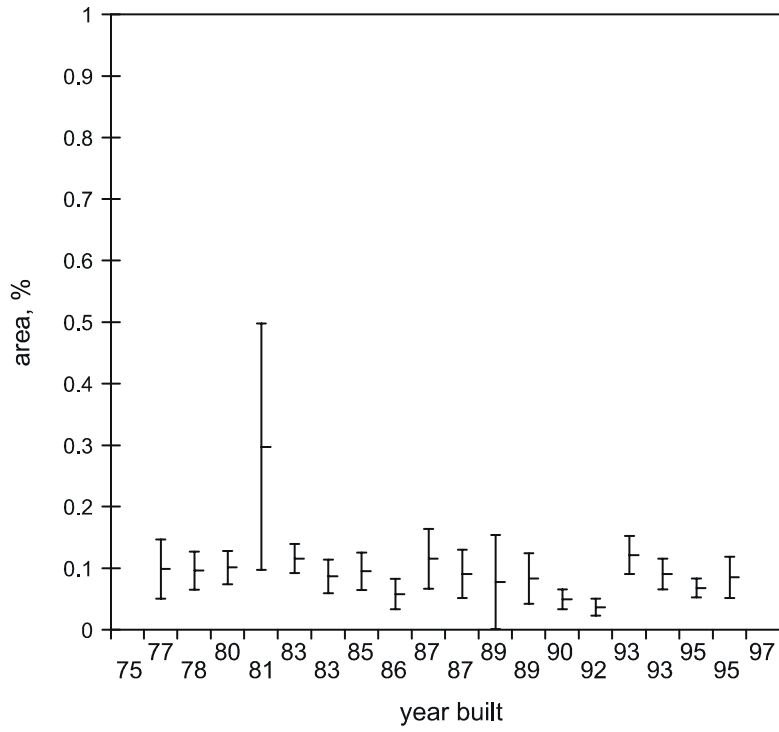


Figure 6. Average Coating Damage, Top Bars

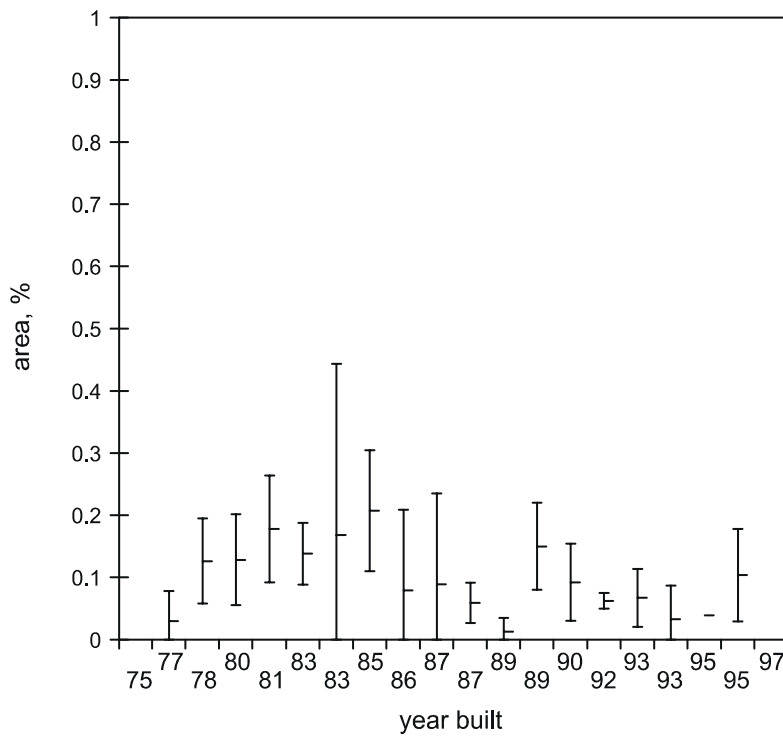


Figure 7. Average Coating Damage, Truss Bars

0.3 m of the bar (Figure 6). The average damage for the top bars was from 0.05% to 0.3%, and that for the truss bars was from 0%, the lower bound condition, to 0.2% (Figure 7).

Holes

The number of holes in the epoxy coating was determined during the visual inspection of the ECR specimens. For the ECR specimens from the top mat, the average number of holes was equal to 0, with the exception of two bridge decks: SN2262 built in 1985 and SN1006 built in 1993, where the average number of holes was 0.07 and 0.34 holes per meter, respectively. For the truss bars, holes were detected in only one set of specimens, from SN2068, built in 1978. The average number of holes was 1.09 holes per meter.

Holidays

The number of holidays per meter was determined for each ECR specimen, and the average number of holidays was computed for each bridge deck. The top bars in SN1056 built in 1997 and SN1020 built in 1983 had 44 and 32 holidays per meter, respectively, and, therefore, did not meet the specification maximum in 1995 of 3 holidays per meter. Bridge SN1020 met the 1981 limit of 6 that was in effect at the time of construction. The average number of holidays per meter for the other 16 bridge decks ranged from 0 to 3 and, therefore, met the 1995 requirements for holidays.

ECR specimens from truss bars had a higher variability in the number of holidays per meter, which is probably related to the small sample size, two or three specimens from each bridge deck. Eight structures met the current specification limit of 3 holidays per meter. The numbers for two structures were slightly above the current specification limit but met the older specification limit, 6 holidays per meter, and the average number of holidays for specimens extracted from the other eight structures exceeded both specification limits. Bridge decks SN1056 built in 1977 and SN1020 built in 1983 had the two highest average numbers of holidays of 246 and 15 per meter of bar, respectively.

Thickness

Coating thickness was determined at 12 locations on each bar, 6 readings between deformations on each bar side. The average coating thickness was then calculated for each specimen and for every bridge deck and compared with the different specification limits.

The average of readings on top bar specimens from 13 of the 18 decks was within the specification range of 175 to 300 μm . The other 5 bridge decks had an average coating thickness between 125 and 175 μm (Figure 8). The average of readings for truss bar specimens from 5 bridges was below the lower specification limit of 175 μm but greater than 125 μm

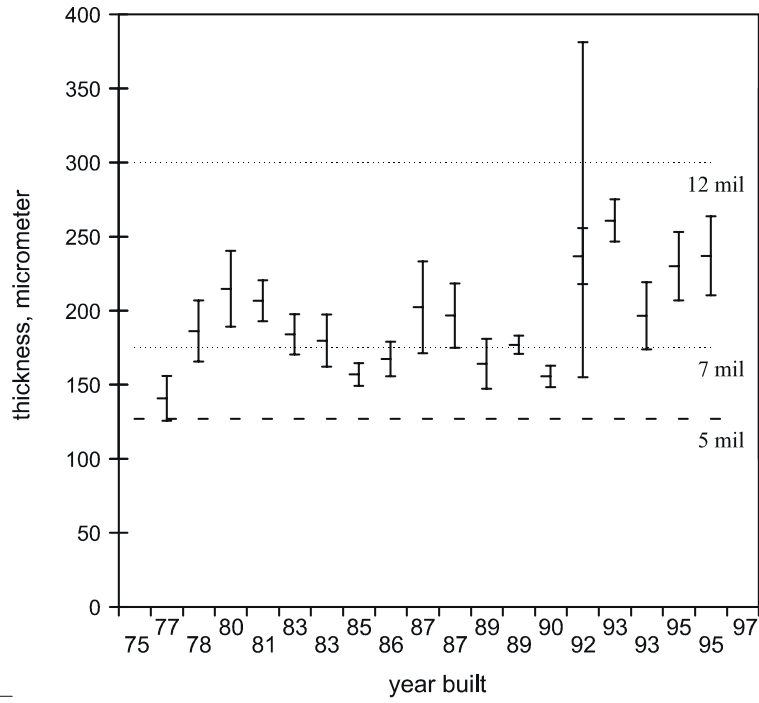


Figure 8. Average Coating Thickness, Top Bars

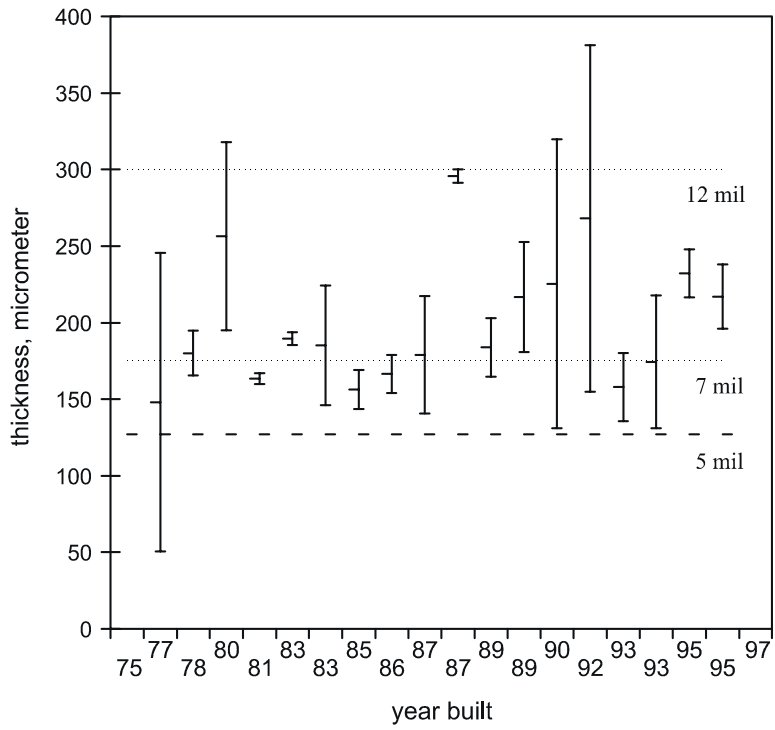


Figure 9. Average Coating Thickness, Truss Bars

Figure 9). Although only two or three cores were taken through the trusses, confidence intervals are presented for consistency.

ASTM A 775-95 specifies a minimum ECR coating thickness of 125 μm . If a single recorded measurement is below the value of 125 μm , the bar should be rejected. Because of this, during the testing procedure, some ECR specimens were discarded because of their low thickness. The bars were rejected so that performance evaluations would not be influenced by bars that did not comply with the specifications. The performance of the rejected bars would be expected to be worse than the performance of bars that complied with the specifications. The percentages of ECR specimens discarded are shown in Figures 10 and 11. The number of rejected bars varied among evaluated bridge decks between 0% and 70% for top and truss bars. No general trend was found. New average coating thicknesses were determined after rejected bars were excluded (Figures 12 and 13). The average coating thickness increased with the elimination of the rejected bars, but the differences in the thickness among bridges remained about the same.

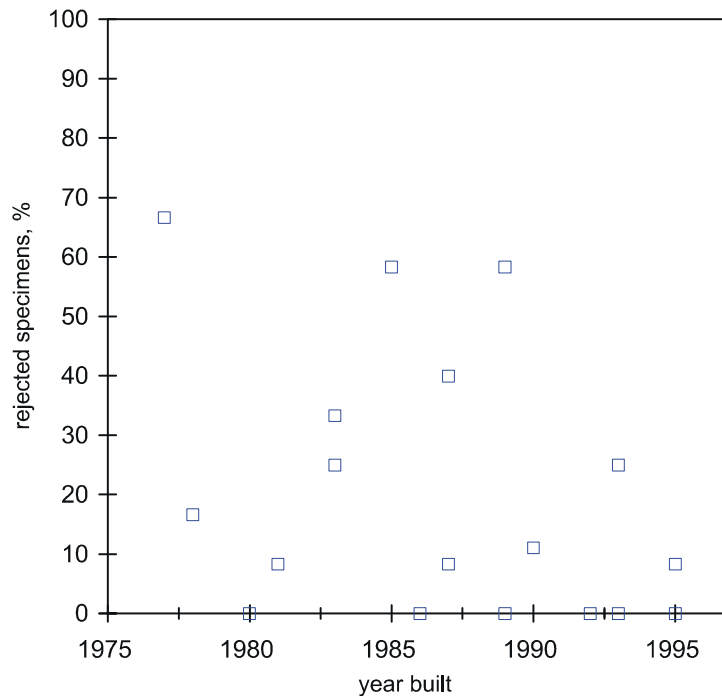


Figure 10. Percentage of Rejected Top Bars

Adhesion

ECR specimens rejected because of their low coating thickness were discarded from adhesion testing. For top bars, only three bridge decks had an average adhesion rating equal to 1, SN1020 built in 1983 and SN6243 and SN1136 built in 1995 (Figure 14). For top bars, average adhesion ratings were equal to or greater than 3 for 7 bridge decks. For truss bars, average adhesion ratings were above 3 for all but 6 bridge decks (Figure 15). The percentage of average adhesion ratings equal to or greater than 3 was also determined (Figures 16 and 17). For top mat

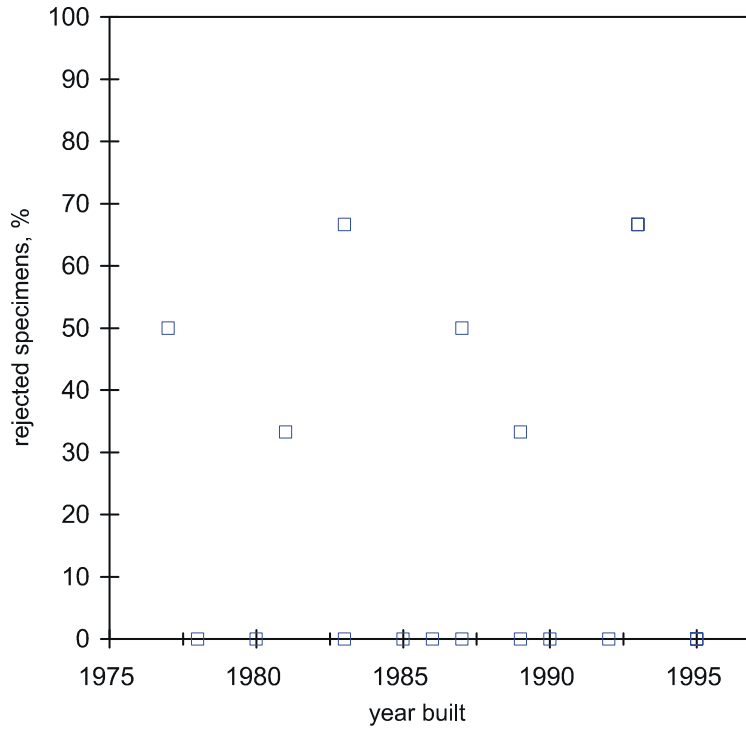


Figure 11. Percentage of Rejected Truss Bars

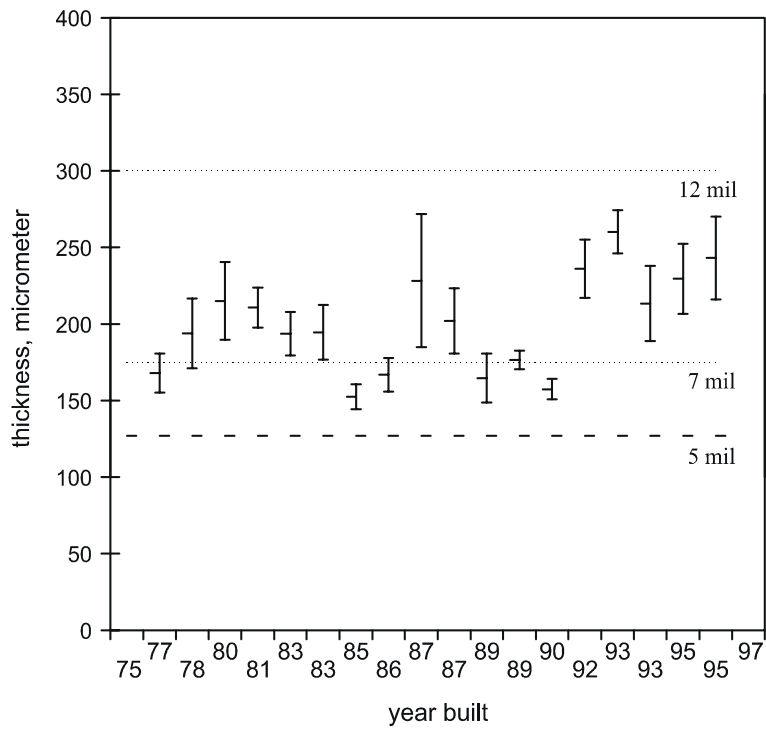


Figure 12. Average Coating Thickness, Accepted Top Bars

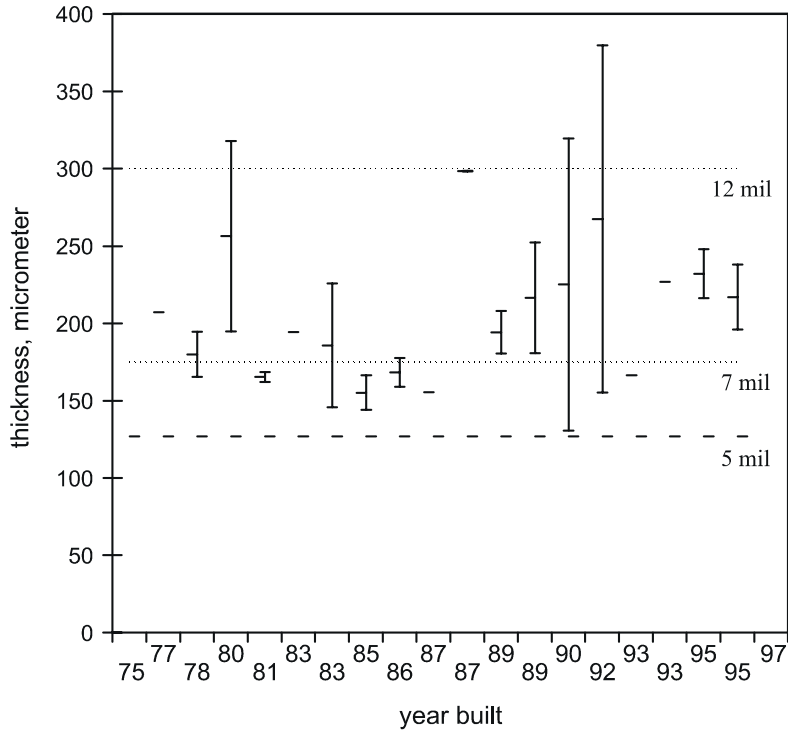


Figure 13. Average Coating Thickness, Accepted Truss Bars

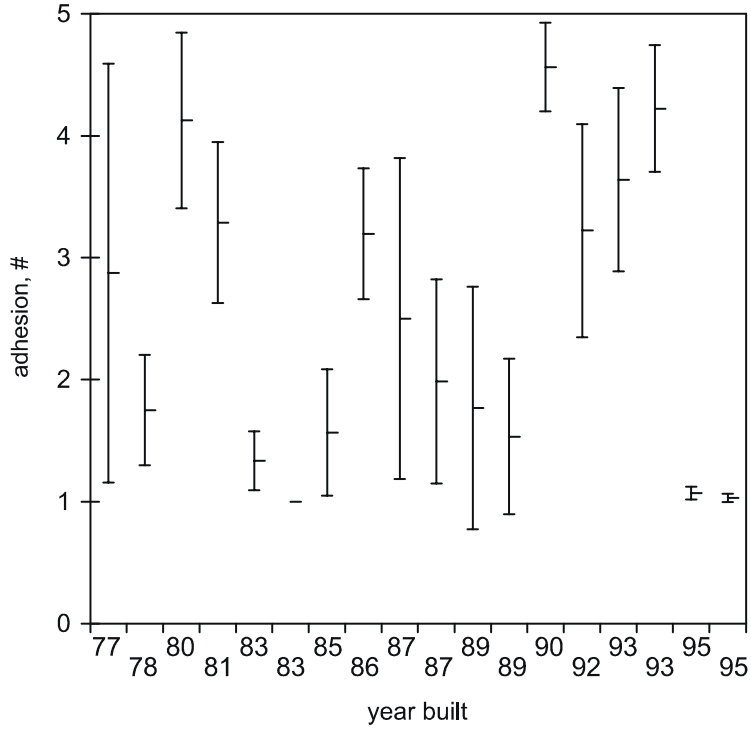


Figure 14. Average Coating Adhesion, Accepted Top Bars

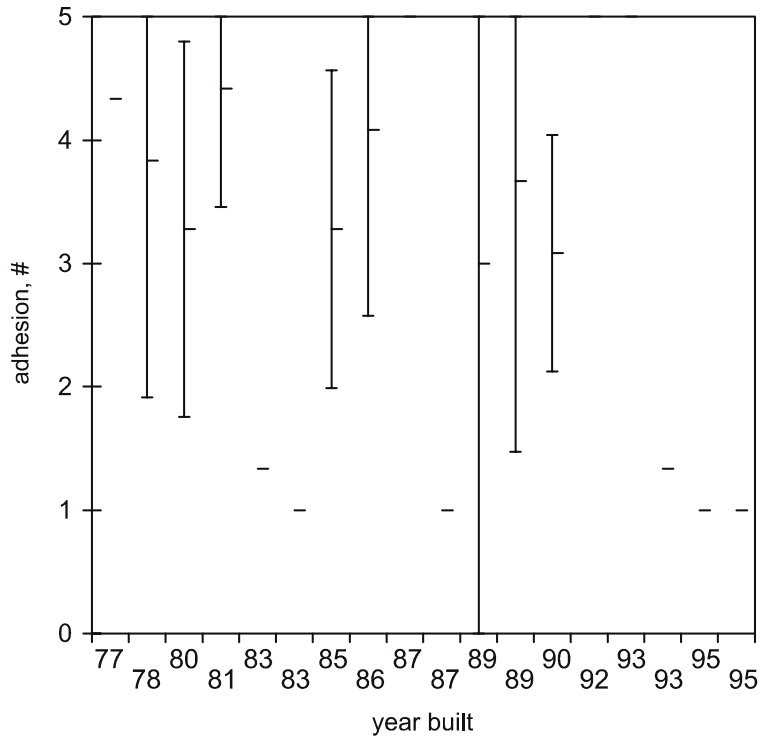


Figure 15. Average Coating Adhesion, Accepted Truss Bars

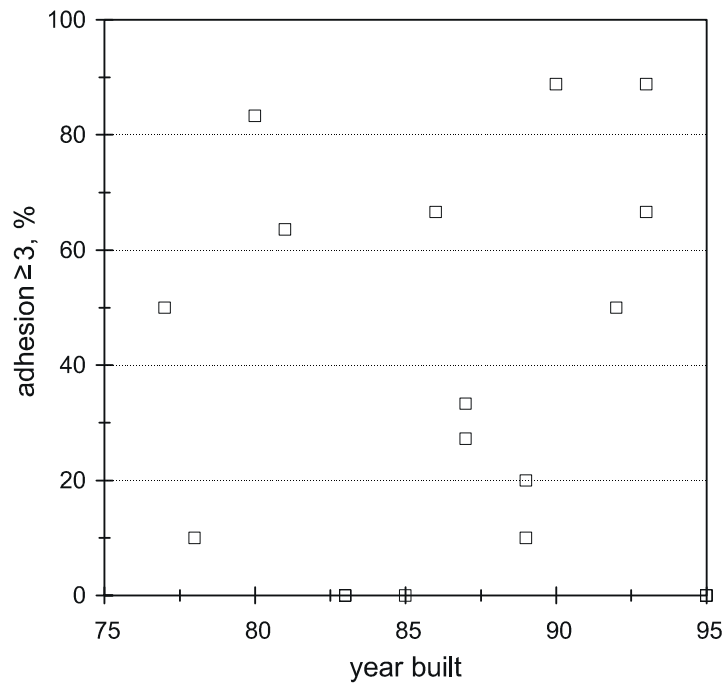


Figure 16. Percentage of Average Adhesion ≥ 3 , Top Bars

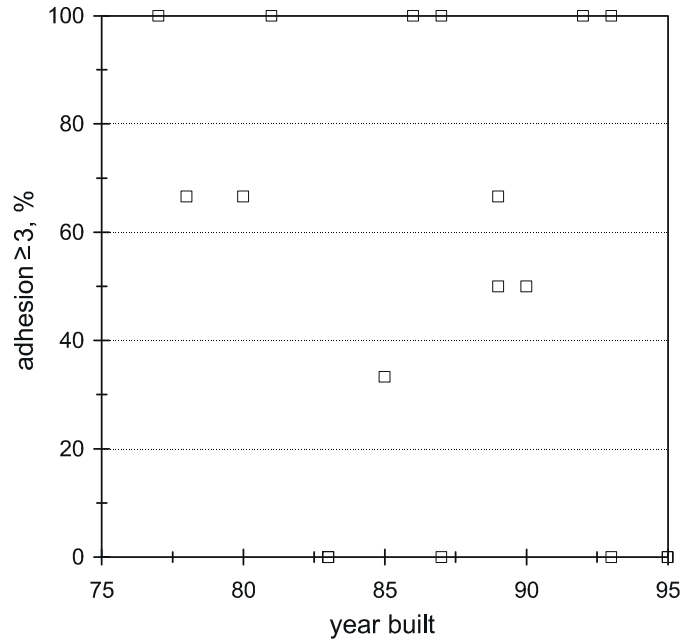


Figure 17. Percentage of Average Adhesion ≥ 3 , Truss Bars

ECR specimens, the percentage of average adhesion ratings equal to or greater than 3 was between 10 and 90, except for 5 structures (Figure 16). Similar observations were made for the truss bars, with the percentage of average adhesion ratings equal to or higher than 3 ranging from 30 to 100, except for 6 bridges (Figure 17).

The color of the steel under the peeled epoxy coating was noted, and a numerical value was assigned to the color (see Table 5). A correlation was established between the color of the steel surface under the coating and the adhesion ratings based on average values for each bridge deck (see Figures 18 and 19). The linear relationship had R^2 values of 0.88 for the top bars and 0.7 for the truss bars. Another relationship between the color of the steel surface and the adhesion based on data points for every top bar sample evaluated (rather than average values for each deck) had an R^2 value of 0.75. Table 5 describes the various colors of the steel surface underneath the coating.

Table 5. Color of Steel Under Coating

Number	Steel Color
0	N/A, Epoxy bonded
1	Shining steel
2	Gray and shining steel
3	Dark gray and shining steel
4	Black and shining steel
5	Black

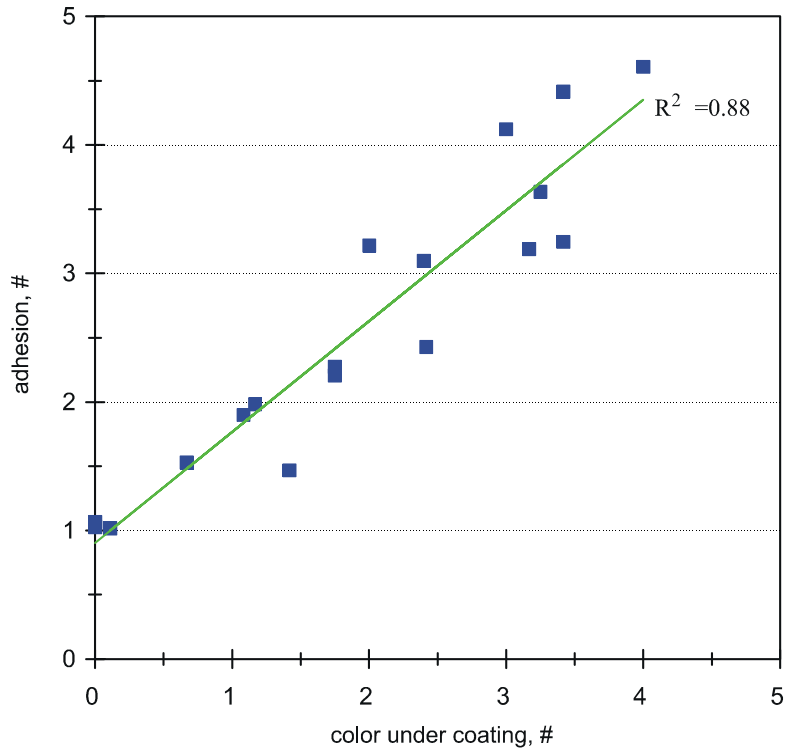


Figure 18. Average Adhesion and Steel Color Relation, Top Bars

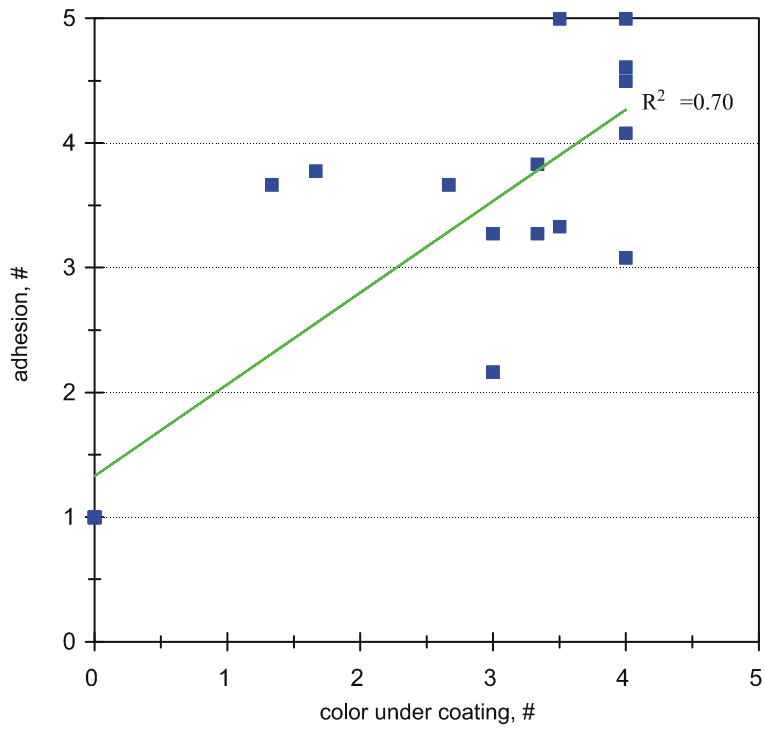


Figure 19. Average Adhesion and Steel Color Relation, Truss Bars

The results of the chemical analysis tests using EDAX and XPS are presented in Tables 6 and 7. XPS samples were approximately 5 nm, about 10 layers of molecules, of the specimen surface. X-ray analyses were made immediately after a fresh cut was made in the epoxy coating. Iron, oxygen, and carbon were the elements with the highest detected concentrations identified in the XPS analysis: 0.6% to 6.3%, 11.9% to 39.3%, and 40.2% to 83.6%, respectively (Table 6). Traces of other metals, i.e., Cu, Sn, and alkalis, i.e., Ca, Na, K, were also present. The thickness of the layer analyzed by EDAX was about 100 μ m, 1,000 times deeper than the XPS penetration. Testing the same type of freshly exposed reinforcing steel surface using EDAX provided similar results, with iron and oxygen having the highest weight percentages, 83.7% to 97.7% and 1.0% to 5.4%, respectively (Table 7). Traces of the following metals and alkalis were also detected: Mn, Cr, Ti, Ni, Cu, Al, Na, Ca, and K. Neither the XPS nor EDAX analyses detected any chlorine under the epoxy coating.

As shown in Table 7, EDAX results showed that the amount of oxidized iron under the epoxy coating increased, decreasing the iron/oxygen ratio, as the color changed from shining steel to black. The XPS results implied the same, as indicated in Table 6.

Table 6. Results of XPS Analysis (% Atomic Concentration)

Element	Shining Steel (1)	Gray and Shining Steel (2)	Dark Gray and Shining Steel (3)	Black and Shining Steel (4)	Black (5)
Fe	6.3	4.5	4.6	0.6	3.9
Cu	0.7	0.7	3.0	<0.2	0.9
Sn	0.7	1.2	0.6	<0.2	<0.2
Si	1.5	0.8	3.7	3.5	8.3
Ca	0.2	<0.2	0.8	<0.2	0.8
Na	<0.2	<0.2	2.0	<0.2	2.9
K	<0.2	<0.2	1.5	<0.2	2.2
O	29.4	25.0	37.9	11.9	39.3
N	1.1	0.6	<0.2	0.4	1.6
C	60.1	67.2	45.9	83.6	40.2
Iron/ oxygen	0.2	0.2	0.1	0.05	0.1

Table 7. Results of EDAX Analysis (% Weight)

Element	Shining Steel (1)	Gray and Shining Steel (2)	Dark Gray and Shining Steel (3)	Black and Shining Steel (4)	Black (5)
Fe	97.7	93.9	92.1	93.6	83.7
Mn	1.7	2.1	1.1	2.3	n/a
Cr	0.7	1.2	0.5	n/a	< 0.2
Ti	n/a	0.7	n/a	0.6	n/a
Ni	n/a	n/a	n/a	0.2	n/a
Al	n/a	n/a	< 0.2	n/a	0.4
Si	n/a	0.3	0.6	n/a	6.7
Cu	n/a	0.4	2.7	0.8	2.5
Ca	n/a	0.3	n/a	n/a	n/a
K	n/a	n/a	n/a	0.4	1.3
O	n/a	1.0	3.2	2.3	5.4
Iron/ oxygen	n/a	93.9	28.8	40.7	15.5

Corrosion

EIS. Nyquist and Bode plots were constructed from the data obtained using EIS. All graphs related to EIS can be found in Pyć (1998). An attempt was made to determine if impedance measurements can indicate the performance of the coating as a protective barrier against chloride-induced corrosion. The possibility of a correlation between the EIS results and the data collected from various testing procedures was also examined. There was considerable variability in the relationship between impedance and the data from the other tests (see Pyć, 1998).

LP. Polarization resistance was determined for all ECR specimens. The values ranged from 10^5 ohm cm^2 to 10^9 ohm cm^2 . Polarization resistance was also compared with the impedance data measured at the lowest frequency of 0.001 Hz. The results are presented in Table 8 and Figure 20.

Corrosion current values calculated from the polarization resistance data were in the range from 0 mA/ cm^2 to 10^{-7} mA/ cm^2 for the majority of tested ECR specimens with the corrosion potential values between -300 mV and -50 mV. Corrosion rates were equal to zero for all ECR specimens.

Table 8. Polarization Resistance Results, Average Values

Structure No.	Year Built	log R_p (ohm cm^2)	log Z @ 0.001 Hz (ohm)
1056	1977	6.57	5.56
2068	1978	6.10	5.12
1032	1980	7.04	6.48
2021	1981	5.84	4.86
1004-3	1983	6.10	5.05
1020	1983	6.76	5.79
2262	1985	7.12	6.19
1029-9	1986	6.84	6.25
1015	1987	5.66	4.90
6161	1987	6.99	6.38
6005	1989	7.23	6.95
2022	1989	7.04	6.18
1019	1990	6.59	5.61
1001	1992	7.37	6.39
1004-6	1993	5.76	4.85
1006	1993	5.66	5.08
6243	1995	7.77	7.93
1136	1995	8.26	7.61

Statistical Analysis of Adhesion

Absorption was excluded from the statistical model based on its perfect correlation with moisture (Figure 21). Three variables, color of the steel under the coating, coating thickness, and coating damage, provided the best fit with a model R^2 value equal to 0.906. The partial R^2 values for individual variables were 0.843 for the color of steel under the coating and 0.031 for the coating thickness and coating damage, respectively (Figures 22 through 24).

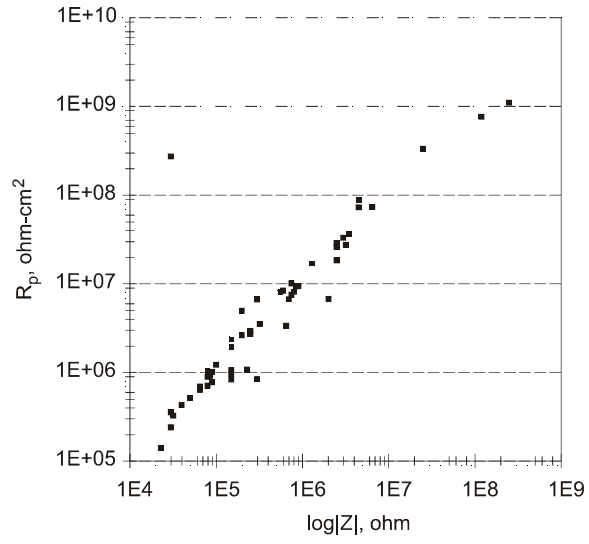


Figure 20. Polarization Resistance vs. Impedance at 0.001 Hz

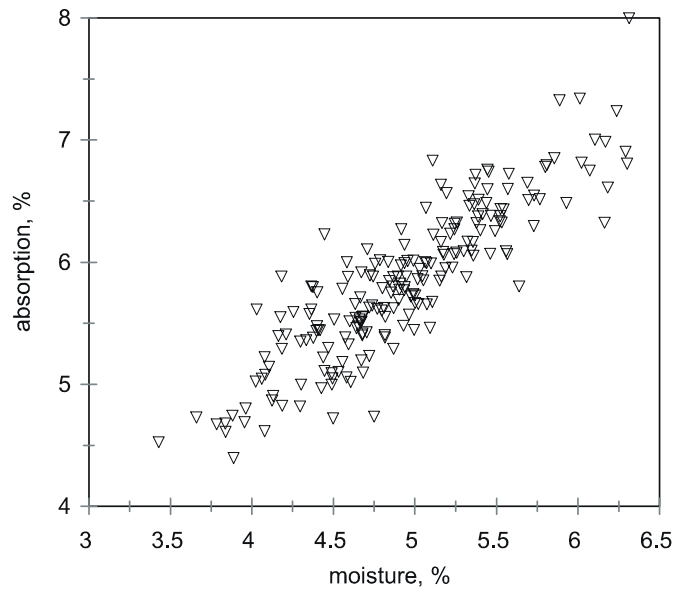


Figure 21. Absorption vs. Moisture, All Data, $R^2 = 1$

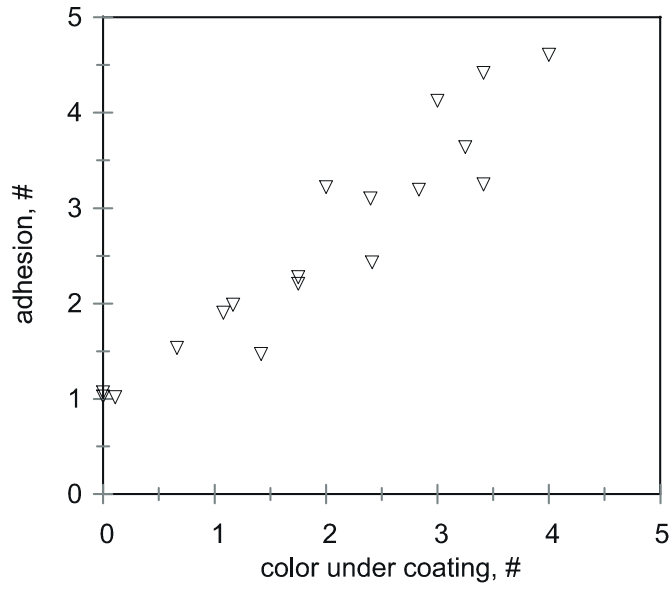


Figure 22. Adhesion vs. Steel Color, Average Values, $R^2 = 0.896$

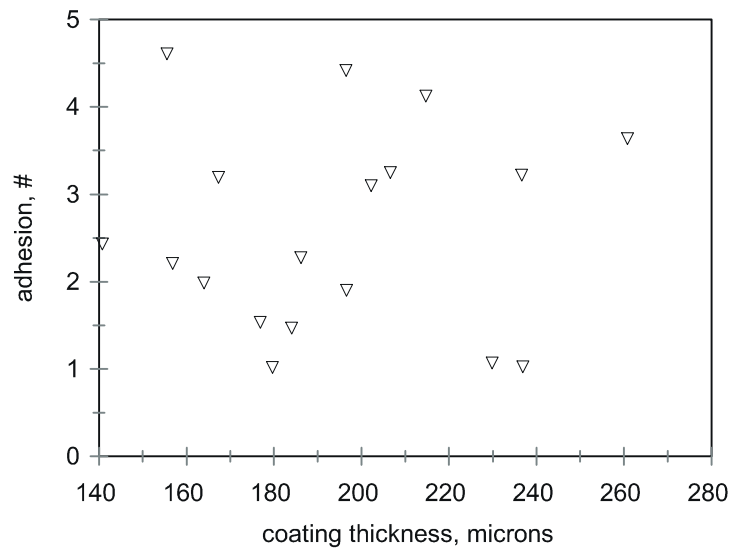


Figure 23. Adhesion vs. Coating Thickness, Average Values, $R^2 = 0.001$

Figure 24. Adhesion vs. Damage, Average Values, $R^2 = 0.008$

Life-Cycle Costs

Table 9 shows initial costs for elements used in bridge deck construction and maintenance. Table 10 shows initial costs for newly constructed bridge decks for various systems. The highest initial costs for the bridge deck, \$126.45/m² (\$11.76/ft²), \$124.19/m² (\$11.55/ft²), and \$124.09/m² (\$11.54/ft²), were for the ECR + low-permeability Class A4 concrete system, the bare steel + low-permeability Class A4 concrete + corrosion inhibitor system, and the ECR + Class A4 concrete system, respectively. The two lowest initial costs were for the bare steel + Class A4 concrete system and the bare steel + low-permeability Class A4 concrete system, \$117.85/m² (\$10.96/ft²) and \$120.22/m² (\$11.18/ft²), respectively (Table 10).

The present value of the life-cycle cost, using a 5% interest rate, for a 75-year design life was determined for all systems (Table 11). Based on service life estimates, several systems require no maintenance during the 75-year design life. Among them were ECR + low-permeability Class A4 concrete, bare steel + low-permeability Class A4 concrete, bare steel + Class A4 concrete + corrosion inhibitor, and bare steel + low-permeability Class A4 concrete + corrosion inhibitor. The ECR + Class A4 concrete and the bare steel + Class A4 concrete systems will need LMC overlays. The service life of an LMC overlay is 24 years (Weyers et al., 1993). The placement times for the LMC overlays varied depending on the system and were based on the calculated service life estimates (Table 11). The present cost of an LMC overlay was obtained using 3.4% inflation rate. The life-cycle cost evaluation of the systems requiring LMC overlays was estimated for two cases: with and without the traffic control costs.

The analysis indicated the highest total cost was \$132.26/m² (\$12.30/ft²) and \$141.40/m² (\$13.15/ft²) for the ECR + Class A4 concrete system, without and with the traffic control, respectively. The lowest total cost of \$120.22/m² (\$11.18/ft²) was determined for the bare steel + low-permeability Class A4 concrete system without the traffic control (Table 11).

Table 9. Initial Costs for Bridge Deck, 1997

Item	Initial Cost (\$)
ECR	1.37/kg (0.62/lb)
Bare steel	1.08/kg (0.49/lb)
Class A4 concrete	464.05/m ³ (355.00/ yd ³)
Low-permeability Class A4 concrete	475.82/m ³ (364.00/ yd ³)
Corrosion inhibitor (DCI-S, 9.90 l/m ³ , 2 gal/yd ³)	1.98/L (7.50/gal)
LMC overlay w/o traffic control	56.13/m ² (5.22/ft ²)
LMC overlay with traffic control	119.35/m ² (11.10/ft ²)

Table 10. Initial Costs for Bridge Deck, Various Systems, 1997

Systems	Initial Cost (\$/m ²)	Initial Cost (\$/ft ²)
ECR + Class A4 concrete	124.09	11.54
ECR + low-permeability Class A4 concrete	126.45	11.76
Bare steel + Class A4 concrete	117.85	10.96
Bare steel + low-permeability Class A4 concrete	120.22	11.18
Bare steel + Class A4 concrete + corrosion inhibitor	121.83	11.33
Bare steel + low-permeability Class A4 concrete + corrosion inhibitor	124.19	11.55

Table 11. Life-Cycle Cost for 75-Year Design Life

System	Initial Cost (\$/m ²)	Initial Cost (\$/ft ²)	LMC Overlay Placement (yr)	Total Cost (\$/ft ²)	
				Without traffic control	With traffic control
ECR + Class A4 concrete*	124.09	11.54	45, 69	12.30	13.15
			50, 75	11.99	12.54
			55	11.90	12.30
			60	11.82	12.13
			65	11.76	12.00
ECR + low-permeability Class A4 concrete	126.45	11.76	n/a	11.76	---
Bare steel + Class A4 concrete	117.85	10.96	40, 64	11.93	13.02
Bare steel + low-permeability Class A4 concrete	120.22	11.18	n/a	11.18	---
Bare steel + Class A4 concrete + corrosion inhibitor	121.83	11.33	n/a	11.33	---
Bare steel + low-permeability Class A4 concrete + corrosion inhibitor	124.19	11.55	n/a	11.55	---

*Assumes 5, 10, 15, 20, and 25 years of added corrosion protection provided by ECR, respectively.

SUMMARY

Bridge Decks

Evaluated bridge decks were in a good overall condition. No severe cracking damage, no carbonation, and no delaminations were detected. The concrete was sound and well consolidated, with a normal amount of entrained and entrapped air. Coarse and fine aggregates were well graded and uniformly distributed. The chloride permeability of the inspected concrete was low to moderate, based on the rapid chloride permeability test results and the chloride concentrations at 13 mm determined by chemical analysis of concrete powdered samples.

Measured cover depths were normally distributed and close to VDOT's specification of a 64-mm clear cover depth (Virginia Department of Transportation, 1997). They also seemed to provide the desired protection for the reinforcing steel. The standard deviation for cover of 9.1 mm is in accordance with the findings of other researchers and demonstrates the capability of current construction techniques to place the steel in bridge decks at the desired location.

ECR

This research supports the conclusions drawn from Phase I of this project and provides more information on the performance characteristics of ECR used in concrete bridge structures in Virginia (Weyers et al., 1997).

Although the coatings on tested ECR specimens were in overall good condition, i.e., the detected damage and measured coating thickness were within VDOT's specification limits, the adhesion ratings raised a concern about the long-term performance of ECR in the concrete environment. The time from the initiation of corrosion to cracking and delamination in bare reinforcing steel is about 5 years in Virginia (Liu & Weyers, 1998). The epoxy coating debondment identified in this study indicated that the epoxy debonded from the reinforcing steel in bridge decks in as little as 4 years. Thus, in Virginia, the epoxy coating will be completely or partially debonded from the steel when the chloride ions arrive at the bar depth in bridge decks. However, the disbondment was not caused by the presence of chloride ions on the steel surface or the excessive coating damage. Instead, the loss of adhesion was related to water penetrating the coating and accumulating at the metal/coating interface, causing peeling stresses exceeding the adhesive bond strength and oxidation of the steel surface (Weyers, 1995; Weyers et al., 1997).

EIS measurements suggested that most of the tested ECR specimens became permeable while in moist concrete bridge decks. In Virginia, concrete exhibits more than 72% saturation. The charge transfer and diffusion-controlled corrosion process have also developed at the metal/coating interface, which would explain the change in color of the steel surface underneath the coating.

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 (1996) reported on four bridges built between 1973 and 1978 in

Minnesota where the overall coating adhesion was considered poor. Thirty-four cores were taken, and the adhesion and chloride content at the depth of the ECR were measured in 31 ECR sections. Of these sections, 25 had a chloride content at the depth of the reinforcing steel of less than 0.71 kg/m^3 . Of the 25 sections, the epoxy coating had debonded (adhesion rating of 3 or greater) from 16 sections, or 64%, of the ECR with a chloride content less than 0.71 kg/m^3 and an adhesion rating of 3 or greater (4 with 3, and 12 with 5).

Sagues et al. (1994) reported on 30 substructures in Florida's marine environments. The lack of coating adhesion was widespread and affected virtually all the structures 4 years or 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 these findings, and summaries of the field performance of ECR may be found in Weyers (1995), Manning (1995), and Smith and Virmani (1996).

Adhesion testing (bond strength testing) of the epoxy coating to the steel surface using the knife-peel test should become a standard procedure for the evaluation of ECR. However, additional research should be performed to determine the adhesion rating at which an epoxy coating will not maintain its protective properties against chloride-induced corrosion.

The authors strongly believe that on the scale of 1 to 5, the adhesion rating of 3 is the limit after which coating disbondment will progress rapidly and, upon the arrival of chloride ions, corrosion underneath the coating will develop in the same manner as with adhesion ratings of 4 and 5. However, a laboratory study monitoring the development and progress of chloride-induced corrosion should be performed to verify this hypothesis.

Statistical Analysis

The multiple regression analyses applied to examine the relation of various ECR and concrete properties to the adhesion of the epoxy coating to the steel surface indicated a high correlation with the chemistry of the steel underneath the coating, as evidenced by the amount of oxidized iron. It is believed that as adhesion is lost, the steel reacts with oxygen and changes color.

ECR and Alternate Systems Based on the Cost-Effectiveness Analysis

The adhesion loss of the epoxy coating to the steel surface in a moist concrete environment before the arrival of chlorides and the possibility of corrosion underneath the coating suggest that ECR will provide little or no additional service life for concrete bridge decks. Other systems that will provide longer protection with a higher degree of reliability against chloride-induced corrosion of steel in concrete should be considered.

The economic analysis of various systems showed that the most cost-effective practice would be the use of bare reinforcing steel with the following type of concrete: low-permeability Class A4 concrete, Class A4 concrete + corrosion inhibitor, or low-permeability Class A4 concrete + corrosion inhibitor. According to the cost analysis presented in this study, these three systems will provide savings for concrete bridge decks, in comparison with the ECR + Class A4 concrete system, of \$12.04, \$10.43, and \$8.06/m² (\$1.12, \$0.97, and \$0.75/ft²), respectively (Table 11). The use of the bare steel + low-permeability Class A4 concrete and bare steel + low-permeability Class A4 concrete + corrosion inhibitor systems instead of the ECR + low-permeability Class A4 concrete system will also provide savings of \$6.24 and \$1.61/m² (\$0.58 and \$0.21/ft²), respectively. These values were obtained for the systems without a latex-modified overlay and do not include the traffic control costs.

Field performance evaluations of other corrosion protection systems suitable for concrete bridge decks should be conducted and compared with the results obtained from the ECR study.

CONCLUSIONS

- Evaluations of ECR in 18 bridge decks in Virginia indicate that the epoxy debonds from the reinforcement in as little as 4 years and long before chlorides arrive at the level of the reinforcement.
- The epoxy debonds in properly constructed bridge decks having good cover over the reinforcement, good quality concrete, and ECR that complies with VDOT's specifications.
- The level of protection provided by the epoxy coating is uncertain because no studies have been done to address the issue of service life extension provided by coatings in various states of adhesion loss.
- Assuming a debonded coating will provide for little additional service life, alternative protection systems such as low-permeability concrete and low-permeability concrete with calcium nitrite will provide for a more cost-effective structure.

RECOMMENDATIONS

1. VDOT should discontinue the use of ECR for extending the service life of bridges and employ alternatives such as the use of low-permeability concrete and corrosion inhibitors and alternative reinforcement.
2. Research should be conducted to estimate the service life extension provided by ECR in various stages of adhesion loss.

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