Final Report VTRC 06-R29

Virginia Transportation Research Council

research report

Summary Report on the Performance of Epoxy-Coated Reinforcing Steel in Virginia

http:/www.virginiadot.org/vtrc/main/online_reports/pdf/06-r29.pdf

RICHARD E. WEYERS, Ph.D., P.E.
Charles E. Via, Jr. Department
of Civil and Environmental Engineering
Virginia Polytechnic Institute & State University

MICHAEL M. SPRINKEL, P.E.
Associate Director
Virginia Transportation Research Council

MICHAEL C. BROWN, Ph.D., P.E.
Research Scientist
Virginia Transportation Research Council

VTRC
Virginia Transportation
Research Council

Virginia Transportation Research Council, 530 Edgemont Road,
Charlottesville, VA 22903-2454, www.vtrc.net, (434) 293-1900

Standard Title Page - Report on Federally Funded Project

1. Report No. FHWA/VTRC 06-R29	2. Government Accession No.	3. Recipient's Catalog No.

4. Title and Subtitle Summary Report on the Performance of Epoxy-Coated Reinforcing Steel in Virginia	5. Report Date June 2006 6. Performing Organization Code
7. Author(s) R. E. Weyers, M. M. Sprinkel, and M. C. Brown	8. Performing Organization Report No. VTRC 06-R29
9. Performing Organization and Address Virginia Transportation Research Council 530 Edgemont Road Charlottesville, VA 22903	10. Work Unit No. (TRAIS) 11. Contract or Grant No. 21720
12. Sponsoring Agencies' Name and Address Virginia Department of Transportation, 1401 E. Broad Street, Richmond, VA 23219 FHWA, P.O. Box 10249, Richmond, VA 23240	13. Type of Report and Period Covered Final August 1992–March 2006 14. Sponsoring Agency Code

15. Supplementary Notes

16. Abstract

From 1992 to 2006, the Virginia Transportation Research Council and its contract researchers conducted a long-term systematic series of investigations to evaluate the corrosion protection effectiveness of epoxy-coated reinforcement (ECR) and to identify and recommend the best and most cost-effective corrosion protection system for Virginia bridge decks. This report summarizes this research and subsequent efforts to implement alternative reinforcement. The work was conducted, and is reported, in this general order:

- review of historical performance of ECR, ECR performance in solutions and concrete, and preliminary field investigations
- investigation of field performance of bridge decks built with ECR
- assessment of alternative corrosion protection methods
- development of probabilistic service life models for bridge decks and laboratory assessment of ECR cores extracted from bridge decks to determine service life extension
- efforts to implement alternative reinforcement.

The series of studies demonstrated that the epoxy coating on ECR naturally degrades in the highly alkaline moist environment within concrete. The subsequent loss of bond, coupled with the inevitable flaws in the coating induced by construction, leads to an estimated service life benefit of ECR of as little as 3 to 5 years. Further, non-critical decks, beams, and substructure elements not exposed to marine environments, particularly on secondary and rural routes, can be cost-effectively constructed and maintained using low-permeability concrete and black reinforcing bar. However, because the Federal Highway Administration requires the use of corrosion-resistant reinforcement, and because ECR cannot provide adequate corrosion protection for structures designed for a 100-year+ service life as currently recommended by FHWA, the report recommends that the Virginia Department of Transportation amend its specifications regarding the use of ECR to require the use of corrosion-resistant metallic reinforcing bars such as MMFX2, stainless steel clad, and solid stainless steel.

17 Key Words Epoxy coated reinforcement, corrosion, reinforced concrete, bridge decks, deicing salts, marine structures	18. Distribution Statement No restrictions. This document is available to the public through NTIS, Springfield, VA 22161.

19. Security Classif. (of this report) Unclassified	20. Security Classif. (of this page) Unclassified	21. No. of Pages 35	22. Price

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

FINAL REPORT

SUMMARY REPORT ON THE PERFORMANCE OF EPOXY-COATED REINFORCING STEEL IN VIRGINIA

Richard E. Weyers, Ph.D., P.E.
Charles E. Via, Jr. Professor of Civil and Environmental Engineering
Virginia Polytechnic Institute & State University

Michael M. Sprinkel, P.E.
Associate Director
Virginia Transportation Research Council

Michael C. Brown, Ph.D., P.E.
Research Scientist
Virginia Transportation Research Council

Virginia Transportation Research Council
(A partnership of the Virginia Department of Transportation
and the University of Virginia since 1948)

In Cooperation with the U.S. Department of Transportation
Federal Highway Administration

Charlottesville, Virginia

June 2006
VTRC 06-R29

DISCLAIMER

ABSTRACT

From 1992 to 2006, the Virginia Transportation Research Council and its contract researchers conducted a long-term systematic series of investigations to evaluate the corrosion protection effectiveness of epoxy-coated reinforcement (ECR) and to identify and recommend the best and most cost-effective corrosion protection system for Virginia bridge decks. This report summarizes this research and subsequent efforts to implement alternative reinforcement. The work was conducted, and is reported, in this general order:

- review of historical performance of ECR, ECR performance in solutions and concrete, and preliminary field investigations
- investigation of field performance of bridge decks built with ECR
- assessment of alternative corrosion protection methods
- development of probabilistic service life models for bridge decks and laboratory assessment of ECR cores extracted from bridge decks to determine service life extension
- efforts to implement alternative reinforcement.

The series of studies demonstrated that the epoxy coating on ECR naturally degrades in the highly alkaline moist environment within concrete. The subsequent loss of bond, coupled with the inevitable flaws in the coating induced by construction, leads to an estimated service life benefit of ECR of as little as 3 to 5 years. Further, non-critical decks, beams, and substructure elements not exposed to marine environments, particularly on secondary and rural routes, can be cost-effectively constructed and maintained using low-permeability concrete and black reinforcing bar. However, because the Federal Highway Administration requires the use of corrosion-resistant reinforcement, and because ECR cannot provide adequate corrosion protection for structures designed for a 100-year+ service life as currently recommended by FHWA, the report recommends that the Virginia Department of Transportation amend its specifications regarding the use of ECR to require the use of corrosion-resistant metallic reinforcing bars such as MMFX2, stainless steel clad, and solid stainless steel.

FINAL REPORT

SUMMARY REPORT ON THE PERFORMANCE OF EPOXY-COATED REINFORCING STEEL IN VIRGINIA

Richard E. Weyers
Charles E. Via, Jr. Professor of Civil and Environmental Engineering
Virginia Polytechnic Institute & State University

Michael M. Sprinkel
Associate Director
Virginia Transportation Research Council

Michael C. Brown
Research Scientist
Virginia Transportation Research Council

INTRODUCTION

In the 1960s during the early period of the construction of the U.S. Interstate Highway System, premature deterioration of reinforced concrete bridge decks in the northern portions of the United States became evident. The time period of the early bridge deck deterioration coincided with the institution of the bare pavement policy that resulted in a fourfold increase in deicing salt application during the winter maintenance months.

In response, the Portland Cement Association, Bureau of Public Roads, presently the Federal Highway Administration, and state highway organizations conducted several research projects to identify the cause(s) of the premature deterioration of concrete bridge decks. The results of these investigations demonstrated that the principal cause of the early deterioration was corrosion of the reinforcing steel initiated and propagated by winter maintenance deicing salts containing chloride. Recommendations to decrease the rate of deterioration of reinforced concrete bridge decks included:

- increasing the clear concrete cover depth
- decreasing the chloride permeability of the concrete by decreasing the maximum allowable water to cement ratio
- minimizing chloride-laden melt water contact periods by improving deck surface drainage characteristics.

Subsequent to implementation of these recommendations in the early 1970s, engineering judgment was that the corrosion deterioration rate of concrete bridge decks was still too high, even though highway engineers did not know what the rate of deterioration was or the influence of implemented changes. Additional research studies were conducted to further improve the corrosion resistant service life of concrete bridge decks. Suggested options included application of concrete sealers, two-course construction, further increases in clear concrete cover depth and reduction in the concrete chloride permeability, use of less corrosive deicing agents, concrete corrosion inhibitor admixtures, and development of metallic and organic coatings for reinforcing steel.

BACKGROUND

The following presents the background on the development, implementation, and corrosion-resistance performance of epoxy-coated reinforcement (ECR) prior to the Virginia Transportation Research Council (VTRC) ECR studies and efforts to implement alternative reinforcement over the 14-year period from 1992 to 2006.

Development of ECR

The landmark research project on organic coatings for reinforcing steel was initiated in 1972, and the final project reports were issued in February 1974.[1,2] The project was sponsored by the Federal Highway Administration (FHWA) and conducted by the National Bureau of Standards (NBS) (now the National Institute of Standards and Technology [NIST]). A total of 47 coatings were tested for coating properties, coated steel characteristics, corrosion resistance, and bond strength between concrete and coated reinforcing steel. Testing consisted of the following:

- *Coating properties:* chloride permeability and water absorption
- *Coated steel:* abrasion resistance, hardness, impact resistance, chemical resistance, bend test, and electrical resistance
- *Corrosion resistance:* applied voltage
- *Bond strength:* pullout and creep.

The testing program was used as qualifying criteria for subsequent inclusion of corrosion-resistance testing of coated reinforcing bar in concrete. Four epoxy coatings were selected as having the best coating, corrosion resistance, and bond strength properties. It was expressed that, relative to corrosion resistance of coated bar in a sodium chloride solution, resistance measurements are probably a more reliable performance indicator than potential measurements, because resistance values are primarily dependent on the integrity of the coating films. However, for the four epoxy coatings that were further tested for corrosion resistance in concrete and further evaluated in experimental bridge decks, the resistance of two were about equal to that of uncoated bar: one decreased the resistance by 3 orders of magnitude to 1.5×10^3 ohms and one increased the resistance by 1 order of magnitude to 2×10^6 ohms.

Uncoated (black) and ECR bar using the four selected powders were cast in concrete with a water-to-cement ratio (w/c) of 0.49 and 25 mm (1 in) of clear concrete cover and partially submerged in a 3.5 percent solution by weight of sodium chloride. After 145 days, the copper copper-sulfate (CSE) potentials of the black bar specimens were -180 mV and -207 mV and, thus, were in the corrosion passive range. Three of the four selected epoxy powders used to coat the ECR bar and placed in concrete had potentials more negative (more active) than those of the black bar specimens. No evidence of cracking of the concrete or rust stains was observed on any of the black bar or ECR concrete specimens. Thus, the corrosion-resistance performance of ECR in concrete was not confirmed at the end of the research project.

The final report for this landmark study for organic coatings for reinforcing steel presented no conclusion on the corrosion protection performance of ECR in concrete, nor was further laboratory testing of the corrosion protection performance of ECR recommended. The recommendation was for the four selected epoxy coatings to be evaluated in experimental bridge decks. As stated in the report, these four epoxy coatings were not specifically designed for coating reinforcing steel, but coatings were expected to become available based on acceptance criteria.

The results of the landmark NBS (NIST) study were used to develop the Interim Specifications for Epoxy-Coated Reinforcing Steel.[3] A special provision for ECR used by Virginia in an experimental installation for two bridge decks in Carroll County may be considered typical of early specifications.[4] The special provision required the following:

- The coating material shall be one of the four same materials and quality evaluated in the NBS study.
- The bar surface shall be cleaned to white metal and coated before visible oxidation of the surface occurs.
- Coating material shall be applied as an electrostatic charged dry powder.
- Coating shall be uniform, smooth coat having a film thickness of 178 ± 50 μm (7 mil ± 2 mil) after cure.
- Coated bars shall be thermally treated to provide a fully cured finished coating.
- Coating shall be free of holes, voids, contamination, cracks, and damaged areas and shall have no more than 2 holidays in a linear foot of the coated bar.
- Flexibility of the coating shall be tested, with no visible evidence of cracking of the coating after bend testing.
- Coating shall be tested for abrasion loss.
- Bars shall be bundled with padded steel bands or slings for shipping.

- Bars whose coatings are severely damaged shall be rejected.

- Minor damage shall be repaired or patched.

- ECR shall be supported in the bridge deck on plastic or coated wire supports.

- Prior to concrete placement, minor damage areas shall be repaired.

Thus, the NBS recommendations were implemented. Interestingly, the resistance measurements of coated bar in chloride solution, which were considered a more reliable measure of corrosion resistance, were not included in the interim specifications or in revised future specifications. Further historical development of ECR specifications, including those of the American Society for Testing and Materials (ASTM) and foreign specification authorities, is presented elsewhere.[5]

These prescription specifications reflect the state of the practice in the 1970s. The prescription specifications were based on the hypothesis that a near continuous, intact, epoxy coating of 178 ± 50 μm (7 ± 2 mil) thickness bonded to steel reinforcing bar will provide long-term corrosion protection in concrete. The use of prescription specifications implies that such a hypothesis has been tested to a sufficient degree that a plausible or scientifically acceptable explanation (theory) has been demonstrated. Prescription specifications also imply that the failure mechanism(s) are sufficiently understood, such that an acceptable level of performance is reasonably ensured. Unfortunately, in the case of ECR, the corrosion protection performance in chloride-contaminated concrete was not demonstrated prior to the implementation of ECR in bridge deck construction. Thus, the corrosion protection performance hypothesis of ECR in chloride-contaminated concrete remained a hypothesis at implementation.

Implementation of ECR

The first bridge deck with ECR was constructed in 1973, prior to the issue of the NBS final project report in February 1974. Four spans of the northbound lanes of the bridge carrying I-476 over the Schuylkill River in West Conshohocken, Pennsylvania, were constructed with ECR in only the top mat of the deck.[3] The four spans of the 15-span structure constructed with ECR were part of the FHWA National Experimental and Evaluation Program, Project No. 16 (NEEP 16). The epoxy material, coating process, and construction procedures were required to meet the FHWA Interim ECR Specifications. By 1976, 40 bridge decks had been constructed by 19 state transportation agencies and the District of Columbia under the NEEP 16 Program using ECR as the top reinforcing mat.

In 1976, Virginia constructed two bridge decks using ECR as the top reinforcing mat in Carroll County.[4] By 1979, ECR became a standard bridge deck construction practice in Virginia. By 1987, at least 41 state departments of transportation (state DOTs) were using ECR for conventional structural concrete bridge decks as the only corrosion protection method.[6]

Corrosion Protection Performance of ECR

In 1980, Virmani et al. investigated the corrosion performance of ECR in slabs at a FHWA outdoor exposure site in Washington, D.C.[7] The ECR was coated in 1977 and stored outdoors for 2 years prior to the fabrication of the 12 test slabs with ECR either as the top mat only or the top and bottom mat. The ECR did not meet the FHWA Interim Specifications, AASHTO specifications, or ASTM specifications. The ECR had more than 25 holidays/foot (82 holidays/meter) and failed the bend test. The study concluded that it would take 12 times longer to consume the same amount of steel for ECR top mat only in concrete than for concrete decks constructed with black bars. For the case where ECR was used in the top and bottom mats, the predicted ratio was 46 to 1. The conclusions were based on measurements of macro-cell corrosion currents and electrical continuity between the top and bottom mats.

The implication of the FHWA conclusions was that if non-conformance ECR provided this level of corrosion protection, ECR that meets specification will provide long-term corrosion protection of at least 50 years. Standard practice by state DOTs was changed to ECR as the top and bottom mats as the primary corrosion protection system.

By 1987, there was still insufficient data on the corrosion protection performance of ECR from field evaluations because most installations were less than 10 years of age.[5] Simply stated, chloride had not penetrated to the depth of the reinforcing steel during this relatively short exposure period. In Virginia, when ECR was adopted as the corrosion protection system, the maximum allowable w/c ratio was simultaneously reduced to 0.45 from 0.47 and the clear concrete cover depth was increased so there would be very little to no reinforcing steel with a clear cover depth of 54 mm (2 in) or less. Clear cover depth specifications for bridge decks increased from 43 mm (1.69 in) to $63\ ^{+13}/_{-0}$ mm ($2.5\ ^{+½}/_{-0}$ in), and contractors received an additional payment for placing an additional 13 mm (½ in) of concrete.

In 1988, the first reported field corrosion protection failures of ECR were reported by the Florida DOT.[8-10] Spalling and delamination had taken place on three of four bridge substructures in the Florida Keys: Long Key, Seven Mile, and Niles Channel bridges. Corrosion damage was limited to 0.61 to 2.44 m (2 to 8 ft) above mean zero sea level. Clear concrete cover depths generally averaged 50 to 70 mm (2 to 3 in). The chloride contents exceeded the often-cited lower conservative limit for black bar of 0.71 kg/m^3 (1.2 lb/yd^3) by a factor of 1.66 to 19.3. The fourth bridge, Channel Number 5, had 102 mm (4 in) clear concrete cover with no evidence of corrosion damage and some structural cracking. ECR samples from Channel Number 5 bents suggested some degree of epoxy coating debondment. Based on field experience, the Florida DOT researchers and transportation officials concluded that active corrosion damage occurs in less time for ECR than for black steel in marine environments, since active corrosion generally occurs in black steel in 12 to 15 years and ECR corrosion had begun in 7 to 9 years.

As some researchers have stated, the "halo" around ECR had been removed. During the next 5 years, mixed performance of the corrosion protection performance of ECR in laboratory and field investigations was reported.[5] Reasons for the mixed corrosion protection performance of ECR included limited or inappropriate evaluation methods, which always accompany a lack of knowledge of the cause(s) of failure, and subjectively defined failure criteria.

In an effort to identify the state of the practice of evaluating the corrosion protection performance of ECR, a full-day technical session on ECR performance was held at the 1993 Transportation Research Board Meeting in Washington, D.C. It was then recognized that the question was not whether ECR would corrode, but when ECR would corrode. The Virginia Department of Transportation (VDOT) personnel who attended the session decided that Virginia needed to initiate an exploratory research project to determine if Virginia was having, or would in the future have, a corrosion protection performance problem with ECR.

The following presents the VTRC research project results, the conclusions drawn, and efforts to implement alternative reinforcement over the subsequent 14-year period from 1992 to 2006.

PURPOSE AND SCOPE

During the period from 1992 through 2003, VTRC conducted a series of investigations of the corrosion resistance performance of ECR to assess its cost-effectiveness in preventing corrosion of reinforced concrete bridge elements. The objective of this report is to present a summary of this research and subsequent efforts to implement alternative reinforcement. The research is presented herein as four separate but interrelated projects and a subsequent section on efforts to implement alternative reinforcement. The following is generally the order in which the work was concluded:

- review of historical performance of ECR, ECR performance in solutions and concrete, and preliminary field investigations[11-14]

- investigation of field performance of bridge decks built with ECR[15]

- assessment of alternative corrosion protection methods[16,17]

- development of probabilistic service life models for bridge decks and laboratory assessment of ECR cores extracted from bridge decks to determine service life extension[18,19]

- efforts to implement alternative reinforcement.

These studies illustrate the systematic, long-term investigation conducted by VTRC to evaluate the corrosion protection effectiveness of ECR and to identify and recommend the best corrosion protection system for Virginia bridges, particularly decks.

HISTORICAL PERFORMANCE REVIEW OF ECR, ECR PERFORMANCE IN SOLUTIONS AND CONCRETE, AND PRELIMINARY FIELD INVESTIGATIONS

Historical Performance Review 1974-93

The historical performance review of ECR sponsored by VTRC addressed the available structural and corrosion protection assessments and the laboratory and field studies completed to date. The review also included transcription of the technical sessions on ECR held at the 1993 TRB Annual Meeting.[11] The review built upon the previous NCHRP 10-37B literature review.[5]

Performance in Solutions and Concrete

In Solutions

ECR samples representing three coaters, Florida Steel (FS), Free State Coaters (FSC), and Lane Enterprises (LNE), that supplied VDOT projects were collected. The sampling procedure for straight No. 5 bars consisted of unannounced sampling visits to three construction sites.[12] The bars were stored outside for a total of 30 days, which included storage at the construction site and outside the laboratory, and then were stored in the laboratory under black plastic sheeting to minimize further UV light damage. Thus, the ECR bars reflected a typical natural environment, transportation to a job site, and handling at a job site. Black bar was also sampled from a single heat at Resco Steel, a bar manufacturing steel mill in Roanoke, Virginia. Subsequent investigation showed that all three coating plants had participated in and passed the Concrete Reinforcing Steel Institute's industry coating quality control inspection program.

ECRs were cut into 152-mm (6-in) specimens. Coating thickness, damage visible to the unaided eye, cracks, holes, mashed areas, scrapes, dented areas, number of holidays, and adhesion were measured. The ECR specimens were separated into two groups: 0 and 1 percent surface area damage. Plastic caps were attached to the cut ends to protect against corrosion during the immersion testing. Black and ECR specimens were preconditioned in a simulated concrete pore water solution at 40°C (104°F) for 7 days prior to the addition of sodium chloride. Sodium chloride was added to the simulated pore water solution for both black and ECR bars at concentrations equivalent to 0.00 (control), 0.73, 1.47, 2.93 and 5.86 kg/m^3 (1.2, 2.4, 4.8, and 9.6 lb/yd^3) of concrete. Immersion solutions were saturated with either oxygen or breathing air after addition of the sodium chloride and weekly throughout the 90-day test period. Specimens were visually examined after 4 weeks and every 2 weeks thereafter. Chloride, hydroxide, and pH were measured throughout the test period.

The results of the investigation were:

- The coating thickness between the ribs averaged 280, 180, and 150 μ (11.0, 7.0, and 5.9 mil) for the FS, FSC, and LNE coaters, respectively. Thus, the coating thickness between the ribs was less than the specified range of 175 to 300 μm (7 to 11 mil) for coater LNE.

- The average number of holidays per 152-mm section was 0.22, 0.19, and 10 for FS, FSC, and LNE, respectively. Thus, LNE coater did not meet the holiday specification of a maximum of 2 per 305 mm (2 per foot).

- Adhesion loss occurred during the immersion testing, even with those specimens in solutions that did not contain sodium chloride. Adhesion values increased from 1 to 3 for FS, 1 to 3 for FSC, and 2 to 5 for LNE. Thus, adhesion loss was greater for the LNE coater, which did not meet thickness and holiday specifications.

- Average corroded area was 0.13 percent for FS, 0.17 percent for FSC, and 0.77 percent for LNE of the surface area, respectively. The average percent surface corrosion for black steel specimens was 2.96 percent of the surface area.

- Black bar corrosion was typically pitting corrosion, whereas corrosion for ECR was more uniform underfilm corrosion. Thus, the ECR corroded area was 23 to 3.8 times greater than that for black steel.

In Concrete

The three field-sampled ECR No. 5 bars, coaters FS, FSC, and LNE, were cast into large, 0.76-m^3 (1-yd^3) inverted U-shaped specimens.[13] Four exposure conditions were represented: a bridge deck (horizontal section), vertical surfaces (column), marine tidal zone, and submersion zone (legs of the inverted U-shaped specimens). The four conditions included wet salt solution and dry cycles. The concrete mixture w/c was 0.45, and the clear cover depth was 25 mm (1 in). Specimens were exposed to either a 3 or 6 percent by weight sodium chloride solution weekly for 2 years. A control specimen using black steel was also cast and exposed to a 6 percent sodium chloride solution. The chloride content at a depth of 25 mm (1 in) of the 0.45 w/c mixture exceed 0.73 kg/m^3 (1.2 lb/yd^3) after 1 year and 1.5 years for the 6 and 3 percent chloride solutions, respectively. After 2 years, a conclusion that is typical for the laboratory testing of new ECR in concrete was reached: "The ECR specimens performed significantly better than the controls (black bar) in the relative short laboratory exposure period even though one of the three ECR samples did not meet specifications."

Preliminary Field Investigation

Because coating damage, thickness, and adhesion may influence the corrosion protection performance of ECR and the difference that may exist between laboratory testing of new ECR and the long-term field exposure of field ECR, a preliminary field investigation was conducted. The objective was to determine if Virginia was experiencing conditions similar to those of other field investigations in northern deicing salt and marine exposures.[14] The following summarizes the results of the preliminary field investigation of three structures in marine and deicing salt environments.

Marine Structures

Two of the structures were 7 years old and one was 6 years old at the time of the investigation in 1996. A total of 23 cores, each containing a No. 3 smooth ECR bar, were evaluated. The cores were taken on a straight section of square prestressed piles between high and low tide elevations. Virginia uses a dual corrosion protection system for marine piles: epoxy coating of the concrete surface and ECR for tie bars. All but one of the pile concrete surfaces were coated with epoxy. The piles were in brackish water with a percent chloride content of 0.87, 1.06, and 1.54. There were no surface cracks in any of the cores; the carbonation depth was essentially zero; the clear cover depth for the three structures ranged from 48 to 66 mm (1.9 to 2.6 in); and the concrete percent saturation ranged between 83 to 89 percent.

- Coating thickness specifications for the ECR were 127 to 229 μ (5 to 9 mil). The measured coating thicknesses, as an average thickness per bar, were somewhat variable, with 2 less than 127 μ (5 mil), 15 between 127 and 229 μ (5 to 9 mil), and 6 greater than 229 μ (9 mil).

- Two ECR bars showed damage and exceeded the damage specification limits because of incomplete coating. Corrosion was visible on one of these bars and most likely occurred prior to the concrete casting because the chloride content in the concrete ECR bar trace was 0.20 kg/m^3 (0.33 lb/yd^3), which is the concrete background content.

- Only two cores had a chloride content in the ECR bar trace greater than 0.72 kg/m^3 (1.2 lb/yd^3): 1.19 and 2.18 kg/m^3 (2.0 and 3.6 lb/yd^3). The highest chloride content in the bar trace was for the core taken on the pile without a concrete surface epoxy coating. The chloride content of the remaining 21 cores in the bar trace was about equal to the concrete background content for this concrete type.

- The holiday counts for the 23 ECR sections (53 mm, 2.0 in) were 0 for 15 ECR, 1 for 2 ECR, 2 for 1 ECR, and continuous for 5 ECR . A 53-mm section with 1 or more holidays exceeded the holiday specification maximum of 2 per 305 mm (12 in). Thus, 8 ECR sections, or 35 percent, exceeded the holiday specification.

- Adhesion values were 5 for 11 ECR, 4 for 1 ECR, 3 for 2 ECR, 2 for 2 ECR, and 1 for 3 ECR sections. Thus, adhesion loss is occurring without the presence of chloride. The adhesion loss is classified as wet adhesion loss in wet environments (moist concrete) where the adhesion of epoxy to steel will occur spontaneously.

- Corrosion products were visually observed under the epoxy coating for 10, or 43 percent, of the 23 ECR sections. Thus, the steel bar appears to be oxidizing under the epoxy coating without the presence of chloride.

Bridge Decks

Three bridge decks that receive about 25 deicing salt applications per winter were evaluated. The decks were built in 1979 with a top mat of ECR and were 17 years old at the time of the investigation. A total of 36 cores, 12 from each bridge deck, were drilled through a No. 5 ECR reinforcing bar. The cores were 100 mm (4.0 in) diameter and were taken randomly within the area of the deck that exhibited the 12th percentile least clear concrete cover depth. The random coring procedure encountered two delaminated areas, both in the same deck. The concrete percent saturation for the three decks was relatively uniform, varying between 83.2 and 84.0 percent. The percent concrete moisture content and percent saturation for the deck and pile concrete were similar. Fifteen of the 36 cores had surface cracks, the majority being transverse cracks generally oriented over the reinforcing bars. Of these, 14 were less than 0.300 mm (0.012 in) wide at the surface, one was 0.504 mm (0.020 in) wide, and the remaining cracks were in the middle of this range. The presence of surface cracking appeared not to influence the chloride content of the bar trace, as the chloride in the bar trace was about equal to the chloride content at the same depth but 50 mm (2.0 in) away from the bar trace. None of the concrete bar traces was carbonated. The clear concrete cover depth ranged from 46 to 66 mm, 66 to 76 mm, and 46 to 66 mm (1.8 to 2.6 in, 2.6 to 3.0 in, and 1.8 to 2.6 in) for the 12 cores taken from the area with the 12th percentile of the smallest clear cover depth for the three decks.

- Coating thicknesses, as an average thickness between ribs, were slightly variable: 4 were less than the specified minimum 127 μ (5 mil), 28 were within the 127 to 229 μ (5 to 9 mil) thickness requirement, and 4 were greater than the specified maximum of 229 μ (9 mil).

- The number of holes in the epoxy coating was low: 25 had 0 holes, 8 between 0.1 and 0.5 percent of the surface area, and 3 greater than 0.5 percent of the surface area.

- Total percent damage, i.e., the sum of the mashed areas, dents, scrapes and holes, of the epoxy coating surface was also relatively low: 17 at 0, 8 between 0.10 and 0.50 percent of surface area, 6 between 0.60 and 1.0 percent of surface area, and 5 greater than 1.0 percent of surface area.

- Eight ECR had an adhesion value of 5, 11 had 4, 6 had 3, 10 had 2, and 1 had 1. Thus, 35 of the 36 ECR specimens appear to have lost adhesion. Of the 25 ECR specimens with an adhesion value of 3 or greater, no corrosion was observed under the coating for 14 ECR specimens.

- Holidays were continuous for 24 of the 36 ECR specimens, 5 specimens had between 10 and 22 holidays, and 7 had between 3 and 9 holidays. Thus, none of the ECR bar sections was less than the holiday specification of 2 per 305 mm (2 per foot).

- Concrete chloride contents in the ECR bar trace exceeded 0.72 kg/m^3 (1.2 lb/yd^3) in 18 of the 36 cores. All 12 chloride contents of the bridge deck that included two delaminated areas were greater than 0.72 kg/m^3 (1.2 lb/yd^3) and ranged between 1.12

and 6.56 kg/m^3 (1.74 and 10.9 lb/yd^3). The two delaminated area chloride contents were 4.95 and 6.56 kg/m^3 (8.25 and 10.9 lb/yd^3).

- Of the 36 ECR specimens, 11 exhibited corrosion under the epoxy coating, 9 of which had chloride contents in the bar trace greater than 0.72 kg/m^3 (1.2 lb/yd^3). Eight of the 25 ECR specimens that did not exhibit corrosion under the epoxy coating had chloride contents in the ECR trace greater than 0.72 kg/m^3 (1.2 lb/yd^3).

INVESTIGATION OF FIELD PERFORMANCE OF BRIDGE DECKS BUILT WITH ECR

Overview

The results of the previous study created two questions that VDOT wished to address:

1. What is the extent of the ECR conditions identified in the preliminary study, including coating adhesion loss and corrosion under coatings, within Virginia bridges?

2. What other corrosion protection system(s) would perform better than ECR in Virginia?

The following presents the results of the two research projects that were initiated in 1997 to address these two questions.

Field Condition of ECR in Bridge Decks

Because Virginia uses a dual corrosion protection system, consisting of a protective epoxy coating on the concrete exterior and epoxy-coated reinforcement within, for piles in marine environments and because of the demonstrated effectiveness of the concrete epoxy surface coating in limiting chloride ingress, further research efforts were limited to bridge decks.

Two survey bridge decks were selected from each of the nine VDOT districts.[15] The decks were built between 1977 and 1995. Two decks also represented each age group of 20, 16, 14, 12, 10, 9, 6, and 4 years old. In addition, two decks were selected for the 1995 construction period that represented the most recent change in ECR coating thickness specifications. The 1995 decks were 2 years old when the field surveys were completed. In general, 12 cores were drilled through top mat ECR No. 5 reinforcing bars and 3 cores through bottom mat ECR No. 5 reinforcing truss bars. Thus, approximately 30 ECR cores represented each age of 20 to 2 years. The ECR specimens extracted from the cores were approximately equal to the core 100 mm (4.0 in) diameter. A total of 256 cores were obtained, 206 from the top mat and 50 from the bottom mat. It needs to be pointed out that alternate deck reinforcing bars are truss bars; thus, when

ECR was used in the top mat only, one of every two bottom mat reinforcing bars in the trough region of the truss was an ECR bar, alternating sequentially.

The laboratory assessment of the deck concrete included rapid permeability, chloride content at a depth of 13 mm (1/2 in), and moisture content and percent saturation. A condition assessment of the ECR sections included coating damage, holes in the coating, holidays, coating thickness, and adhesion. The following presents the concrete and ECR evaluation results.

Concrete

The evaluation of the concrete yielded the following results:

- Average rapid permeability for the 18 bridge decks ranged between very low to moderate: 8 very low, 6 low, and 4 moderate. The highest average charge passed in Coulombs (hereafter "Coulomb value") was 2600 (moderate).

- Average chloride content at a depth of 13 mm (1/2 in) ranged between 0.74 and 5.77 kg/m^3 (1.2 to 6.7 lb/yd^3), with an average of 2.2 kg/m^3 (3.7 lb/yd^3). Average chloride at a depth of 13 mm (1/2 in) was less than 1.0 kg/m^3 (1.7 lb/yd^3) for 4 decks, between 1.0 and 3.0 kg/m^3 (1.7 and 5.0 lb/yd^3) for 10 decks, and greater than 3.0 kg/m^3 (5.0 lb/yd^3) for 4 decks.

- Concrete moisture content at the depth of the top ECR ranged between 4.2 and 5.5 percent, and percent saturation ranged between 73 and 90 percent.

ECR Sections

The evaluation of the ECR sections yielded the following results:

- The percent surface damage of the coating was the sum total of the areas with mashed, dented, and scraped spots and cracks, and blisters. The average percent damage ranged between 0.05 and 0.3 percent for the top ECR sections: 16 were less than 0.1 percent, 1 was 0.14 percent, and 1 was 0.3 percent. The bottom ECR sections had similar epoxy-coating surface damage; the range of the averages was 0.02 to 0.2 percent.

- The number of holes in the epoxy coating of the ECR top sections was 0 for 16 decks and 0.07 and 0.34 per meter (0.02 and 0.10 per foot) for the other 2 decks. For the bottom ECR sections, only 1 ECR deck set of sections had holes in the epoxy coating, 1.09 holes per meter (0.33 per foot).

- The average coating thickness for the 41 top ECR sections for the 14 decks built before 1995 complied with coating thickness specifications of 127 to 229 μ (5 to 9 mil); the average coating thickness for 2 decks was greater than 229 μ. The 2 decks built in 1995 had average ECR sections taken from the top mat of reinforcement that ranged between the new specifications of 178 to 305 μ (7 to 12 mil). The average top ECR coating thickness for these 2 decks was 225 and 230 μ (8.8 and 9.0 mil). The bottom ECR section thickness was approximately the same.

- The thickness for the adhesion-tested top ECR sections ranged from 140 to 240 μ (5.5 to 10.2 mil). The average adhesion for the top ECR sections ranged between 1 and 4.5: 7 were greater than 3, 2 were between 3 and 2, 8 were between 2 and 1, and 1 was 1. The bottom ECR sections were associated with similar adhesion results.

ASSESSMENT OF ALTERNATIVE CORROSION PROTECTION METHODS

Systems Evaluated

The corrosion protection systems evaluated included low-permeability concretes, fly ash, ground-granulated blast furnace slag and microsilica mixtures, microsilica plus calcium nitrite corrosion inhibitor, and three corrosion inhibitors in a bridge deck portland cement mixture.[16,17] The large U-shaped specimens and exposure conditions used in this study were the same as in the previous ECR corrosion protection laboratory study of three Virginia coaters.[13]

Results demonstrated that bridge decks built with black steel and low-permeability concrete or black steel, low-permeability concrete, and the calcium nitrite corrosion inhibitor would not have to be overlaid within a 75-year design life. The results were demonstrated using a clear concrete cover depth of 50 mm (2.0 in), surface chloride content of 4.05 kg/m^3 (6.8 lb/yd^3), corrosion initiation concentration of 0.89 kg/m^3 (1.5 lb/yd^3), and chloride diffusion constant of 7.7 mm^2/yr (0.01 in^2/yr).

Cost-Effectiveness

Cost-effectiveness was based on a 75-year service life, 5 percent interest rate, and VDOT construction bid prices.[15] Table 1 presents the life cycle present value (1999) per unit surface area for five bridge deck systems.

The question that remained, because there had been no definitive study of field performance, was: What is the service life extension provided by ECR in Virginia?

Table 1. Cost Comparison of Alternative Corrosion Protection Systems

System	Total Cost $/m^2 ($/ft^2)
Black steel + PCC	128.25 (11.93)
ECR + PCC; 10 yr additional 20 yr additional 25 yr additional	128.90 (11.99) 127.06 (11.82) 126.42 (11.76)
Black steel + LPC	120.18 (11.18)
Black steel + PCC + CI	121.80 (11.33)
Black steel + LPC + CI	124.16 (11.55)

To address properly the question of the use of ECR or any other corrosion abatement system, the following must be addressed:

- expansion of deterministic corrosion models that include the initiation and corrosion periods to include the variability of the input parameters for a bridge deck or bridge system

- identification of the percentage and location of decks that require a corrosion protection system and what system is required must be identified.

DEVELOPMENT OF PROBABILISTIC SERVICE LIFE MODELS FOR BRIDGE DECKS AND LABORATORY ASSESSMENT OF ECR CORES EXTRACTED FROM BRIDGE DECKS TO DETERMINE SERVICE LIFE EXTENSION

Probabilistic Service Life Models

Corrosion life models are typically separated into two distinct, additive stages: time to initiate corrosion and time from corrosion initiation to cracking and spalling of the cover concrete. The project concentrated on the initiation phase because this is typically the longest time period and the active corrosion period may be simply added to the initiation time period. [18] The initiation time period variables are corrosion initiation concentration, surface chloride content, apparent chloride diffusion constant, and clear concrete cover depth.

Two approaches may be used in developing a statistically based model, depending on whether the probability density functions are known or are not fully developed: the parametric bootstrap method, where each variable is described by a continuous probability density function and sample values are generated at random using the function, or the simple bootstrap method, where individual values are selected from a finite group of measured values. It was known based on field data that the cover depth distribution is normally distributed and the surface chloride and apparent diffusion constant may be gamma distributions; little was known of the functional relationship for the corrosion initiation concentration distribution. Thus, Monte Carlo simulation was used as the computational tool because the iterative sampling and calculation procedure does not require a definitive knowledge of the probability density functions.

A computer program was developed using Monte Carlo simulation. The number of iterations was determined to minimize the computation error. Using field-collected data for the cover depth, surface chloride content, and apparent diffusion constant from 10 Virginia bridge decks that were distributed geographically and thus representative of chloride exposure conditions and corrosion initiation values, a probabilistic corrosion service life distribution for Virginia was developed. The field data compiled from the 10 bridge decks represented decks built between 1981 and 1995. These decks are representative of the population of decks built under the VDOT specification of a maximum w/c = 0.45 and a cover depth of 63 mm minus zero, plus 13 mm (2.5 in minus 0.0 in, plus 0.5 in). The statistically based computational model was then validated through comparison with the performance of a random sample of 128 bridge decks built between 1968 and 1972.

Subsequent analysis determined that less than 25 percent of all bridge decks built since the advent of ECR in 1979 and the simultaneous reduction of the maximum allowable w/c to 0.45 from 0.47 and the increase in clear concrete cover depth from 43 mm (1.69 in) to 63 mm (2.5 in) minus zero, plus 13 mm (0.5 in) [2.5 to 3.0 in] regardless of reinforcing bar type (black, ECR, galvanized) would need to be rehabilitated within 75 years.[18] In addition, critical chloride deicing salts zones were identified based on the deicing salt usage in Virginia for the three winter seasons of 2001, 2002, and 2003. The critical regions are the VDOT districts of Northern Virginia, Staunton, and Culpeper. Decisions for specific bridge locations are easily determined by measuring the chloride content at 12 to 15 locations in the two wheelpaths of the most traveled lane. The surface chloride contents serve as the input data to determine time-to-repair and rehabilitation for select corrosion protection systems.

Laboratory Assessment of ECR Cores Extracted From Bridge Decks to Determine Service Life Extension

The objectives of the research project were five fold: estimate the corrosion protection service life of ECR; identify factors influencing ECR performance; determine the influence of cracks on corrosion protection service life; determine the cost-effectiveness of corrosion protection methods; and make recommendations on the selection of corrosion protection systems in Virginia.[19]

Selection of Study Bridge Decks and Coring

Ten bridge decks were selected for inclusion in the study. Eight were built with ECR, and two with black steel. The ECR decks were built between 1981 and 1995 and simultaneously met the representative characteristics of in-place residual adhesion of 1, 2, 3, and 4; surface chloride contents of about 1 to 6 kg/m^3 (1.7 to 10 lb/yd^3); and the geographic distribution that included multiple coater marketing areas. The two black steel decks were built in 1983-84 using the same concrete and cover depth specifications as were used with the ECR decks. Cores drilled through a single No. 5 ECR reinforcing bar were taken adjacent to cores taken in the previous ECR study.[14] In general, 12 cores were taken adjacent to previous core locations at areas with surfaces that were not cracked, and 3 cores over surface crack locations. A total of 141 drilled cores, 113 ECR and 30 black steel, were obtained and used in the corrosion service life study. The cores were 102 mm (4.0 in) in diameter and about 127 to 152 mm (5 to 6 in) in length.

The top section of each core was removed by dry cutting, resulting in a uniform concrete cover above the reinforcing bar, ECR and black steel, of 13 mm (0.5 in). Following additional preparation, cores were ponded with a 3 percent by weight sodium chloride solution. The weekly ponding cycle was 2 days ponding followed by 3 to 5 days drying under laboratory conditions. Electrochemical impedance spectroscopy (EIS) was used to assess the corrosion activity or lack of it immediately after the fourth week ponding cycle. Simultaneous shifts in corrosion potential, impedance at 1 mHz, and phase angle identified the initiation of corrosion. The end of the time-to-cracking (propagation) period, from corrosion initiation to visual evidence of a surface crack above the reinforcing bar, was also determined. Following the end of the time-to-cracking period, the cores were destructively examined (autopsied) to determine the corroded condition of the bar and chloride content above and at the corroded bar sites. Chloride contents at corrosion initiation were back-calculated using a solution of one-dimensional diffusion through a semi-infinite medium, according to Fick's Second Law.

Figures 1 and 2 present the percent probability curves for the initiation and time-to-cracking periods. As shown, all the black steel was corroding after about 10 months. After 36 months, 70 percent of the ECR specimens were corroding. Fifty percent of the black bars were corroding after 2 months, and 50 percent of the ECR specimens were corroding after 14 months. However, corrosion initiated in a small percentage of the ECR specimens prior to initiation in any of the black bar specimens. The mean chloride content for the ECR specimens was greater than that for the black bar at corrosion initiation, 4.63 and 2.94 kg/m^3 (7.7 and 4.9 lb/yd^3), respectively. However, because of the larger variability in the chloride corrosion initiation values for both the ECR and black bars, statistically there appears to be no difference.

As shown in Figure 2, black steel specimens cracked prior to any of ECR specimens. At the end of the 36-month study, 26 percent of the ECR specimens had cracked whereas all but 1 of the black steel specimens had cracked in about 22 months. At the beginning of the observed cracking, the difference was about 4 months but increased to about 16 months at the 25th percentile level. Of interest is the 12th percentile level, the deterioration level when bridge decks are overlaid.[21] The difference at the 12th percentile level is about 12 months. The mean chloride content at cracking was 7.37 and 9.52 kg/m^3 (12.3 and 15.9 lb/yd^3) for black and ECR specimens, respectively. However, again because of the large variability of the chloride concentrations for both black and ECR specimens at cracking, it was not possible to state that there was a significant difference between the two groups.

Because of the nature of accelerated laboratory ponding tests, further analysis was warranted to relate to field performance. An effort was made to compare the corrosion propagation phase of the two reinforcing materials based on a relationship developed by Torres-Acosta and Sagues.[24] The method relates the propagation period, t_p, to a critical depth of corrosion penetration to cause cracking, x_{CRIT}, and to the rate of corrosion, C.R., as expressed in Equation 1. The critical depth of corrosion penetration is a function of the length of bar along which the corrosion is concentrated, as well as the clear cover depth and the bar diameter.

$$t_p = x_{crit} / \text{C.R.} \qquad \text{[Eq. 1]}$$

A prior analysis using this methodology inadvertently employed an equation referenced from a publication containing a misprint, giving potentially misleading results.[19] The analysis was revised using the appropriate equation, and results follow.

When the observed average degree of surface area corroded at the time of cracking for the two bar types was used, 25 percent for black and 80 percent for ECR, the average corrosion rate for black steel appeared to be approximately twice that of ECR, as shown in Table 2.

Since the laboratory studies were confined to single bars of 4-in length, it is not possible to forecast the relative surface areas directly that would corrode (indicating degree of corrosion localization) in the field for longer segments of the two bar types. However, if the surface area corroded at the time of cracking is assumed to be the same for both types, the depth of penetration is not the critical factor, and the expected corrosion propagation time extension can be estimated to range from 2.7 to 5.7 additional years for ECR beyond that expected for bare steel, with an average of the shallowest 20th percentile depths of about 4 years additional service, as shown in Table 3.

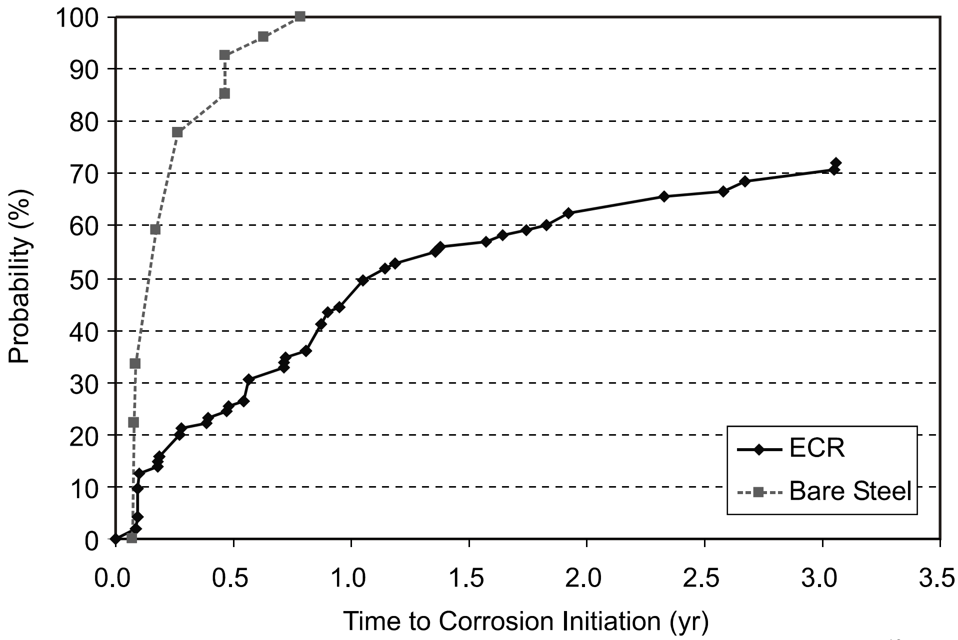

Figure 1. Comparison of Time to Initiation under Laboratory Exposure.[19]
Note: The experimental data for the period 1.94 to 3.06 years were provided by M.C. Wheeler.[20]

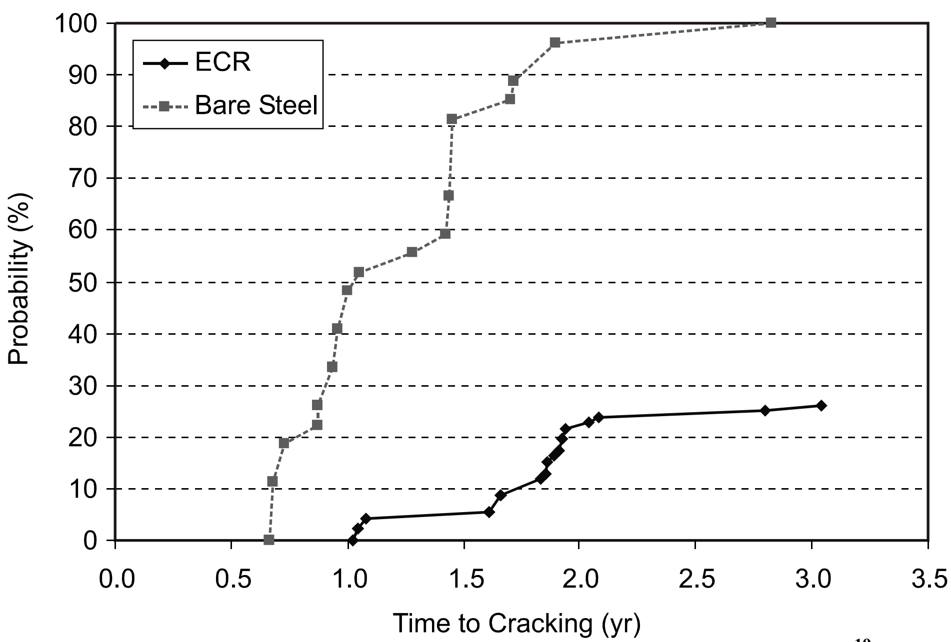

Figure 2. Comparison of Time to Cracking under Laboratory Exposure[19]
Note: The experimental data for the period 1.94 to 3.06 years were provided by M.C. Wheeler.[20]

Table 2. Corrosion Rate Comparison from Laboratory Tests

Bar Diameter (mm)	L_{BARE} (mm)	L_{ECR} (mm)	$x_{crit(BARE)}$ (mm)	$x_{crit(ECR)}$ (mm)	$t_{p(BARE)}$ (yr)	$t_{p(ECR)}$ (yr)	$CR_{(BARE)}$ (μm/yr)	$CR_{(ECR)}$ (μm/yr)	CR Ratio $^{BARE}/_{ECR}$
16	25	80	0.019	0.012	0.35	0.49	54.3	23.9	2.27
16	25	80	0.019	0.012	0.46	0.50	41.3	23.4	1.76
16	25	80	0.019	0.012	0.58	0.67	32.7	17.5	1.87
16	25	80	0.019	0.012	0.58	0.68	32.7	17.2	1.90
16	25	80	0.019	0.012	0.61	0.85	31.1	13.8	2.26
								Average	2.01

Table 3. Propagation Times Under Field Corrosion

Bar Diameter (mm)	L_{BARE} (mm)	L_{ECR} (mm)	$x_{crit(BARE)}$ (mm)	$x_{crit(ECR)}$ (mm)	$t_{p(BARE)}$ (yr)	$t_{p(ECR)}$ (yr)	ECR Time Extension (yr)
16	100	100	0.065	0.065	3.00	6.81	3.81
16	100	100	0.071	0.071	3.50	6.16	2.66
16	100	100	0.081	0.081	4.00	7.49	3.49
16	100	100	0.083	0.083	4.00	7.60	3.60
16	100	100	0.088	0.088	4.50	10.16	5.66
				Average	3.80	7.64	3.84

The assessment is an estimate based on currently available information. The expected service life extension may vary depending on the actual values of *L*, for which there is inadequate evidence at present. In addition, a number of potential issues may influence the estimation of critical corrosion penetration depth. For example, the presence of the coating may introduce physical factors that may influence the relationship between degree of localization (corroded length) and critical penetration depth. Concrete is known not to bond as well to the epoxy coating as to bare steel, and this may influence the transition zone between the bar and the cement paste, changing the volume that must be filled by corrosion product before expansive pressures can develop. Conversely, the presence of the coating may confine corrosion products, where bare steel may permit some corrosion product to migrate into the void structure of the cement paste, also influencing the development of expansive pressures.

Influence of Cracks

Crack surveys of 3,346 m^2 (10,978 ft^3) resulted in a unit crack frequency of 0.23 m/m^2 (0.12 ft/ft^2). Eighty-seven percent were longitudinal cracks, and 13 percent were transverse/diagonal cracks. In addition, 87 percent of the measured surface crack width was less than or equal to 0.30 mm (0.012 in), and 13 percent was greater 0.30 mm (0.012 in).

Although 30 field cores were drilled through visual surface cracks over an ECR bar, subsequent laboratory assessment of all the cores demonstrated that 40 cores contained surface cracks. Crack widths of the cores ranged from 0.07 to 0.33 mm (0.003 to 0.013 in). Only 2.5 percent of the cores had a surface crack greater than 0.30 mm (0.012 in). Crack depths ranged from 3 to 162 mm (0.12 to 6.4 in), with 12.5 percent of the cracks penetrating to at least the depth of the reinforcing steel. Analysis of the chloride contents in the cracked and uncracked cores demonstrated that the chloride content at 19 mm (0.75 in) above the bar showed that the cracks influenced (increased) the chloride content at 19 mm above the bar 61 percent of the time.

For a worst-case scenario, crack frequency of 0.38 m/m^2 (0.12 ft/ft^2), crack-corrosion influence zone of 130 mm (5.12 in), and 61 percent of the cracks increasing the chloride content at the bar depth, the resulting deck area affected is projected at 3.0 percent.

Factors Influencing ECR Performance

An effort was made to identify the differences among the three groups of ECR specimens that existed within the 36-month ponding period, corrosion initiated and cracked (CR), corrosion initiated not cracked (CI), and corrosion not initiated (CNI). After 22 months of ponding, 17 ECR were randomly selected from the two CI and CNI groups. EIS measurements indicated that corrosion had not initiated in 9 of the 17; visual observations of the ECR specimens verified that no corrosion was present on these ECR specimens. Chloride initiation concentrations were investigated by considering direct and interactive relationships to various coating and concrete parameters. Coating parameters considered were holidays, percent surface damage, and thickness. Concrete parameters included absorption and degree saturation prior to the first ponding cycle, apparent field diffusion constants, and the original core clear concrete cover depth. No reliable relationships were found. Of interest is that the percent damage to the coating or number of holidays could not be correlated within this small subset of cores. This may have been the result of the low percent of damage, averaging 0.2 percent, and the number of holidays, averaging less than 2 per meter (0.6 per foot).[19,20]

After 36 months, additional cores were analyzed to assess the concrete and coating parameters that correlated with corrosion of ECR. Additional testing was conducted on a total of 22 ECR specimens: 3 CNI, 3 CI, and 16 CR. Additional concrete parameters and coating parameters after ponding were evaluated. Concrete parameters included were moisture content and degree of saturation of the concrete during treatment. Additional coating parameters included moisture content, glass transition temperatures, and degree of micro-cracking.[20] Micro-cracking in the coating was observed under the scanning electron microscope (SEM) and rated visually on a scale of 1 to 4, with 1 indicating no observed cracking and 4 indicating severe map cracking. Figure 3, from a related study, illustrates the most severe of the four degrees of micro-cracking observed, and Figure 4 illustrates the width of observed cracks, which were about 4 orders of magnitude greater than a water or chloride molecule, suggesting that chloride could enter and partially or completely breach the epoxy coating.[19,20,22] Cracks were observed in 59 percent of the ECR specimens: 27, 18, and 14 percent for ratings 2, 3, and 4, respectively. The moisture content of the epoxy coating, as presented in Figure 5, ranged between 0.38 and 1.39 percent, with a mean of 0.81 percent, which would be considered typical for epoxy.[19,20]

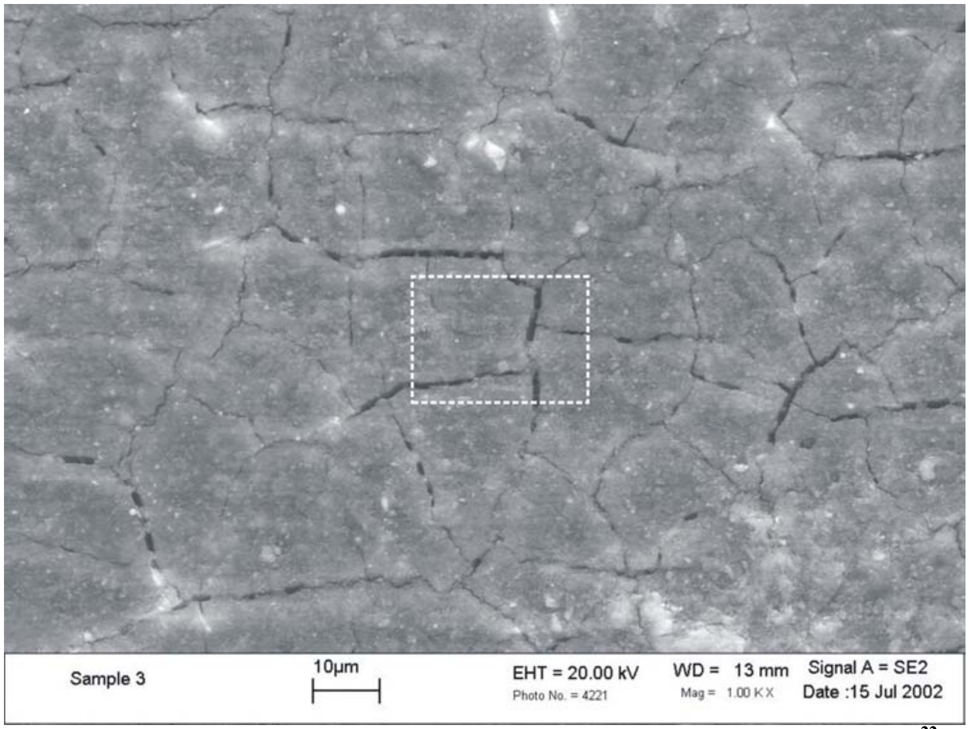

Figure 3. Observed Cracks in Coating under SEM: Visual Rating = 4 (2000x) [22]

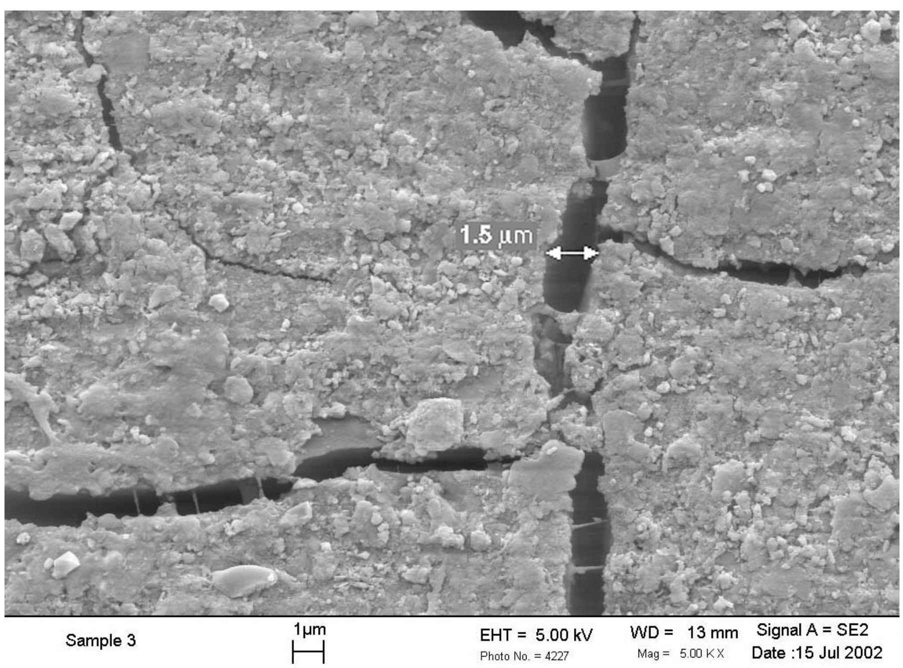

Figure 4. SEM Photomicrograph of Surface Crack in Epoxy Coating (5000x) [22]

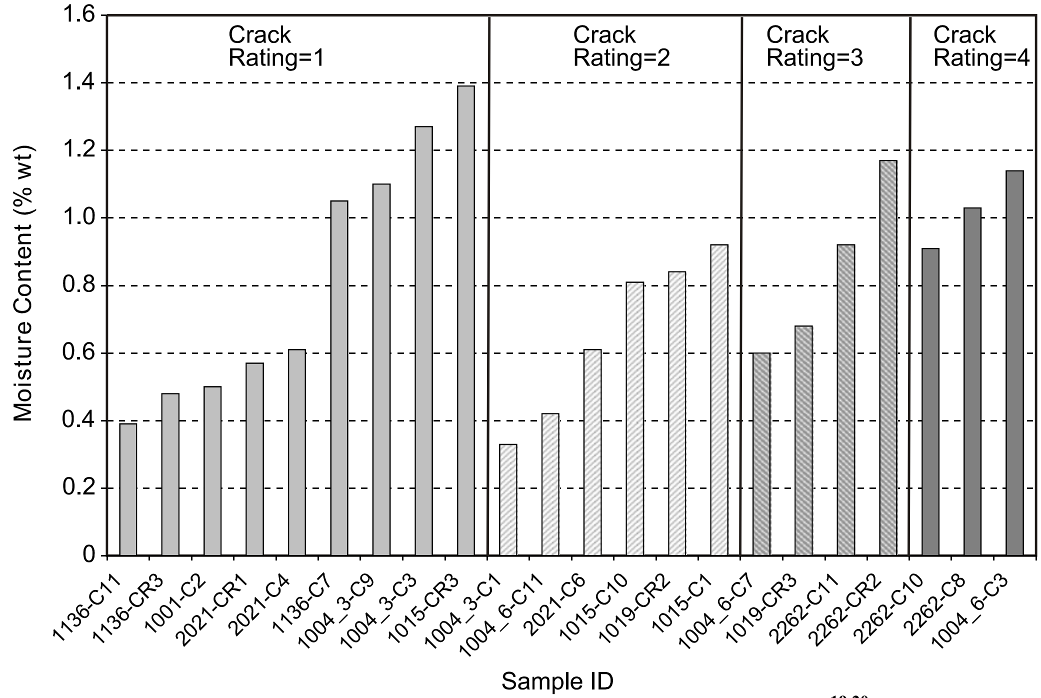

Figure 5. Histogram of Epoxy Coating Moisture Content[19,20]

The epoxy coating glass transition temperature was measured in two states: as removed from the concrete core and after the coating was fully cured. The as-removed and fully cured glass transition temperatures ranged from 64.54ºC to 102.5 ºC, and 90.9 ºC to 116.5 ºC, respectively. The change in glass transition temperature (the difference between glass transition temperature after full curing versus glass transition temperature as-removed from the concrete core) ranged from as little as 5.86 ºC to as much as 50.0 ºC.[19,20] The results indicate that production line ECR being placed in bridge decks in Virginia is not fully cured.[19,20]

Statistical analysis revealed no reliable relationship, direct or interrelated, between the corrosion initiation and corrosion propagation period and concrete and coating parameters. However, moderate relationships were demonstrated between the epoxy coating moisture content and change in glass transition temperature, crack rating and change in glass transition temperature, and crack rating and epoxy coating moisture content. Moisture content was found to increase with increasing change in glass transition temperature, which is in agreement with the principle that the lower the degree of curing, the more open the epoxy matrix and the more water the coating may absorb. The severity of cracking in the epoxy coating increases with increasing change in epoxy coating glass transition temperature. The observation is in agreement with the concept that a lower degree of curing results in lower strength of the epoxy; thus, the more prone the coating would be to cracking as a result of volume changes after manufacturing. As can be observed for crack ratings 2, 3, and 4 in Figure 5, a weak relationship exists where as the severity of surface cracking of the epoxy coating increases, the coating moisture content increases, suggesting that a more open epoxy matrix is more prone to cracking and ingress of moisture.[19,20]

EFFORTS TO IMPLEMENT ALTERNATIVE REINFORCEMENT

Meeting with the Federal Highway Administration

A meeting of staff from FHWA, VDOT's Structure & Bridge Division, and VTRC was held at VTRC on December 8, 2005. The objective of the meeting was to identify alternatives to ECR that VDOT could use in FHWA-funded construction and to develop plans to phase out VDOT's use of ECR.

Discussions indicated that approximately 13 years of research by VTRC had found that a number of deck protection systems were more cost-effective than ECR. These systems included uncoated, MMFX2, stainless steel clad, and solid stainless steel reinforcement. FHWA indicated that to receive funding for structures, VDOT is required to specify and use a concrete protection system and a reinforcement protection system. Low-permeability concrete and 2.5 in of concrete cover qualifies as a concrete protection system. Epoxy-coated, MMFX2, stainless steel clad, and solid stainless steel reinforcement qualify as reinforcement protection systems. FHWA indicated that for applications in which ECR is specified in Section 8.22.6 of the November 2004 edition of the VDOT Modification, Highway Bridges of the AASHTO Design Standards; epoxy-coated, MMFX2, stainless steel clad, and solid stainless steel reinforcement may be used.[23] Uncoated plain bar will continue to be used in structural elements in non- corrosive environments where it has been approved for use in the past in accordance with Section 8.22.

Discussions further indicated that of the alternatives, the initial cost of MMFX2 is most likely to be competitive with that of ECR. Consequently, MMFX2 will be specified for the typical applications where ECR has been used in the past, but contractors can also bid and use stainless steel clad and solid stainless steel with the approval of the bridge engineer. The sole source issue may or may not have to be addressed. Although there is one source of MMFX2 reinforcement, many fabricators have supplied MMFX2 reinforcement for both DOT and commercial contracts. The competition between fabricators and the alternatives of stainless steel clad and solid stainless steel should eliminate any sole source issues.

FHWA indicated that galvanized reinforcement can be used for bridges off the National Highway System that are less than 500 ft in length. VDOT would use galvanized reinforcement or other alternative reinforcement (except ECR) in these bridges.

Lap length and the higher yield strength of MMFX2 were discussed, and it was agreed that VDOT would meet with representatives from MMFX to discuss these design issues. VDOT representatives were not concerned about lap length and the higher yield strength for most VDOT applications.

It was agreed that VDOT should document the type of reinforcement used in bridge elements in the as-built plans so the information would be available for future use.

Discussions indicated that July 1, 2006, was a reasonable goal for issuing plans and specifications requiring the use of MMFX2 reinforcement. The plan to phase out VDOT's use of

ECR as developed during the meeting would be forwarded to VDOT's State Structure & Bridge Engineer for approval.

Plan to Phase Out Use of ECR

The following plan to phase out the use of ECR was sent to VDOT's State Structure & Bridge Engineer on March 6, 2006.

- VDOT will pursue the use of MMFX2 reinforcement in structural elements in corrosive environments where ECR has been approved for use in the past in accordance with Section 8.22.6 of the November 2004 edition of the VDOT Modification, Highway Bridges of the AASHTO Design Standards.

- MMFX2 will be specified for the typical applications where ECR has been used in the past. There is no sole source issue because many fabricators have supplied MMFX2 reinforcement for both DOT and commercial contracts. However, contractors can also bid and use stainless steel clad and solid stainless steel with the approval of the bridge engineer as these alternatives are approved for use by FHWA on FHWA-funded contracts.

- In addition to MMFX2 and stainless steel clad and solid stainless steel, galvanized reinforcement may be specified for use in "off National Highway System" bridges that are less than 500 ft in length.

- The type of reinforcement used in bridge elements should be documented in the as-built plans so the information will be available for future use.

- July 1, 2006, is a target date for issuing plans and specifications requiring the use of MMFX2 reinforcement and allowing the use of stainless steel clad and solid stainless steel.

Life Cycle Cost Analysis

Cost and service life information for uncoated carbon steel, ECR, and MMFX2 reinforcement was included with the plan. Cost data obtained from VDOT bid tabulations in 2004 and 2005 and from MMFX are shown in Table 4. It can be seen from the data in Table 4 that the differences in the cost between ECR and MMFX2 reinforcement is small, and with today's changing prices it is reasonable to assume that the cost of MMFX2 reinforcement is approximately the same as that of ECR. The cost to construct structures with MMFX2 reinforcement should not be significantly different from the cost to construct structures with ECR. Consequently, to compare the life cycle costs, only the time to cracking of decks reinforced with each material needs to be considered. The initial cost of stainless steel clad and solid stainless reinforcement is much greater than that of MMFX2, and consequently use of these reinforcements would likely be limited to special structures in which their higher cost could be justified.

Table 4. Reinforcement Bid Prices ($/lb installed)

Type Reinforcement	Black	ECR	MMFX2	MMFX2*
Average 2004 VDOT bid tabs	0.89	1.10	0.85	0.90
Average 2005 VDOT bid tabs	0.99	1.12	-	1.17
Average VDOT bid tabs	0.94	1.11	0.85	1.04

*U.S. and Canada project data from MMFX.

Figure 6 shows the time to cracking for typical concretes used by VDOT over the past 20 years when the concrete deck is reinforced with carbon steel, ECR, and MMFX2 reinforcement. The times to cracking for 12 percent of the population are 43, 47, and 263 years, respectively. The figure is based on research by Clemena and Virmani that indicated MMFX2 reinforcement is 4.5 times more corrosion resistant than straight carbon reinforcement and research by Weyers et al. that indicates the time to cracking for ECR is 4 to 5 years longer than for uncoated carbon reinforcement.[19,25,26] Based on the times to cracking, MMFX2 reinforcement is worth 5 times more than ECR.

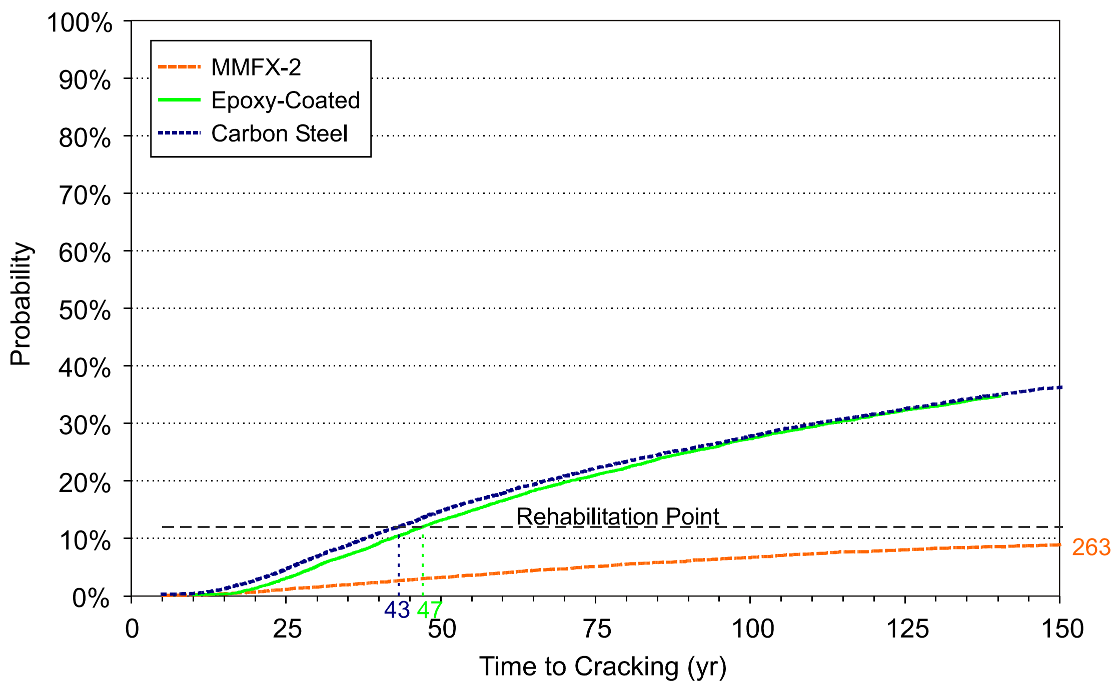

Figure 6. Service Life Comparison, Percentile of Population Corroded and Cracked

Final Considerations

Cost-effectiveness evaluations of current and proposed solutions generally include initial construction costs and future maintenance, repair, and rehabilitation costs. However, such evaluations should also consider the influence of the prevention method in question on future maintenance, repair, and rehabilitation options. Is there a greater risk involved in adopting a corrosion prevention strategy for which the benefit is not clearly quantified and the costs and difficulties involved in mitigating it, should it not perform, are less well understood?

Findings regarding projected time-to-corrosion for decks containing bare steel suggest that the severity of corrosion-related deterioration issue has been overestimated, at least for Virginia bridge decks. For example, a recent analysis of a random sample of 129 bridge decks built 30 years ago in Virginia showed that only 13 percent had received a rehabilitation concrete overlay and 25 percent had received a polymer concrete maintenance overlay.[18] These decks were built under the VDOT specification in place from 1968 to 1972 of a clear concrete cover depth of 43 mm (1.69 in) and a maximum w/c of 0.47.

With the introduction of ECR in Virginia in the late 1970s, VDOT cover depth and maximum w/c specifications were upgraded to a clear cover depth of 63 to 73 mm (2.5 to 3.0 in) and a maximum w/c of 0.45. In addition, in the 1980s, Virginia began using high-performance, low-permeability concretes primarily to prevent alkali-silica reaction. Thus, the threat presented by reinforcement corrosion in Virginia bridge decks is limited to less than 25 percent of the decks that are projected to initiate corrosion over an area sufficient to require rehabilitation within 100 years after construction.[19]

At present, federal guidelines require the use of a corrosion prevention method in bridge deck construction. Currently, ECR is the most widely used method. Best estimates have shown that ECR is not cost-effective in Virginia bridge decks. A more prudent approach to application of limited highway construction and maintenance funds would be to differentiate between regions or structures based on severity of environment and traffic exposure. Structures in less severe exposures are sure to benefit from improvements in concrete quality and increased cover depth, whereas high risk areas might be more effectively addressed on a case-by-case basis by implementing a more reliable method, such as stainless steel reinforcement. As previously stated, the more severe chloride exposure areas in Virginia are the Northern Virginia, Staunton, and Culpeper districts and the coastal line substructures.

CONCLUSIONS

Historical Performance Review of ECR, ECR Performance in Solutions and Concrete, and Preliminary Field Investigations

Historical Performance Review

- ECR laboratory investigations were short-termed projects, less than 3 years, and demonstrated that ECR was a suitable corrosion protection method. However, one longer term study, 7 years, showed that significant corrosion occurred when chloride was added to the concrete at a concentration of 18.9 kg/m^3 (32 lb/yd^3).

- Corrosion products were observed mainly at areas where the coating lost adhesion (coating debondment).

- The main cause of coating debondment is believed to be galvanic corrosion and surface damage.

- The ECR corrosion process is controlled cathodically.

- Epoxy coating debondment may result from a continuously wet environment.

- For the first 15 years of use of ECR, the assumed primary failure mechanism of macro-cell corrosion is in error.

- For high-quality ECR, adhesion failure is followed by underfilm corrosion followed by significant macro-cell corrosion.

- For poor quality ECR, the mechanism is the same with macro-cell corrosion occurring at critical chloride concentration levels.

- Fusion-bonded ECR will not provide long-term, 50 years or more, corrosion protection in northern U.S. and Canadian deicing salt environments.

- For typical field quality ECR, a deck service life of 3 to 6 years beyond that of black bar can be expected.

- Two ECR corrosion protection theories were proposed based on laboratory testing: One was that the epoxy coating prevents chloride from coming in contact with steel surfaces; the other was that the epoxy coating acts as a high-resistance coating reducing macro-cell corrosion between neighboring coated steel locations. Regardless of which theory is applicable, the best corrosion protection performance will occur with coatings that adhere well, have adequate uniform thickness, and have a low number of defects.

ECR Performance in Solutions and Concrete

- The following corrosion mechanism applies to ECR in concrete: Concrete pore water solution penetrates the coating and debonds the epoxy coating at weak adhesion areas, and the pH of the solution under the coating at that point is about 12. Chlorides arrive and initiate the corrosion process under the coating; the pH decreases to 5 as corrosion proceeds. This corrosion process agrees with other observations, including field observations.[9]

Preliminary Field Investigations

- The total epoxy coating damage (mashed, dents, scraps, and holes) of in-place ECR appears to be low.

- The number of holidays of in-place ECR appears to be excessive.

- The epoxy coating adhesion loss is occurring in moist concrete and is the result of wet adhesion loss without the presence of chloride.

- ECR is corroding in Virginia bridge decks.

- The corrosion protection period for ECR beyond that of black bar was estimated at 5 years for the two delaminated sites.

Investigation of Field Performance of Bridge Decks Built with ECR

- Epoxy coating on reinforcing steel debonds from the steel surface in as little as 4 years in bridge decks and long before chlorides arrive at the depth of the reinforcement.

- Epoxy coating on reinforcing steel in bridge decks debonds from the steel surface in properly constructed bridge decks having good clean concrete cover, good quality concrete, and ECR that complied with VDOT prescription specifications.

Assessment of Alternative Corrosion Protection Methods

- Bridge decks built either with black steel and low-permeability concrete (sometimes called high performance concrete) or with black steel, low-permeability concrete and the calcium nitrite corrosion inhibitor would not have to be overlaid within a 75-year design life and would provide a more cost-effective solution than ordinary portland cement concrete or low-permeability concrete in combination with ECR.

Development of Probabilistic Service Life Models for Bridge Decks and Laboratory Assessment of ECR Cores Extracted from Bridge Decks

- Based on assessments of chloride concentrations in field specimens associated with induced corrosion and cracking of concrete cover, in conjunction with comparisons of corrosion rates, the estimated service life extension for ECR beyond that of black steel was projected to be about 4 to 5 years. The 5-year estimate is in agreement with the researchers' initial estimate.[14]

- The estimated 2.5 to 3.0 percent damage that was projected to result from typically observed degrees of deck cracking would be associated with the deterioration level at first repair. With about 12 percent of the worst span lane being considered as the rehabilitation damage level, deck cracking may influence the time-to-first repair but not the time to rehabilitation.

- Efforts to assess factors influencing ECR performance revealed that production line ECR coatings being placed in bridge decks in Virginia are not fully cured.[19,20] The researchers concluded that observed micro-cracking in the surface of ECR coatings is related to the epoxy coating moisture content after long-term exposure in concrete and measurable change in glass transition temperature under testing, which indicates incomplete curing.

Efforts to Implement Alternative Reinforcement

- The cost to construct structures with MMFX2 reinforcement should not be significantly different from the cost to construct structures with ECR. Consequently, to compare the life cycle costs, only the time to cracking of decks reinforced with each material needs to be considered. Based on the times to cracking, MMFX 2 reinforcement is worth 5 times more than ECR. VDOT will pursue the use of MMFX2 reinforcement in structural elements in corrosive environments where ECR has been approved for use in the past in accordance with Section 8.22.6 of the November 2004 edition of the VDOT Modification, Highway Bridges of the AASHTO Design Standards. [23]

RECOMMENDATIONS

Based on 14 continuous years of research of corrosion protection methods and in consideration of the lowest risk of implementation, four recommendations were presented to VDOT's Structure and Bridge Division for implementation:

1. For critical structures, such as those structures on interstates and U.S. routes in VDOT's Northern Virginia, Staunton, and Culpeper districts and other bridge locations with a high rate of deicer salt usage, use stainless steel (316 LN) reinforcing bar as a bridge deck corrosion protection system.

2. Use stainless steel (316 LN) reinforcing steel also in coastal substructures subjected to a marine environment.

3. Use low-permeability concrete (<1500 coulomb per ASTM C1207-97) and black bar for all other non-critical decks, for beams, and for substructure elements not exposed to marine environments.

4. Develop repair procedures and remediation strategies for structures built with ECR.

As a result of recent efforts to implement alternative reinforcement that is eligible for funding by FHWA, the four recommendations were refined to the following:

1. VDOT should pursue the use of MMFX2 reinforcement in structural elements in corrosive environments where ECR has been approved for use in the past in accordance with Section 8.22.6 of the November 2004 edition of the VDOT Modification, Highway Bridges of the AASHTO Design Standards.

2. MMFX2 should be specified for the typical applications where ECR has been used in the past. However, contractors can also bid and use stainless steel clad and solid stainless steel with the approval of the bridge engineer as these alternatives are approved for use by FHWA on FHWA-funded contracts.

3. In addition to MMFX2, stainless steel clad, and solid stainless steel; VDOT may specify galvanized reinforcement for use in bridges off the National Highway System that are less than 500 ft in length.

4. VDOT should document the type if reinforcement used in bridge elements in the as built plans so the information will be available for future use.

5. By July 1, 2006, VDOT should begin issuing plans and specifications requiring the use of MMFX2 reinforcement and allowing the use of stainless steel clad and solid stainless steel.

COSTS AND BENEFITS ASSESSMENT

The 1999 life-cycle cost analysis was upgraded to 2002 considering an annual inflation rate of 2.13 percent. Evaluations included low-permeability concrete with black, ECR, and stainless steel (SS) bar based on the cumulative percent deterioration of bridges in Virginia over time. Total expenditures were summed over a 75-year service life. Costs included initial costs for materials (low-permeability concrete and black, ECR, or SS bar), installation costs, and later age costs for overlays including traffic control costs. Table 5 summarizes the 75-year service life total expenditures for the various systems.

Table 5. Life Cycle Costs for Reinforcement Alternatives

Virginia Bridge Deck Percentile Corrosion	Total Expenditures Over 75-Year Life of Decks $\$/m^2$ ($\$/ft^2$)			
	Black	ECR	MMFX2	SS
5	902 (83.90)	995 (92.50)	144 (13.30)	237 (22.00)
10	902 (83.90)	995 (92.50)	144 (13.30)	237 (22.00)
15	506 (47.20)	557 (51.70)	144 (13.30)	237 (22.00)
20	506 (47.20)	557 (51.70)	144 (13.30)	237 (22.00)

As shown, in no case is ECR the most cost-effective solution for corrosion protection of bridge decks in Virginia. For decks that will require an overlay within the 75-year service life, MMFX2 is the most cost-effective solution. The cost of one overlay with traffic control, $\$129/m^2$ ($\$12/ft^2$), is greater than even the SS cost of $\$100/m^2$ ($\$9.30/ft^2$). For decks that will not require an overlay within the 75-year service life, black bar is the most cost-effective solution.[19] Since FHWA will not fund decks with black bar, MMFX2 is clearly the most cost-effective reinforcement for the majority of VDOT decks. The higher cost of solid stainless can be justified for some special structures requiring a longer lasting bar.

Another way to look at the relative life cycle costs is to consider the cost of future rehabilitations when designing a deck for a life of 100 years or more. MMFX reinforcement costs about the same as ECR but can extend the life of a deck approximately 5 times longer. Consequently, most decks constructed with MMFX reinforcement would not require a protective overlay for more than 200 years. Decks constructed with ECR would need an overlay in approximately 40 years. The life cycle cost of a deck constructed with MMFX reinforcement is many times less than that of a deck constructed with ECR. The recommended change in the specification will reduce future annual expenditures on bridge rehabilitation by several million dollars.

REFERENCES

1. Clifton, J.R., Beeghly, H.F., and Mathey, R.G . 1974. *Nonmetallic Coatings for Concrete Reinforcing Bars.* FHWA/RD/74-18. Federal Highway Administration, Office of Research and Development, Washington, D.C.

2. Clifton, J.R., Beeghly, H.F., and Mathey, R.G . 1974. *Nonmetallic Coatings for Concrete Reinforcing Bars.* NBS Building Science Series 65. National Bureau of Standards, Center for Building Technology, Institute for Applied Technology, Gaithersburg, Md.

3. Kilareshi, W.P. 1977. *Epoxy Coatings for Corrosion Protection of Reinforcing Steel.* STP 629: Chloride Corrosion of Steel in Concrete. American Society for Testing and Materials, West Conshohocken, Pa..

4. McKeel, W.T., Jr. 1977. *Evaluation of Epoxy-Coated Reinforcing Steel.* VTRC 77-R56. Virginia Transportation Research Council, Charlottesville.

5. Weyers, R.E. 1995. *Protocol for In-Service Evaluation of Bridges with Epoxy-Coated Reinforcing Steel.* NCHRP 10-37B. Transportation Research Board, Washington, D.C.

6. Baber, K., and Hawkins, N.M.. 1988. *Evaluation of Bridge Deck Protection Strategies.* National Cooperative Highway Research Program Report 297. Washington, D.C.

7. Virmani, Y.P., Clear, K.C., and Pasko, T.J.. 1983. *Time-to-Corrosion of Reinforcing Steel in Concrete Slabs, Vol. 5, Calcium Nitrite Admixtures or Epoxy-Coated Reinforcing Bars as Corrosion Protection Systems.* FHWA/RD83/012. Federal Highway Administration, Washington, D.C.

8. Kessler, R.J., and Powers, R.G. 1988. Corrosion *of Epoxy-Coated Rebar, Keys Segmental Bridges.* Florida Department of Transportation, Materials Office, Corrosion Research Laboratory, Gainesville.

9. Sagúes, A.A., Lee, J.D., Chang, X., Pickering, H., Nystrom, E., Carpenter, W., Kranc, S.C., Simmons, T., Boucher, B., and Hierholzer, S. 1994. *Corrosion of Epoxy-Coated Rebar in Florida Bridges.* University of South Florida, College of Engineering, Tampa.

10. Smith, L.L. 1993. *Corrosion of Epoxy-Coated Rebar in a Marine Environment.* Transportation Research Circular 403. Transportation Research Board, Washington, D.C.

11. Zemajtis, J., Weyers, R.E., Sprinkel, M.M., and McKeel, W.T., Jr. 1996. Epoxy-Coated Reinforcement: A Historical Performance Review. VTRC 97-1R1. Virginia Transportation Research Council, Charlottesville.

12. Pyć, W.A., Weyers, R., and Sprinkel, M.M. 1998. *Corrosion Protection Performance of Corrosion Inhibitors and Epoxy-Coated Reinforcing Steel in a Simulated Concrete Pore Water Solution.* VTRC 98-R42. Virginia Transportation Research Council, Charlottesville.

13. Zemajtis, J., Weyers, R.E., and Sprinkel, M.M. 1998. *An Evaluation of the Performance of Epoxy-Coated Reinforcing Steel in Concrete Exposure Specimens.* VTRC 99-CR2. Virginia Transportation Research Council, Charlottesville.

14. Weyers, R.E., Sprinkel, M.M., Pyc, W., Zemajtis, J., Liu, Y., and Mokarem, D. 1997. Field Investigation of the Corrosion Protection Performance of Bridge Decks and Piles Constructed with Epoxy-Coated Reinforcing Steel in Virginia. VTRC 98-R4. Virginia Transportation Research Council, Charlottesville.

15. Pyć, W.A., Weyers, R.E., Mokarem, D., Zemajtis, J., Sprinkel, M.M., and Dillard, J.G. 2000. *Field Performance of Epoxy-Coated Reinforcing Steel in Virginia Bridge Decks.* VTRC 00-R16. Virginia Transportation Research Council, Charlottesville.

16. Zemajtis, J., Weyers, R.E., Sprinkel, M.M. 1999. *Corrosion Protection Performance Evaluation of Low Permeable Concrete in Exposure Specimens.* VTRC 99-CR5. Virginia Transportation Research Council, Charlottesville.

17. Zemajtis, J., Weyers, R.E., and Sprinkel, M.M. 1999. *Performance Evaluation of Corrosion Inhibitors and Galvanized Steel in Concrete Exposure Specimens.* VTRC 99-CR4. Virginia Transportation Research Council, Charlottesville.

18. Kirkpatrick, T.J., Anderson-Cook, C., Brown, M.C., Sprinkel, M.M. *A Model to Predict the Impact of Specification Changes on the Chloride-Induced Service Life of Virginia Bridge Decks.* FHWA/VTRC 03-CR4. Virginia Transportation Research Council, Charlottesville.

19. Brown, M.C., Weyers, R.E., and Wheeler, M.C. 2003. *Corrosion Protection Service Life of Epoxy-Coated Reinforcing Steel in Virginia Bridge Decks.* FHWA/VTRC 04-CR7. Virginia Transportation Research Council, Charlottesville.

20. Wheeler, M.C. 2003. *Parameters Influencing the Corrosion Protection Service Life of Epoxy Coated Reinforcing Steel in Virginia Bridge Decks.* Virginia Polytechnic Institute & State University, Blacksburg.

21. Weyers, R.E., Prowell, B.D., Sprinkel, M.M., and Vorster, M. 1993. *Concrete Bridge Protection, Repair, and Rehabilitation Relative to Reinforcement Corrosions: A Methods Application Manual.* SHRP-S-360. Strategic Highway Research Program, National Research Council, Washington, D.C.

22. Taylor, J.N., Jr., and Weyers, R.E. 2002. *The Influence of Concrete Pore Water on the Epoxy Coating of Reinforcing Steel.* Report to the National Science Foundation Science and Technology Center for High Performance Polymeric Adhesives and Composites and to the Department of Chemistry, Virginia Polytechnic Institute & State University, Blacksburg.

23. American Association of State Highway and Transportation Officials. *Highway Bridges, Design Standards.* Washington D.C. Modified by the Virginia Department of Transportation, Richmond, November 5, 2004.

24. Torres-Acosta, A.A., and Sagues, A.A.. 2000. Concrete Cover Cracking with Localized Corrosion of Reinforcing Steel. In *Proceedings of the 5th CANMET/ACI International Conference on Durability of Concrete, Barcelona, Spain,* Vol. SP-192, V.M. Malhotra, ed. ACI International, Farmington Hills, Mich., pp. 591-612.

25. Clemena, G.G., and Virmani, Y.P. 2004. Comparing the Chloride Resistances of Reinforcing Bars. *Concrete International,* Vol. 26, No. 11, pp. 39-49.

26. Clemena, G.G. 2003. *Investigation of the Resistance of Several New Metallic Reinforcing Bars to Chloride-Induced Corrosion in Concrete.* VTRC 04-R7. Virginia Transportation Research Council, Charlottesville.