

Corrosion Resistance of Concrete Reinforcement

By

Elizabeth Ward-Waller

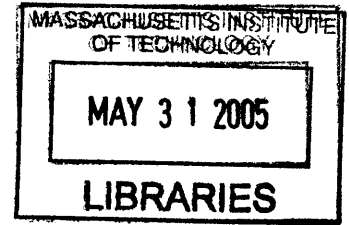
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Elizabeth Ward-Waller

Submitted to the Department of Civil and Environmental Engineering
on May 6, 2005 in Partial Fulfillment of the
Requirements for the Degree of Master of Engineering in
Civil and Environmental Engineering

ABSTRACT

The objective of this thesis is to investigate the mechanism of corrosion of steel reinforcement in concrete and epoxy coated reinforcing bars as corrosion resistant alternatives. Several case studies explore the durability and deterioration issues for epoxy-coated bars discovered through 30 years of implementation in reinforced concrete structures. The methods for predicting the end of functional service life for structures reinforced with uncoated reinforcing bars and with epoxy-coated reinforcing bars are detailed and tested in a design problem in the final section of this report.

Thesis Supervisor: Oral Buyukozturk

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INTRODUCTION

For thousands of years, humans have taken advantage of ductile materials with high tensile strength in the reinforcement of brittle materials with high compressive strength. The ductile reinforcement transfers tensile loads in the structure, allowing the brittle material to crack without causing failure of the structure. Throughout the last two centuries, concrete has been developed into a construction material with ever increasing potential to support compressive forces. As the compressive capacity of concrete has increased and with it demands to support longer and larger and taller structures, stronger, more ductile, and more tensile reinforcement has been required.

Steel has been used to reinforce concrete since nearly its advent as a modern construction material, and is manufactured in the form of bars, plates, wire, and mesh. Ductility, strength, and chemical bond to concrete are just a few of the advantages steel provides as a reinforcing material. Unfortunately, steel is subject to corrosion in wet and salty environments, and the resulting damage causes the steel to weaken and lose some of its valuable properties. Encasing the steel in concrete increases the length of time before initiation of corrosion by forcing the chlorides to diffuse through the concrete to the depth of the steel. In the last fifty years, even this technique has proved to be inadequate, and corrosion induced deterioration of reinforced concrete structures has become a major issue. A Federal Highway Agency (FHWA) report from 1999 stated that “the cost of repairing or replacing deteriorated structures estimated to be more than \$20 billion and to be increasing at \$500 million a year has become a major liability for highway agencies.”

One technique that has been developed and implemented during the last thirty years involves coating the steel reinforcement with an epoxy polymer before the reinforcement is cast in the concrete. The coating provides an additional barrier that is resistant to corrosive elements and significantly increases the service life of reinforced concrete structures. Unfortunately minor drawbacks to this technique have also been encountered in evaluation of structures reinforced with epoxy-coated rebar in extremely harsh exposure conditions.

This paper will begin by briefly outlining the history and development of traditional steel rebar, the mechanism for and effects of corrosion, and the method for

predicting the service life of a reinforced concrete structure subject to corrosive agents. The next section will describe the properties of and applications for epoxy-coated rebar, the issues encountered in its use, and initial exploration of potential repair and rehabilitation strategies for protecting structures reinforced with epoxy-coated rebar. Finally, a design problem will explore the service life extension provided by substituting epoxy-coated rebar in the place of standard black steel rebar in the reinforcement of a bridge pier in a high exposure environment.

CHAPTER 1: DEVELOPMENT AND CORROSION VULNERABILITY OF CARBON STEEL REBAR

Traditional Reinforcement: Steel Reinforcing Bars

Steel was developed in the 19th century as a stronger and more ductile alternative to iron. The advantages these improved properties would provide in concrete reinforcement were recognized and steel in the form of reinforcing bars (rebars) became an effective method of providing ductility and tensile strength to concrete. Modern steel rebar typically has a Young's modulus of 29×10^6 psi and behaves in an elastic-plastic manner.

Another advantage steel provides in reinforcing concrete is the ability to bond with the cement mortar matrix of the concrete. This bond strength is a significant property of reinforcing steel, allowing forces to be transferred through the steel and distributed evenly to the surrounding concrete material. Bond strength between steel and concrete is a product of the adhesion between the two material surfaces, the pressure or gripping effect provided by the concrete after drying shrinkage, physical interlocking of the concrete aggregate and bar deformations, and mechanical anchors within the concrete at the rebar ends. The quality of the concrete, its strength in tension and compression, and the diameter, shape, and spacing of the rebar determine the bond properties above. The bond stress that develops at the material interface when either material is subject to stress is expressed as local shearing stress per unit area of the bar surface, and the bond stress limit can be determined by pullout tests of the rebar imbedded in concrete. Much of the research and development on rebar throughout its history attempted to determine and increase the bond strength of reinforcing elements in concrete.

One early development that increased the bond strength between rebar and concrete came in the form of projections or "deformations" rolled onto the bars in order to increase the bond surface area. In addition to increasing the bond surface, the deformations provided a physical mechanism for interlocking the bars with the concrete aggregates. As early as the late 1800s, Thaddeus Hyatt tested deformed bars to show their increased bond performance. Ernest Ransome first patented a deformed bar, made by twisting a square section, in 1884, and in the 1940's the deformed bar was

standardized by ASTM with technical specifications for the height and spacing of bar deformations. Deformation properties are still being tested and improved on, and within the last decade experiments have shown that bond strength can be increased by enlarging the relative area of the deformations to the bar surface.

Several drawbacks of traditional steel rebar include loss of strength due to corrosion from moisture and chloride, and lack of resistance to severe heat and fire damage. These material weaknesses have led to American Concrete Institute (ACI) specifications that require rebar to be entirely encased in concrete, with a minimum concrete cover and spacing between bars. The concrete cover provides limited fireproofing and corrosion resistance to the steel. Due to aggregates in concrete as large as ¾" in diameter, the distance between parallel bars must be greater than 1" or the diameter of the bar. For longitudinal bars in columns, the spacing must be greater than 1.5" or 1.5 times the diameter of the bar. For cast-in-place beams and columns, the minimum concrete cover must exceed 1.5", and for bridge decks and elements with high exposure to corrosive agents the cover must exceed 2". If the reinforcing tendons are prestressed and will be exposed to chlorides, the minimum cover required should be increased by 50%.

Over time, moisture and chloride contamination infiltrate and degrade the concrete, and the concrete cover no longer provides adequate protection from corrosion to the reinforcement. In addition, poor construction practices often result in less concrete cover than the ACI code specifies, creating greater potential for concrete damage and infiltration of corrosive agents. Other methods are necessary to provide corrosion and damage resistance to the reinforcement in high exposure environments. Epoxy coating is a modern development that provides a physical barrier for reinforcing steel subject to significant moisture and chloride content.

Corrosion: Carbon Steel's Big Drawback

As discussed in the previous section, corrosion of steel rebar is a significant obstacle to its long-term use under conditions of exposure to moisture and chloride. Prior to the use of deicing salts on roadways in the 1950s, reinforced concrete (R/C) was

thought to be a maintenance-free material. Within a decade corrosion of reinforcement emerged as a major issue for R/C bridge decks and other infrastructure elements. Testing performed on rebar used in marine substructures has found corrosion damage due to chloride contamination within twenty years of construction. Corrosion resistance is particularly important for bridge piers and foundations, marine and coastal structures, roads and parking garages, and other structures that are constantly subject to severe environments, weather, and use.

Corrosion can be described simply as the process by which steel releases the energy imparted during the milling process and returns to its pre-milled iron ore state. This mechanism requires the presence of water, oxygen, and chloride ions, and is therefore highly dependent on the permeability, electrical resistivity, and temperature of the concrete. Decreasing the water/cement ratio of the concrete mix reduces the permeability and void ratio of the concrete, and therefore reduces the amount of water and oxygen that can be contained within the voids of the concrete. ACI code specifies a maximum water/cement ratio of 0.45 (and compressive strength 4500 psi) for R/C subject to deicing salts, and a max water/cement ratio of 0.40 (compressive strength 5000 psi) for R/C subject to marine environments. Reducing the moisture content of the concrete also reduces the electrical conductivity of the chloride ions in the concrete.

In order for corrosion to take place, chloride ions must first penetrate the concrete to the level of the steel. The period of time required for conduction of the ions is called the “initiation” or “incubation” stage of the corrosion process, and can be predicted using Fick’s 2nd Law of diffusion. Fick’s Law describes the chloride ion concentration, $C(x,t)$, at a specific concrete depth x (in.) and time t (years) from initial ion penetration of the concrete surface.

$$C(x,t) = C_o * \{1 - \text{erf}[x/(2*(D_{ac} t)^{1/2})]\} \quad (1)$$

Fick’s Law requires the concentration of chloride measured ½ inch below the concrete surface, C_o (typically ~14 lb/yd³), and the diffusion constant, D_{ac} (measured in in²/yr). The diffusion constant is a function of the permeability and condition (i.e. presence of cracking) of the concrete, environmental factors such as exposure and climate, care of

construction, and subjection to wearing and use. The value of the diffusion constant under high exposure conditions is typically ~ 0.05 in²/yr. Tables 1 and 2 list values for the surface concentration and diffusion constant specified by several states' departments of transportation.

Table 1: Accepted Surface Chloride Constants for Various States

	Low (lb/yd³)	Moderate (lb/yd³)	High (lb/yd³)	Severe (lb/yd³)
C_o	$0 < C_o < 4$	$4 \leq C_o < 8$	$8 \leq C_o < 10$	$10 \leq C_o < 15$
Mean	3.0	6	9.0	12.4
States	Kansas, California	Minnesota, Florida	Delaware, Iowa, West Virginia, Indiana	Wisconsin, New York

5. Center for Transportation Research and Education and the Civil, Construction, and Environmental Engineering Department at Iowa State University (sponsored by Iowa DOT and Iowa Highway Research Board). "Summary of the Impact of Concrete Cracking on Bridge Decks Constructed Epoxy-Coated Reinforcing Bars." CRSI, 2003.

Table 2: Mean Diffusion Constants for Various States (CTRE paper)

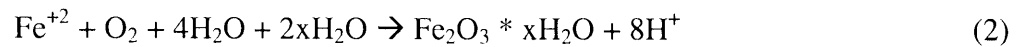
	Mean
California	0.25
Delaware	0.05
Florida	0.33
Indiana	0.09
Iowa	0.05
Kansas	0.12
Minnesota	0.05
New York	0.13
West Virginia	0.07
Wisconsin	0.11

5. Center for Transportation Research and Education and the Civil, Construction, and Environmental Engineering Department at Iowa State University (sponsored by Iowa DOT and Iowa Highway Research Board). "Summary of the Impact of Concrete Cracking on Bridge Decks Constructed Epoxy-Coated Reinforcing Bars." CRSI, 2003.

The corrosion rate predicted by Fick's Law can change continuously subject to environmental conditions, but contributes to estimating the length of time a concrete member can be exposed to chloride contamination before the reinforcement begins to corrode. Wet-dry cycles increase the rate of corrosion, causing the highest rates to generally be found during the spring. For black steel rebar a rate below $0.1 \mu\text{A}/\text{cm}^2$ qualifies as an essentially passive diffusion rate, a rate of $0.1 - 0.5 \mu\text{A}/\text{cm}^2$ qualifies as a

low to moderate rate, a rate of 0.5 – 1.0 $\mu\text{A}/\text{cm}^2$ qualifies as a moderate to high rate, and a rate greater than 1.0 $\mu\text{A}/\text{cm}^2$ qualifies as a very high rate.

Once the chloride ions have penetrated to the level of the steel stage two of the corrosion process, chemical corrosion, begins. In the presence of water and oxygen steel will naturally form an iron oxide (“rust”) film around the bar. Iron oxide is a product of the total chemical reaction below, which is actually a series of four smaller reactions.



The first mechanism of the reaction is the oxidation of the steel to form iron cations and the reduction or capture of the free electrons by oxygen. The oxygen reacts with water to form hydroxyl, and then reacts with the metal ions to produce a ferrous hydroxide. This reacts with water and oxygen again to further oxidize metal ions and form ferric hydroxide. Through dehydration ferric hydroxide becomes ferric oxide, also known as iron oxide or “rust”.

The chloride ions initiate damaging corrosion by passing through the iron oxide film surrounding the steel and reacting with the iron to form a soluble iron-chloride complex. This complex then diffuses away from the bar and reacts again with the iron to form the ferrous hydroxide, and finally frees the chloride ions to continue corrosion.

When two layers of reinforcement are present in a R/C element – such as the top and bottom mats of rebar used to provide tension and compression reinforcement in a typical bridge deck – the layers tend to acquire an electrical potential difference that serves to accelerate the corrosion of the steel. The top mat, which is first subject to chloride ion penetration and corrosion, shifts from a positive to negative potential as it corrodes. The bottom mat maintains its positive potential, and the potential difference induces an electric current through the concrete. As a result, ion transfer through the concrete is accelerated and the corrosion rate is increased.

Typically chloride-induced corrosion is negligible until a particular chloride content or “threshold” is reached at the level of the steel. The threshold level can be described by the critical chloride content (CCC) as a percent of cement weight or the chloride concentration threshold by concrete volume (lb/yd^3 or kg/m^3). The Federal

Highway Agency (FHWA) specifies CCC as 0.4 – 1% by cement weight for typical carbon steels. A typical corrosion threshold used by state DOTs is 1.2 lb/yd³ (300 ppm), which corresponds to a CCC of 0.2% of cement weight of concrete. In the presence of higher alkalinity of the concrete and the water in the voids, the iron oxide film protecting the steel from the initiation of chloride corrosion will provide more protection to the steel and increase the corrosion threshold. Figure 1 illustrates the diffusion of chloride ions through the concrete as concentration of chloride versus the depth from the surface of the concrete. The solid line at the bottom of the plot shows the chloride threshold for initiation of corrosion (~50 mm = 2” or typical rebar depth).

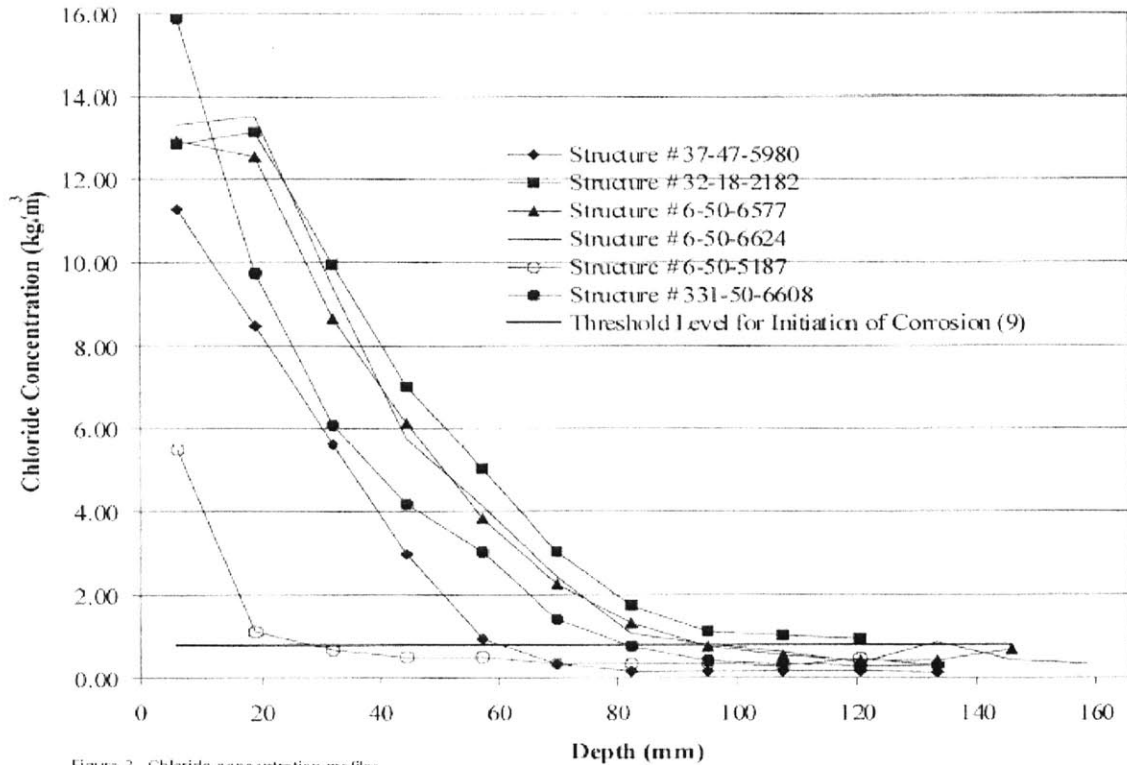


Figure 3. Chloride concentration profiles

Figure 1: Chloride Concentration v Concrete Depth

9. Samples, Lisa M. and Ramirez, Julio A (Purdue University School of Civil Engineering). "Field Investigations of Existing and New Construction Concrete Bridge Decks." Concrete Reinforcing Steel Institute (CRSI), 1999.

The products of corrosion can increase the volume of the bar by 3-6 times its original volume, applying significant pressure and stresses to the surrounding concrete and resulting in cracking, spalling (Figure 2), and delamination. This damage caused to the concrete provides new pathways for the corrosive agents to reach the steel.

Eventually the corrosion-induced damage in a concrete element becomes significant enough to require repair and maintenance. The point at which the concrete requires maintenance to remain in service is termed the end of functional service life of the element.



Figure 2: Concrete Spalling with Exposed Reinforcement

9. Samples, Lisa M. and Ramirez, Julio A (Purdue University School of Civil Engineering). "Field Investigations of Existing and New Construction Concrete Bridge Decks." Concrete Reinforcing Steel Institute (CRSI), 1999.

Several experimenters have developed models to determine the end of the functional service life for a reinforced concrete member subject to chloride contamination. One model in widespread use is Weyers' 1994 diffusion-spalling model that predicts the service life of a bridge deck reinforced with black steel rebar. This model uses Fick's 2nd Law to calculate the length of the initiation period. Approximately 4-5 years are then added for the corroded steel to build up corrosion product to a level that causes significant damage to the deck and requires maintenance.

Weyers concluded by a survey of bridge engineers that the end of functional service life for a R/C bridge deck is reached when 9.3-13.6% of the worst traffic lane (typically the right or outside lane) surface area or 5.8-10% of the whole deck surface

area is spalled, delaminated, and patched. The Iowa Department of Transportation uses the Weyers model and similar end-of-life criteria. Iowa DOT requires 9-14% of the worst traffic lane or 8-10% of the whole deck to be spalled or delaminated for the end of functional service life to be reached.

A number of methods have been developed to determine the condition of R/C elements in order to determine the time to the end of their functional service lives. A study conducted by the Iowa DOT listed the following as current methods used to determine the condition of R/C bridge decks:

- Visual inspection of the surface and undercarriage of the deck
- Delaminations survey – Conducted by sounding the deck with a steel hammer or rod, or dragging a chain across the deck. The vibrations reverberate within the concrete, and the resulting sound can be interpreted as the presence or lack of delamination. A sharp ringing sound means no delaminations are present; while a dull hollow sound means delaminations exist. (This method is not effective on bridges with an asphalt surface.)
- Depth of cover measurements – A nondestructive pachometer – also called a “covermeter” – can be used to measure variations in magnetic flux caused by the location of steel (the size and orientation of the steel bar must be known). Alternatively small diameter holes can be drilled to exposes rebar for direct measurement, or cores can be extracted for direct measurement. Obviously the nondestructive method is preferred because it results in the least damage and the most data.
- Determination of chloride content – A specific ion probe can be used in the laboratory or in the field. In order to use the probe, concrete powder at a specific depth must be drilled out and collected. 3 grams of the powder is placed in 20 ml of digestion solution with 80 ml of stabilizing solution. The specific ion probe is then inserted into the solution and takes a voltage measurement that it converts automatically to % chloride by weight of concrete (CCC). Alternatively an X-ray fluorescence spectrometer can be used to determine CCC in samples.

- Electrical continuity tests and corrosion potential mapping – Using a half-cell, electrical connections are made to the rebar and voltage readings are taken across the rebar. The voltages can be interpreted using the Numeric Magnitude Technique (ASTM C876) to the corrosion state of the rebar (the existence of corrosion is measured as a negative shift in the half-cell potential). If enough readings are taken over the whole deck surface the condition of the deck can be “mapped” as shown in Figure 3. (These methods are performed only on uncoated steel rebar.)
- Corrosion rate measurements – A corrosion rate device induces small currents or voltages into the rebar and measures the response. The electrical response measured can be converted mathematically into corrosion rates.
- Determination of rebar cross-section loss – Direct measurement can be taken of the effective cross-section of rebar from a cored concrete sample.
- Petrographic analysis – Examining drilled cores directly and with a microscope reveals details of the concrete condition.
- Rebound # test – A standardized spring driven hammer can be dropped on the surface of the concrete to measure the distance it rebounds. The rebound distance indicates the hardness of the concrete, which correlates to its compressive strength.
- Penetration test – A special gun drives a small rod into the concrete and the distance of penetration of the rod also indicates the compressive strength of the concrete.

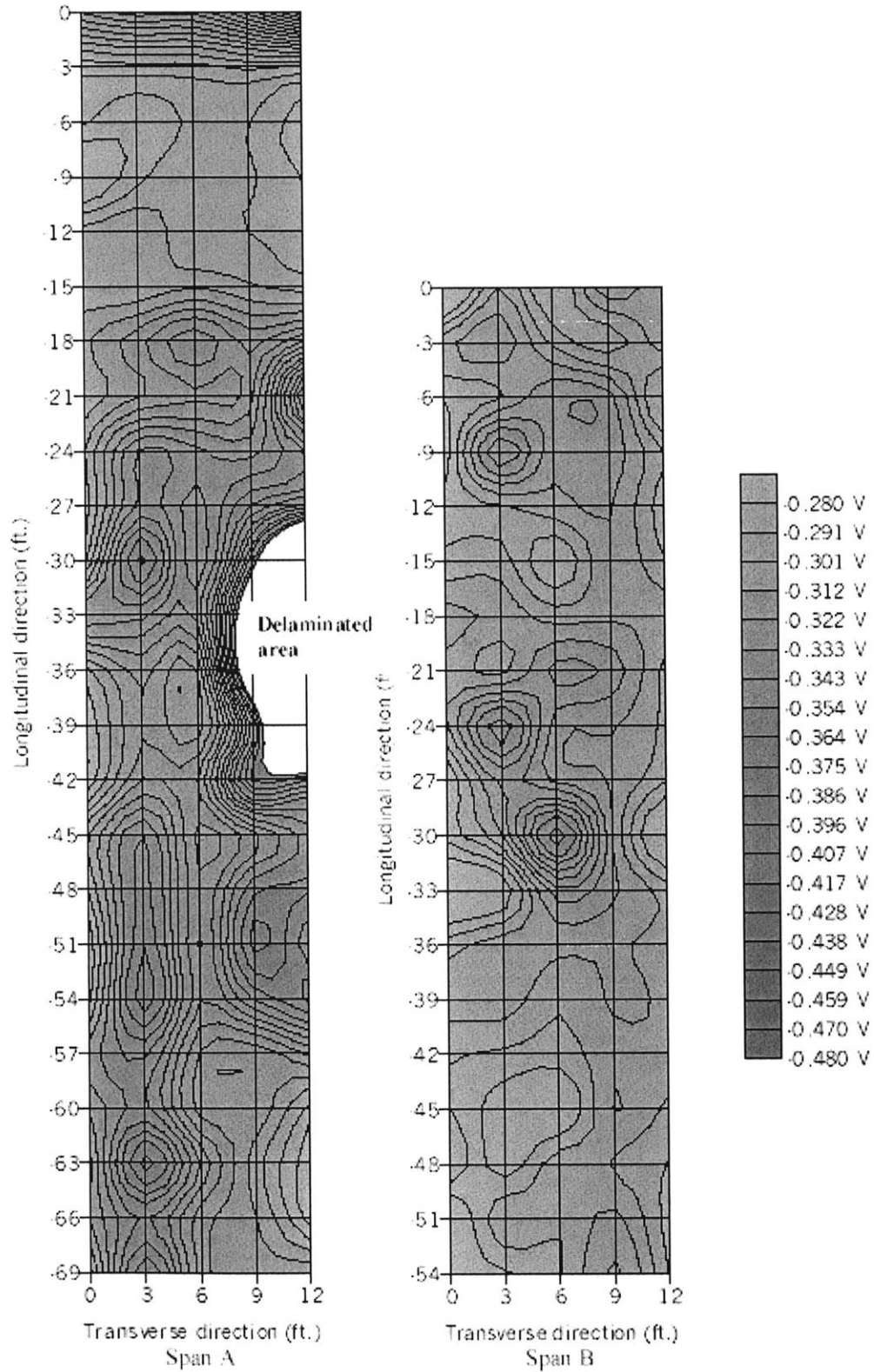


Figure 3: Electrical contour mapping

16. Wiss, Janney, Elstner Associates, Inc. "Service Life Extension of Northern Bridge Decks Containing Epoxy-Coated Reinforcing Bars," (WJE Report No. 2003.0510.0). Concrete Steel Reinforcing Institute (CRSI), 2003.

To establish a comprehensive survey of the deck condition several of the methods listed above must be used in conjunction. Once the deck condition is established, the overall percent of deck surface damage can be calculated and possible repair and rehabilitation strategies can be considered. These strategies may include patching and coating small deck areas or in severe cases overlay of the entire deck surface. FHWA requires inspection of R/C bridges every two years to maintain their safety and serviceability.

CHAPTER 2: EPOXY-COATED STEEL REBAR

Properties and Applications

Recent developments including the use of epoxy coating as a protective outer layer for steel rebar have improved the corrosion resistance of reinforcement. This new product has provided considerable benefits over standard reinforcement, and was thought to completely eliminate the problem of corrosion susceptibility of steel rebar. Unfortunately, some minor issues have arisen since the implementation of epoxy-coated rebar (ECR), such as corrosion on rebars in areas of cracked concrete, and some coating defects due to damage in construction. ECR have still proved to be a cost effective option to black steel rebar by reducing the maintenance required over a long service life.

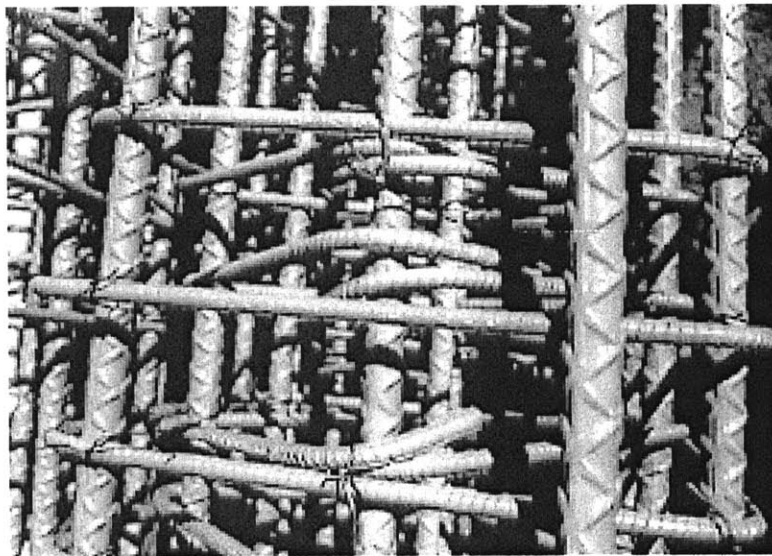


Figure 4: Epoxy Coated Rebar

12. Smith, Jeffrey L. and Virmani, Yash Paul, (Federal Highway Administration). "Performance of Epoxy-Coated Rebars in Bridge Decks." Concrete Reinforcing Steel Institute (CRSI), 1999.

Designed to provide a physical barrier between chlorides and oxygen absorbed in the concrete and the reinforcing steel, epoxy coating consists of organic epoxy resins combined with curing agents. Epoxy coating is a thermoset material; meaning that it is not subject to damage by high temperatures once it is cured. The coating starts out as a dry powder, and is heat-treated to melt the powder and catalyze the chemical reaction that

allows epoxy coating to adhere to the steel. EC is then applied by either spraying it directly on to steel or dipping the steel into epoxy baths.

The surface of the steel must be cleaned and roughened with abrasive material in order to provide an uneven surface for the epoxy to bond to mechanically as well as chemically. The steel is then heated and passed through a sprayer which charges the epoxy powder and causes it to evenly coat the surface of the steel. The heated steel melts the powder on contact, initiating the chemical reaction that forms complex polymers in the epoxy and bonds the epoxy molecules to each other and the rough steel surface.

In addition to providing a barrier for corrosive agents, epoxy coating also has a high electrical resistance, and prevents the flow of electrons that contribute to electrochemical corrosion. EC also possesses the necessary mechanical properties for use in coating steel, i.e. ductility, negligible shrinkage after application, and good heat resistance. Epoxy coating is also durable to rough handling and weathering, and is flexible enough to expand and contract with the steel. Epoxy coating is environmentally friendly thanks to a lack of hazardous materials and solvents and very efficient methods of manufacture and application to the steel.

Epoxy-coated reinforcement is generally required in roadways and bridge decks where deicing salts cause significant chloride contamination to the concrete. The ACI code lists ASTM standards A775 and A934 as the accepted specifications for epoxy-coated steel reinforcing bars.

ECR was first used to reinforce a four-span bridge deck over Pennsylvania's Schuylkill River in 1973. By 1975 ten states had implemented ECR in bridge deck construction, and within a decade nearly all states with freeze-thaw cycle climates or salty coasts had adopted the material. The Canadian Ministry of Transportation of Ontario standardized the use of ECR in bridge decks in 1978, and for bridge substructure components in 1981. Today EC is the most widely used rebar corrosion protection method for bridge deck reinforcement.

The use of ECR quickly expanded from bridge deck reinforcement to parking garages and marine structure reinforcement. In the 1980s chloride damage in parking garages became a significant problem due to deicing salts carried in on cars and ECR was implemented for new garage construction. The ECR garages are still performing well

today, though more time is required to discern the actual service life of ECR in this context. Unfortunately, corrosion of ECR in marine structures was discovered by the Florida Department of Transportation after only 5-10 years of exposure. Further research is required on the durability of ECR in this application, but ECR has clearly not proved to be a corrosion- and maintenance-free material.

Corrosion of ECR

Weyers model for service life prediction can also be used to predict the end of functional service for ECR, using a higher corrosion threshold and an alternate system for rating the corrosion level of the reinforcement. The corrosion level is determined by visual inspection and rated on a 5-point scale: 5 – no evidence of corrosion, 4 – a number of small, countable corrosion, 3 – corrosion area less than 20% of total ECR surface area, 2 – corrosion area between 20 to 60% of total ECR surface area, and 1 – corrosion area greater than 60% of total ECR surface area (Singh, ed.). The corrosion rating $r(t)$ can be predicted after a certain length of time t (years) depending on the presence of cracking in the concrete by the following polynomials.

Cracked concrete locations:

$$r(t) = 5.00 + .0038t - .0031t^2 \quad (3)$$

Uncracked locations:

$$r(t) = 5.00 + .0135t - .00134t^2 \quad (4)$$

A corrosion rating of 3 indicates noticeable corrosion (Figure 5), and ECR with ratings of 3 were extracted from concrete with chloride concentrations around 7.5 lb/yd³. This signifies an approximate corrosion threshold for ECR around 7.5 lb/yd³ and a conservative estimate approximately half of this value or 3.6 lb/yd³. The end of functional service life for R/C elements reinforced with ECR is associated with a corrosion rating of 1, plus 5-8 years for the corrosion product to build up and create spalls and delaminations in the concrete. Using this method of prediction for the end of

functional service life, ECR has been estimated to have about three times the service life of uncoated black steel. In the presence of cracks, ECR will still have about twice the service life of uncoated black steel.

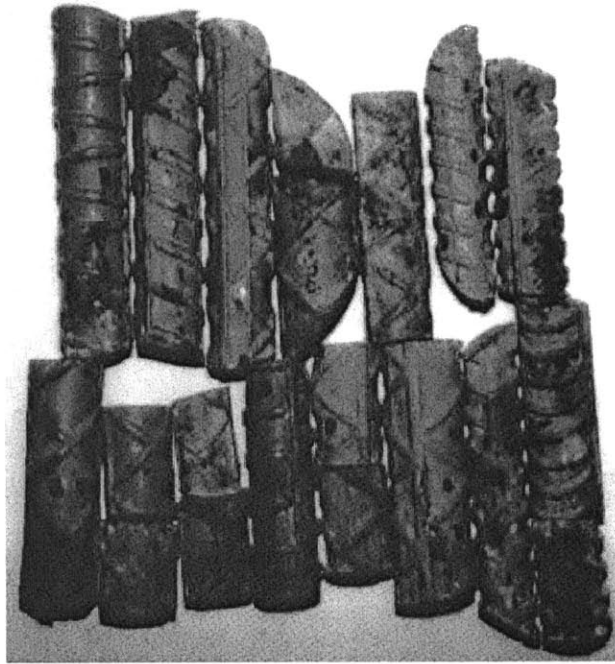


Figure 5: ECR Showing Significant Corrosion

6. Fanous, Fouad. "Impact of Deck Cracking on Durability," (Iowa DOT Project TR-405, CTRE Management Project 97-5). Center for Transportation Research and Education and the Civil and Construction Engineering Department at Iowa State University (sponsored by Iowa DOT and Iowa Highway Research Board), 2000.

As previously discussed, when reinforcement occurs in two layers or mats within a concrete element, the mats tend to gain a potential difference and accelerate the rate of corrosion of the steel. ECR was originally used only in the top mat close to the exposed surface of the concrete with uncoated steel remaining in the lower mat in order to reduce the amount of ECR required in the element. The corrosion threshold of the ECR in this arrangement is between 3.6 – 7.5 lb/yd³ (average 4.7 lb/yd³) as previously described. Further exploration revealed that if damage points exist in the EC, the current can still be induced between the mats and the benefit of the coating is reduced. Substituting ECR in both mats resisted the current induced by the potential difference and significantly increased the corrosion threshold to an average of 15 lb/yd³.

Additional Drawbacks of ECR

A disadvantage of substituting epoxy-coated reinforcement in the place of uncoated steel is that epoxy has poor chemical adhesion to the cement mortar matrix, resulting in lower bond strength between the rebar and concrete. Epoxy coating also reduces the size of the rebar deformation ribs and provides less friction to resist bond slip. The reduction in bond strength is a product of the rebar size, with larger bar diameters resulting in lower relative bond strength. Bond slip creates cracking in the concrete, and epoxy coated bars were found to create fewer but wider cracks in concrete as a result of bond slip. After repeated loading, the bond slip due to epoxy did not appear to significantly increase the deflection of reinforced concrete members. The suggested modification factor for epoxy coated bars to account for bond slip is 1.35.

Reduced bond strength between ECR and concrete affects the strength and development length of laps and splices, as well as the capacity of plastic hinges in reinforced concrete. A 20 – 50% longer anchorage length is required by the ACI code to fully develop the bond strength of ECR due to the reduced bond strength at the epoxy/concrete interface.

In addition to the reduced bond strength between the epoxy surface and the concrete, the bond between the epoxy coating and the steel rebar also tends to deteriorate over time and exposure to moisture. The EC adhesion to steel deteriorates at an even higher rate in the presence of cracks in the concrete. One of the most common types of EC, 3M's Scotchkote 213, was found to be inadequate in marine structures due to debonding of the coating, and was eventually removed from the market by an OSHA regulation of its curing agent. Reports of coating disbondment vary from coating that can be removed easily with a fingernail to coating that can only be removed by scraping with a knife.

Small defects or "holidays" in the EC can be created during fabrication or placing of the rebar and must be patched with additional epoxy. Post-fabrication bending of the rebar for hooks and splices can crack, delaminate, or reduce the thickness of the coating in a particular area, creating additional defects that will render the rebar susceptible to corrosion. The effectiveness of the coating as a corrosion resistant barrier is highly

dependent on a lack of defects over the surface of the rebar. A coating thickness must be specified to reduce the chances for damage and defects during fabrication and placement of the ECR.

Holidays are most often the result of the concrete casting process, when wet concrete is pumped from a few feet above the ECR mats, and the aggregates in the concrete fall onto the rebar and damage the coating. One study determined that on average 40 holidays were created per meter of bar during the casting process, but that the number of holidays could easily be reduced by lowering the head of the concrete pump and increasing the thickness of the coating (9. Samples).

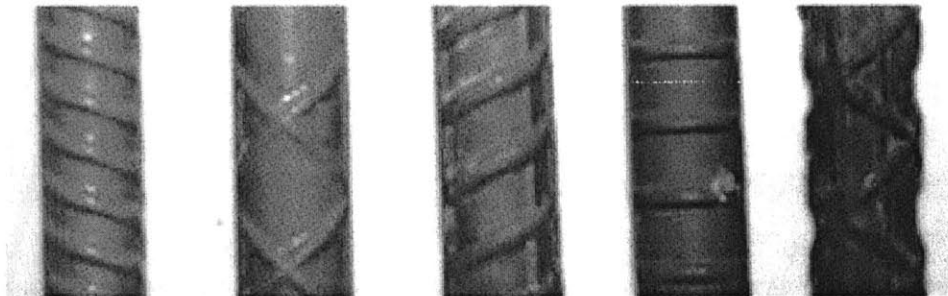


Figure 6: ECR Showing Discoloration

6. Fanous, Fouad. "Impact of Deck Cracking on Durability," (Iowa DOT Project TR-405, CTRE Management Project 97-5). Center for Transportation Research and Education and the Civil and Construction Engineering Department at Iowa State University (sponsored by Iowa DOT and Iowa Highway Research Board), 2000.

EC also becomes discolored (Figure 6) after prolonged exposure to chloride contamination. A closer look at a severely discolored ECR with a scanning electron microscope (Figure 7) revealed a network of micro-cracks that had developed on the coating surface. At this time little is understood about the impact of these cracks, and for now they are assumed to be superficial and negligible.

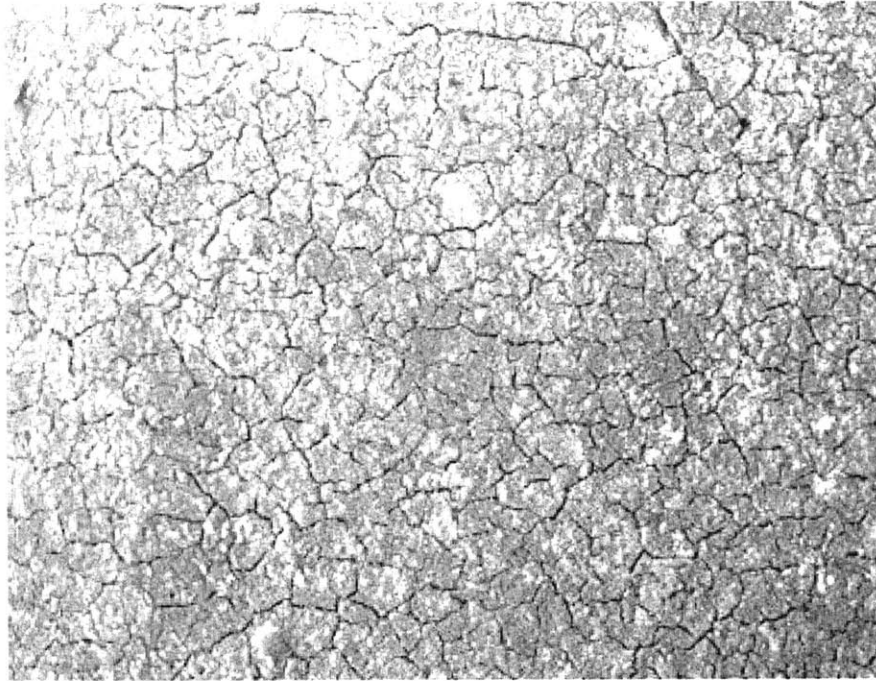


Figure 7: Micro-cracks in Discolored Epoxy Coating

6. Fanous, Fouad. "Impact of Deck Cracking on Durability," (Iowa DOT Project TR-405, CTRE Management Project 97-5). Center for Transportation Research and Education and the Civil and Construction Engineering Department at Iowa State University (sponsored by Iowa DOT and Iowa Highway Research Board), 2000.

These limitations and drawbacks of ECR have not proved considerable enough to discourage the widespread use of ECR in construction of R/C elements in high exposure areas. These structures have been in service for thirty years or less, so unanticipated and more significant durability issues may arise after more time has passed. In the last decade numerous case studies have been completed to evaluate the condition and continued serviceability of ECR-reinforced structures. The case studies described in the following sections were prepared for the Concrete Reinforcement Steel Institute (CRSI) and several state DOTs.

ECR Case Study 1: Minneapolis Parking Garages

The City of Minneapolis specifies a high water/cement ratio (.38 - .45) and corrosion inhibitors in the concrete mix to be used in conjunction with ECR for parking garage construction. Parking garages built around 1980 with black uncoated rebar

exhibited significant corrosion (Figure 8) and required extensive maintenance within a few years of construction. Minneapolis' Government Center parking garage required \$3 million to repair and replace more than 30% of the concrete after only 12 years of service. The repair cost eventually exceeded the cost of construction. Part of the problem at Government Center was that 1.5" of concrete cover for the rebar had been specified and poor construction resulted in an average of only 1" of cover. Quality of construction is often an unreliable variable and provisions should always allow a factor of safety to account for possible shoddy work.



Figure 8: Extensive Black Steel Rebar Corrosion on Parking Garage Ceiling

2. Brown, Dan. "Epoxy-Coated Reinforcing Steel in Parking Garages." Concrete Reinforcing Steel Institute (CRSI), 1997.

ECR Case Study 2: Navy Marine Structures

The US Navy owns thousands of marine structures in which the major mechanism of deterioration is corrosion of the reinforcement (Figure 9). In addition to ACI standards, the Navy requires a water/cement ratio of 0.4 and 3" of concrete cover for reinforcement used in R/C structures with "splash zone exposure." These marine structures are often subject to more severe conditions than the typical bridge deck would due to tidal wetting with salt water and drying and higher temperatures in subtropical

locations that speed the rate of corrosion. As in the Minneapolis garages, inspection of the Navy structures has shown that the specified level of quality was not always maintained, and as little as ½” of concrete cover was discovered in several cases. Chloride content where only ½” of cover existed was measured at levels reaching 10 kg/m³, with only 1 kg/m³ required to initiate corrosion in steel.



Figure 9: Navy Marine Structure with Extensive Corrosion Damage

3. Burke, Douglas F. (Naval Facilities Engineering Service Center). "Performance of Epoxy-Coated Rebar, Galvanized Rebar, and Plain Rebar with Calcium Nitrate in a Marine Environment." Concrete Reinforcing Steel Institute (CRSI), 1994.

ECR Case Study 3: Iowa Bridge Decks

A study conducted at Iowa State University for the Iowa DOT attempted to describe the relationship between deterioration of ECR in cracked and uncracked concrete, as well as the improvement provided by placing ECR in both the top and bottom mats of bridge decks as opposed to strictly the top mat.



Figure 10: Dry Knife Adhesion Test

6. Fanous, Fouad. "Impact of Deck Cracking on Durability," (Iowa DOT Project TR-405, CTRE Management Project 97-5). Center for Transportation Research and Education and the Civil and Construction Engineering Department at Iowa State University (sponsored by Iowa DOT and Iowa Highway Research Board), 2000.

The study made use of two experiments in addition to visual inspection to determine the condition of the epoxy coating in cracked and uncracked locations: the pencil hardness test and the dry knife adhesion test. The pencil hardness test is used to determine the hardness of the EC, which tends to increase with age. The dry knife adhesion test requires a particular knife blade to cut a V-shaped groove in the EC to the steel (Figure 10), then a microscope to closely inspect the adhesion between the EC and the steel. The adhesion is rated on a 3-point scale developed by the Pennsylvania DOT and detailed in Table 3 below. As described previously and reinforced in this study, the adhesion of EC to steel tends to decrease with age and exposure. The results of these two tests for the Iowa bridge decks are plotted in Figure 11 below.

Table 3: Epoxy Coating Adhesion Rating

Rating	Description
3	Well adhered coating that cannot be peeled or lifted from the substrate steel
2	Coating that can be pried from the substrate steel in small pieces but cannot be peeled off easily
1	Coating that can be peeled from the substrate steel easily, without residue

6. Fanous, Fouad. "Impact of Deck Cracking on Durability," (Iowa DOT Project TR-405, CTRE Management Project 97-5). Center for Transportation Research and Education and the Civil and Construction Engineering Department at Iowa State University (sponsored by Iowa DOT and Iowa Highway Research Board), 2000.

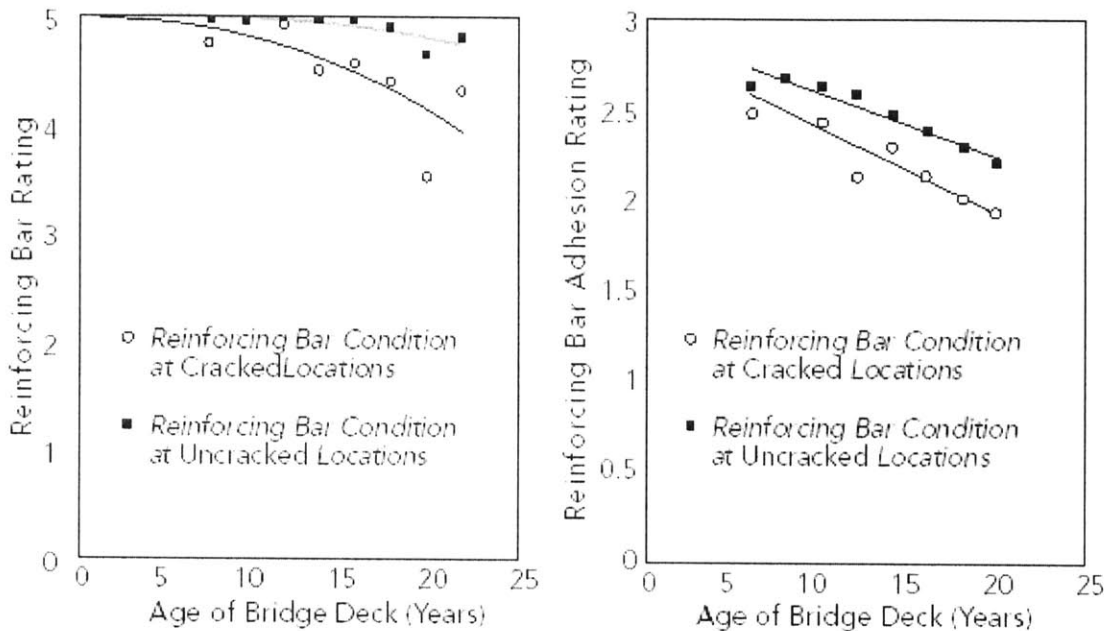


Figure 11: ECR Corrosion Rating and Adhesion Rating in Cracked v Uncracked Locations Varying with Age

6. Fanous, Fouad. "Impact of Deck Cracking on Durability," (Iowa DOT Project TR-405, CTRE Management Project 97-5). Center for Transportation Research and Education and the Civil and Construction Engineering Department at Iowa State University (sponsored by Iowa DOT and Iowa Highway Research Board), 2000.

As shown in the plots, the corrosion rating of ECR tends to be lower in cracked locations, and decrease with age. The EC also tends to become more brittle with age and increased exposure to chlorides. The adhesion rating is also lower in cracked locations, and decreases with age. Cracks in the concrete of Iowa bridge decks clearly resulted in weaker EC adhesion, lower corrosion rating, and higher rates of corrosion. In uncracked locations even with very high chloride concentrations (7.6 kg/m^3) in the concrete, the ECR exhibited little or no corrosion.

In 1976 Iowa bridges began using ECR in the top mat only, but in the 1980s began implementing ECR in both mats. Placing ECR in both mats increased the electrical resistance between the mats by 4-7 times. Essentially no corrosion-induced damage was observed in Iowa bridge decks that had been constructed with two mats of ECR. The lack of damage can be partially attributed to the younger age of the two-mat decks, which were constructed after more stringent DOT standards had been specified. Two-mat bridge decks were found to have a larger number of concrete cracks than single-mat decks, but no serviceability issues were created or maintenance required upon this discovery. The predicted extension in service life provided by the ECR for the bridge decks is 40 years when placed in the top mat only and 82 years when placed in both mats.

Repair and Rehab Strategies for Epoxy-Coated Rebar Structures

Modern studies have identified some of the potential issues involved in the long-term use of ECR in concrete structures. While these structures may not yet require rehabilitation, eventually maintenance will have to be performed to lengthen their service lives. No current guidelines exist for the repair and rehabilitation of structures with ECR, the guidelines instead have been focused on the repair and rehab of structures with black steel rebar that already require significant maintenance. A study prepared for the National Cooperative Highway Research Program investigated potential strategies and their effectiveness for repair and rehab specifically of R/C structures reinforced with ECR.

This study proposed the following repair and rehab methods for cracking not induced by rebar corrosion, for cracking induced by corrosion, and for delaminations resulting from corrosion. For cracking unrelated to corrosion repair options include epoxy injection into the cracks, flooding the cracks with corrosion inhibitor, or a combination of both methods. For corrosion induced cracking repair options include epoxy injection of cracks and flooding cracks with corrosion inhibitor, in addition to applying corrosion inhibitor directly to the surface of the concrete. In the more severe case of corrosion induced delaminations repair strategies include recoating the exposed steel with epoxy, patching the delaminated area with new concrete, mixing corrosion

inhibitors in the concrete patch material, applying corrosion inhibitors to the surface of the patched area, and a combination of all four of these options. A final rehab strategy tested was the use of cathodic protection on the rebar in the area of any type of cracking or delamination. The patching and protection materials used in this study are listed in Table 4.

Table 4: Materials Used for Repair and Rehab of ECR Structures

Material	Description
Epoxy Injection	bisphenol A and polyamine curing agent
Patching Material A	pre-bagged Portland cement concrete
Patching Material B	pre-bagged polymer modified silica fume concrete
Patching Material C	Class III Portland cement concrete, Florida DOT standard specifications
Coating Material A	epoxy coating
Coating Material B	water based epoxy resin/Portland cement coating
Coating Material C	water based alkaline coating with corrosion inhibitor
Corrosion Inhibitor A	water based amine and an oxygenated hydrocarbon for surface application
Corrosion Inhibitor B	calcium nitrite based inhibitor for surface application
Corrosion Inhibitor C	water based amine and an oxygenated hydrocarbon for use as admixture
Corrosion Inhibitor D	calcium nitrite based inhibitor for use as admixture
Corrosion Inhibitor E	multi-component admixture with corrosion inhibitor and concrete densifier

13. Sohahngpurwala, Ali Akbar; Scannell, William T.; Hartt, William H. "Repair and Rehabilitation of Bridge Components Containing Epoxy-Coated Reinforcement," (NCHRP Web Document 50, Project D10-37C). National Cooperative Highway Research Board, 2002.

The specimens used to test the repair strategies were subject to 31 months of two-week wet/dry cycles with a concentrated salt solution. The types of specimens used in the study are detailed in Table 5.

Table 5: Types of Specimens Used to Test Repair Strategies

Specimen Type	Description
NCD Macrobeams	concrete beams with simulated cracks of various widths over ECR
G109 Macrobeams	concrete beams containing ASTM G-109 specimens with ECR
FHWA Slabs	concrete slabs constructed with ECR in a previous FHWA study
Bent Bar Slabs	concrete slabs with straight and bent ECR from a previous study
PCS Beams	beams with sections manufactured using poor quality concrete and ECR
Cathodic Protection Slabs	concrete slabs with impressed current cathodic protection system installed
Duanesburg Bridge Deck	ECR bridge deck located in Daunesburg, New York
7 Mile Bridge	ECR columns of the 7 Mile Bridge located in the Florida Keys

13. Sohanghpurwala, Ali Akbar; Scannell, William T.; Hartt, William H. "Repair and Rehabilitation of Bridge Components Containing Epoxy-Coated Reinforcement," (NCHRP Web Document 50, Project D10-37C). National Cooperative Highway Research Board, 2002.

All of the repair strategies – with the exception of cathodic protection – were found to be deficient in protecting the EC in cracked areas. The ECR continued to deteriorate in the vicinity of the cracks. Recoating with coating materials A and C and patching of the corrosion induced delaminations proved to be a successful strategy by halting the corrosion and reversing the current in the ECR. The most effective combination of materials for repairing delaminations made use of the silica fume patch material and coating similar to and compatible with the original EC. Testing of impressed current cathodic protection successfully mitigated corrosion in all cases in which the steel was electrically continuous, but is an expensive option for repair and was the primary source of disbondment when applied to ECR.

Using the results of the NCHRP study, decision matrices were developed to identify repair strategies for different elements exhibiting a variety of rebar and concrete properties under various exposure conditions. The matrices additionally considered future propensity of the elements for corrosion, and the technical viability and cost effectiveness of the strategies. The decision matrices are extremely dependent on the current condition of the R/C component and the resulting probability for future corrosion

and damages. These conditions were determined by the probable failure of ECR extracted from the concrete and the average exposure of the material to chlorides. The decision matrices developed appear below (Tables 6 and 7) specifically for repair of bridge structures subject to deicing salt and marine environments.

Table 6: Decision Matrix for Repair and Rehab of Bridge Structures Subject to Deicing Salts

Element	Sub-environment	Damage Category	Probability for Corrosion			
			Low	Localized	Medium	High
Substructure						
Piles & Columns	Deicing Salt Splash or Contaminated Water Runoff	Negligible	Do Nothing			
		Crack	EI	Do Nothing		
		Partial	Patch, Patch+EC	Patch: Patch+Zn-Rep, Patch+barrier(1)		Patch+Zn-adh
		Full Surface	Shotcrete: Jacket		Jacket: Jacket+Zn-Rep: Overlay+barrier(1)	Overlay+Zn: Overlay+Ti: Jacket+Zn: Jacket+Ti
Pile Caps, Struts, & Hammer Heads	Contaminated Water Runoff	Negligible	Do Nothing			
		Crack	Do Nothing			
		Partial	Patch, Patch+EC		Patch: Patch+Zn-Rep	Patch+Arc-Zn: C-Paint or Zn-adh
		Full Surface	Shotcrete		Shotcrete+Zn-Rep: Shotcrete+barrier(1)	Shotcrete+Arc-Zn: C-Paint or Zn-adh: Replace
Superstructure Elements						
Deck Top Surface	Direct Application of Salt	Negligible	Do Nothing			
		Crack	Do Nothing			
		Partial	Patch, Patch+EC	Patch+Zn-Rep: overlay: asp-mem		Overlay+Ti: Patch+aspsh-coke: Hydro
		Full Surface	Overlay: Patch+aspsh-mem		Hydro: overlay+aspsh-mem	Overlay+Ti: Hydro: Replace
Deck Soffit	Contaminated Water Runoff	Negligible	Do Nothing			
		Crack	Do Nothing			
		Partial	Full Depth Patch		Full Depth Patch+Zn-Rep	Full Depth Patch + overlay-Ti
		Full Surface	Replace entire deck or effected areas			
Primary & Secondary Members	Contaminated Water Runoff	Negligible	Do Nothing			
		Crack	EI	Do Nothing		
		Partial	Patch, Patch+EC	Patch+Zn-Rep	Patch+Zn-Rep	
		Full Surface	Shotcrete	Shotcrete+Ti		Replace

Notes:

1 Install some waterproofing breathable sealer or membrane to prevent future contamination from chloride ions.

Abbreviations

Rt-grt: Route and grout cracks.

barrier: Coatings, membranes, overlays, and sealers.

EC: Epoxy coating of the rebar prior to the installation of the patch.

EI: Epoxy injection of cracks.

Zn-adh: Zinc sheet anode with adhesive for attachment to concrete surface.

Overlay+Zn: Zinc mesh anode in concrete jacket/overlay functioning as a galvanic anode.

Overlay+Ti: Titanium anode in concrete jacket/overlay functioning as an impressed current anode.

Jacket+Zn: Zinc mesh anode in fiber glass jacket filled with a cementitious fill functioning as a galvanic CP system.

Jacket+Ti: Titanium anode in fiber glass jacket filled with a cementitious fill functioning as an impressed current CP system.

Zn-Rep: Zinc anode installed in repair areas to serve as a galvanic CP system.

Arc-Zn: Arc sprayed zinc galvanic cathodic protection system.

C-Paint: Conductive paint cathodic protection system.

Hydro: Hydromolition of the top contaminated layer and replacement with a concrete layer.

aspsh-mem: Ashpalt overlay with a waterproofing membrane.

13. Sohahngpurwala, Ali Akbar; Scannell, William T.; Hartt, William H. "Repair and Rehabilitation of Bridge Components Containing Epoxy-Coated Reinforcement," (NCHRP Web Document 50, Project D10-37C). National Cooperative Highway Research Board, 2002.

Table 7: Decision Matrix for Repair and Rehab of Bridge Structures Subject to Marine Environment

Element	Sub-environment	Damage Category	Probability for Corrosion			
			Low	Localized	Medium	High
Substructure						
Piles and Columns	Splash Zone (elevation 0 to 6 feet)	Negligible	Do Nothing			
		Crack	Not feasible due to severity of exposure.	Jacket+Zn; Jacket+Ti; Overlay+Zn; Overlay+Ti; Patch+Zn-adh(1)		
		Partial				
		Full Surface				
	Above Splash Zone (elevation 6 to 12 feet)	Negligible	Do Nothing			
		Crack	EI	Do Nothing		
		Partial	Patch+Zn-Rep+EI		Patch+Arc-Zn; Zn-adh(1)	
		Full Surface	Jacket; overlay	Jacket or overlay + Zn-Rep	Jacket+Zn; Jacket+Ti; Overlay+Zn; Overlay+Ti	
	Above Splash Zone (elevation 12 feet and above)	Negligible	Do Nothing			
		Crack	EI	Do Nothing		
		Partial	Patch+Zn-Rep+EI; Patch+EC+EI			Patch +Zn-Rep
		Full Surface	Jacket; overlay			Jacket+Zn; Jacket+Ti; Overlay+Zn; Overlay+Ti
Pile Caps & Struts	Splash Zone (elevation 0 to 6 feet)	Negligible	Do Nothing			
		Crack	Not feasible due to severity of exposure.	Shotcrete+Zn-Rep; Arc-Zn		
		Partial				
		Full Surface				
Pile Caps & Struts	Above Splash Zone (elevation 6 to 12 feet)	Negligible	Do Nothing			
		Crack	EI	Do Nothing		
		Partial	Do Nothing; Patch; Patch+EC	Patch+Zn-Rep+EI; Patch+Zn-adh		
		Full Surface	Shotcrete+Zn-Rep		Shotcrete+Arc-Zn; Zn-Rep	
	Above Splash Zone (elevation 12 feet and above)	Negligible	Do Nothing			
		Crack	EI	Do Nothing		
		Partial	Do Nothing; Patch; Patch+EC	Patch+Zn-Rep+EI; Patch+Zn-adh	Patch+Arc-Zn; Zn-Rep	
		Full Surface	Shotcrete	Shotcrete+Zn-Rep	Shotcrete+Arc-Zn; Zn-Rep	
Superstructure Elements						
Deck Top Surface	All elevations	Negligible	Do Nothing			
		Crack				
		Partial	Patch	Patch+Zn-Rep	Overlay+Ti asph-coke	
		Full Surface	Overlay	Overlay; Hydro	Hydro; Overlay+Ti; asph-coke; Ductaco	
Deck Soffit	All elevations	Negligible	Do Nothing			
		Crack				
		Partial	Patch; Patch+EC	Patch+Zn-Rep	Patch+Arc-Zn	
		Full Surface	Replace Deck			
Primary & Secondary Members	Splash Zone (elevation 0 to 6 feet)	Negligible	Do Nothing			
		Crack				
		Partial	Patch; Patch+EC	Patch+Zn-Rep	Patch+Arc-Zn	
		Full Surface	Shotcrete	Shotcrete+Arc-Zn		
	Above Splash Zone (elevation 6 feet and above)	Negligible	Do Nothing			
		Crack	EI	Do Nothing		
		Partial	Patch; Patch+EC	Patch+Zn-Rep	Patch+Arc-Zn; Patch+Zn-adh	
		Full Surface	Shotcrete	Patch+Zn-Rep	Shotcrete+Arc-Zn	

Notes:

1. If and only if it can be sealed properly to prevent water infiltration into the adhesive. Manufacturer claims successful installation on one s

Abbreviations

Zn-adh: Zinc sheet anode with an ionically conductive adhesive for attachment to concrete surface functioning as galvanic anode.

Patch: Concrete removal using standard practice and patch with cementitious mix.

EC: Epoxy coating of the rebar prior to the installation of the patch.

Jacket+Zn: Zinc mesh anode in a fiberglass jacket filled with a cementitious mix functioning as a galvanic CP system.

Jacket+Ti: Titanium mesh anode in a fiberglass jacket filled with a cementitious mix functioning as an impressed current CP system.

Overlay+Zn: Zinc mesh anode in a concrete jacket/overlay functioning as a galvanic CP system.

Overlay+Ti: Titanium mesh anode in a concrete jacket/overlay functioning as an impressed current CP system.

Arc-Zn: Arc sprayed zinc galvanic CP system.

Zn-Rep: Zinc anodes installed in repair areas prior to patch or overlay placement to give localized cathodic protection against ring anodes.

EI: Epoxy injection.

Jacket: Fiberglass jacket filled with an epoxy or cementitious mix.

Overlay: Concrete overlay or a concrete jacket.

Hydro: Hydromolition of the top contaminated layer and replacement with a concrete layer.

asph-mem: Asphalt overlay with a waterproofing membrane.

asph-coke: Coke breeze conductive impressed current CP system with asphalt overlay.

13. Sohngphurwala, Ali Akbar; Scannell, William T.; Hartt, William H. "Repair and Rehabilitation of Bridge Components Containing Epoxy-Coated Reinforcement," (NCHRP Web Document 50, Project D10-37C). National Cooperative Highway Research Board, 2002.

CHAPTER 3: ESTIMATION OF SERVICE LIFE OF EPOXY-COATED REINFORCEMENT FOR MYSTIC RIVER BRIDGE PIER

Problem Definition

1. Using a previous design (Figure 12) for the reinforcement of a bridge pier using ASTM standard black steel rebar, estimate the service life of the pier. The pier cross section measures 6' wide by 12' deep, and is reinforced with 27 ASTM No. 14 bars at each end. Two inches of concrete cover as well as two inches of spacing is provided between all rebar and the exterior of the concrete section.

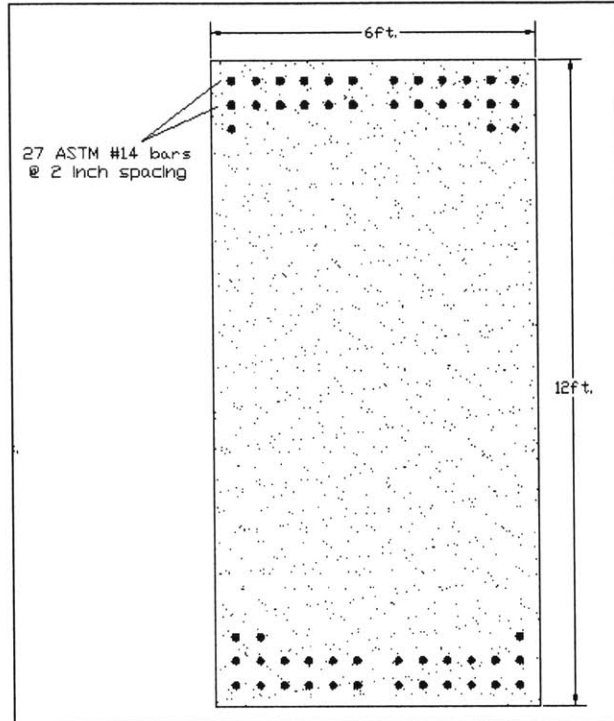


Figure 12: Bridge Pier Cross Section

14. Ward-Waller, Elizabeth; Georgakopolus, Phillip; Nasr, Mohsen. "Proposal for a New Mystic River Crossing." M. Eng. Design Project, 2005.)

2. Substitute ECR for the black bars, and estimate the service life extension of the pier provided by the ECR.

Solution

1. The service life of the bridge pier can be determined using Weyers' model described in the section of Chapter 1 labeled "Corrosion: Carbon Steel's Big Drawback." Weyers' model uses Fick's 2nd Law of diffusion to predict the time required for chloride ion concentration to reach a threshold value at the depth of the steel rebar. In this case, the depth of the rebar, x , is 2 inches, the surface chloride concentration C_o is assumed to be 14.0 lb/yd³, the diffusion constant, D_{ac} , is assumed to be 0.05 in²/yr, and the chloride threshold for initiation of corrosion

in the steel, $C(x,t)$, is assumed to be 1.2 lb/yd³. (The diffusion constant, surface concentration, and chloride threshold are based on averages used in the Iowa DOT study.) In order to determine the time required for chloride diffusion to depth x and concentration $C(x,t)$, substitute the values for $C(x,t)$ and C_o into equation (1) and solve for $\text{erf}[x/(2*(D_{ac} t)^{1/2})]$:

$$C(x,t) = C_o * \{1 - \text{erf}[x/(2*(D_{ac} t)^{1/2})]\} \quad (1),$$

$$\Rightarrow 1.2 \text{ lb/yd} = 14.0 \text{ lb/yd} * \{1 - \text{erf}[x/(2*(D_{ac} t)^{1/2})]\},$$

$$\Rightarrow \text{erf}[x/(2*(D_{ac} t)^{1/2})] = 0.914$$

Using a table to determine the value of the error function equivalent to 0.914, the following equation is obtained:

$$x/(2*(D_{ac} t)^{1/2}) = 1.215$$

Finally substituting the values for x and D_{ac} and solving for t ,

$$2'' / (2 * (0.05 \text{ in}^2/\text{yr} * t)^{1/2}) = 1.215,$$

$$\Rightarrow t = 13.5 \text{ years}$$

This calculation revealed that after approximately 13.5 years, the chloride concentration at the level of the steel will reach the threshold level and the steel will begin to corrode. Assuming a worst-case scenario, the time from initiation of corrosion for the steel to build up enough corrosion products to cause spalling and delamination in the concrete requires an additional 4 years. Therefore the total time from construction until the end of functional service life for the bridge pier is approximately 17.5 years.

2. The goal of this step is to calculate the service life extension provided by substituting ECR for black steel rebar using the polynomial given in the section labeled “Corrosion of ECR” in Chapter 2. We will assume for the sake of optimization that the concrete is uncracked, therefore we will use the following variation of the polynomial:

$$r(t) = 5.00 + .0135t - .00134t^2 \quad (4)$$

Assuming that the rebar must reach a rating, $r(t)$, of 1 before it begins to build up corrosion product and solving for t :

$$t = (-0.0135 \pm (0.0135^2 - 4(4.0)(-0.00134))^{1/2}) / (2 * -0.00134),$$

$$\Rightarrow t = 66.3 \text{ years.}$$

From this calculation approximately 66.3 years will pass before the ECR will begin to build up corrosion product. Again at least 4 years will be required to build up significant corrosion product and cause spalling and delamination in the concrete, therefore the total time to end of functional service life for the ECR is 70.3 years. The service life extension provided by the ECR in this scenario and predicted by these models is greater than 50 years!

CONCLUSION

This thesis provided a comprehensive description of issues and solutions involved in reinforcing concrete structures with steel rebar. Steel reinforcing bars were implemented because of their high strength in tension and compression, ductility, and bond strength with concrete. As the variety of applications and conditions for reinforced concrete construction have increased, corrosion of rebar in harsh environments emerged as a significant problem.

Corrosion occurs when concrete is exposed to moisture and chlorides, which infiltrate the concrete cover to the depth of the steel. The chlorides, water, and oxygen then react chemically with the metal to form iron oxide. The products of the corrosion reaction take up more volume than the original bar and cause the concrete to crack and delaminate. In order to prevent corrosion damage in new construction, epoxy coating was invented to protect the bars before they are cast in concrete.

Epoxy coating has proved to be an effective method for resisting rebar corrosion during the past thirty years in most applications. Predictions for the extension of service life for concrete reinforced with epoxy-coated rebar instead of black steel rebar estimate that epoxy coating lengthens the structure's service life by 2 to 3 times. Unfortunately, studies in the past ten years have revealed that epoxy coating is not completely corrosion and maintenance free. Coating defects, loss of adhesion, reduction of bond strength with concrete, and increased incidence of concrete cracking have all resulted from epoxy-coated rebar subject to high concentrations of corrosive agents over time. Maintenance strategies are already in development to anticipate and repair deterioration in ECR structures.

The effectiveness of epoxy-coated rebar in extending the service life of reinforced concrete structures was demonstrated in the bridge pier reinforcement problem. Weyers' model predicted a service life of 17.5 years when black steel reinforcement was used, and 70.3 years when epoxy-coated rebar was used. Using this prediction model, epoxy-coated rebar provides an extra 50 years of service to the pier.

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