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Epoxy-Coated Reinforcement in Highway Structures

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EPOXY-COATED REINFORCEMENT IN HIGHWAY STRUCTURES

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Preface

The papers presented in this *Circular* were reviewed by members of the Transportation Research Board Committee A3C15, *Corrosion* and presented in Sessions 153 and 182, "Epoxy-Coated Rebars for Reinforced Concrete Structures" at the Transportation Research Board's 72nd Annual Meeting held in Washington, D.C., January 10-14, 1993. The papers were accepted for publication by the Transportation Research Board's peer review process established according to procedures approved by the Governing Board of the National Research Council. Reviewers were selected among committee members and other outside experts. With a minimum of three reviews a decision to publish was based on reviewer comments and resultant author revisions.

The information presented should be of interest to federal, state and local engineers responsible for design, construction or maintenance of reinforced concrete structures. The first three papers provide information on activities outside the United States. In the first paper, Erdoğdu and Bremner present the results of a Canadian study of corrosion of epoxy-coated reinforcing steel in concrete exposed to chloride-contaminated environments. McKenzie describes the results of an on-going United Kingdom study on corrosion of straight and bent bars with uncoated ends, repaired ends, and holes in the epoxy-coating. Salt was introduced through ponding and mixed in the concrete. The third paper by Schiessl and Reuter provides an overview of research projects, standards/guidelines, and uses of epoxy-coated reinforcement in Europe. The remaining papers contain information on epoxy-coated rebars in the United States. Smith, et. al., describe the use and current restrictions on epoxy-coated rebar in Florida. Hasan and Ramirez present the findings from an on-going investigation on the effects of static and repeated loadings on decks and slabs reinforced with epoxy-coated bars in Indiana. Pfeifer, et. al., provide a summary of corrosion research sponsored by the Concrete Reinforcing Steel Institute from 1982 to 1992. Appendix A contains an executive summary of research conducted under C-SHRP and Appendix B a bibliography of epoxy-coated rebar literature.

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FIELD AND LABORATORY TESTING OF EPOXY-COATED REINFORCING BARS IN CONCRETE

Şakir Erdoğdu and Theodore W. Bremner*

Corrosion of reinforcement in concrete exposed to chloride-contaminated environments is a common problem. If the corrosion process proceeds undetected, the safety of the reinforced concrete structures may be diminished. Therefore, information about the corrosion activity of reinforcing steel is essential. This paper presents the results of a study of the corrosion of epoxy-coated reinforcing steel in concrete exposed to chloride-contaminated environments using common test procedures including open-circuit potentials, linear polarization measurements, and AC impedance measurements. Testing was carried out both in the laboratory and in the field. Laboratory testing was accelerated in a simulated marine environment with four wetting and drying cycles each day using small-scale concrete slabs containing a single U-shaped epoxy-coated rebar. Corrosion activity was monitored continuously while specimens were exposed over a two year period. For field testing, concrete slabs containing U-shaped epoxy-coated reinforcing bars were placed at a natural marine environment exposure station in Treat Island, Maine. Electrochemical monitoring indicated that the corrosion rate of epoxy-coated rebar was negligible regardless of the degree of damage to the coating. Similarly epoxy-coated rebars removed from the slabs at the end of one and two years of exposure showed no propensity to cause cracking and spalling due to corrosion products and no visible signs of corrosion were found on the surface of concrete.

Keywords: corrosion, chloride ingress, oxygen access, diffusion, epoxy-coated reinforcement, pH, polarization, passivity, resistance, impedance, current density, corrosion rate

INTRODUCTION

Reinforcing steel embedded in uncarbonated, chloride-free concrete does not corrode due to the presence of alkali hydroxides in the Portland cement matrix. It is assumed that the high alkalinity of the concrete passivates the steel (*1,2*). Aggressive ions such as chloride are capable of destroying this passivity causing the steel to corrode at localized areas (*3,4,5*). The aggressiveness of the environment is also a function of available oxygen with which steel interacts to form corrosion products (*6,7,8*). The conditions which must be fulfilled if corrosion of steel is to be sustained are as follows:

- The relative concentration of chloride ions to hydroxyl ions at the steel surface must be sufficient to break down the passivity.
- The availability of oxygen at the cathodic areas must be sufficient to sustain a reaction. If oxygen is limited, the cathodic polarization curve indicates that the current passing from the anodic area to the cathodic area will be negligible. This is normally the case for reinforcing steel properly isolated from the surrounding environment.

A progressive accumulation of rust at the steel-concrete interface produces tensile stresses often higher than the ultimate stress of the concrete. Eventually , this will result in cracking and spalling of the concrete.

The ability to measure the rate of corrosion of rebar in concrete would be useful in determining the time for repair or replacement of structures in a given environment. Within the past decade there have been important advances in the development of electrochemical techniques for measuring the corrosion rate of reinforcement in concrete. These methods include open-circuit potentials (*9,10*), linear polarization technique (*10,11,12*), and AC impedance spectroscopy (*13,14,15*). However, monitoring the corrosion activity of rebar in concrete, particularly when it is covered with an epoxy compound, is very tedious and, in some cases, can be misleading. The work described in this paper concerns the applicability of commonly used electrochemical techniques to measure the corrosion rate of steel in concrete exposed to chloride-contaminated environments. The significance of this research is that the corrosion rate as determined by linear polarization is an effective method for predicting the behavior of reinforced concrete structures in aggressive environments.

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EXPERIMENTAL PROCEDURE

Program and Exposure Conditions

The investigation outlined in this paper was carried out in the field and under accelerated exposure conditions in the laboratory. For laboratory testing, a Marine Environment Simulated Setup (MESS) was used. For field work, a severe exposure site in the Bay of Fundy at Treat Island, Maine, was used.

All the concrete slabs tested were made with Type 10 Canadian Portland cement (ASTM Type I). The concrete was machine mixed and then tamped and vibrated during placement. The maximum aggregate size used was 12 mm (0.5 in). An air entraining agent was added to the mixes to produce 6.5±0.5% air content. After casting, the concrete slabs were moist cured for two weeks at 20±2°C (68±4°F) and a relative humidity of not less than 95% before exposure to simulated seawater and a natural marine environment.

Four series of mixes (mix-C, D, E, and F) with a water to cement ratio of 0.60 were cast. Concrete slabs from mix-C were exposed to simulated seawater in the laboratory while mix-D through mix-F were exposed to a natural marine environment at Treat Island. The slabs from mix-D were placed at mid-tide level on the beach while the slabs from mix-E and mix-F were placed at 0.5 m (1.6 ft) below high tide and 0.5 m (1.6 ft) above high tide level. Each mix yielded 12 concrete slabs containing U-shaped rebars. Four of the slabs in each series contained an uncoated bar and eight contained a coated bar. All the coated bars were initially tested with a holiday detector in conformance with ASTM G-62/79 and patched with an epoxy compound if a bare spot was noted. One half of the epoxy-coated bars were cast in the concrete without damage to the coating. The coating was damaged on two of the bars by removing 1% of the total coating area and on two other bars by removing 2% of the total area. The testing program is summarized in Table I. To obtain a 1% and 2% damage to the coating, 7 and 14 patches of epoxy in size 6 x 6 mm (0.24 x 0.24 in) were removed, respectively. The patches were between the ribs and evenly distributed over the surface of each rebar. The U-shaped bars fabricated from regular reinforcing steel were 15 mm (0.6 in) in diameter.

The slabs were 55 x 200 x 300 mm (2.2 x 8 x 12 in.). The concrete cover over the rebar was 20±2 mm (0.8±0.1 in) in all directions as shown in Figure 1. To keep concrete cover the same in all directions for all concrete slabs, the U-shaped rebars were adjusted so the legs of the bar were parallel to each other and in one plane. For corrosion rate measurements, a 100 mm (4 in) stainless steel rod (303 SS) was centrally located in each slab as a counter electrode. Wires were connected to the stainless steel rod and the rebar prior to casting the concrete. The electrical connections were coated with an epoxy compound.

Table I LAYOUT OF EXPERIMENTAL PROGRAM

Type of Exposure		Number of Concrete Slabs Cast			
		Uncoated Rebar	Epoxy-Coated Rebar		
			No Damage	1% Damage	2% Damage
ASTM Seawater (MESS) (SERIES-C)		4	4	2	2
Treat Island	Mid-Tide (SERIES-D)	4	2	2	2
	0.5 m below High-Tide (SERIES-E)	4	4	2	2
	0.5 m above High-Tide (SERIES-F)	4	4	2	2

Marine Environment Simulated Setup (MESS)

The apparatus used to simulate a moist chloride environment consisted of two fiberglass-coated wooden tanks, a pump, heaters and blowers. The tanks were hooked up such that the upper tank would be a container for the specimens while the bottom tank was the reservoir for water. A computer was programmed to engage and later disengage each of the electrical devices to generate the required wet-dry cycling process. In this manner, automatic cycling 24 hours a day over extended periods was possible. To accelerate the corrosion process, a two-hour period for the wet cycle portion and a four-hour period for the dry cycle portion were chosen so four complete cycles were performed each day. The wet portion of the cycle was operated at a temperature of 32±2°C (90±3°F) while the dry portion was at 69±3°C (156±6°F) for the entire test period. The concrete slabs were placed in the upper tank in such a way that during the dry cycle one-third and during the wet cycle two-thirds of each slab was submerged to simulate tidal effects. The seawater used to simulate a marine environment was a modified version of ASTM D-1141. The composition of seawater used is given in Table II. The solution was changed periodically to maintain a consistent pH.

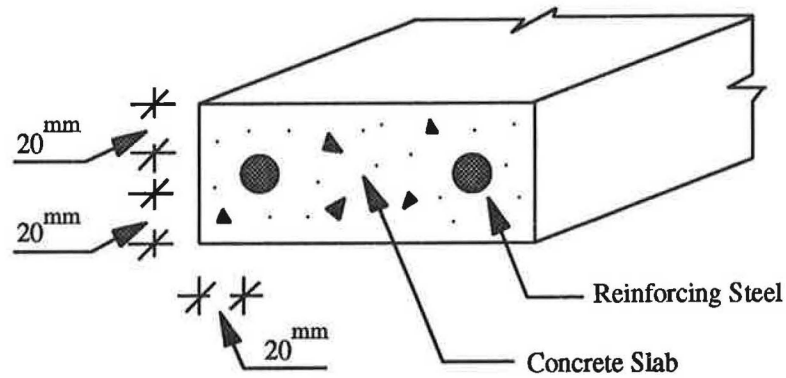

Figure 1 Location of the U-shaped rebar in concrete slab.

Table II COMPOSITION OF SEAWATER USED IN THE MESS

Ion	ASTM Seawater Concentration, mg/l	Synthetic Seawater Concentration, mg/l
Cl^-	19,500	17,500
Mg^{++}	1,329	1,020
$SO_4^=$	7,634	4,020

Natural Marine Exposure Station at Treat Island

The facility used for field testing is a severe exposure site in the Bay of Fundy at Treat Island near Eastport, Maine. The exposure station has been in use since 1936 and maintained by the U.S. Army Corps of Engineers. An average of 130 freeze-thaw cycles per year are experienced at the exposure site (*16*).

Instrumentation and Monitoring

The instantaneous corrosion rate of rebar in concrete was measured with a device designed using linear polarization principles. The computer-based data acquisition system is battery operated for field applications. The corrosion measuring system includes a Zenith Turbosport 386E laptop computer, Zenith Extender Chassis, Data Translation DT2801 data acquisition board and a 12-volt battery.

The system is able to polarize the rebar (working electrode) in concrete by impressing a voltage through a counter electrode. The impressed voltage causes current to flow between the working electrode and the counter electrode. A schematic layout of the setup is illustrated in Figure 2.

After the connections were made between the data acquisition system and the corroding system, the open-circuit potential (E_{corr}) of the reinforcing steel was determined. Scanning was then initiated. This was accom-

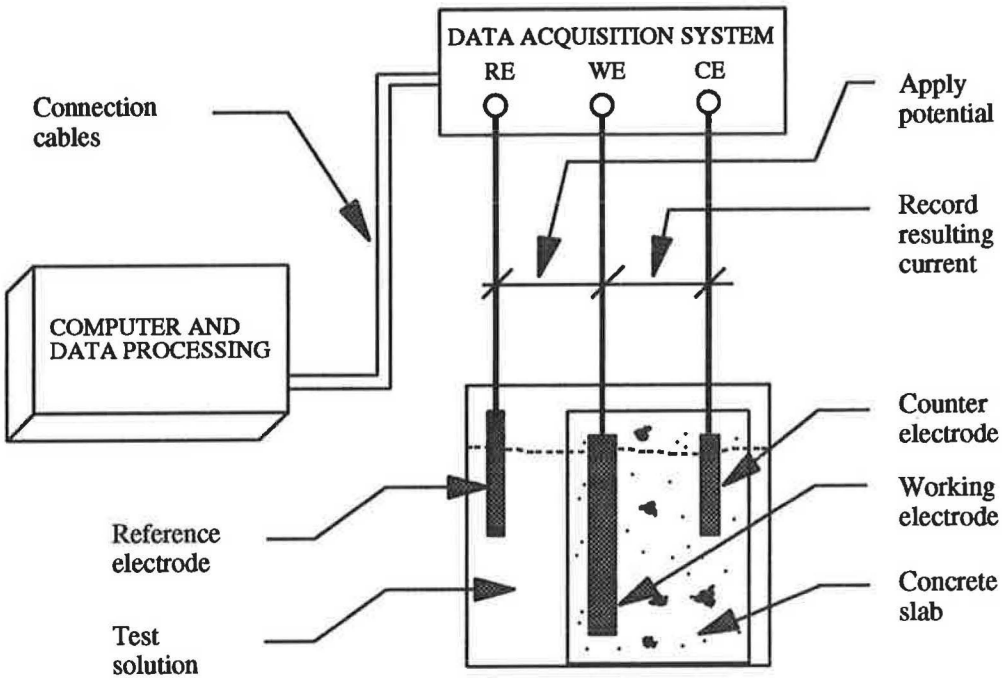

Figure 2 Schematic layout of corrosion rate measuring instrument.

plished automatically, anodically and cathodically, by changing the potential of the steel in the concrete in a range of ±10 mV about E_{corr}. The scan rate can be adjusted by modifying the software. During scanning, the difference in potential and the corresponding current was recorded each 15 seconds, and each data point was plotted on the screen of the computer. The relationship, though not perfect, is linear. The test could be restarted if abnormalities exist. The data at the end of the test was processed and the slope of the relationship calculated. This can be correlated to the corrosion current density using the Stern-Geary relationship:

$$i_{corr} = \frac{B}{R_p} [\mu A/cm^2] \qquad (1)$$

where, $R_p = \frac{\Delta E}{\Delta I} [ohm\text{-}cm^2]$ and, $B = \frac{\beta_a \beta_c}{2.303(\beta_a + \beta_c)}$

R_p is the slope of the linear portion of the polarization curve close to the open-circuit potential and it is often called linear polarization resistance. B is referred to as the proportionality constant which can be calculated using the Tafel slopes (β_a and β_c).

The corrosion intensity calculated from Equation (1) was then converted to corrosion rate (CR) in terms of steel consumption per year using Faraday's law:

$$CR = \frac{0.13 i_{corr}(EW)}{\rho} [MPY] \qquad (2)$$

where i_{corr} is the corrosion current density ($\mu A/cm^2$), EW is the equivalent weight (g/eq.) and ρ is the density of the steel (g/cm^3). The only inputs for the calculation of current density and corrosion rate are Tafel constants, equivalent weight, density, and area of the steel under test.

The AC impedance measurements were obtained using a Solartron Model 1255 Frequency Response Analyzer and a Solartron Model 1286 Electrochemical Interface. The frequency response analyzer used is capable of generating frequencies in the range of 10 μHz to 20 MHz. A 10 mV amplitude sinusoidal signal was used. The Solartron Model 1255 can measure signals as small as 1 mV. The data can be saved automatically for further use and manipulation.

At the end of testing the polarization resistance or charge transfer resistance (R_t) was either obtained

from a Nyquist plot or calculated empirically. The current density and corrosion rate were then calculated following the steps described in the linear polarization technique.

RESULTS AND DISCUSSION

Laboratory Testing

Open-circuit Potential Measurements

The open-circuit potential of rebar was measured against a saturated calomel electrode. The corresponding illustrations of rebars in the concrete slabs exposed to simulated seawater are presented in Figures 3 through 6. The open-circuit potentials were measured at 25-cycle intervals up to 1,000 cycles and then measurements were repeated at 50-cycle intervals for another 1,000 cycles. The measurements were taken at 100-cycle intervals thereafter.

Figure 3 illustrates the thermodynamic behavior of the uncoated bars in the concrete slabs for two years. The bars exhibited a similar potential change with time. After an initial drop in potential at the end of 275 cycles, the open-circuit potential of the rebars began to increase. The potential drop was greater for rebars in concrete slabs C2 and C4 than those slabs C1 and C3. Repassivation continued for about 200 cycles, then, a sudden drop in potential was observed for all bars. Following this depassivation, active corrosion started and continued for the remainder of the test with a consistent potential trend. The first drop in potential is indicative of the arrival time of chloride ions at the steel surface.

Figure 4 shows the potential variation of the undamaged epoxy-coated reinforcing steel in the slabs. Initially, all bars showed a gradual increase in potential. This increase continued up to 650 cycles for the bars in the concrete slabs C5 and C6, and 900 cycles for bars in slabs C7 and C10. Following a period of fluctuation in potential, all bars showed a consistent potential variation thereafter. This is a good example the fact that properly coated steel in concrete remains passive in the absence of cathodic areas where reduction of oxygen is supposed to take place.

Figure 5 shows the thermodynamic behavior of 1% damaged epoxy-coated reinforcing steel in the concrete slabs. Up to 300 cycles, a continuous increase in the potential of both rebars was observed. Then, following a fluctuation period, both rebars displayed

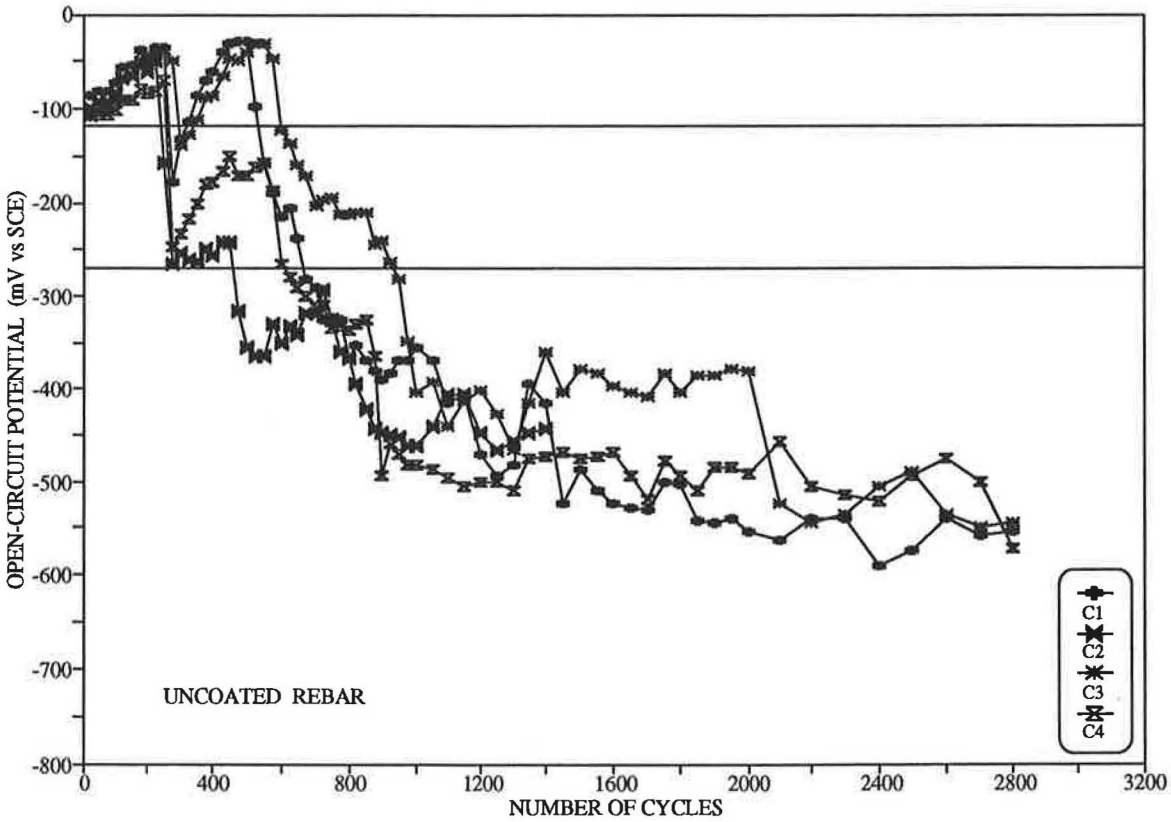

Figure 3 Potential of the uncoated rebar in concrete slabs exposed to simulated seawater.

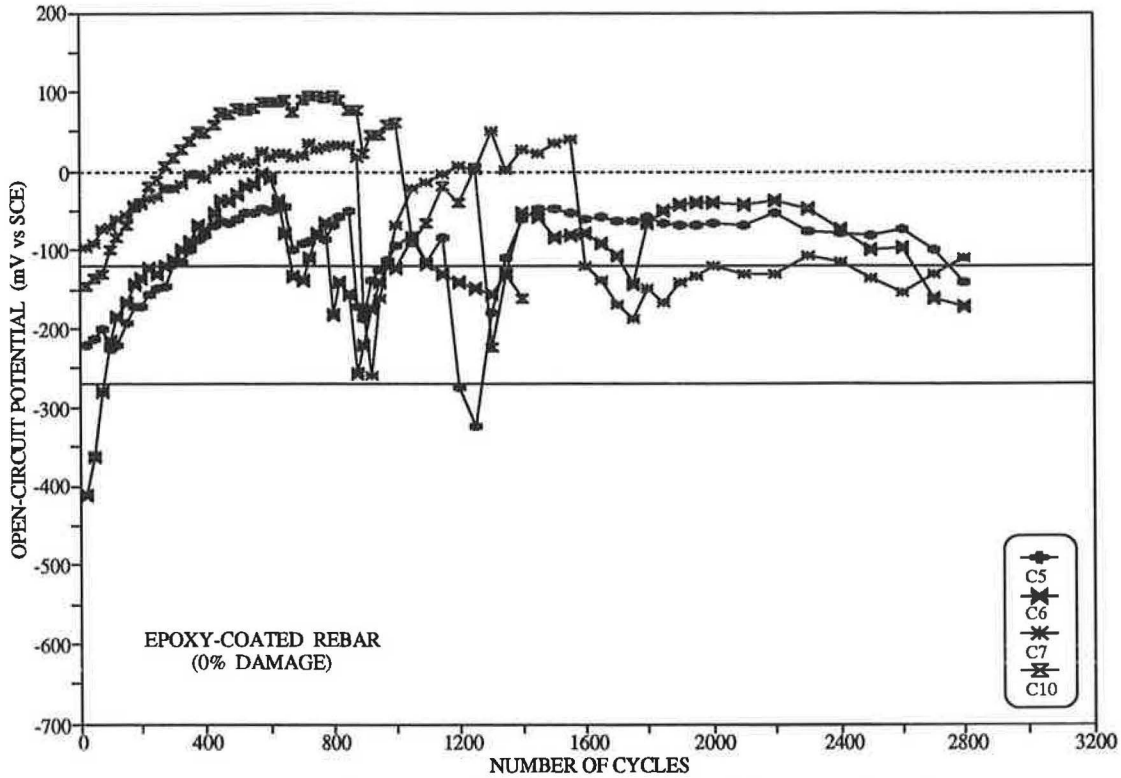

Figure 4 Potential of the undamaged epoxy-coated rebar in concrete slabs exposed to simulated seawater.

consistent potential variation. The fluctuations indicate that the corrosion activity was controlled by the oxygen necessary for cathodic reaction and the amount of chloride ions available at the steel surface to break down the passivity.

Figure 6 illustrates the thermodynamic behavior of 2% damaged epoxy-coated rebar in concrete slabs with time. The potential variation is similar to that obtained from 1% damaged epoxy-coated bars. Following an initial increase in potential, both bars went from a passive to an active state by a series of intermediate steps depending on the availability of chloride ions at the reaction front. Then, both bars remained active for the remainder of the test. This indicates there were sufficient amounts of chloride ions available at the damaged areas of the steel surface to break down the passivity. As a result, the potential of the bars remained active without microcathode polarization owing to the presence of oxygen at the damaged areas of the steel.

Corrosion Rate Measurements

The corrosion rate measurements using the linear polarization technique were repeated periodically to measure the trend of current density over time. The polarization resistance (R_p) was directly determined by employing the linear polarization technique discussed earlier. Then, corrosion current density was calculated using Equation (1). For the proportionality constant B, a value of 52 mV was used to calculate the corrosion current density for the undamaged epoxy-coated rebar since the corrosion activity was passive, and a value of 26 mV was used for the rest of the cases since the corrosion process was active. Equation (2) was used to calculate the steel consumption per year in mils per year (MPY). The area considered in the calculation was the total area of the rebar (245 cm^2, 38 in^2) in the concrete slab. A sweep rate of 4 mV/min was applied during the linear polarization.

The average polarization resistances and current densities obtained from bars are summarized in Table III. The corresponding illustration is given in Figure 7. Although it differed from rebar to rebar, the magnitude of the average current density increased with time. There was a slight increase in the resistance of the undamaged epoxy-coated bars exposed to the same environment. The average polarization resistances (ohm-area) were over 10,000 $k\Omega\text{-}cm^2$ (1,550 $k\Omega\text{-}in^2$) for the entire test period. This resulted in negligibly low

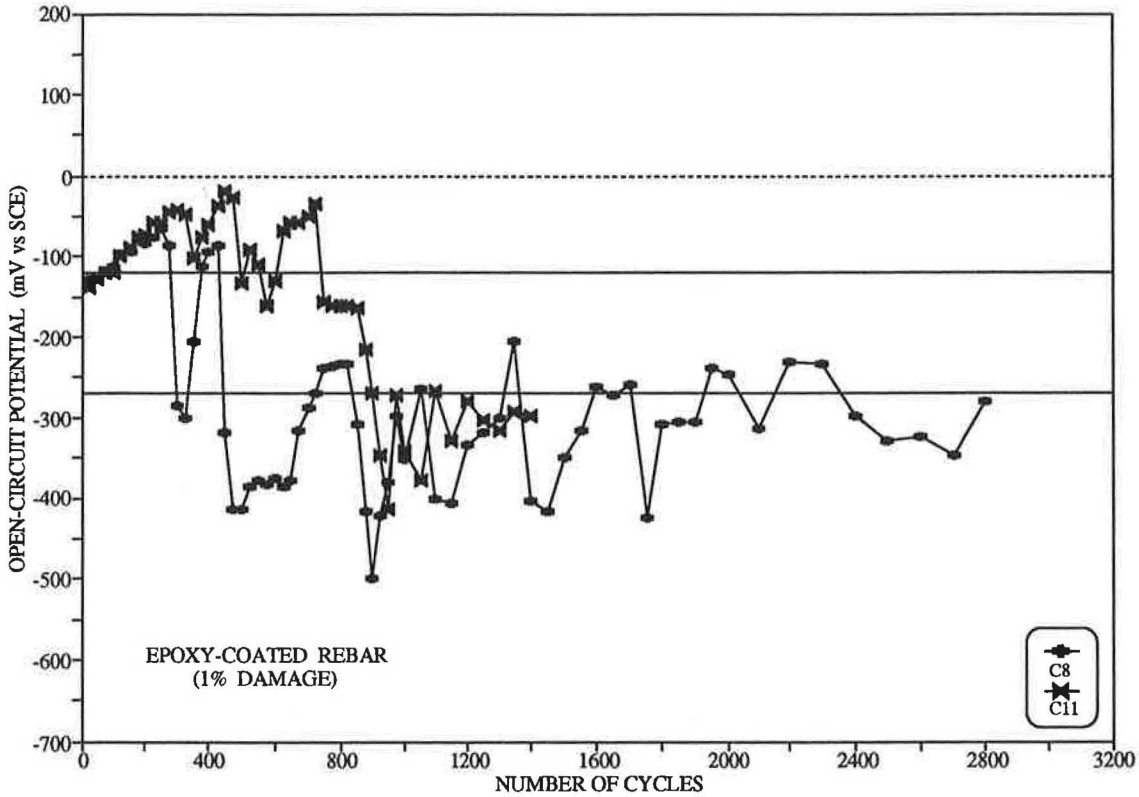

Figure 5 Potential of 1% damaged epoxy-coated rebar in concrete slabs exposed to simulated seawater.

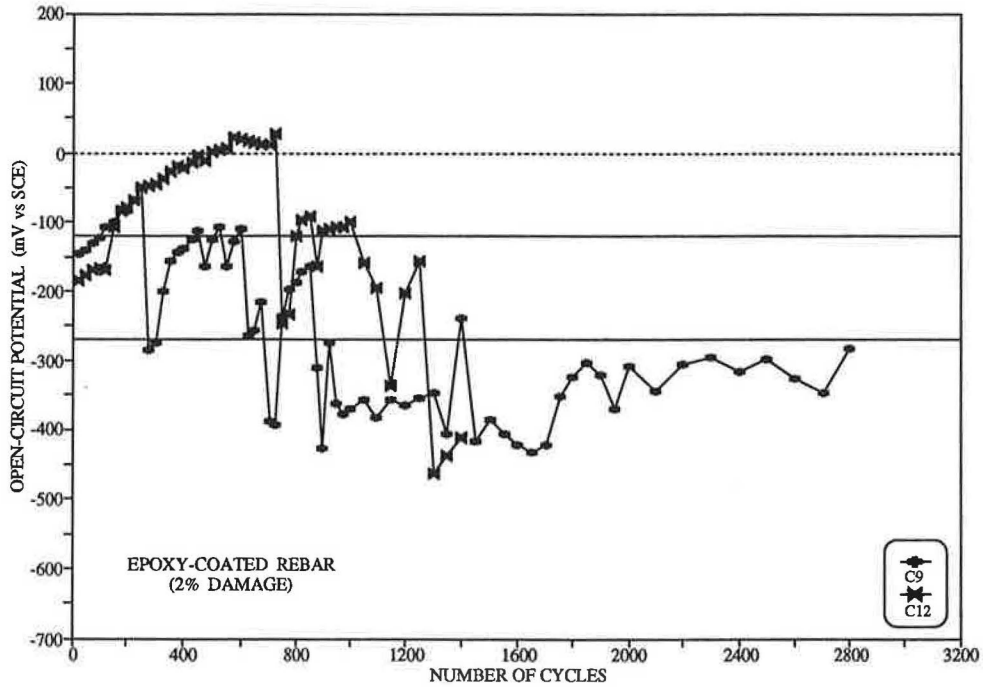

Figure 6 Potential of 2% damaged epoxy-coated rebar in concrete slabs exposed to simulated seawater.

Table III AVERAGE CURRENT DENSITIES OBTAINED FROM REBARS IN CONCRETE SLABS EXPOSED TO SIMULATED SEAWATER USING THE LINEAR POLARIZATION TECHNIQUE

Exposure Time (Months)	Polarization Resistance, KΩ-cm^2 (Current Densities, μA/cm^2)			
	Uncoated Rebar	Epoxy-Coated Rebar		
		No Damage	1% Damage	2% Damage
7	118 (0.32)	43,700 (0.00)	4,160 (0.01)	10,600 (0.00)
11	27.5 (1.82)	198,000 (0.00)	2,510 (0.02)	2,400 (0.02)
13	11.9 (3.94)	60,700 (0.00)	939 (0.03)	559 (0.05)
15	10.8 (5.15)	199,000 (0.00)	827 (0.03)	500 (0.05)
17	10.2 (7.87)	263,000 (0.00)	1,340 (0.02)	667 (0.04)
19	4.45 (7.42)	313,000 (0.00)	1,420 (0.02)	833 (0.03)
21	4.51 (7.50)	179,000 (0.00)	1,120 (0.02)	800 (0.03)
24	2.63 (10.1)	190,000 (0.00)	1,510 (0.02)	869 (0.03)

current densities. Obviously, this positive contribution for corrosion resistance is attributable to the dielectric characteristics and the high resistance of the epoxy coating to ionic diffusion.

From Figure 7, 1% and 2% damage epoxy-coated bars showed similar corrosion trends over time. There was an increase in the average polarization resistance measured for both groups of rebars. The magnitude of corrosion current density was higher than the 0.01 μA/cm^2 (0.01 mA/ft^2) which is recommended as the maximum current density normally considered for long-term maintenance-free performance.

Corrosion rate measurements based on the AC impedance technique are summarized in Table IV. Five bars were tested at the end of 7.5 months of exposure to simulated seawater. Two of them were uncoated and the rest were epoxy-coated with one undamaged, one 1% damaged and one 2% damaged. The value of charge transfer resistance (R_t) which corresponds to R_p was determined from the AC impedance responses of individual rebars. The current densities and corrosion rates were calculated using Equation (1) and Equation (2).

The uncoated rebars in concrete slabs C2 and C4 had similar AC responses. Since the corrosion cells were resistive, the AC responses of the bars were flat. The rebar in slab C7 had a semicircle with a large diameter. The corrosion rate was negligible due to a high resistance. Rebars in slabs C8 and C9 indicated similar AC impedance responses with small semicircles. Following these semicircles, at low frequencies, they exhibited a linear segment with a slope smaller than 45 degrees. This was a good indication that the corrosion taking place at the damaged areas was small in magnitude.

Visual Observations

At the end of one and two years of exposure some concrete slabs were cut open to expose the bars for visual and microscopic examination of corrosion products formed at the rebar-concrete interface. At the end of two years, the uncoated rebars were heavily corroded. Because of the accumulation of corrosion products at the steel-concrete interface, cracks formed in the concrete slabs and extended to the surface where they were visible to the eye. The corrosion products formed at the steel surface were mainly black-green-brown in color.

The undamaged epoxy-coated rebar showed no signs of corrosion even at the end of two years of exposure to simulated seawater. However, the damaged rebars showed some degree of corrosion at the damaged areas. The rust products were usually black in color and formed as localized pits at the damaged spots. There was no sign of cracking of the concrete cover when examined at 100X with a stereo-microscope.

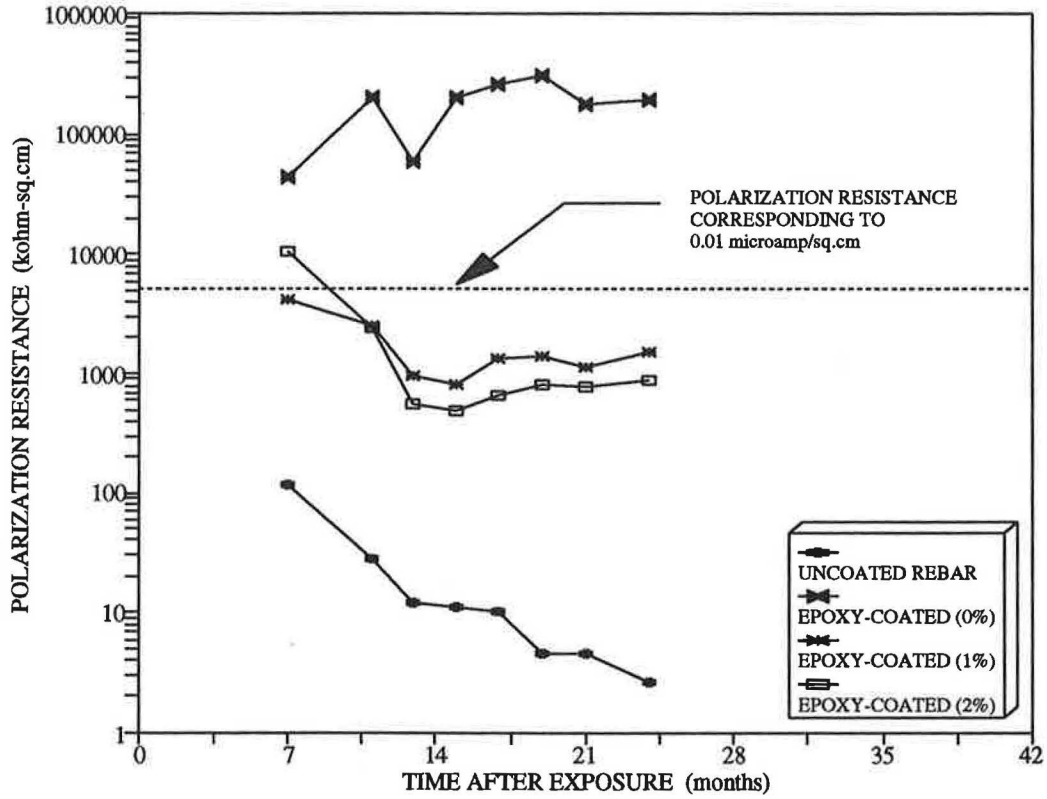

Figure 7 Polarization resistance of reinforcing steel in concrete slabs exposed to simulated seawater.

Field Testing

Corrosion Rate Measurements

The average polarization resistances obtained from rebars in the concrete slabs exposed to natural marine environment using the linear polarization technique are presented graphically in Figures 8 and 9. They illustrate the average polarization resistances measured at the end of one and two years of exposure. These graphs allow comparisons between the polarization resistance of rebars with regard to location at the exposure site. At the end of one year, the difference in polarization resistance of the damaged and undamaged epoxy-coated rebar, was not discernable regardless of location within the exposure site. The magnitude of current density was negligible. However, at the end of two years, the damaged rebars showed high current densities with respect to those obtained at

Table IV CORROSION RATE OBTAINED FROM REBARS IN CONCRETE SLABS EXPOSED TO SIMULATED SEAWATER USING THE AC IMPEDANCE TECHNIQUE (7.5 MONTHS EXPOSURE)

Parameter	Uncoated Rebar		Epoxy-Coated Rebar		
			No Damage	1% Damage	2% Damage
	C-2	C-4	C-7	C-8	C-9
R_t (kΩ-cm^2)	8.91	63.1	355,000	562	501
i_{corr}(μA/cm^2)	2.92	0.41	0.00	0.05	0.05
CR (MPY)	1.35	0.19	0.00	0.02	0.02

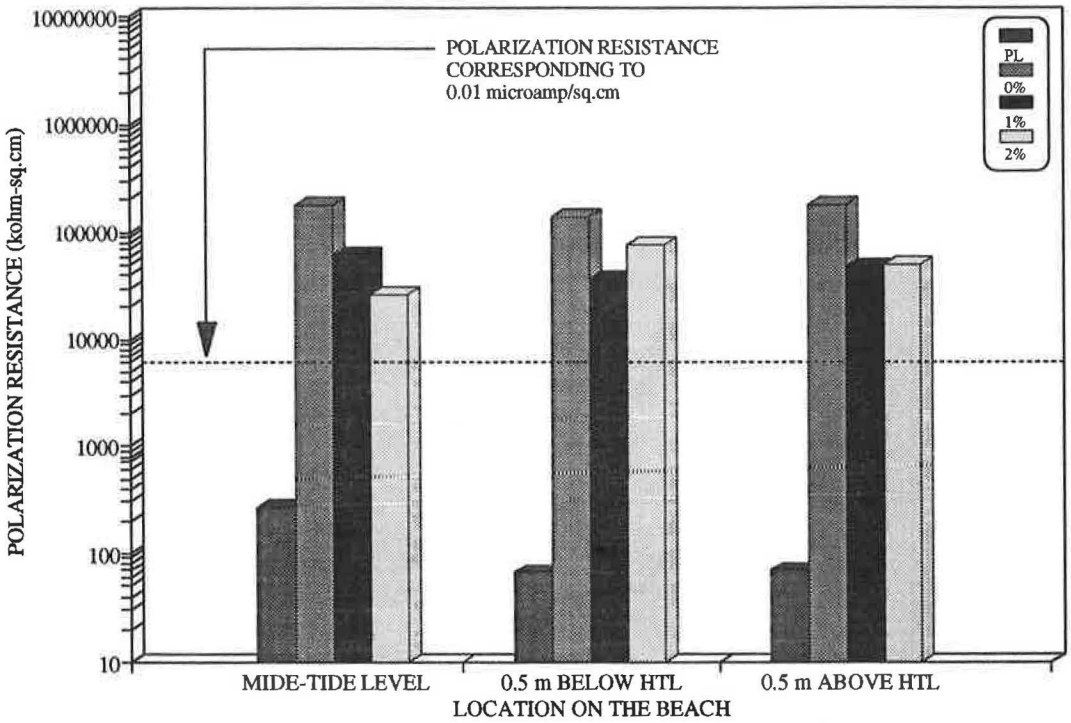

Figure 8 Polarization resistance of reinforcing steel in concrete slabs exposed to natural marine environment for one year.

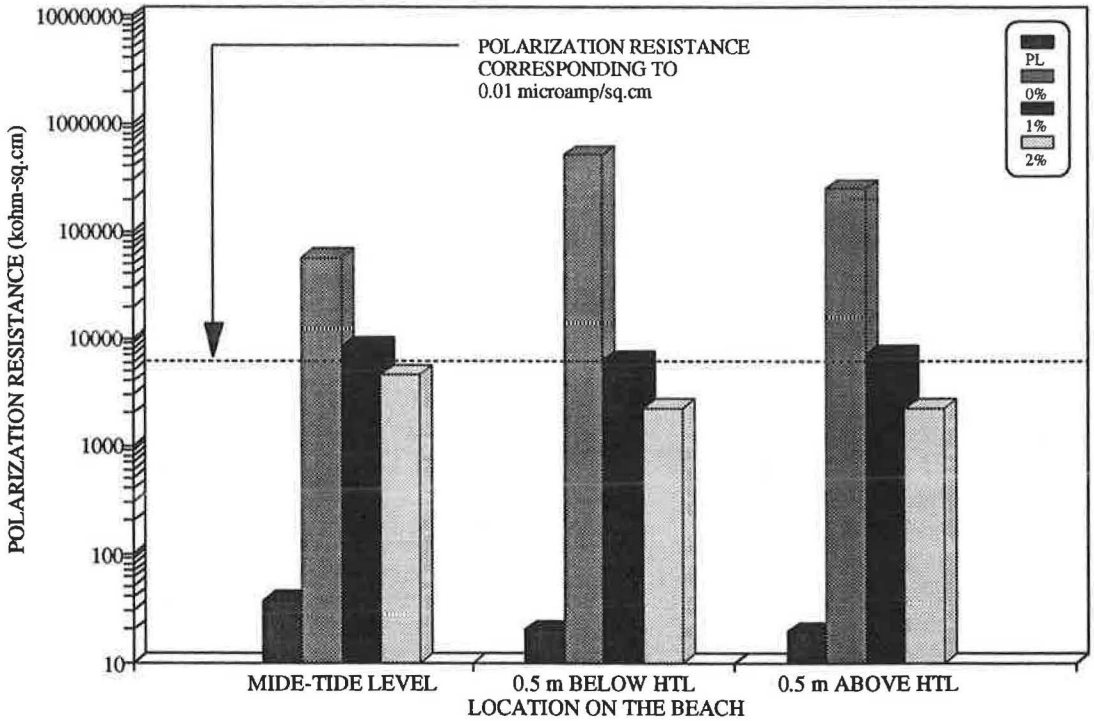

Figure 9 Polarization resistance of reinforcing steel in concrete slabs exposed to natural marine environment for two years.

the end of one year. Moreover, the two percent damaged bars showed an average current density slightly higher than 0.01 $\mu A/cm^2$ (0.01 mA/ft^2). A simple comparison revealed that the average current density obtained from the uncoated rebar at the end of two years was 2 to 3 times the average current density measured at the end of one year. Another discernable point is that the corrosion rate at the mid-tide level was slightly lower than that observed at the other locations on the exposure site.

Visual Observations

At the end of one and two years, some slabs from each location were cut open to expose the rebar for visual and microscopic examination of the corrosion products on the rebar surface. Regardless of the location at the exposure site, the uncoated bars had localized corroded spots over 10 to 15% of the total surface area of the bar. The rust products were usually black-green-brown in color. The undamaged epoxy-coated rebars were unchanged while the damaged epoxy-coated rebars were in good condition as well. The only corrosion products detected were some small pits at the damaged areas.

CONCLUDING REMARKS

The main conclusions from this investigation are as follows:

- Time-to-active corrosion initiation for the uncoated rebar exposed to simulated seawater was about 5.5 months according to the criteria quoted in ASTM C-876.
- Long-term open-circuit potentials revealed that epoxy-coated rebar in concrete exposed to simulated marine environment remains passive. This was also confirmed by the linear polarization measurements as no measurable corrosion was obtained for the entire test period.
- The overall average corrosion current density of the uncoated steel was 2.88 $\mu A/cm^2$ (1.33 MPY) at the end of one year of exposure to simulated seawater, and 10.1 $\mu A/cm^2$ (4.65 MPY) at two years. The average corrosion current densities measured at the end of one year of exposure to a natural marine environment was 0.39 $\mu A/cm^2$ (0.18 MPY), and 1.19 $\mu A/cm^2$ (0.55 MPY) at two years. The current density at the end of two years increased more than three times the value obtained at the end of one year of exposure for both exposure conditions.
- One percent damaged epoxy-coated rebars exposed to a natural marine environment for two years showed a current density slightly higher than the 0.01 $\mu A/cm^2$ which is the recommended maximum current density for long-term maintenance-free performance.
- The average current density of the uncoated rebars in concrete slabs placed at the high tide region was twice the current density experienced at mid-tide level.
- After two years of exposure to marine environments in the tidal zone, there was no indication of rust stains or cracks on the surface of concrete slabs due to the corrosion of the epoxy-coated reinforcing steel regardless of the degree of the damage to the coating and exposure conditions.

ACKNOWLEDGEMENTS

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THE EFFECT OF DEFECTS ON THE DURABILITY OF EPOXY-COATED REINFORCEMENT

Malcolm McKenzie*

The effect of defects in commercially produced UK epoxy-coated reinforcement is being assessed using concrete test specimens. Conditions being studied include uncoated ends, repaired ends, holes in the coating, and bent bars. The experiment involves both salt ponded specimens and specimens with salt added to the concrete mix. Corrosion currents are being measured between bars connected with a resistor. After 2 years the experiment has shown that corrosion does spread from the defects under the epoxy coating in salt contaminated concrete. Corrosion was also detected beneath repaired coating and beneath the coating on bent bars. Corrosion of the epoxy-coated reinforcement was light surface rusting without loss in bar section, or peeling or blistering of the coating, and less than that of uncoated steel. Half-cell potential measurements on epoxy-coated reinforcement were more variable than on uncoated steel. Due to this variability, periodic readings should be made to detect significant changes in potential. Testing is continuing at the Transport Research Laboratory in the UK.

INTRODUCTION

The ingress of chlorides into bridge concrete can lead to corrosion of the reinforcement. One approach to preventing such corrosion is to coat the reinforcement with a fusion bonded layer of epoxy resin. This provides a continuous barrier with low permeability to oxygen, water and chloride ions. Tests (*1*) have shown that epoxy coatings on reinforcement can reduce the rate of deterioration of concrete specimens containing high levels of chloride. However, corrosion did develop at faults in the rebar coating.

In practice, there will be some defects in epoxy-coated bridge reinforcement, i.e., pinholes in the coating when it leaves the factory, and damage during transport to the site, placement of the reinforcement, and pouring and vibrating the concrete (*2*). Such damage should be repaired, but locating all of the faults is difficult and very time consuming. The after-production cut-ends and damaged areas are coated with repair-epoxy rather than the fusion bonded material. In addition, bent bars, although visually undamaged might have lower corrosion resistance than straight bars (*3*). The long term durability of structures employing epoxy-coated reinforcement will depend on the progress of corrosion at defects. If the number of defects is limited and corrosion does not spread beneath the coating, then long term performance should be possible.

The Transport Research Laboratory (TRL) in the United Kingdom (UK) set up an exposure test to study the effect of defects and potential areas of reduced corrosion resistance on the performance of epoxy-coated reinforcement. The factors considered were repaired and unrepaired coating damage, and bent bars. The corrosion performance of epoxy-coated reinforcement with such defects was compared with uncoated reinforcement using test specimens exposed in an outdoor rural environment. To encourage corrosion salt was added to the concrete mix during casting of some specimens. Others were ponded with salt solution after exposure. The epoxy-coated reinforcement used was produced commercially and in compliance with the recently developed British Standard (*4*). Although epoxy-coated reinforcement has been used in bridges in North America since 1973, it has seen little use so far in bridges in the UK. The first UK manufacturer commenced production in 1987 which encouraged the development of the British Standard.

This report deals with the results after the first 2 years of the exposure tests. Additional specimens remain on exposure for future examination.

EXPOSURE TESTS

Concrete Test Specimens

Two shapes of concrete test specimen were used - beams and slabs, see Figure 1. The beam samples were for visual assessment of performance and the slab specimens to allow for electrochemical measurements, such as half-cell potential and galvanic currents. The concrete mix used for all specimens consisted of 330 kg (728 lbs) of Portland Cement, 980 kg (2,161 lbs) of Thames Valley Aggregate and 802 kg (1,768 lbs) of medium sand. The water/cement ratio was 0.58. The mean 28 day cube strength was 42 MPa (6,100 psi). Chloride additions,

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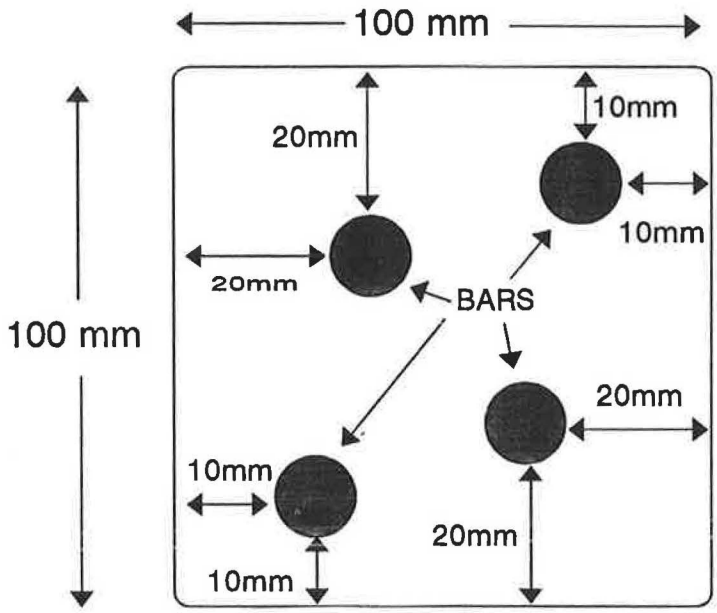
CROSS SECTION OF BEAM



PLAN VIEW OF SLAB

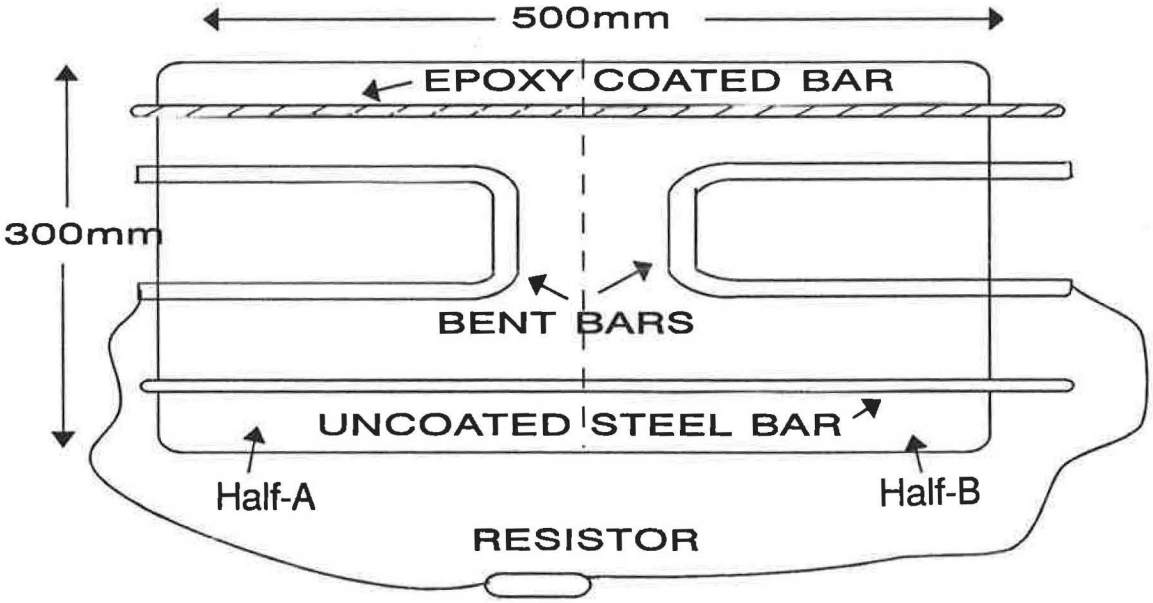


Figure 1 Prism and slab test specimens.

where made, were 3.2% by weight of cement. All the epoxy-coated reinforcement was commercially produced in the UK.

Beam specimens. Each concrete beam was 400 x 100 x 100 mm (15.75 x 3.93 x 3.93 in) and contained four 10 mm (0.39 in) diameter reinforcing bars. The bars were positioned and electrically isolated using plastic spacers with two bars having a minimum of 10 mm (0.39 in) cover and two bars having 20 mm (0.79 in) cover. Bars in a given beam were either epoxy-coated or uncoated. Where bars were epoxy-coated, one bar at each cover depth was damaged at four specific positions using an Ericsohn Paint Borer to make a small hole of about 1 mm (0.04 in) diameter in the coating. In addition, one of the cut-ends of the epoxy bar was left bare. The other bar of each pair was embedded with both cut-ends coated with standard twin pack epoxy repair material supplied by the coating manufacturer. Sets of beam specimens were cast both with and without salt added to the concrete mix. The numbers of each type of specimen are given in Table I.

Slab specimens. Each slab specimen was 500 x 300 x 60 mm (19.69 x 11.81 x 2.36 in) and contained one straight epoxy-coated bar, one straight uncoated bar, and two bent bars either coated, uncoated or one of each. The minimum concrete cover was 20 mm (0.79 in). Some specimens had salt added to the concrete mix. Others were cast in two halves so one of the bent bars was in chloride contaminated concrete while the other was in chloride free concrete. A lip was cast in the top of all the slab specimens to allow half of some initially chloride free slabs to be ponded with salt solution. The number of specimens for each combination of bent bar, cast-in chloride or ponded chloride is given in Table II. All bars protruded from the edge of the specimens for electrochemical measurement connections. The protruding sections of bar and the concrete edges were coated to prevent corrosion of the exposed steel. The bent bars within each specimen were permanently connected through an external resistor, see Figure 1. Any current flow that developed between the two bars could be measured from the potential drop across the resistor. The resistors used varied between 0.1 and 10k Ω (Ohms) and were chosen to be low when compared with the internal resistance of the galvanic cell created by combining the two bent bars.

Table I NUMBER OF BEAM SPECIMENS FOR EACH CHLORIDE CONTENT AND BAR TYPE

Chloride Content, % (Cl^- by weight cement)	Number of Specimens	
	Uncoated Bars	Coated Bars
0	10	10
3.2	10	20

Exposure Conditions and Routine Monitoring

Both beam and slab specimens were placed outdoors at TRL, a rural site in Southern England with low levels of atmospheric sulphur compounds and chlorides. Half-A of four out of the original five slab specimens from each chloride free set were ponded with a 3% solution of sodium chloride weekly. The salt solution was contained on Half-A of each specimen by a ponding lip. No attempt was made to maintain the chloride solution on the surface during the week. Depending on the weather conditions the solution evaporated leaving salt crystals on the surface or was diluted by rain. Water present in the ponding depression was removed before the chloride solution was poured in each week. Slab specimens with cast-in chlorides and the beam specimens were not ponded with chloride solution.

Both slab and beam specimens were visually inspected every three months for signs of cracking and rust staining. On the slab specimens, measurements of half-cell potential were made on the straight bars using a silver/silver chloride reference cell positioned over Half-A (the chloride contaminated or ponded section). Current flow between the bent bars was also measured.

Destructive Examination

At 1 and 2 years, some beam and slab specimens from each of the individual sets were opened and the reinforcing bars removed for detailed examination. Ten beam specimens (four from the set with epoxy coatings and cast-in chloride and two from each of the other three combinations, see Table I) were destructively examined along with one slab specimen from each of the sets given in Table II. The bars were examined for the type and extent of corrosion present. In some cases the epoxy coating was stripped from sections of the bar to check for underfilm corrosion. Stripping was carried out either mechanically or by soaking the bars in methylene chloride.

Table II NUMBER OF SLAB SPECIMENS FOR EACH COMBINATION OF CHLORIDE CONTENT AND BENT BAR COMBINATION

Location of Cast-In Chloride[a]	Number of Specimens For Each Bent Bar Combination (combination shown as Half A - Half B)			
	Epoxy-Epoxy	Steel-Steel	Epoxy-Steel	Steel-Epoxy
None[b]	5	5	5	5
Half-A	5	5	5	5
Both Half- A & B	5	5	5	-

a 3.2% by weight of cement.
b Four specimens from each set were ponded with salt solution on Half-A, see Figure 1.

RESULTS

Cracking and Rust Staining

During the first two years outward signs of deterioration on the beams containing epoxy-coated reinforcement were apparent on only one chloride contaminated specimen with a single fine crack and minor rust staining. In contrast, the chloride contaminated beam specimens with uncoated bar rapidly developed rust staining and significant levels of cracking. After one year, this was apparent above bars with 10 mm (0.39 in) cover and after two years was present above bars with 20 mm (0.79 in) cover. At two years, all 10 specimens with cast-in chloride were showing rust staining and fine cracking. More extensive cracking was present in six of these specimens. After two years, there was very little outward sign of deterioration on any of the beams containing uncoated bar without chloride in the mix.

For the slab specimens rust staining and cracking developed above some uncoated steel in specimens with cast-in chloride within one year. No staining or cracking was apparent above the epoxy-coated steel during the first two years.

Electrical Measurements On Slab Specimens

Half-cell potential measurements on chloride free slabs were between +50 mV and -300 mV (Ag/AgCl reference cell) for both coated and uncoated bars. On slabs with cast-in chloride, potentials were more negative for uncoated bars and showed wide variation with time for coated bars. For slabs ponded with chloride, potentials of both coated and uncoated straight bars became more negative as time passed but, as in the specimens with cast-in chloride, there was more variation in the potentials measured on the coated bars. Typical results from single slabs are shown in Figure 2.

Current measurements between the connected bent bars were made at three month intervals. Current was termed positive if the direction of flow agreed with expectations, i.e., if the bar expected to act as the anode did so. The largest currents were found in uncoated-uncoated bar combinations where only one bar was in chloride contaminated concrete. Where both bars were in chloride contaminated concrete currents were lower and in some cases changed direction as time passed. For coated bars where the coated bar was acting as the anode, the highest currents were measured for coated-uncoated bar combinations where the coated bar was in chloride contaminated concrete and the uncoated bar was in chloride free concrete. However, the magnitude of the current was lower than for the equivalent uncoated-uncoated combination. Average currents for all slabs in the uncoated-uncoated and coated-uncoated sets with cast-in chloride in one side only are shown in Figure 3. The maximum current measured for coated-uncoated bar combinations was about 1 μA, some 50 times lower than the maximum measured on uncoated-uncoated bar combinations.

Condition of the Reinforcement from Visual Inspection Before Removing the Epoxy Coating

Beam specimens. After removing the bars from the concrete, the extent of visible corrosion was assessed. Only the bars from the chloride contaminated beams showed signs of significant corrosion. At one year, up to 10% of the bar surface was corroded for uncoated bars with both 10 mm (0.39 in) and 20 mm (0.79 in) cover. At two years, the area of corrosion had increased for the 10 mm (0.39 in) cover bars, see Figure 4.

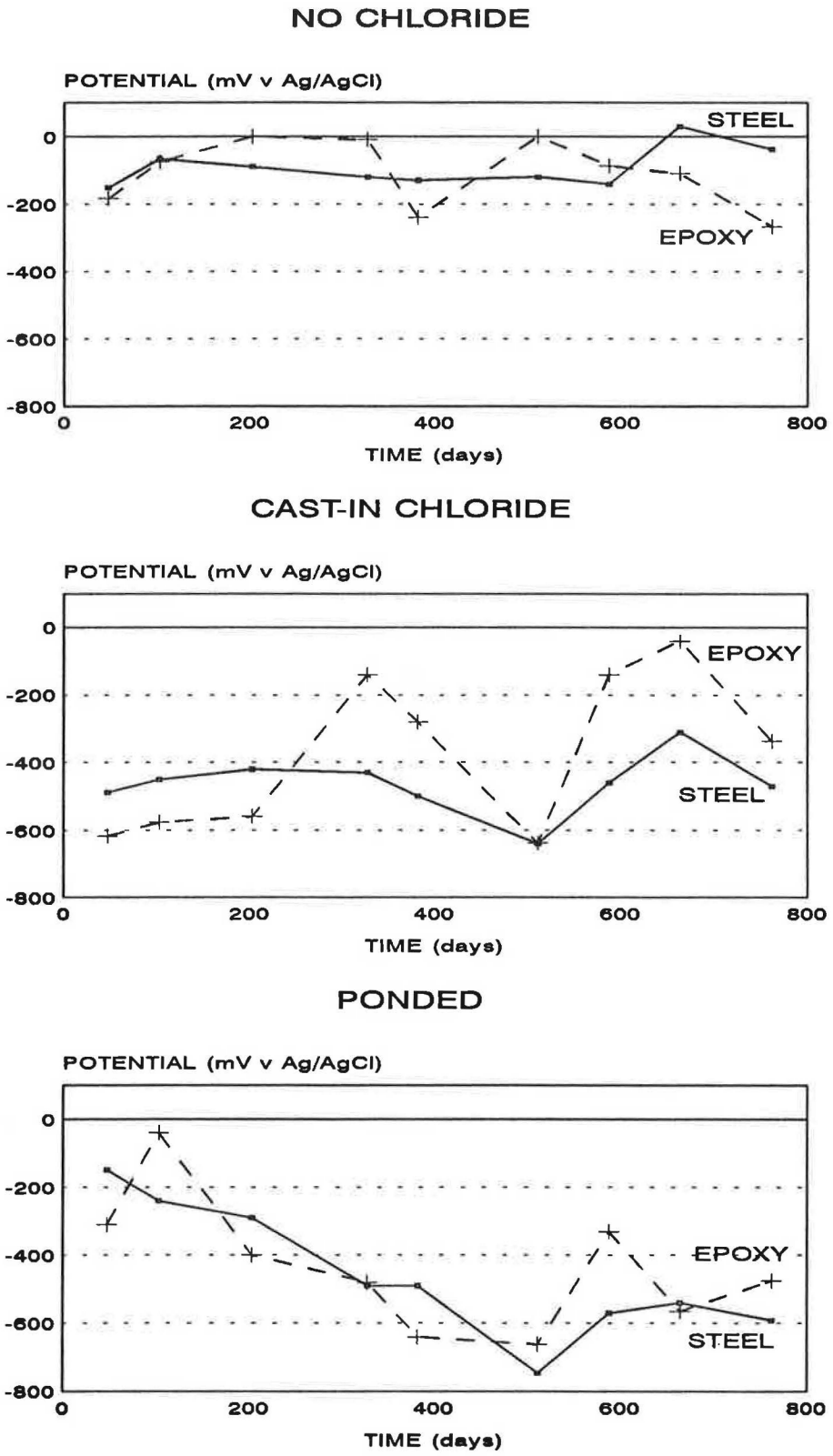


Figure 2 Typical half cell potential measurements on single slab specimens.

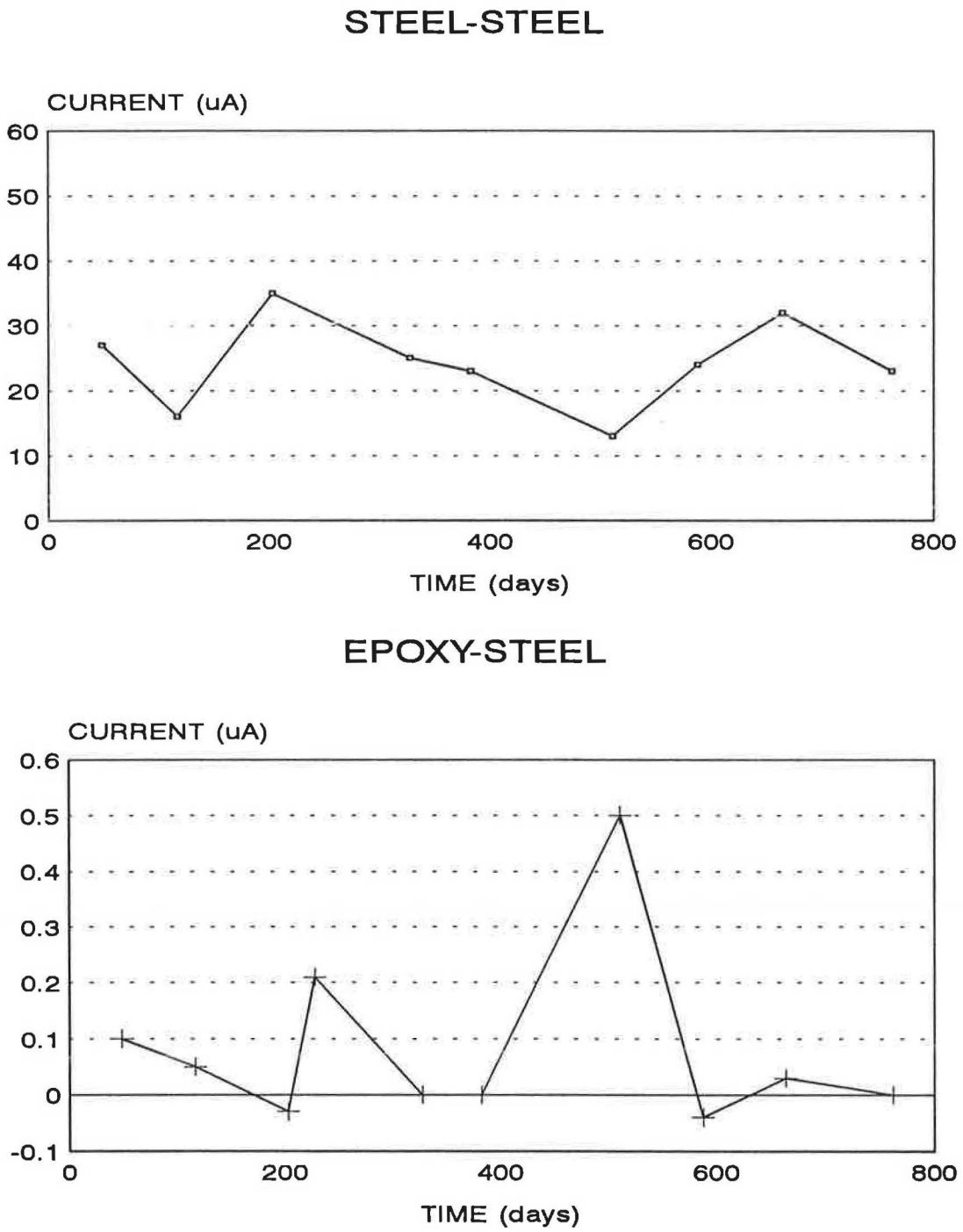


Figure 3 Average corrosion current measured between bent bars in slab specimens containing cast-in chloride in one side only. [First named bar is in the chloride side].

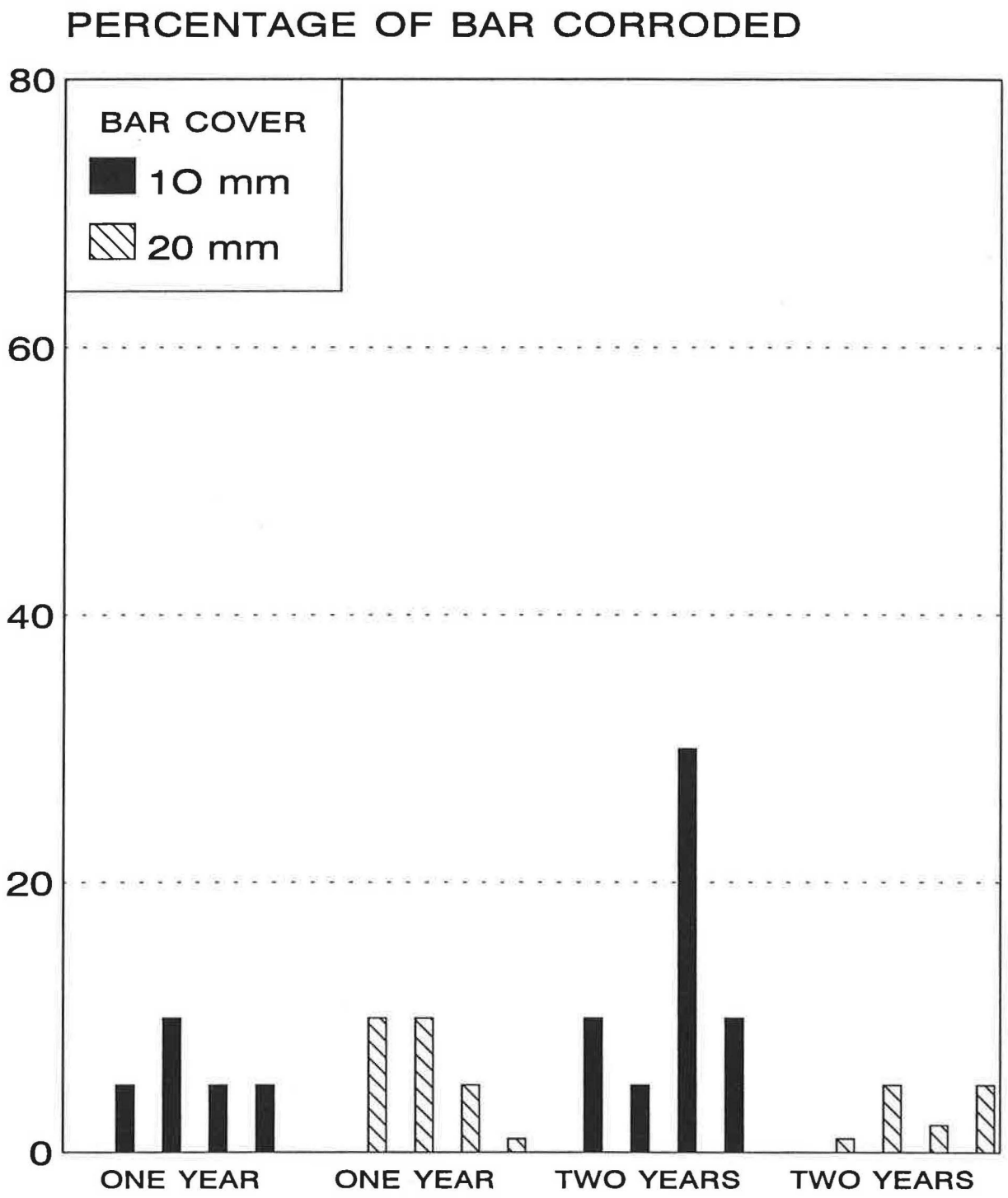


Figure 4 Percentage of surface corroded for uncoated bars removed from chloride contaminated beam specimens after one and two years exposure.

Epoxy-coated bars showed little sign of corrosion. There was rust spotting at some introduced holes after one year and after two years rust spotting at other positions was visible. However, this was all very slight and the unrepaired cut-ends remained rust free.

Slab specimens. The straight epoxy-coated bars used in the slab specimens were all visually undamaged when cast into the concrete. On removal after one and two years they appeared to have undergone very little deterioration. The maximum area of bar corroded was about 1% and this was found on only two of the 11 bars examined each year. The corrosion was light surface rusting with no loss in section of the bars. The uncoated bars however showed extensive corrosion, 50% of the bar in one case, see Figure 5.

The bent epoxy-coated bars were visually in good condition with no signs of corrosion after one year. Uncoated bent bars had up to 70% of the bar corroded after the same period, see Figure 6. After two years, four coated bars showed signs of corrosion over 1 to 2% of the bar. Uncoated bars showed higher levels of corrosion, see Figure 7. In some cases uncoated bent bars in non-chloride zones in the ponded and half chloride specimens had corroded. This might have resulted from diffusion of the chloride from the ponded or chloride half. The level of corrosion found on uncoated bars was more severe than on coated bar with noticeable loss in section in some bars.

Examination Of Epoxy-Coated Bars after Removal of the Coating

The epoxy coating was removed from around both repaired and unrepaired cut-ends on some epoxy-coated bars extracted from the beam specimens after two years. A scalpel blade was used to remove the coating. Although there was no visible signs of corrosion before the coating was removed, corrosion was found beneath one repaired cut-end. Underfilm corrosion was also found in regions adjacent to unrepaired cut-ends. Crevice corrosion was taking place with the uncoated cut-end acting as a cathode in the corrosion cell. In one case the corrosion had spread some 15 mm (0.59 in) from the cut-end. The corrosion consisted of light surface rust with no loss in bar section.

To enable more of bar to be examined chemical stripping of the epoxy coating was carried out on a selection of bars from both beam and slab specimens. This was done by soaking the bars overnight in methylene chloride. On straight bars with signs of corrosion at defects, underfilm corrosion was spreading from the defects. The extent of the spread was small - only a few millimeters (0.08 in) - and formed a light surface rust coating with no loss in bar section. There was an area of corrosion on a bent bar where there had been little sign of corrosion before the coating was removed. The corrosion covered some 15% of the bar surface, similar to that found on some uncoated bars. However, the corrosion beneath the coating was light surface rusting only.

Only a few bars were examined at this stage of the experiment and it is not appropriate to identify the underfilm corrosion as significant. More detailed assessments need to be conducted at the next destructive examination.

DISCUSSION

The main objective of this experiment was to assess the effect of defects on the corrosion performance of epoxy-coated reinforcement. The results have shown that corrosion starting at defects in the coating can spread beneath the epoxy coating. Corrosion can be active beneath repaired coating and beneath visually undamaged coating. It must be emphasized that the severity of such corrosion was much less than that found on uncoated bar. The underfilm corrosion was light surface rusting with no loss in section of the reinforcing bar while corrosion on the uncoated bar had led to loss in bar section. The measurements of corrosion current indicate that the rate of underfilm corrosion over the first 2 years was low and there was no peeling or blistering of the epoxy coating.

The results obtained to date highlight some difficulties in assessing the performance of epoxy-coated reinforcement. Concrete specimens containing epoxy-coated reinforcement have shown less cracking than those containing uncoated bars, and corrosion on the coated bars has been less severe than on uncoated bar. Epoxy-coated reinforcement can reduce the deterioration caused by corrosion in chloride contaminated concrete. However, will the low rate of corrosion continue as time passes? Future destructive examinations of the specimens should provide the answer. If the coating debonds, the corrosion rate might rise to the levels found on uncoated bar. The consequences of this will depend on the extent of the disbondment and how this influenced the serviceability and strength of the structure.

The question also arises as to how the extent and severity of corrosion in epoxy-coated reinforcement can be detected in in-service structures since corrosion is initially likely to be localized without signs of stain, cracking or spalling. Half-cell potential mapping is one

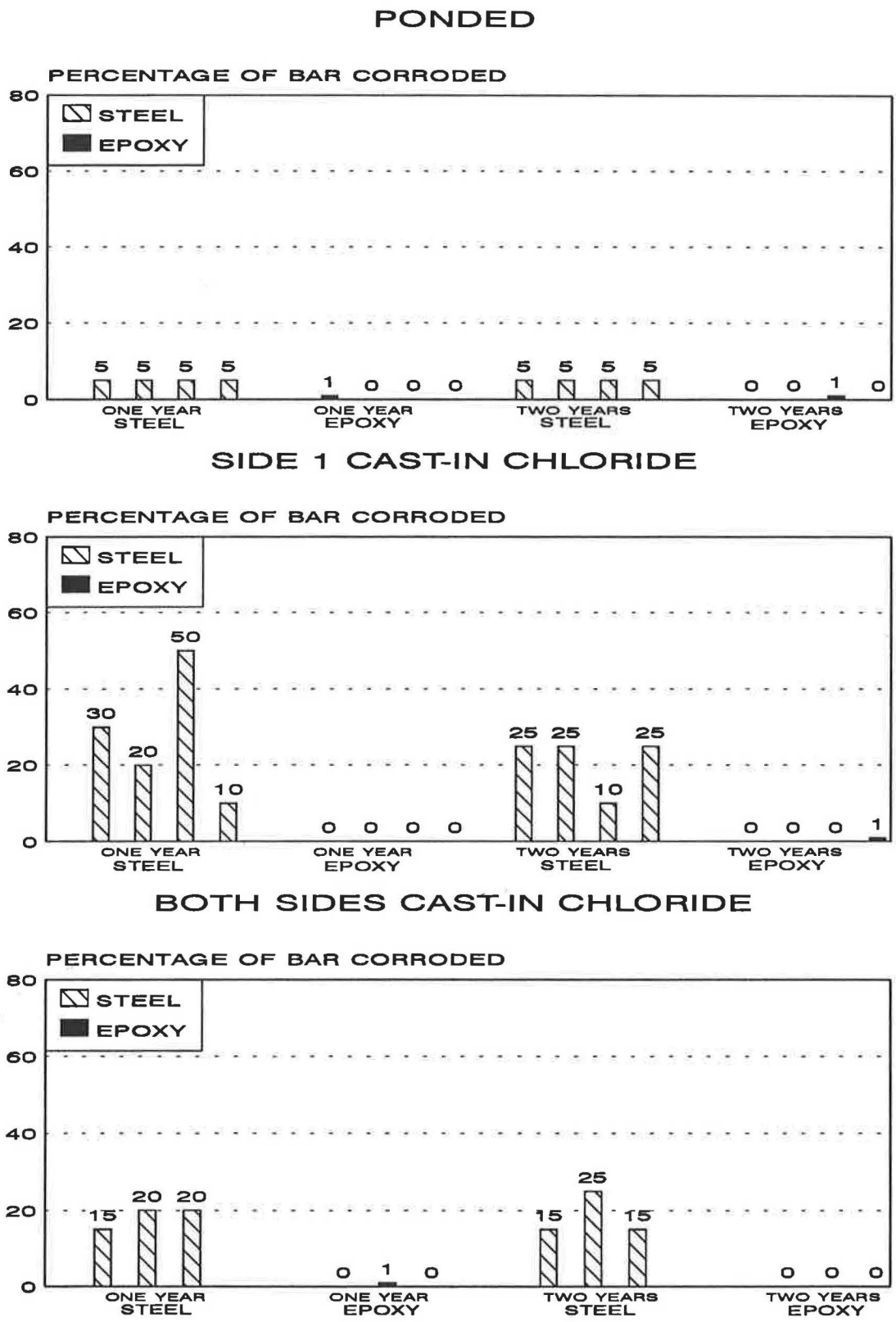


Figure 5 Percentage of surface corroded for straight bars removed from slab specimens after one and two years. [Numbers within each figure show the actual value].

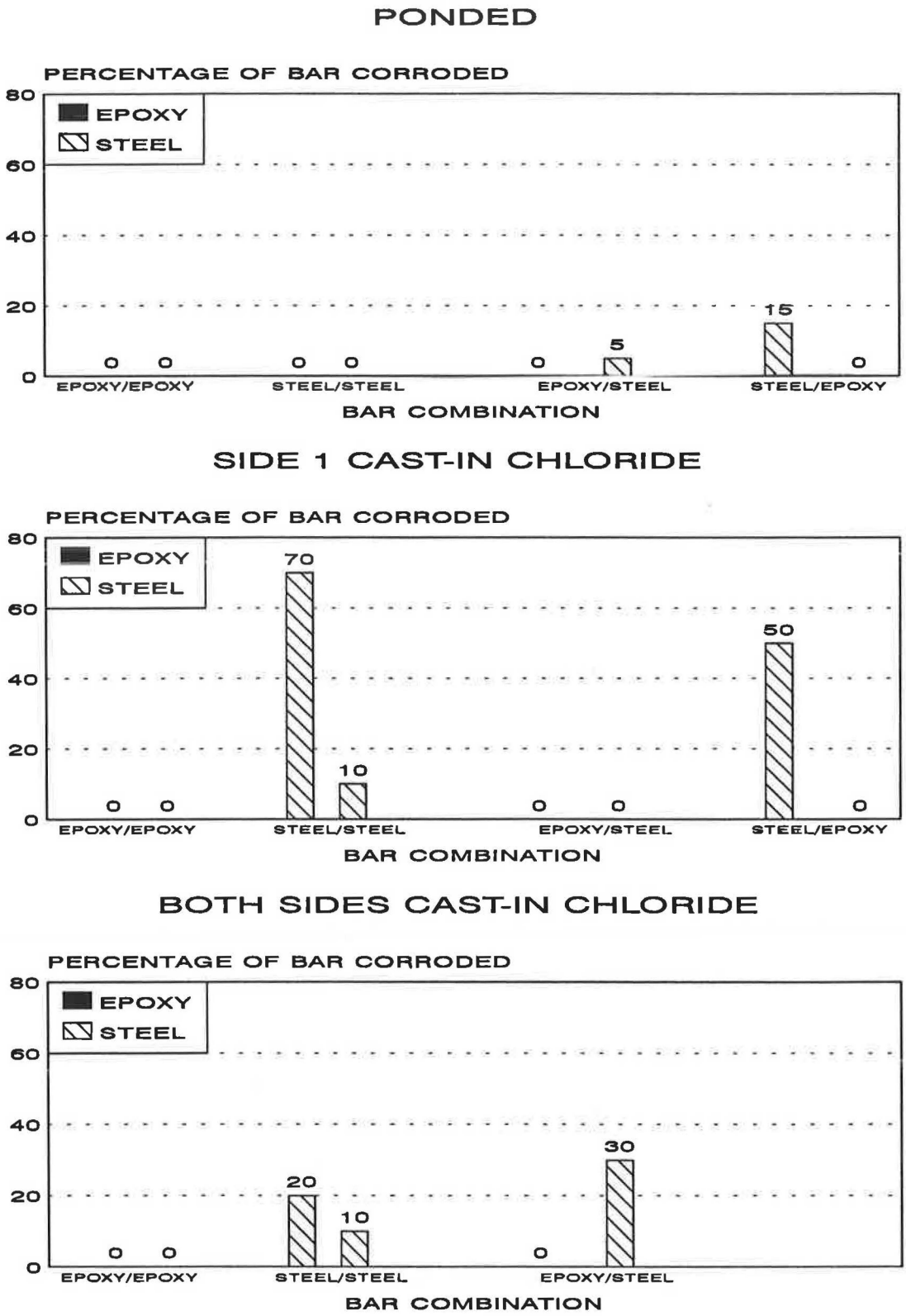


Figure 6 Percentage of surface corroded on bent bars removed from slab specimens after one year. [Where appropriate, the first named bar of the combination was in the chloride side].

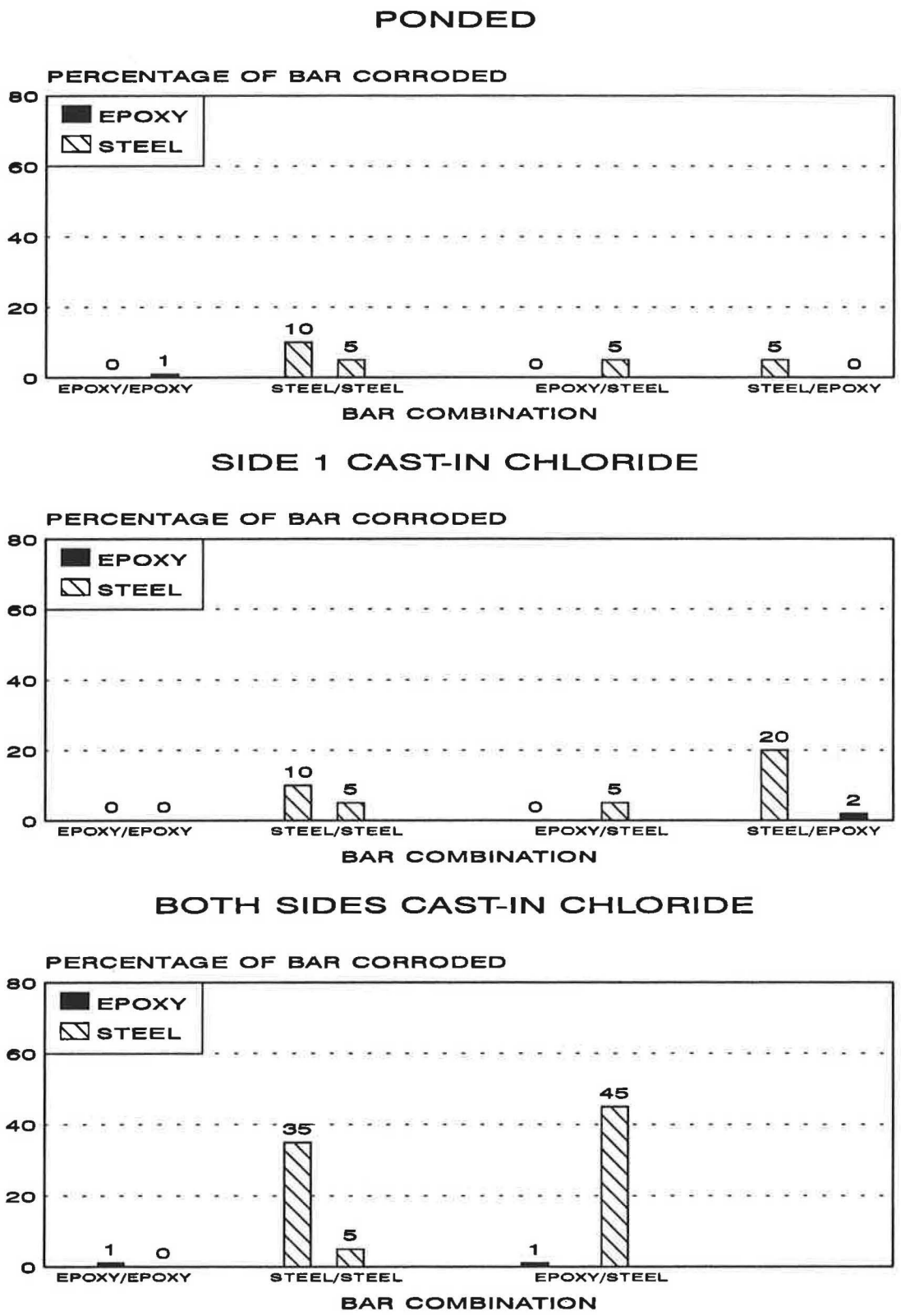


Figure 7 Percentage of surface corroded on bent bars removed from slab specimens after two years. [Where appropriate, the first named bar of the combination was in the chloride side].

of the standard methods of testing in-situ reinforcement for early signs of corrosion. There are practical difficulties in using this technique on epoxy-coated reinforcement because a separate electrical connection needs to be made to each bar of interest. For in-service structures, it is worth considering making electrical connections to bars in critical areas during construction to facilitate the use of this or other electrical techniques. Measurements of half-cell potential made during this test showed that results on epoxy-coated reinforcement can be more variable than on uncoated steel. This makes interpretation difficult from a single set of measurements. If this technique is to be used, it is necessary to conduct periodic surveys to detect significant changes in potential.

CONCLUSIONS

The extent of concrete cracking and the severity of reinforcement corrosion were reduced for epoxy-coated reinforcement in chloride contaminated test specimens in comparison with specimens containing uncoated reinforcement over a two year period. However, corrosion did spread under the coating from defects in the epoxy. Corrosion was also detected beneath repaired coating and beneath the coating on bent bars. The corrosion found beneath the epoxy coating was light surface rusting without loss in bar section, or peeling or blistering of the coating. During the same period, there was noticeable loss in bar section due to corrosion of the uncoated bar. Half-cell potential measurements on epoxy-coated reinforcement were more variable than on uncoated steel.

ACKNOWLEDGEMENTS

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EPOXY-COATED REBARS IN EUROPE: RESEARCH PROJECTS, REQUIREMENTS AND USE

Peter Schiessl and Carola Reuter*

The first European pilot construction projects with epoxy-coated reinforcing steel have been completed, some fifteen years after the first applications in North America. Epoxy-coated rebars were used for the first time on a large scale in Denmark, 1990, as part of the Storebaelt Tunnel project and on some other projects in Switzerland, Germany and Great Britain. Initial tests in Germany in 1983 identified the conditions under which epoxy-coated rebars (ECR) should be used. These test results were used as the basis for a German guideline on the use of epoxy-coated rebars. The requirements in these guidelines are similar to those in the ASTM A 775 Standard, but are generally more stringent. Limited preliminary tests investigating the behavior of damaged epoxy-coated rebars in different chemical environments with cathodic and anodic polarization indicate the need to examine the causes of unexpected material performance problems such as disbonding and under-rusting. Additional tests are needed to obtain a more accurate view of the mechanisms involved and their correlation with decisive influencing parameters in the long-term performance of ECR in aggressive environments.

INTRODUCTION

In 1983 the first test in Europe to examine the suitability of reinforcing steel coatings as a corrosion protective measure were initiated at the Institute for Reinforcing Steel and Reinforced Concrete Structures (IBS) in Munich, Germany. Some 40 different coatings were examined using standard test methods employed in Europe and the North America in the pipe-coating and rebar coating field. The fusion-bonded epoxy-coatings achieved the best performance with respect to corrosion protection of rebars.

In the early 1980's tests with epoxy-coated and PVC-coated rebars were performed at the University of Stuttgart. In the mid-1980's, tests with different powders were started at the Institut für Bauforschung (Building Research Institute - IBAC) of the RWTH Aachen. These tests were conducted in cooperation with powder coating manufacturers to provide coated reinforcing steels with improved quality characteristics for the German market.

In 1987 an independent German expert group was formed at the Institut für Bautechnik in Berlin (Institute for Building Technology (IFBT) -- the national authority responsible for approval of building materials not covered by national standards) to develop technical guidelines for the production and use of epoxy-coated rebars (ECR). The technical guidelines developed by this expert group are based on a review of the literature in Europe and North America, and rebar testing. The basic finding of this group was that the requirements for ECR should be the highest possible. Their test results showed that high powder quality and adhesion of the coating film to the steel surface were the most important parameters for corrosion protection. Adhesion is dependent primarily on the cleanness of the steel surface, i.e., effectiveness of the blasting procedure. Based on these tests, three epoxy resins and one PVC-system have been certified by the IFBT. In Germany, two coating companies that produce welded wire meshes and steel bars have IFBT certification. The PVC-system was never produced and marketed.

Similar studies were begun in the 1980's in other European countries, including Great Britain, the Netherlands and Norway. Coating plants for rebars have been constructed in Norway, Great Britain and Switzerland.

In the late 1980's discussions between Swiss, Dutch and German experts were initiated to harmonize guidelines and standards for ECR. The Swiss and German guidelines have been published and are very similar. The Dutch guidelines have just recently been published. All of the guidelines are similar to those in ASTM A 775 Standard with generally stricter requirements.

REQUIREMENTS FOR COATED REBARS AND COMPARISON OF GUIDELINES

The design engineer should use the same design and dimensioning characteristics for coated reinforcing steels as for uncoated steels. The organic coating should provide permanent protection against corrosion for the

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reinforcement. Table I compares the requirements imposed for ECR in various international guidelines or standards (*1*). The German and Swiss guidelines have been harmonized as a result of close cooperation between the two countries, and thus, the German requirements represent both in Table I.

The quality of adhesion of the coating to the steel surface is one of the dominating parameters with respect to the effectiveness of epoxy-coated rebars, and is dependent primarily on the cleanness of the steel surface (i.e., effectiveness of the grit blasting procedure). The roughness of the steel surface after blasting is another factor. Investigations have shown that the hot water test, immersion of coated rebars in 90°C (194°F) demineralized water, is an excellent test for quality of adhesion and to a certain extent the permeability of the coating film. This test is accepted in Germany and Switzerland as a quality criterion in the pipe coating business, and is one of the main quality control tests in the German guideline. Recent studies indicate that permeability is also an important property in long-term corrosion protection. Besides the hot water test, the cathodic disbonding test has been used to evaluate the quality of adhesion and the quality of application at the coating plant. This test is required in European guidelines/standards. The ASTM standards do not have a selective adhesion test requirement.

Requirements on other points shown in Table I, such as bendability, corrosion resistance, maximum permissible defect size and freedom from holidays are very stringent in the German guideline. The German regulations are also strict in terms of conditions of use, such as minimum spandrel diameter and permissible defects. The German and Swiss guidelines recommend exclusive use of coated reinforcement in any single structural component. Where coated and uncoated reinforcement are combined, suitable binder wires and spacers must be employed to exclude any electrically conductive contact. The Swiss guideline prescribes electrical resistance measurements between coated and uncoated bars where mixed reinforcing systems are used.

In Germany, an association for coated reinforcing steels has been founded with the objectives of assuring a quality standard that exceeds the requirements of the guidelines, and initiating research projects to extend scientific knowledge in coated reinforcing steels.

EXPERIENCE IN USE

European experience in the use of ECR in construction projects is limited. Some projects are presented below.

Spiez Bridge, Switzerland. This is the first civil engineering structure in Switzerland to incorporate epoxy-coated reinforcing steel in the barrier walls of the four-section bridge 57 m (187 ft) long by 11.8 m (38.7 ft) wide. The exposed bridge is subjected to extensive weathering and road salt. The project was completed in 1988 with close collaboration between the Bundesamt für Straßenbau (Federal Highway Construction Authority), the Institut für Baustoffe, Werkstoffchemie and Korrosion (Institute for Building Materials, Materials Chemistry and Corrosion) of the ETH Zürich, and a coating company. The objective of the project was to acquire experience in the handling and installation of epoxy-coated reinforcing steels and to test the long-term behavior of the coating and the rebars. For comparison, half of each of the two barrier walls was constructed with coated and half with uncoated reinforcement. Inspection areas, 0.5 to 5 cm^2 (0.1 to 0.8 in^2), were constructed at two different heights from the road deck on selected coated steel elements to allow monitoring of corrosion behavior. One half of the inspection area steel is insulated, and the other half is connected externally to the uncoated reinforcement of the road deck. This permits periodic measurement of the corrosion potential, the electrical resistance and the macro-element current. Up to now (October 1992), all coated-rebars are in the passive state, as is the uncoated reinforcement.

Cooling Towers for a Cell Re-cooling Plant in Ludwigshafen, Germany. The first large-scale use of epoxy-coated reinforcing steels in Germany was in the cooling towers of a cell re-cooling plant for the chemical industry. Roughly 18 t (20 tons) of coated reinforcing steel were used for the site-mixed concrete cover of the re-cooling works and the prefabricated shell elements of the diffusors. Since neither a national standard nor a suitability test was available for the epoxy resin powder, individual certification by the supervising authority IFBT, was necessary. Studies on the suitability of the powder for the specific application were conducted at the IBAC.

Reinforced Concrete Elements of the Storebaelt Tunnel, Denmark. The most extensive European pilot project for ECR is the 7.3 km (4.5 mi) railway tunnel beneath the Great Belt forming part of the rail connection between the Danish Islands of Funen and Zealand. The reinforcing steel for the concrete segments, approx-

TABLE I COMPARISON OF REQUIREMENTS FOR EPOXY-COATED REBAR IN DIFFERENT STANDARDS/GUIDELINES

REQUIREMENT	GERMAN GUIDELINE IFBT -RULE 05.90	ASTM A 775/90	BRITISH STANDARD 09.90
POWDER IDENTIFICATION	• DSC, TGA, IR	-	-
DIFFUSION*	• H_20-Diffusion • H_20-Absorption	• Chloride • Diffusion	- -
CHEMICAL RESISTANCE	• 10% NaOH • Bars Embedded in Concrete • Outdoor Exposure (Free)	• Distilled Water** • 3 M $CaCl_2$** • 3 M $Ca(0H)_2$**	• 3 M $CaCl_2$** • 3 M NaOH** • 3 M $Ca(0H)_2$**
ADHESION QUALITY	• Hot-Water Test • MIBK Test • Cathodic Disbonding	- - -	• Cut Test** - • Cathodic Disbonding
COATING THICKNESS	• 130 to 300 μm	• 130 to 300 μm	• 200 ± 50 μm
BENDABILITY d_{br} = mandrel diameter d_s = steel diameter	• Testing $d_{br} = 4\ d_s$, (d_s < 20 mm) $d_{br} = 6\ d_s$, ($d_s \geq 20$ mm) Additional Bending After 6 Months Outdoor Exposure • Application $d_{br} = 6\ d_s$, (d_s < 20 mm) $d_{br} = 8\ d_s$, ($d_s \geq 20$ mm) Note: T = +5°C	• Testing and Application $d_{br} = 8\ d_s$, (d_s < 43 mm) $d_{br} = 10\ d_s$, (d_s > 43 mm) Note: T = 20 to 30°C	• Testing and Application $d_{br} = 6\ d_s$ Note: T ≤ 15°C
HOLIDAY TEST	• Zero (0) Holidays: Acceptance Test for Powder and Coating Firms. • Maximum 6 Holidays/m: Run Production	Maximum 6 Holidays/m	Maximum 5 Holidays/m
DAMAGE	• < 25 mm^2 • < 0.5% : Manufacturer • < 1.0% : Building Site	< 60 mm^2 < 2%	< 10 mm^2/m and maximum 4 pieces/m
CORROSION TEST	• Salt-Spray Test • Cathodic Disbonding	• Current Test	• Salt-Spray test • Cathodic Disbonding
MECHANICAL DEFECTS	• Impact Test** • Free-Fall Test	• Impact Test**	• Impact Test**
BOND	• Pull-Out Test (Short and Long Time)	-	-
COATING TESTS	- -	• Vickers Hardness** • Abrasion Test**	• Vickers Hardness** • Abrasion Test**

* Tests on resin films.
** Tests on epoxy-coated steel plates.

imately 65,000 segments equal to 22,000 t (24,000 tons) of reinforcing steel, was coated on site in a coating plant using the fluidized bed dipping process. Following blasting and preheating, the welded reinforcing cages 4.20 x 1.65 x 0.40 m (13.78 x 5.41 x 1.31 ft) were coated with epoxy resin in a dip tank and passed through a cooling section. The cycle time for this fully-automated coating plant, including continuous quality control on coating thickness, freedom from holidays and curing was 3 to 4 minutes. The on-site production process minimized later bending, and hence, weakening of the coating film. The coated reinforcement is part of a multi-barrier protective system. Besides excellent concrete quality, alternative cathodic protection systems have been provided if the protective coating should prove insufficient.

Schießbergstraße Bridge in Leverkusen, Germany. This bridge, 53 m (174 ft) long by 9 m (30 ft) wide, was completed at the end of 1990 and is second largest in Germany. The cell cooling tower project is the largest. The coated reinforcing steel, 8 t (9 short tons), is located in the pier caps. One side was constructed with hot-coated reinforcing steel bars that were bent after coating and the other with cold-coated welded wire mesh bent before coating. Inspection areas were provided for observation of corrosion behavior. The corrosion behavior of any macro-element can be measured continuously via electrically conductive connections with uncoated reinforcing steels.

Epoxy-resin coated reinforcements have also been used on other European construction projects in Switzerland (sewage treatment plants & bridge extensions), the Netherlands (part of a port structure in Rotterdam) and in England (salt storage silo, bridges sea walls).

RESULTS OF RECENT LIMITED TESTING

Factors that may initiate and propagate corrosion of straight epoxy-coated rebars include the chemical environment of the rebar, e.g., before and after embedding in concrete, and damage to the coating. As damage is generally unavoidable during production and handling, it is accepted in the American and European standards for ECR. Electrochemical effects, such as cathodic disbondment, is very important in these circumstances.

To investigate the behavior of damaged epoxy-coated rebars under different chemical environments, limited tests were conducted at the IBAC in which epoxy-coated rebars were exposed to various liquid chemicals. Epoxy-coated rebars from a coater operating under ASTM requirements (Coater A) and a coater operating under the German guidelines (Coater B) were used. The test solutions chosen were:

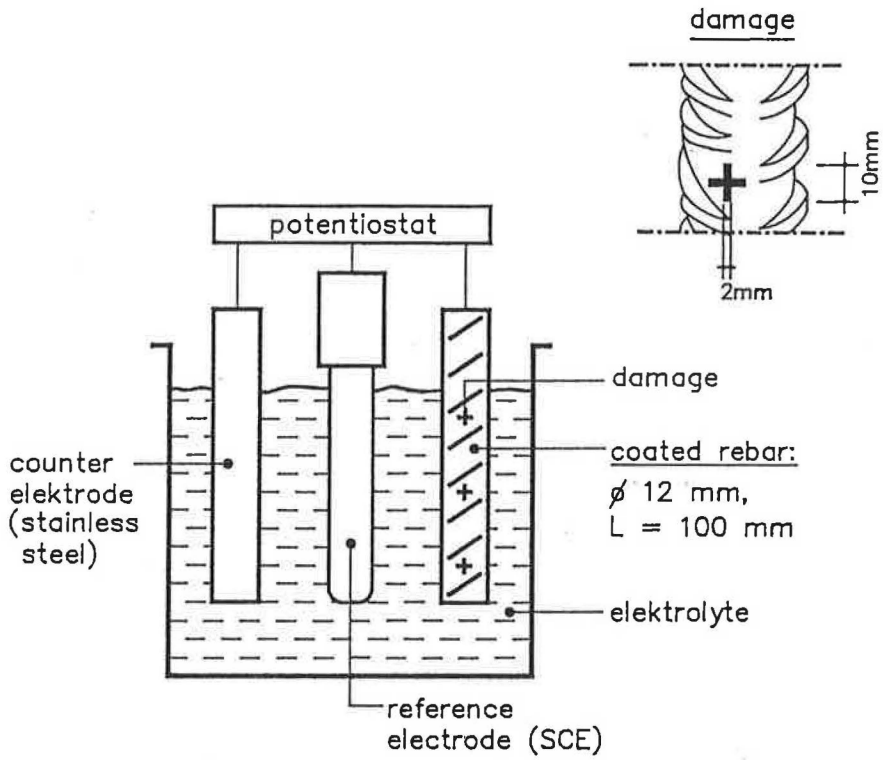
- NaCl,
- simulated pore water of concrete with KOH + NaOH, and
- a mixed solution of KOH + NaOH and NaCl.

The intent was to evaluate the respective effects of storing unbedded coated rebars in a marine environment (NaCl-solution), a chloride-free environment (KOH+NaOH-solution) and a chloride-containing concrete (KOH + NaOH + NaCl-solution). In addition to freely corroding conditions, open circuit (OC) potential, the rebars were tested under potentio-static control ranging from -1800 mV (cathodic polarization) to +100 mV (anodic polarization). A stainless steel counter electrode was used, see Figure 1. Three cuts, nominally 2 by 10 mm (0.1 by 0.4 in), were made in the coating of each bar with a knife. After exposure for 30 days, the specimens were dried and the area of coating that could be easily removed by sliding a knife was determined. Figures 2 to 4 show the amount of disbondment expressed as a fraction of the total surface of the bar in contact with the liquid.

Exposure to 3.5% NaCl-solution caused corrosion under anodic polarization at +100 mV in the coated bars from both producers, see Figure 2. The bar from Coater A showed pitting corrosion with metal loss in the form of circular craters in the steel. By comparison the bar from Coater B showed less corrosion pits, but a greater amount of coating disbondment immediately surrounding the pits. The bar from Coater A suffered disbondment at -1000 mV (cathodic polarization) while no disbondment was observed on the other bar.

Epoxy-coated bars exposed to an alkalinic solution of KOH + NaOH (pH=13.5) suffered considerable disbondment under cathodic polarization, see Figure 3. At -1800 mV the coating of the bar from Coater A could be removed completely, while the bar from Coater B showed a disbondment of 50%. Neither bars showed signs of corrosion under anodic polarization and OC potential.

Exposure to the mixed solution of KOH + NaOH and NaCl caused disbondment under cathodic and corrosion under anodic polarization in both bars. There was more disbondment and pitting corrosion on the bars from Coater A than on the other bars, see Figure 4.



Series	Polarisation	Potential (mV) (SCE)	Electrolyte
1	anodic open circuit cathodic	+ 100 − ~600 − 1000	3,5%NaCl
2	anodic open circuit cathodic cathodic	+ 100 − ~100 − 1000 − 1800	0,3nKOH+0,05nNaOH
3	anodic open circuit cathodic	+ 100 − ~500 − 1000	0,3nKOH+0,05nNaOH+3,5%NaCl

Figure 1 Test set-up (schematic) and test parameters.

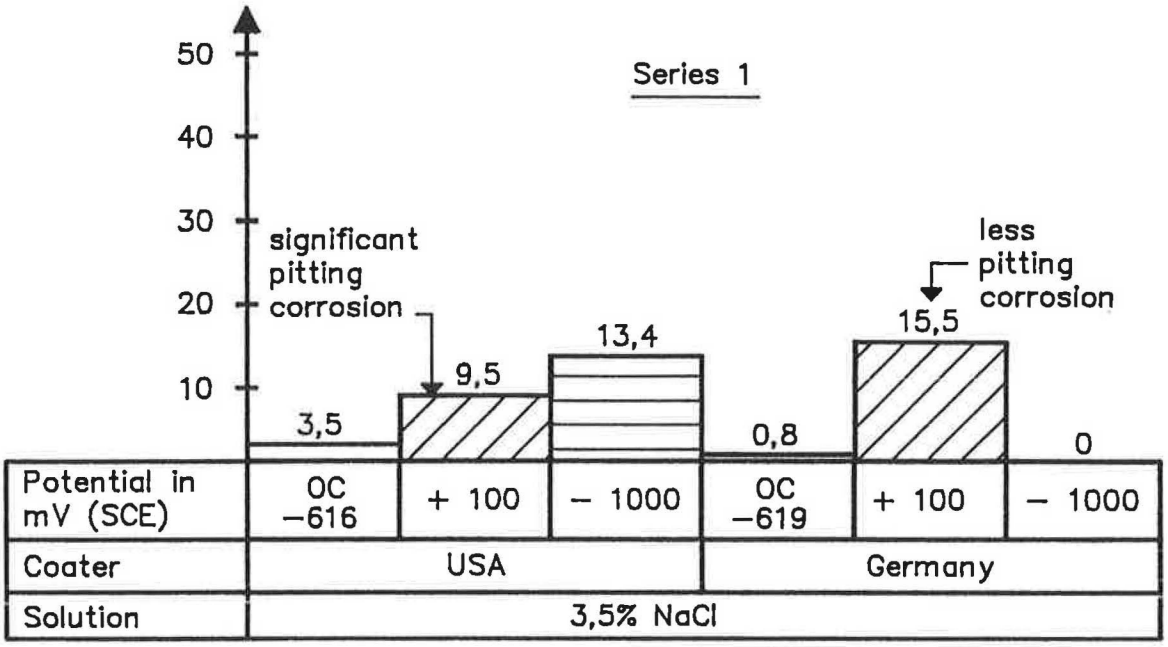


Figure 2 Disbondment (%) of coating after 30 days exposure in NaCl solution at different potentials.

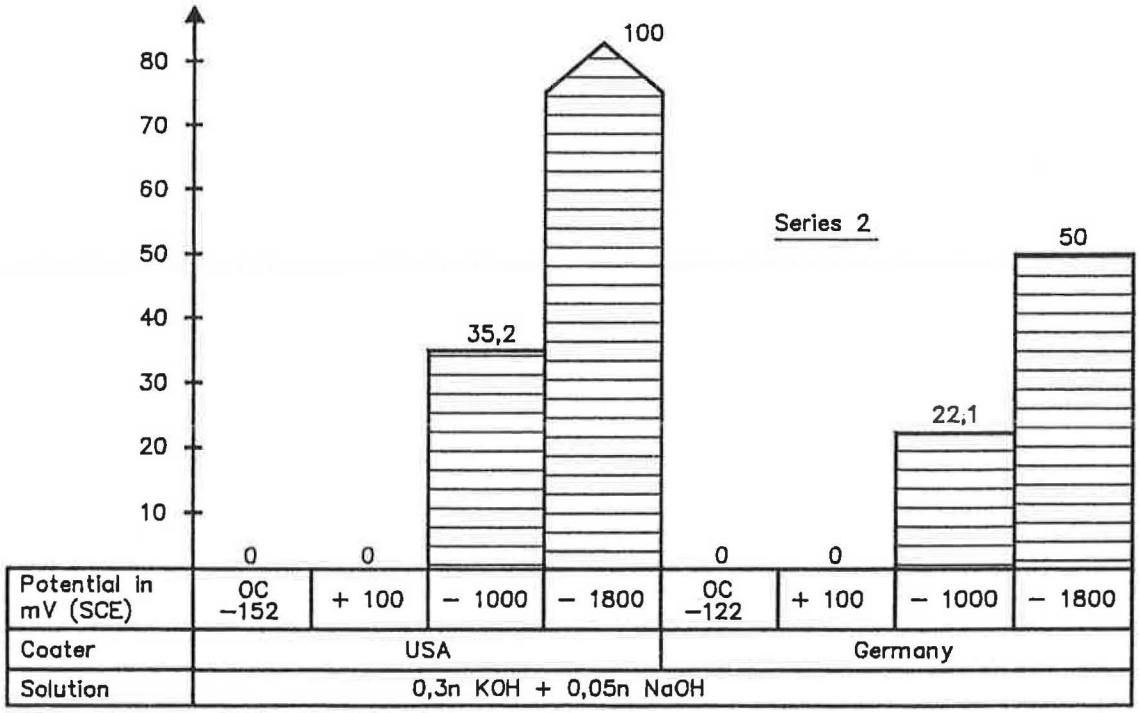


Figure 3 Disbondment (%) of coating after 30 days exposure in KOH + NaOH solution at different potentials.

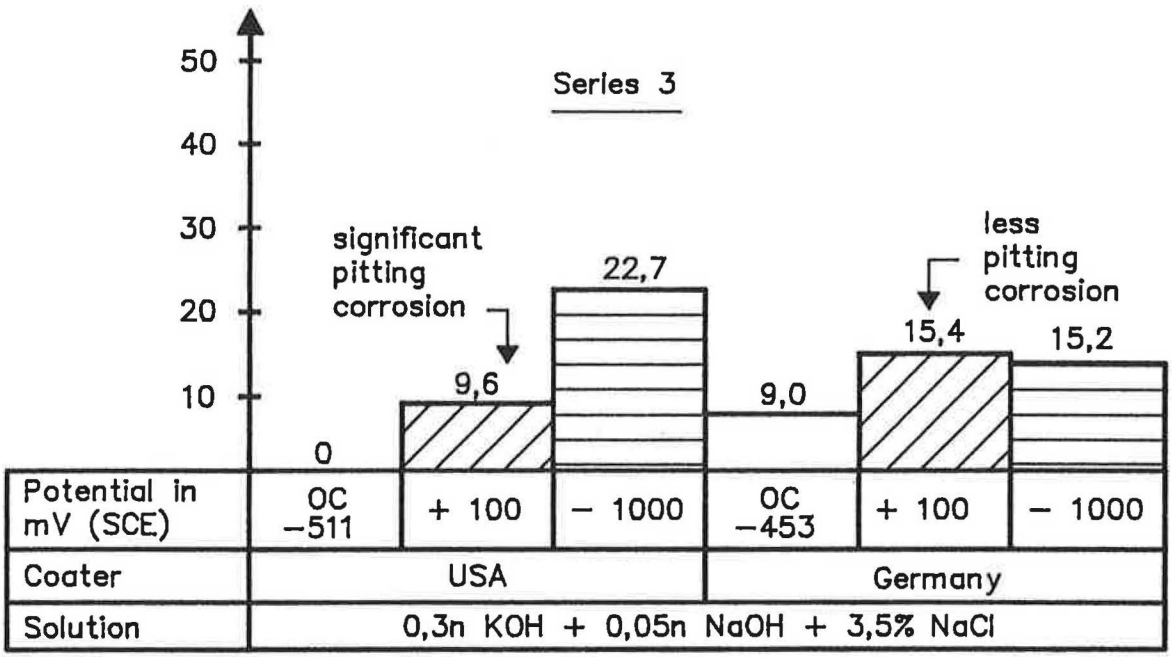


Figure 4 Disbondment (%) of coating after 30 days exposure in KOH + NaOH + NaCl solution at different potentials.

CONCLUSION

The preliminary investigation presented here, based on a single bar, represents an initial attempt to examine the causes of unexpected material performance problems such as disbonding and under-rusting. Additional tests based on a representative sample of test specimens are needed to obtain a more accurate view of the mechanisms involved and their correlation with decisive influencing parameters. Nevertheless, the laboratory experiments suggest that particular attention needs to be paid to the long-term performance of ECR in aggressive environments.

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CORROSION OF EPOXY-COATED REBAR IN A MARINE ENVIRONMENT

L. L. Smith, R. J. Kessler and R. G. Powers*

In 1986 the first signs of corrosion of epoxy-coated rebar were observed in the Long Key Bridge Substructure. Further investigations revealed extensive corrosion of epoxy-coated rebars in four of the five major structures in the Florida Keys. A Task Force was established to investigate the problem by conducting in house research, contract research and a statewide survey of bridges with epoxy-coated rebar substructures. As a result of the investigation it was concluded that epoxy-coated rebar is unsuitable for marine splash zone corrosion protection.

INTRODUCTION

In 1976 a technical group composed of members of the Florida Department of Transportation (FDOT) and the Federal Highway Administration (FHWA) established criteria for corrosion protection for bridges constructed in the Florida Keys. At that time the FHWA was investigating corrosion of reinforcing steel and suggested that epoxy-coated rebars be considered along with other protective measures on federally funded projects. The technical group's resolution was that epoxy-coated rebars should be used throughout the structures built in the Florida Keys since these structures would be located in a corrosive marine environment. The first use of epoxy-coated reinforcement in Florida was on the 17th Street Bridge over Indian River at Vero Beach in October 1977. Two other bridges followed, one at Destin over East Pass in December 1977, and the other at Ormond Beach over Halifax River in May 1981.

In 1981, the FHWA adopted epoxy-coated steel reinforcement as the primary means of corrosion control in bridge decks. As a result, epoxy-coated steel became the primary protection method for all bridge reinforcement in the State of Florida. The FDOT specification for epoxy-coated rebar was fashioned after prevailing ASTM and AASHTO standards. The specifications allow two holidays per 30.5 cm (2/ft) at the production line and 2% damage per 30.5 cm (2%/ft) at the construction site.

Inspection of epoxy coating by the FDOT has always included an initial plant approval and an approved quality control plan. The majority of all epoxy-coated rebars used on FDOT projects was shop inspected by FDOT personnel or by commercial testing laboratories. Job site control has been accomplished through visual examination by construction personnel. Since 1986, FDOT has performed full-time in line inspections at a local epoxy-coated rebar facility to insure compliance with existing specifications. The FDOT has always followed and implemented the national standards and recommendations for the manufacture control and use of epoxy-coated rebar. In 1986, however, the substructure of the Long Key Bridge in the Florida Keys began to show signs of corrosion only six years after construction *(1)*.

KEYS BRIDGES FINDINGS

An investigation was conducted on seven structures in the Florida Keys. Five of these are major structures with lengths greater than 610 m (2,000 ft) *(2,3)*. Significant corrosion of the epoxy-coated rebars was found in four of the five major bridge substructures. The corrosion found in these four bridges was limited to 0.6 to 2.4 m (2 to 8 ft) above the mean high water mark on the substructure (marine splash zone). The bridges and approximate number of piers exhibiting corrosion are shown in Tables I and II.

Corrosion was found in both fabricated and straight epoxy-coated rebar, Figure 1. Visual inspection indicated that corrosion may have started in the fabricated rebars and then progressed to the straight bars. Underneath the coating, water with a pH of 5 was commonly found, indicating that acidic conditions had developed. Cores taken from the structures on opposite sides from the corrosion spalls indicate that corrosion had progressed beyond the spalled area into sound

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Table I THE FIVE MAJOR FLORIDA KEYS BRIDGES

Bridge	Construction Period	No. of Piers In Water	First Signs of Corrosion
Long Key	1979-1981	101	1986
Seven Mile	1979-1982	263	1988
Niles Channel	1981-1983	37	1988
Channel Five	1981-1983	34	None
Indian Key	1979-1981	18	1990

concrete.

The spalled areas were larger than those previously experienced with bare rebar. For example, spalls as large as 0.6 x 1.2 m (2 x 4 ft) have been observed in locations where corrosion was confined to a length less than 30 cm (12 in) of rebar. Some spalls occurred in areas deficient in concrete clear cover and where the concrete components segregated during construction. However, the majority of the corrosion was observed in locations containing sound concrete with up to 10 cm (4 in) of cover. Large delaminations of concrete were observed in areas without visible cracking, indicating advanced stages of corrosion of epoxy-coated rebar.

Determining the initial condition and the original quality of the epoxy coating was difficult. Samples of epoxy-coated rebar were extracted from 2.4 m (8 ft) above the mean high water mark for investigation. Inspection of these samples indicated complete coating disbondment from the substrate. The coating itself appeared to be of adequate quality. Coated rebar samples taken from the Long Key Bridge and analyzed by a producer of epoxy powder were said to be of "good quality." Damage of the epoxy-coated rebar during construction could not be quantified, but appeared to be less than the 2% allowed by specifications.

Table II PROGRESSION OF CORROSION IN FOUR KEYS BRIDGES BY NUMBER OF PIERS AFFECTED

Bridge	1986	1987	1988	1989	1990	1991
Long Key	1	3	17	--	31	31
Seven Mile	--	--	8	--	58	60
Niles Channel	--	--	17	*	17**	17
Indian Key	--	--	--	--	2	2

* All corrosion spalls repaired.
** New corrosion spalls.

RESULTING INVESTIGATIONS

On February 1, 1990 the Department established a Task Force to investigate the epoxy-coated rebar problem statewide. The objectives of the Task Force were:

- Determine the extent of the problem.
- Define causes.
- Define short term solutions.
- Develop long term solutions.
- Develop standards and design criteria.

This report discusses the first two objectives. Discussions on the other objectives can be found elsewhere *(4,5,6,7,8)*. To accomplish the above objectives, in-house research, contract research and a statewide survey were initiated.

IN-HOUSE RESEARCH

In-house research consisted of 1) a laboratory study to evaluate fabrication and damage effects, 2) a field study of test piling, and 3) documentation of the life cycle of an epoxy-coated rebar from coating plant to construction site.

Fabrication And Damage Effects

Fabricated bare, pre-fabricated epoxy-coated and post-fabricated epoxy-coated rebar samples were cast into structural quality concrete and exposed in a high chloride solution for 30 months *(9)*. The samples were characterized according to producer and coating quality. Quality was based on the performance of the coating in the standard bend test. Resistance measurements, voltage potential and visual inspection were used to evaluate the performance of the coating for each specimen.

Figure 1 Typical corrosion deterioration common to Seven Mile Bridge observed in 1987.

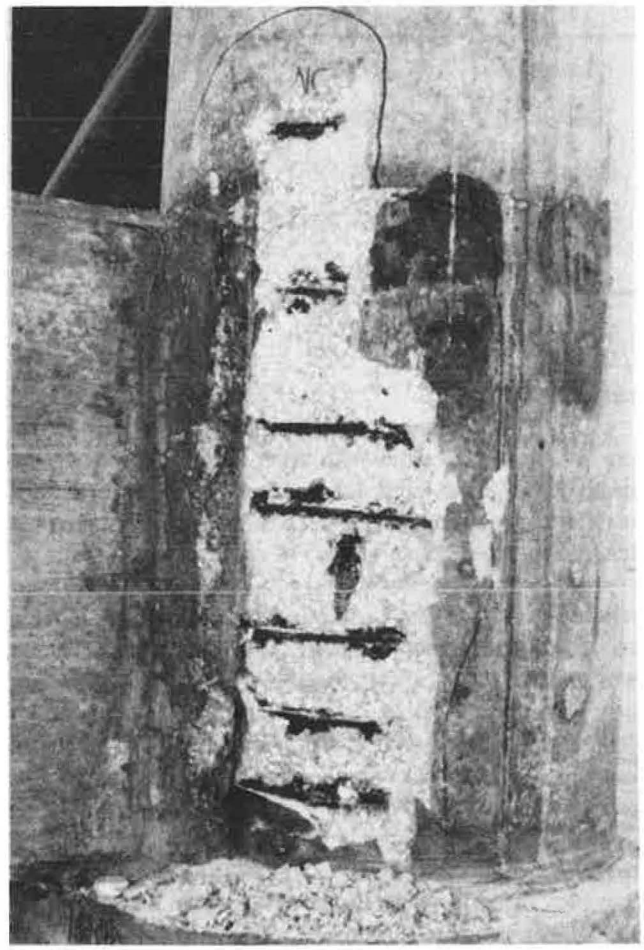

Figure 2 Typical corrosion deterioration common to Niles Channel Bridge observed in 1987.

The results of this investigation showed that coating after fabrication did not significantly improve corrosion resistance. Disbondment occurred in "perfect" condition bars and in the bars coated after fabrication, Figure 2. Coating disbondment can lead to serious problems since the initial passivating effect of the concrete is isolated from the rebar. The disbondment of the epoxy allows the chloride laden electrolyte to reach and progress along the rebar surface. The corrosion observed in these tests was not as severe as noted in the Florida Keys structures. This is believed to be due to the sample configuration that allowed a large anode to small cathode ratio and the short exposure period.

Field Study At Matanzas Inlet

A test site established at Matanzas Inlet (Intracoastal Waterway) on the east coast of Florida was used to evaluate epoxy-coated and bare rebar test piles *(9)*. The test piles were installed in 1979 in the corrosive marine environment, Figure 3. After approximately nine years of exposure, three test piles were removed for examination, one with epoxy-coated steel rebars, one with bare rebars and one with galvanized steel. The epoxy-coated rebar pile outperformed the bare rebar piling. However, the epoxy-coated rebars used in this study were not typical of those supplied to construction projects. The

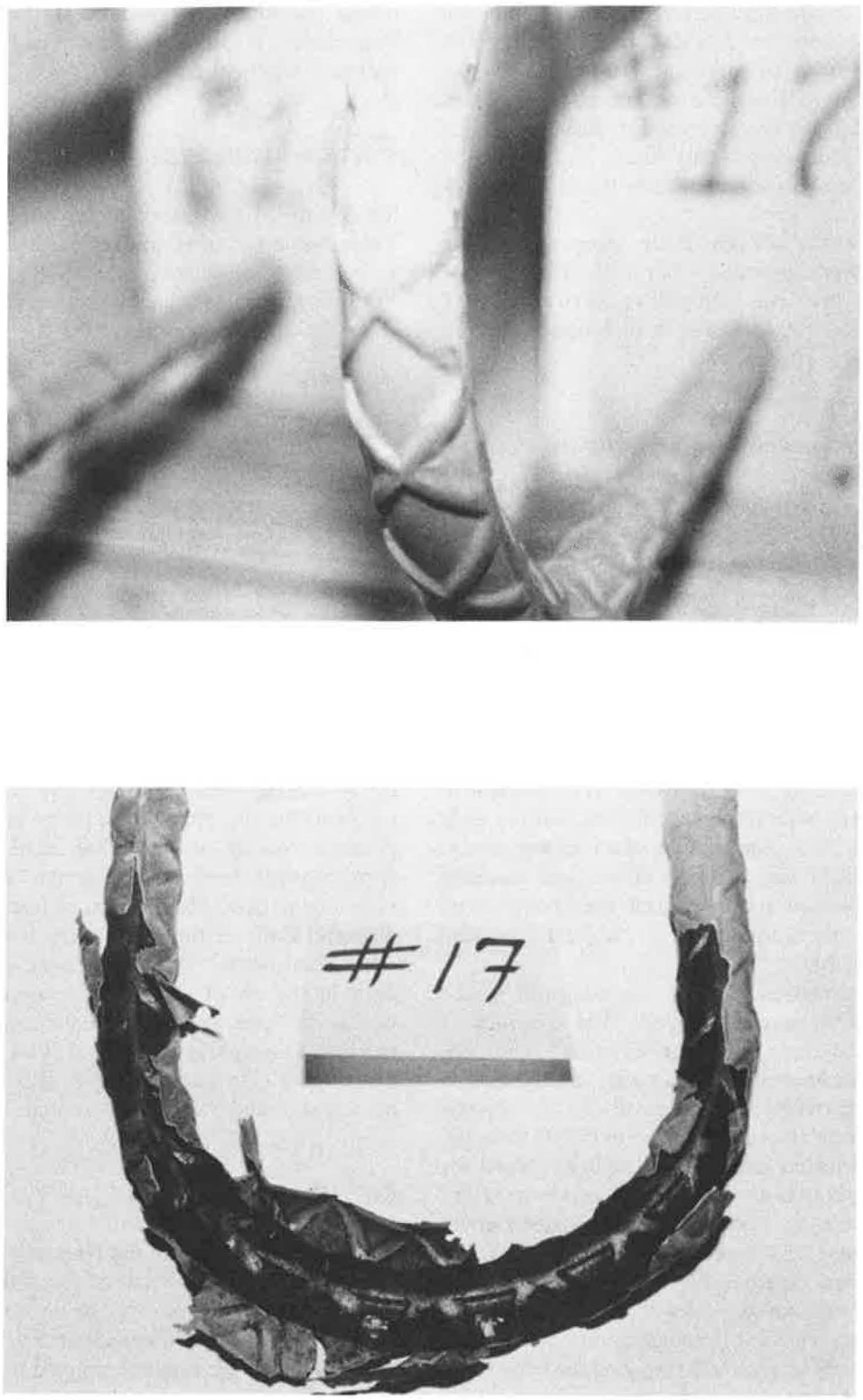

Figure 3 Sample #17 with small defects present before test (top) and extreme disbonding, localized and general corrosion after test (bottom).

epoxy-coated rebars used in this study were "laboratory perfect" bars. "Laboratory perfect" bars may be useful for establishing baseline data but results from these should not be used to imply performance in actual usage. Conditions such as pre-exposure (ultra-violet rays or salt spray), fabrication, and damage during construction were not addressed in this study. The galvanized rebars showed no corrosion and only traces of zinc oxide.

The results of this study compared to the performance of epoxy-coated rebar in the Florida Keys emphasized the problems with past research conducted. These results lead to a change in direction for future research on epoxy-coated rebar.

Life Cycle of an Epoxy-Coated Rebar for a Construction Project

Documentation was made of the life of an epoxy-coated rebar from plant to finished structure. An epoxy coating plant was visited where careful handling and rigid inspection by the manufacturer and FDOT personnel were exercised. Blasting operations, temperature measurements, coating process, curing, holiday detection and adhesion testing were all followed in accordance with an existing quality control plan. All handling was done by padded straps and wood cribbing to minimize damage. The cutting and fabrication were handled in the same manner with all damaged areas and cut ends immediately patched. Each bundle of rebars was placed on wooden pallets and carefully lifted onto shipping vehicles with padded straps. When the epoxy-coated rebars left the plant, they were in excellent condition with minimal damage.

A construction site that was using this plant's epoxy-coated rebar was also visited. The condition of the epoxy-coated rebar was quite different. The construction site was located on a causeway crossing Tampa Bay, a highly corrosive marine environment. Epoxy-coated rebars were stored less than 9 m (30 ft) from the saltwater. Fabricated rebars that had been stored for over a month showed small corrosion products at the root of deformations, Figure 4. Epoxy coating could easily be removed with a pocket knife along the fabricated (bent) areas, Figure 5. Fabricated units were seen on barges that were on the water for six weeks. Epoxy coating on rebars stored at the construction site for over a year, could easily be removed revealing the beginnings of corrosion products. All the epoxy-coated rebars examined were within specification requirements of 2% damage per 30.5 cm (2%/ft). These findings at the construction site showed that even epoxy-coated rebar within specifications will incur sufficient damage and degradation of bond such that the long term performance is highly questionable.

STATE-WIDE SURVEY

To determine the extent of the problem statewide, a listing was made of all bridges constructed using epoxy-coated rebars since the Keys Bridges were built. From this information, bridges were selected for investigation using the following criteria:

- Bridges constructed since 1979,
- Bridges five years or older,
- Bridges with substructures located in marine waters, and
- Bridges with pier-type substructure foundations.

A total of 29 bridges were selected for investigation. Bridge number and site number for the 29 bridge sites are shown in Figure 6. A typical inspection summary is shown in Figure 7. At each bridge site, cores of concrete and epoxy-coated rebar samples are extracted for examination and testing. To date, 14 sites have been inspected. In all but one instance, gross disbondment of epoxy coating from the rebar was reported. In each instance, the disbondment appears independent of the chloride content at the rebar level. No significant corrosion has been noted on the additional bridges inspected to date. However, in all instances thus far, the chloride levels at the rebar were not high enough to initiate corrosion. Since the epoxy coating disbonded early in the life of the structure, corrosion is likely to initiate as soon as chloride levels at the rebar depth increase to a significant amount (0.71 kg per m^3 or 1.2 lbs per yd^3). The causes of rebar disbondment are being investigated under contract research.

CONTRACT RESEARCH

A research contract with the University of South Florida (USF) under the direction of Dr. Alberto Sagüés was initiated. This research is being conducted in close coordination with the statewide survey. Samples extracted from the bridge sites are shipped to USF for detailed examination. The primary objectives of the research are to define the cause of corrosion of epoxy-coated rebar and aid in short and long term solutions to the problem.

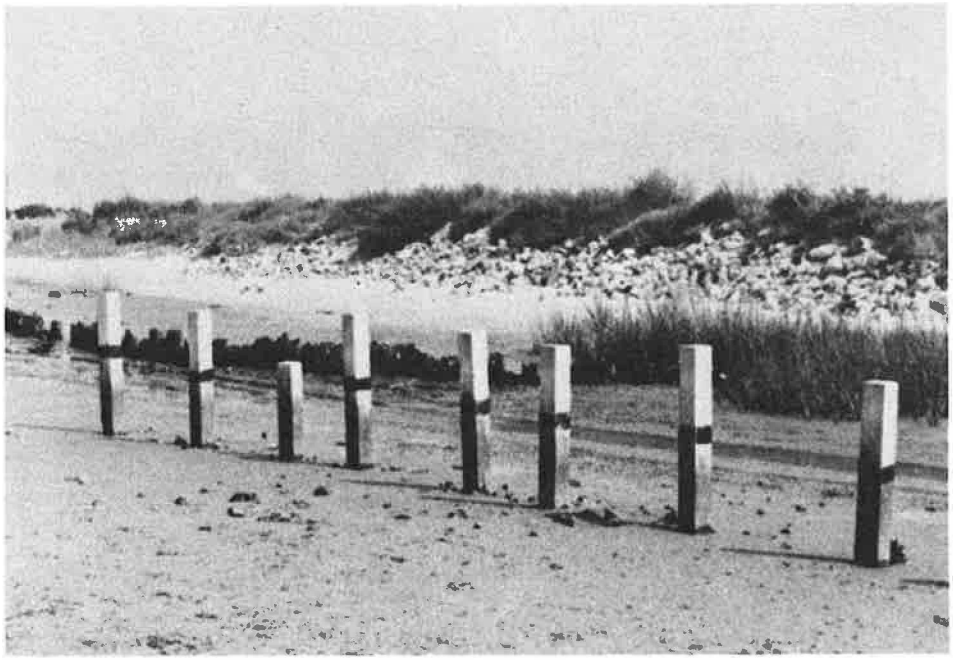

Figure 4 Matanzas Inlet test site after placement of test pilings in 1979 with application of coal-tar epoxy bands around areas containing the rebar support chairs.

These are being accomplished by investigating the corrosion mechanism, parameters affecting the corrosion, and causes and effects of coating disbondment. The results of this research are reported elsewhere *(4,5,6,7)*.

IMPLICATION AND CONCLUSIONS

The investigations conducted to date have convinced the Florida Department of Transportation that epoxy-coated rebar is inappropriate for marine corrosion protection. The disbondment of the epoxy coating from the rebar in the absence of chlorides is the most significant and compelling finding in this investigation. Epoxy-coated rebar is intended to protect against corrosion by providing a thin barrier film. If the bond is lost, the protection characteristics are diminished.

To date, research has not found that the manufacturing process of epoxy-coated rebar was improper or out of specification for any of the FDOT bridges examined. There appear to be no significant differences between present day coating technology and standards and those employed for the Florida Keys Bridges. Due to the severe disbondment problems observed, bolstering of the specifications is not considered an appropriate solution to preventing the corrosion of epoxy-coated rebar. FDOT has concluded that epoxy-coated rebar will not provide suitable long term protection against corrosion in a marine splash zone environment.

In December 1988, FDOT stopped specifying the use of epoxy-coated rebar in bridge substructures. At that time, alternatives such as penetrant sealers, high range water reducers, specification improvement for the quality control of concrete, and certain design features were implemented. Silica fume concrete, coated prestressed strands and ground slag cement were incorporated on a limited experimental basis. Further research was required so protective measures such as stainless steel rebars, galvanized rebars, fiberglass rebars, latex modified concrete, calcium nitrite, and new organic coatings could be evaluated.

In July 1992, the Florida Department of Transportation discontinued the use of epoxy-coated rebar in all construction. Investigations and research conducted since 1988 has lead to the experimental implementation of silica fume concrete for substructures and calcium nitrite for superstructures located in extremely aggressive environments. It is anticipated that these alternative corrosion control features will be adopted as FDOT's standards for long term corrosion control in the marine environment.

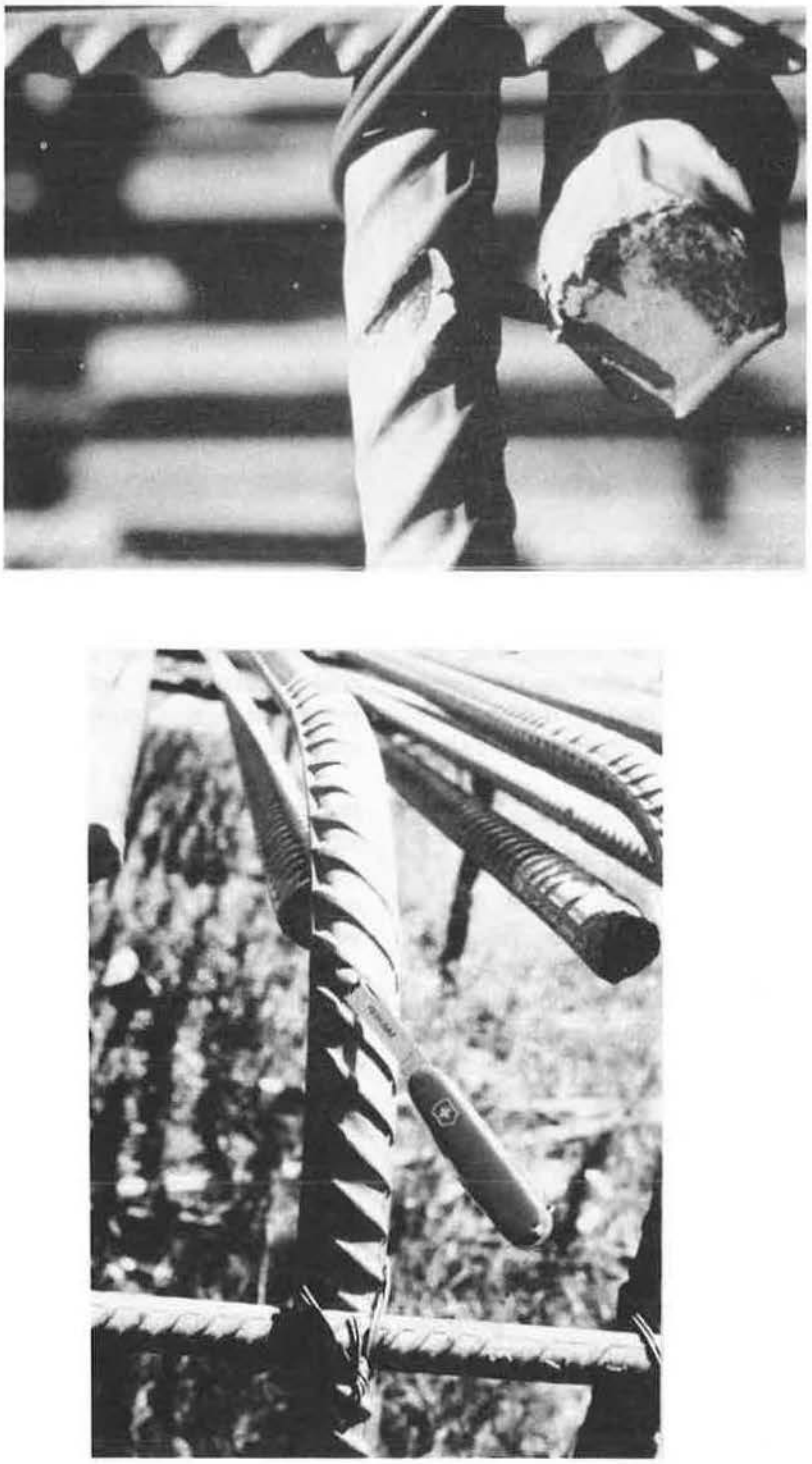

Figure 5 Epoxy coating easily removed with pocket knife along fabricated (bent) areas.

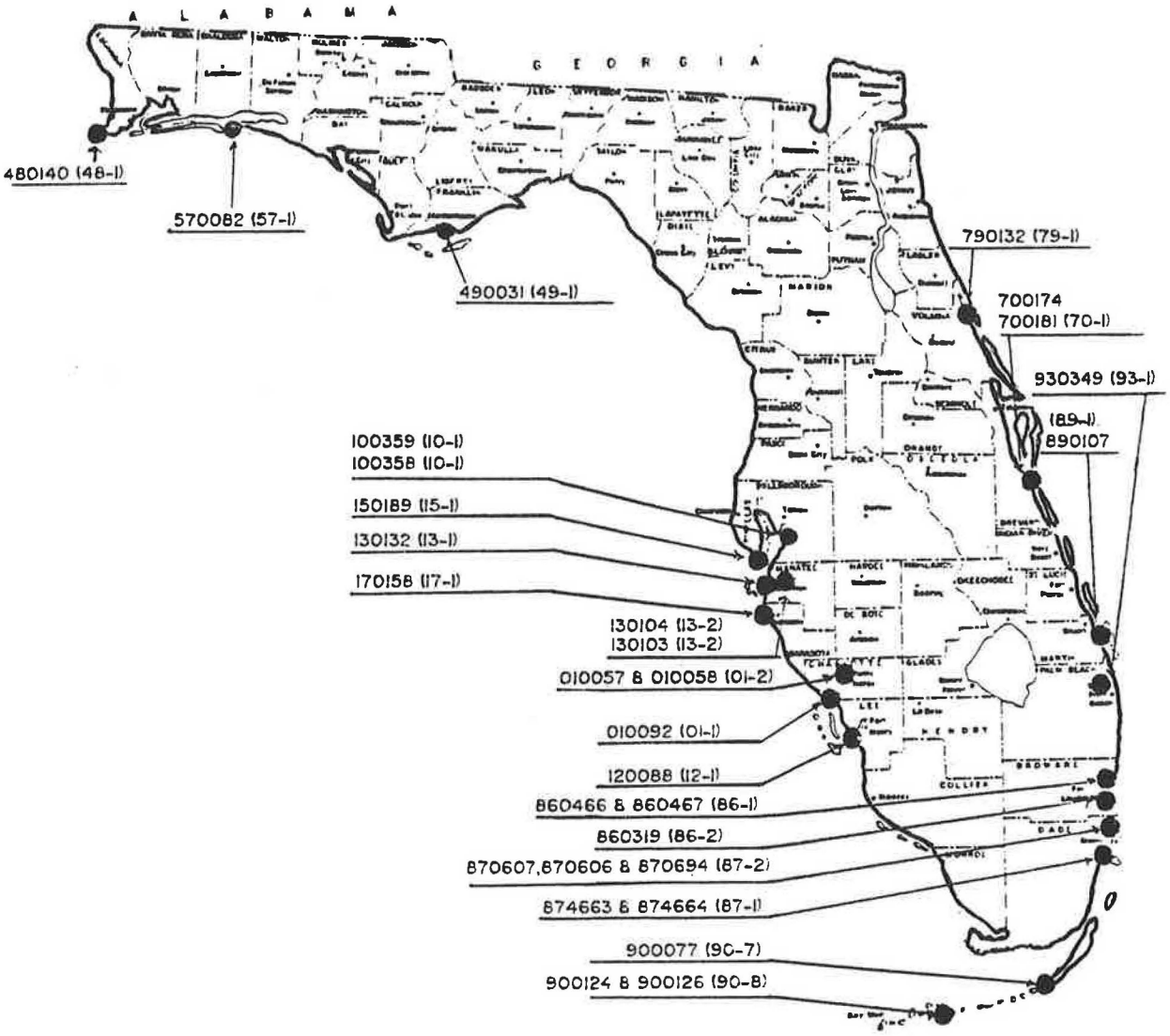

Figure 6 Bridge and site numbers for the 29 bridges selected for the epoxy-coated rebar investigation.

FLORIDA DEPARTMENT OF TRANSPORTATION
EPOXY-COATED REBAR INVESTIGATION
INSPECTION SUMMARY

DATE 05/22/91 **TEMP.** _____
BRIDGE # 790132 **R.H.** _____
CONSTRUCTION DATE 1986
LOCATION (SR#, CITY) SR-40 in Volusia County (City of Ormond Beach)
CONDUCTED BY: Lasa, Langley, Cerlanek, and Petrin
SUBSTRUCTURE DESIGN Round columns bearing on square footers
SUBSTRUCTURE REBAR Epoxy (brown color)
CONSTRUCTION QUALITY Good
NO. OF PIERS EVALUATED Three

NO. OF CORES OBTAINED	PIER NO. 5	PIER NO. 13	PIER NO. 17	PIER NO. ---
FOOTER	1	1	1	---
COLUMN	5	5	5	---
STRUT	---	---	---	---

CONCRETE RESISTIVITY (COLUMN)
LOWER LEVEL 13.6 kΩ **MID-LEVEL** 14.1 kΩ **UPPER LEVEL** 20.2 kΩ
CHLORIDE CONTENT AT REBAR COVER (SPLASH ZONE) _____
NUMBER OF REBARS EXPOSED 15
TYPICAL REBAR COVER 9 to 14 cm (3-1/2 to 5-1/2 in)
NUMBER OF REBAR SAMPLES OBTAINED 6 pieces (2 each column)
REBAR CONTINUITY (%) 80%
AVERAGE REBAR RESISTANCE _____
NO. OF LINEAR POLARIZATION TEST CONDUCTED: 3
MAX. CURRENT 6 mA **MIN. CURRENT** 20 μA
$CuSO_4$ POTENTIAL: LOWER COLUMN -.600 v **UPPER COLUMN** -.068 v

MACROCELL TEST	PIER NO. 5	PIER NO. 13	PIER NO. 17	PIER NO.
10 SEC.	Continuity	5.5 mA	53 mA	---
10 MIN.	Continuity	1.3 mA	10 mA	---

AVERAGE EPOXY BOND Disbonded
BRIDGE CONDITION SUMMARY This bridge is located on SR-40 over the Intercostal Waterway. The pH of the water is 7.8 and the resistivity is 34 ohms and the chloride content is 10, 493 ppm. The bridge was built in 1986 (5 years old). Concrete resistivity falls in the low-medium range and in some instances few high half cell potentials were observed (-.600 v). Linear polarization tests do not indicate any serious corrosion activity. Continuity was observed in 80% of the bars tested. Rebars had adequate cover and construction quality is average. No cracking, or spalling was found. Epoxy disbonding from bars on all tested samples.

Figure 7 Typical FDOT Epoxy-Coated Rebar Investigation inspection summary.

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EFFECTS OF STATIC AND REPEATED LOADINGS ON CONCRETE BRIDGE DECKS AND SLABS REINFORCED WITH EPOXY-COATED BARS

Hendy Hasan and Julio A. Ramirez*

This paper presents findings from an ongoing laboratory and field investigation on the effects of static and repeated loadings on concrete bridge decks and slabs reinforced with epoxy-coated bars in the State of Indiana. Comparisons are presented of the load-deflection behavior, flexural crack pattern and width, and bond strength of companion concrete specimens reinforced with coated and uncoated steel. Current laboratory findings indicate the average concrete crack width is larger in specimens with epoxy-coated reinforcement than in companion specimens with uncoated reinforcement. The findings from one of the five bridge decks in the field investigation are also included.

Keywords: reinforcement bond to concrete, concrete cracking, epoxy coatings, deformed reinforcement, concrete durability, fatigue, lap connections, repeated loading and static loading.

INTRODUCTION

One cause of concrete bridge deck deterioration is corrosion of the reinforcing steel. Corrosion is often attributed to concentrations of chloride ions from de-icing salts in the concrete. These concentrations of ions serve as the electrolyte in the corrosion process. During the corrosion process the volume occupied by the reinforcing steel increases causing pressure on the surrounding concrete leading to eventual spalling and, in extreme cases, loss of structural integrity. Since the early 1970's epoxy-coated reinforcement has been used to minimize rebar corrosion (*1*). An epoxy powder is electrostatically applied to heated reinforcement forming a protective layer that restricts ion contact with the reinforcing steel.

This paper reports on an ongoing HPR-Part II research study, "Behavior of Concrete Bridge Decks and Slabs Reinforced with Epoxy Coated Bars," sponsored by the Indiana Department of Transportation (INDOT) and the Federal Highway Administration. A laboratory and a field phase are being conducted in this research study. The laboratory phase consists of a test program designed to compare the behavior of slab type concrete members containing coated reinforcement with that of otherwise identical companion concrete specimens reinforced with uncoated steel. Comparisons are made of the load-deflection behavior, flexural crack widths and patterns, and the bond strength under static and repeated loading. Test variables include size of the reinforcement bar, ratio of concrete cover to bar diameter, reinforcement splice length, thickness of epoxy coating, number of applied load repetitions, stress range and peak stress. The field phase includes assessment of concrete strength, chloride content, delamination survey, crack patterns, concrete cover, and condition of the reinforcement for five bridge decks reinforced with epoxy-coated steel in the State of Indiana.

LABORATORY PHASE

The dimensions and loading arrangement for the concrete test specimens with the No. 7 (22 mm, 7/8 in) bars and with No. 11 (35 mm, 1-1/4 in) bars are shown in Figure 1. No. 3 bars (10 mm, 3/8 in) spaced at 152 mm (6 in) on centers were used as transverse reinforcement in all specimens. The physical and mechanical properties of the reinforcement are given in Table I. The concrete test specimens were designed to fail at the lap splices before yielding of the steel, see Figure 1 The specimens were loaded such that the lap splices were placed in a constant moment region. The primary variables of the experimental test program are summarized in Table II. Test sets include identical companion beams, one reinforced with epoxy-coated steel and the other with uncoated reinforcement.

The results of the first 24 tests are reported in this paper. Six sets of companion specimens containing No. 7 bars and six sets containing No. 11 bars were tested under repeated loading. The test specimens were initially cracked by application of 2 to 3 monotonic load cycles up to the peak stress used in the repeated load

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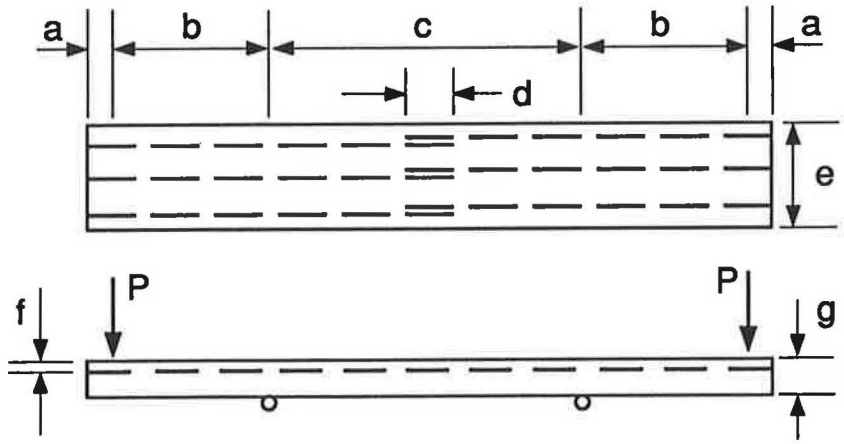

bar size (mm)	a (mm)	b (mm)	c (mm)	d (mm)	e (mm)	f * (mm)	g (mm)	stirrups
22	152	1219	1219	305	610	64	203	10 mm @152 mm
35	152	1219	1219	711	711	64	305	10 mm @152 mm

*f=clear cover above the reinforcement

Figure 1 Specimen dimensions.

Table I PHYSICAL AND MECHANICAL PROPERTIES OF REINFORCEMENT

Reinforcement Property	No. 7 bars (22 mm)		No. 11 bars (35 mm)	
	Uncoated	Epoxy-Coated	Uncoated	Epoxy-Coated
Average Gap (mm)	6.94	6.53	7.39	7.19
Average Spacing (mm)	13.85	13.85	21.77	21.77
Average Height (mm)	1.15	1.13	1.96	1.79
Variation in Weight (%)	-3.9	-4.1	-4.4	-4.4
Yield Stress (MPa)	454	471	515	477
Tensile Stress (MPa)	759	707	747	747
% Elongation in 200 mm	14	13	11	14
Rib Bearing Area (mm^2/mm)	3.54	3.71	7.50	6.05
Related Rib Area	0.051	0.053	0.068	0.055
Rib Bearing Area Ratio (1/mm)	0.0092	0.0096	0.0075	0.0060

Table II TEST VARIABLES

Specimen Designation	Bar Size	Cover (mm)	Splice Length (mm)	Concrete Strength (MPa)	Peak Stress (MPa)	Stress Range Below Peak (MPa)	# Cycles
U7241	No. 7	64	305	20.7	166	60	1,000,000
E7241	No. 7	64	305	20.7	166	60	1,000,000
U7242	No. 7	64	305	32.4	166	60	1,000,000
E7242	No. 7	64	305	32.4	166	60	1,000,000
U7361	No. 7	64	305	35.9	248	60	1,000,000
E7361	No. 7	64	305	35.9	248	60	1,000,000
U7362	No. 7	64	305	36.6	248	60	1,000,000
*E7362	No. 7	64	305	36.6	248	60	600,000
U7363	No. 7	64	305	27.6	248	103	1,000,000
E7363	No. 7	64	305	27.6	248	103	1,000,000
U7301	No. 7	64	305	20.7	207	60	1,000,000
E7301	No. 7	64	305	20.7	207	60	1,000,000
U11241	No. 11	64	711	20.7	166	60	1,000,000
E11241	No. 11	64	711	20.7	166	60	1,000,000
U11242	No. 11	64	711	32.4	166	60	1,000,000
E11242	No. 11	64	711	32.4	166	60	1,000,000
U11243	No. 11	64	711	20.7	166	103	1,000,000
E11243	No. 11	64	711	20.7	166	103	1,000,000
U11301	No. 11	64	711	35.9	207	60	1,000,000
E11301	No. 11	64	711	35.9	207	60	1,000,000
U11302	No. 11	64	711	36.6	207	60	1,000,000
E11302	No. 11	64	711	36.6	207	60	1,000,000
U11303	No. 11	64	711	27.6	207	103	1,000,000
*E11303	No. 11	64	711	27.6	207	103	336,000

* Specimen failed in fatigue.

tests (described later). After the initial loading, the beams were subjected to cycles of load between the maximum and minimum stress levels with a hydraulic pulsator at a rate of 260 cycles per minute in blocks of approximately 100,000 cycles up to a total of 1,000,000 cycles. If failure did not occur during the repeated load phase of the test, the specimen was unloaded and tested monotonically to failure.

The number of cracks in the concrete in the constant moment region and the total width of these cracks are given in Table III. The data presented are from the second and one millionth load cycles. For beams E7362 and E11303 the data are the last measured values before failure in fatigue after 600,000 and 336,000 cycles respectively. The data were recorded at the peak repeated load. This load was selected for comparison because all of the flexural cracks had formed at this level. The values presented in Table III show that the beams with uncoated reinforcement had more flexural cracks than the beams with epoxy-coated reinforcement. The cracks were more widely spaced with epoxy-coated reinforcement which implies a longer transfer length

Table III CRACK WIDTHS IN THE CONSTANT MOMENT REGION

Specimen Designation	Number of Cracks	Total Crack Width (mm)		E/U Ratio of Total Crack Width		E/U Ratio of Average Crack Width	
		2nd Cycle	10^6 Cycle	2nd Cycle	10^6 Cycle	2nd Cycle	10^6 Cycle
U7241	8	1.75	2.24	1.20	1.05	1.37	1.21
E7241	7	2.11	2.36				
U7242	6	2.08	2.41	0.94	0.95	1.41	1.42
E7242	4	1.96	2.29				
U7361	6	3.40	4.09	0.96	0.99	0.96	1.00
E7361	6	3.28	4.06				
U7362	6	3.68	4.01	0.71	0.87	1.07	1.30
*E7362	4	2.62	3.48				
U7363	6	2.90	3.45	1.57	1.49	1.57	1.49
E7363	6	4.55	5.16				
U7301	6	3.40	4.06	1.13	1.12	1.13	1.12
E7301	6	3.86	4.55				
U11241	7	1.35	1.80	1.08	0.93	1.25	1.10
E11241	6	1.45	1.68				
U11242	5	1.57	2.06	1.16	1.04	0.97	1.03
E11242	6	1.83	2.13				
U11243	7	1.47	1.98	1.24	1.17	1.45	1.37
E11243	6	1.83	2.31				
U11301	7	1.75	2.01	1.28	1.32	1.12	1.15
E11301	8	2.24	2.64				
U11302	6	1.40	1.88	1.22	1.12	1.46	1.35
E11302	5	1.70	2.11				
U11303	7	2.29	2.57	1.11	1.21	1.11	1.21
*E11303	7	2.54	3.10				

* Specimen failed in fatigue.

existed with epoxy-coated bars. The ratio of the total crack width of the epoxy-coated to uncoated (E/U) was 1.09 for the second cycle and 1.08 for the one millionth cycle in the specimens reinforced with No. 7 bars. The ratio was 1.18 for the second cycle and 1.13 for the one millionth cycle in the specimens with No. 11 bars. The E/U ratio of average crack width for the second cycle was 1.2 for No. 7 bars and 1.25 for No. 11 bars. After one million cycles the change in the ratio of average crack width was negligible. Although there were fewer cracks in the specimens reinforced with epoxy-coated bars, the width of the individual cracks was larger than with uncoated steel.

The total deflection for each specimen was recorded at the same loads used in the crack width comparisons. The total deflection is defined as the sum of the upward movement at the centerline and the downward movement at the ends of the specimen. At the end of the second cycle, the specimens reinforced with epoxy-coated steel averaged total deflections 5% greater than the beams with uncoated reinforcement. After one million cycles, the E/U ratio for deflections

remained at 1.05 for the specimens containing No. 7 bars and decreased to 0.98 in the specimens with No. 11 bars.

Shown in Table IV are the failure load, failure stress and deflections for the 24 specimens reported in this paper. The failure stress was calculated assuming a linear stress distribution in the concrete compression zone and neglecting the tensile strength of the concrete. The bond ratio given in Table IV, is the ratio of the average stress to failure for the specimen reinforced with epoxy-coated steel to its companion specimen reinforced with uncoated steel. All the specimens reported failed before yielding of the tensile reinforcement. Specimen E7362 failed during the repeated loading phase after 600,000 and E11303 after 336,000 cycles. For these specimens, the reported failure stress is the peak stress in the repeated load cycle phase. The average bond ratio for the specimens with No. 7 bars was 0.80, and 0.76 for No. 11 bars.

Two factors that significantly effected the splitting phenomena associated with bond strength reduction in epoxy-coated reinforcement were the concrete cover to bar diameter ratio and the rib bearing

Table IV FAILURE LOADS, STRESSES AND DEFLECTIONS

Specimen Designation	Failure Stress (MPa)	Bond Ratio	Load (kN)			Deflection (mm)		
			Split	Failure	Difference	Split	Failure	Difference
U7241	269	.95	24.5	26.6	2.1	31.4	35.7	4.3
E7241	256		24.5	25.3	0.8	34.4	34.4	0.0
U7242	310	.82	31.1	31.4	0.3	35.8	35.8	0.0
E7242	256		25.6	25.6	0.0	30.4	30.4	0.0
U7361	397	.73	40.9	40.9	0.0	46.2	46.2	0.0
E7361	290		29.4	29.4	0.0	36.4	36.4	0.0
U7362	381	.65	38.3	39.1	0.8	42.8	42.8	0.0
*E7362	248		24.5	24.5	0.0	32.2	32.2	0.0
U7363	392	.84	37.8	40.0	2.2	43.6	47.0	3.4
E7363	330		30.0	33.4	3.4	35.3	38.3	3.0
U7301	301	.81	24.5	30.0	5.5	31.9	37.2	5.3
E7301	244		24.0	24.0	0.0	34.5	34.5	0.0
U11241	290	.85	102.3	133.4	31.1	16.5	22.9	6.4
E11241	248		89.0	113.7	24.7	14.9	20.0	5.1
U11242	339	.79	124.5	163.0	38.5	16.2	21.2	5.0
E11242	267		122.3	124.8	2.5	17.9	20.2	2.3
U11243	311	.82	89.0	143.4	54.4	16.9	27.4	10.5
E11243	254		103.1	116.2	13.1	17.5	19.7	2.2
U11301	379	.75	106.8	177.9	71.1	16.3	28.4	12.1
E11301	284		97.9	132.9	35.0	15.1	21.7	6.6
U11302	360	.76	112.2	169.1	57.9	16.4	27.8	11.4
E11303	274		97.9	127.8	29.9	16.6	21.5	4.9
U11303	353	.59	121.4	164.6	43.2	19.4	27.8	8.4
*E11303	207		94.5	94.5	0.0	13.4	15.6	2.2

* Specimen failed in fatigue.

area ratio. The ratio of concrete cover to bar diameter was 2.86 for the No. 7 bars and 1.77 for No. 11 bars. The rib bearing area ratio is the ratio of the rib bearing area per unit of bar length minus the area of the longitudinal rib to the nominal cross sectional area of the bar. As the bar size increased in the test specimens, the concrete cover to bar diameter ratio and the rib bearing area ratio decreased.

The peak repeated stress influenced the bond ratio for both No. 7 and No. 11 bar specimens. For the No. 7 bar specimens subjected to a stress range of 60 MPa (8.7 ksi) below the peak stress, the average bond ratio was 0.89 for a peak stress of 166 MPa (24 ksi), 0.81 with a peak stress of 207 MPa (30 ksi), and 0.69 for a peak stress of 248 MPa (36 ksi). The specimens with No. 7 bar subjected to a stress range of 103 MPa (15 ksi) below the peak stress of 248 MPa (36 ksi) had a bond ratio of 0.84. For the No. 11 bar specimens subjected to a stress range of 60 MPa (8.7 ksi) below the peak stress, the average bond ratio was 0.82 with a peak stress of 166 MPa (24 ksi), and 0.76 with a peak stress of 207 MPa (30 ksi). The specimens reinforced with No. 11 bars and subjected to a stress range of 103 MPa (15 ksi) below the peak stress, had bond ratio of 0.82 for a peak stress of 166 MPa (24 ksi) and 0.59 for a peak stress of 207 MPa (30 ksi).

Shown in Table IV are the load and corresponding deflection at first sign of splitting and at failure. The average additional load carrying capacity after splitting in the specimens reinforced with No. 7 bars was negligible for both coated and uncoated reinforcement. For specimens reinforced with No. 11 bars, the average additional load beyond splitting was 49.4 kN (11.1 kips) with uncoated steel and 17.6 kN (4.0 kips) with coated steel. In regards to additional deflection beyond splitting the specimens with No. 7 bars showed little increase in the deflection for either type of reinforcement. In the specimens reinforced with No. 11 bars the average post-splitting deflection was 9 mm (0.35 in) in the specimens with uncoated steel and 3.9 mm (0.15 in) with coated steel.

The work in the laboratory phase is continuing with future tests of specimens at a maximum peak stress of 248 MPa (36.0 ksi), monotonic single cycle baseline tests, coating thickness tests, and bar deformation patterns tests.

FIELD PHASE

This section describes the structures being evaluated in the field phase of the research study and the results of the field evaluation of one structure. The remaining structures are scheduled for evaluation in the future. The field phase is aimed at the condition assessment of concrete bridge decks and slabs reinforced with epoxy-coated steel in Indiana. The field evaluation includes structures throughout the state reflecting a cross section of environmental conditions, traffic and intensity of salt application. It also addresses the performance of decks supported on flexible systems (steel girders) as well as more rigid support conditions (precast prestressed girders) and concrete slabs. A total of five sites have been selected for evaluation. The site selection has been fully coordinated with personnel from the INDOT.

The first structure selected for evaluation is located in Indianapolis and consists of a six-span continuous composite steel box girder bridge with a concrete slab. This bridge deck was built in 1985 and has a maximum span length of 62.8 m (206 ft). This bridge represents the case of a deck on a flexible superstructure in the southern part of the state subjected to heavy urban traffic and severe salt exposure. The bridge cross section is shown in Figure 2 and the plan view in Figure 3. The second structure is located in South Bend. The structure is a four span continuous bridge deck supported on precast prestressed AASHTO sections and represents the case of a concrete deck built on a more rigid support system. The structure was built in 1983 and the maximum span length is 27.4 m (90 ft). This bridge is located in the northern part of the state in an urban area with significant traffic and severe salt exposure. The third structure selected is located south of South Bend in the northern part of the state. The structure consists of a three-span continuous welded steel beam with a composite concrete deck. The structure was built in 1980 and has a maximum span length of 18.9 m (62 ft). This structure is subjected to heavy truck traffic and heavy salt application. The fourth structure is located in southern part of the state and consists of three span skewed continuous reinforced concrete slab bridge. The bridge was built in 1985 and has a maximum span length of 14.0 m (46 ft). This structure is subjected to moderate traffic and moderate salt application. The fifth structure is located in the northern part of the state in Gary. The structure is a three span continuous bridge deck supported on a continuous steel beam. This bridge was built in 1980 with a maximum span length of 19.8 m (65 ft). The concrete deck was built using stay-in-place metal forms. The bridge is subjected to heavy industrial traffic with heavy de-icing salt exposure.

The deck evaluation at each of the five sites will include a deck survey for delamination as well as a

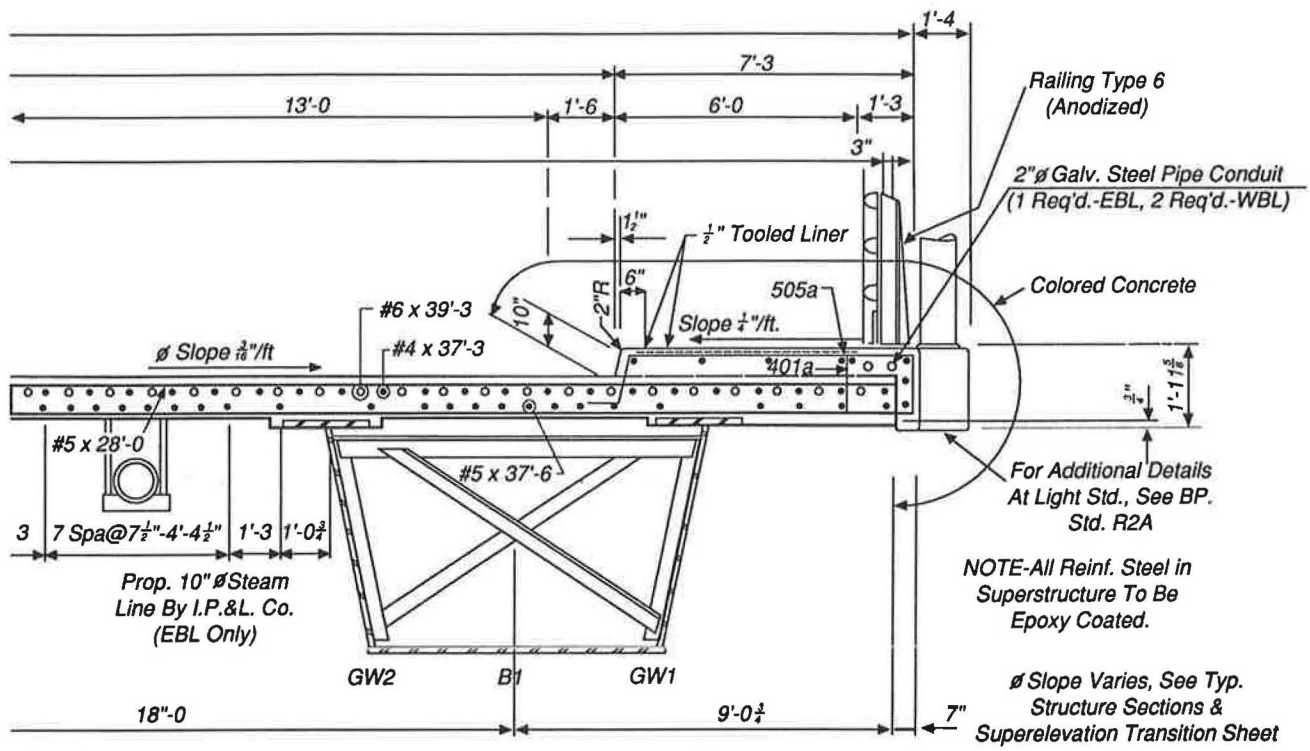

Figure 2 Bridge cross section.

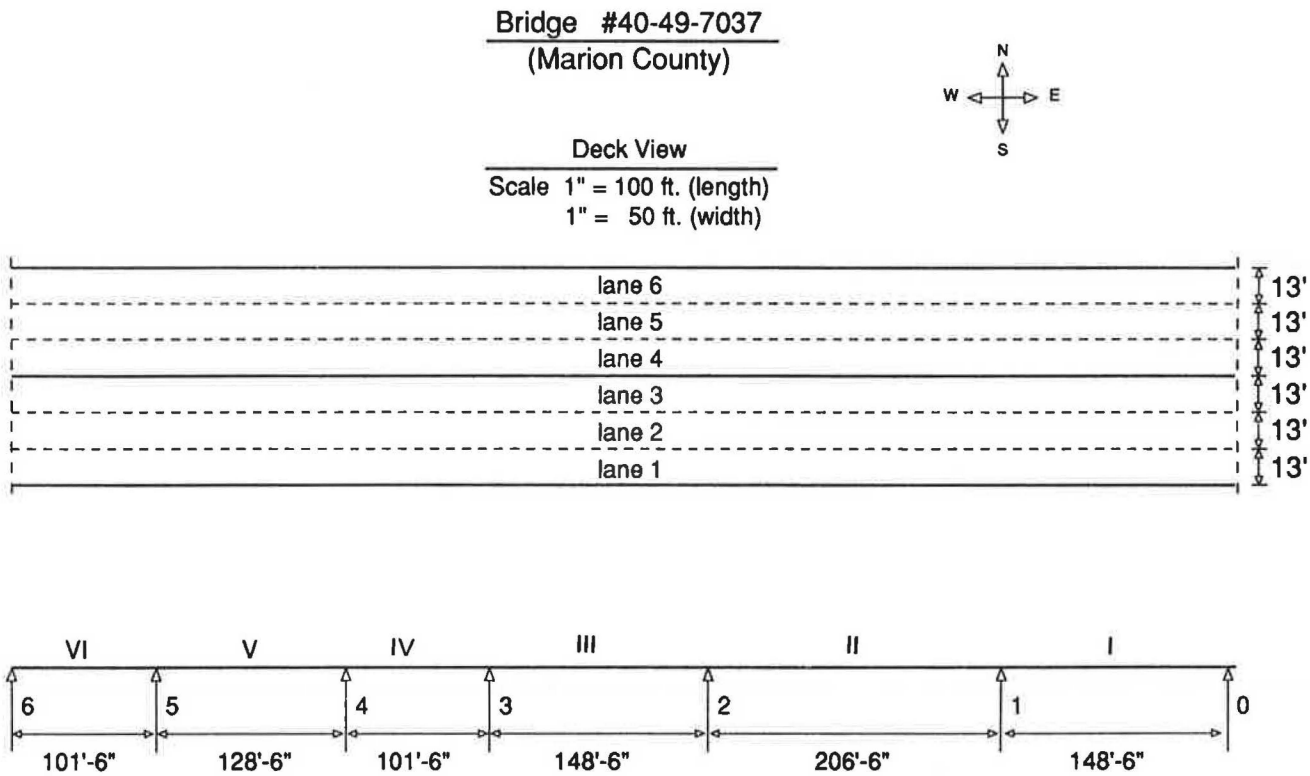

Figure 3 Continuous composite steel box beam bridge.

detailed mapping of the observed cracking. Core samples and chloride samples will be taken. The concrete cover will be evaluated using the R-meter (focused electromagnetic field) as well as coring. The reinforcement condition evaluation will include coating condition, thickness of coating and deformation pattern. In addition, the evaluation will include factors such as: (a) environment, (b) traffic, (c) degree of salt application, (d) storage methods, (e) local practices, sources and specifications, (f) coating process, and (g) type of epoxy material.

The Division of Materials and Tests of the INDOT conducts a series of tests on samples taken from every 1,360 kg (3,000 lb) of epoxy-coated steel used on bridge decks built in the state. These tests include yield and ultimate strength, elongation, 180 degree ASTM bend test, ASTM-Deformation, epoxy thickness AASHTO M-284, and 120 degree bend test AASHTO M-284. No checks are made for holidays, this is left to an on-site INDOT project engineer walk-through visual survey. Coating thicknesses typically exceed minimum requirements. In general, construction practices depend on the contractor's quality control emphasis and level experience working with epoxy-coated rebars, and the level of State inspections. For most jobs the bars are stored for short periods before placement in the structure.

The findings from the field investigation of the bridge structure located in Indianapolis are described below. The evaluation of the bridge deck was conducted on the outside lanes (1 & 6) as shown in Figure 3. Typical crack patterns are shown in Figure 4. The number of cracks in each span are shown in Figure 5 and the average crack width in Figure 6. The deck concrete compressive strength was determined using 127 mm (5 in) cores. The measured concrete cover, the results of compression tests, and the chloride content at depths of 25.4, 50.8, 76.2 and 101.6 mm (1, 2, 3 and 4 in) are shown in Table V.

The average flexural crack width was less than 0.410 mm (0.016 in). The deck concrete compressive strength adjusted for a test core height/diameter of 1.0 resulted in an average strength of 35 MPa (5.13 ksi). The measured average cover over the top layer of steel was 61 mm (2.4 in) with a maximum of 76 mm (3.0 in) and a minimum of 41 mm (1.6 in). The average chloride content was 1.29 kg/m^3 at 51 mm (2.18 lb/yd^3 at 2 in) below the surface of the deck and 0.88 kg/m^3 at 76 mm (1.48 lb/yd^3 at 3 in). No signs of concrete delamination were observed. The epoxy-coating on the rebar sections extracted from the deck showed no signs of damage.

DESIGN IMPLICATIONS

The surface roughness of uncoated bars and the irregularities along the steel concrete interface caused by adhesion of particles of concrete to the steel, found with uncoated reinforcement provide important components to the bond mechanism between concrete and steel. The absence or the reduction of these features eliminates or reduces the friction contribution in the coated bars. Lack of friction leads to higher rib-bearing forces, larger slip, higher bar strains at flexural crack locations and lower bond strength. Some of these deficiencies could be overcome by using deformation patterns with larger rib-bearing areas and steeper rib angles (*2*).

The stress range as well as the other variables in the laboratory phase of this study were selected to be typical of service conditions in a bridge deck. Repeated loading over the stress ranges, number of cycles and concrete strength evaluated in this study were more detrimental to the specimens with uncoated reinforcement. Although the total crack width in the constant moment region was approximately the same for both types of reinforcement, the average width of a single crack was larger for the specimens with epoxy-coated reinforcement. The wider cracks could lead to increased deterioration due to freeze-thaw action and could be of concern if epoxy coatings do not provide the protective barrier that has been assumed. The inspection of epoxy-coated bars after failure in the laboratory specimens found no visible damage to the coating due to the repeated loading.

Due to the larger crack opening, reinforcement stresses at crack locations will be higher for epoxy-coated bars. Hence radial stresses will be higher as well. Thus, adequate confinement must be provided by sufficient concrete cover. Larger cover to bar diameter ratios are recommended in harsh environments and should not be reduced with the expectation that the epoxy coating will be the sole system of corrosion protection. The extra cover provides improved anchorage for the bars. Furthermore, durability depends on careful design, good construction practices and adequate material selection. Improvements in any of these areas will reduce the problem, but individually will not provide an effective solution. Providing adequate cover is an example of good design strategy. Adequate inspection, finishing and curing represent solid construction practices and will lead to durable concrete. The use, proper manufacturing and handling of epoxy-coated bars are but a few of the aspects related to durable concrete bridge decks.

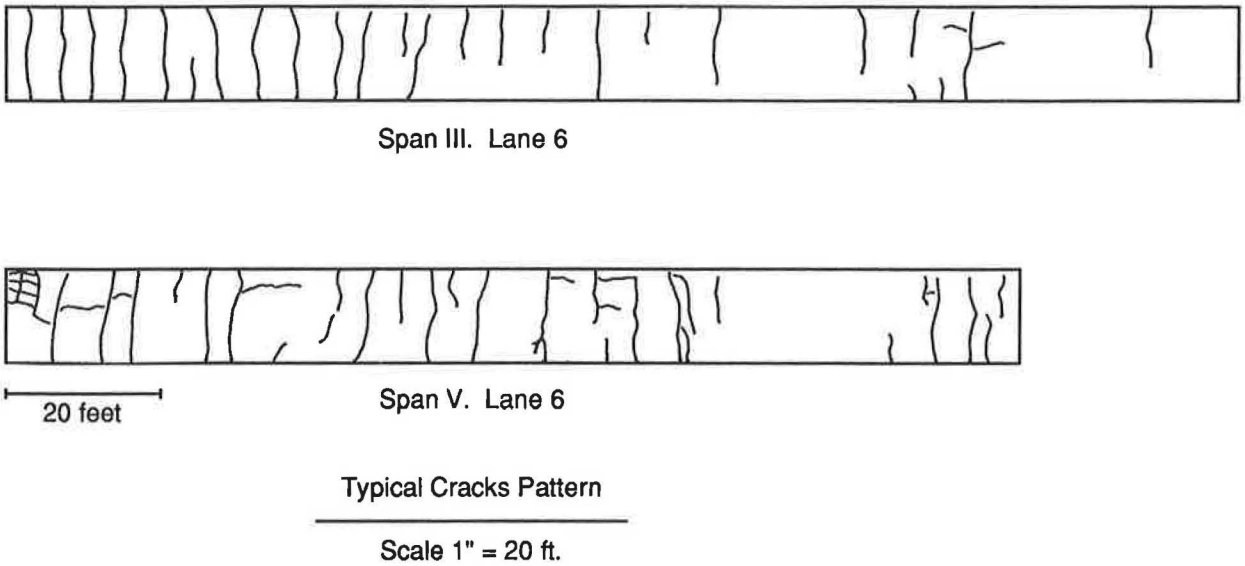

Figure 4 Typical crack patterns.

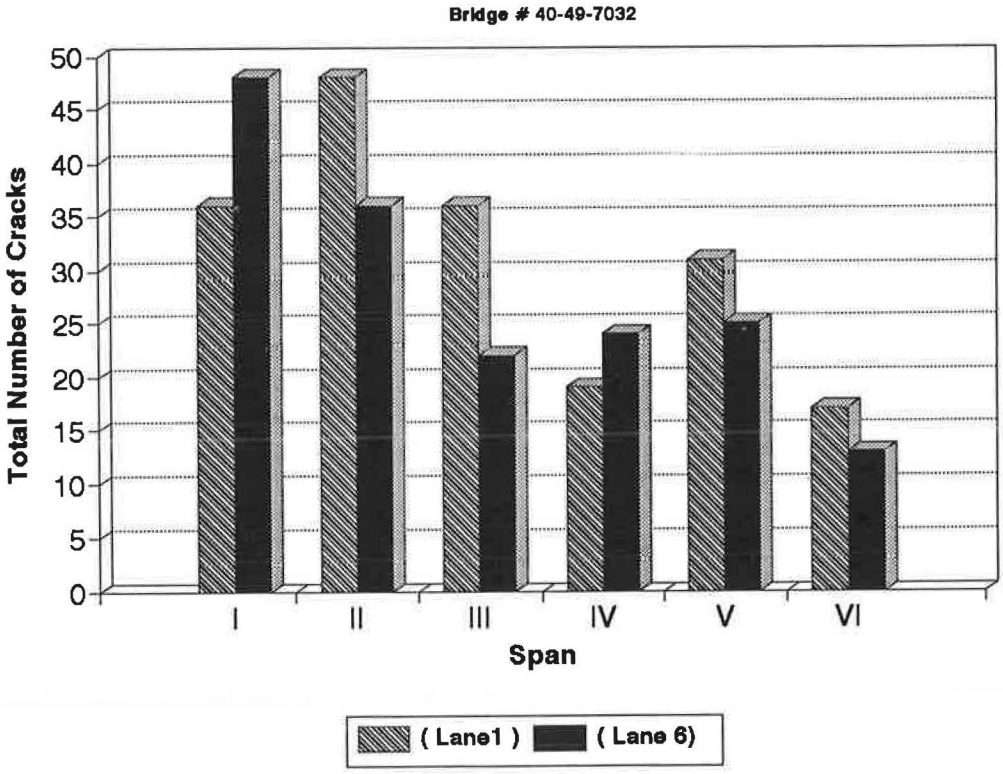

Figure 5 Total number of cracks.

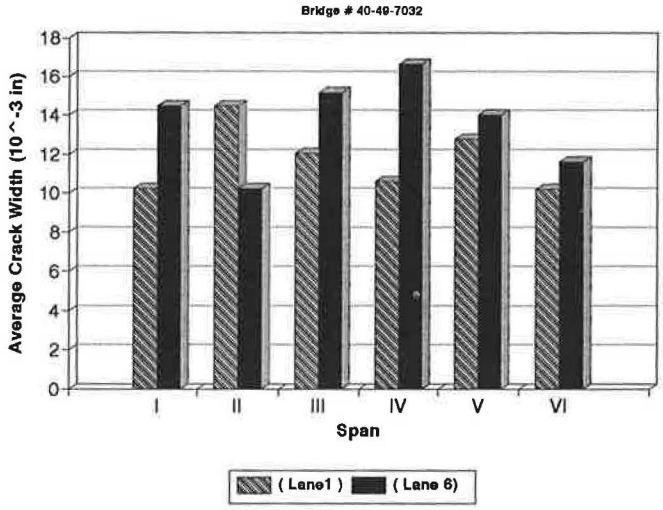

Figure 6 Average crack width.

Table V COMPRESSIVE STRENGTH, UNIT WEIGHT AND CHLORIDE CONTENT OF CONCRETE CORES FROM A BRIDGE IN MARION COUNTY INDIANA

Core No.	Span No.	Minimum Compression Strength (MPa)	Unit Weight (Kg/m^3)	Concrete Cover (mm)	Sample No.	Span No.	Chloride Content (Kg/m^3) at Depth (mm) of			
							25.4	50.8	76.2	101.6
1	I	--	--	--	1	VI	2.11	0.72	0.89	0.56
2	I	41.92	2329	68.6	2	V	2.06	1.41	0.79	0.54
3	I	43.57	2372	55.9	3	IV	7.29	3.80	1.47	0.36
4	II	42.06	2340	55.9	4	III	5.39	1.17	0.81	0.68
5	II	33.99	2283	61.0	5	II	2.72	0.58	0.74	0.65
6	IV	43.64	2392	71.1	6	I	4.00	1.51	0.69	0.64
7	IV	43.64	2390	76.2	7	I	2.46	0.52	0.46	0.55
8	V	35.16	2315	68.6	8	II	2.32	0.71	0.58	0.73
9	V	48.40	2356	63.5	9	III	5.54	1.98	0.53	0.78
10	VI	42.82	2281	50.8	10	IV	1.33	0.64	0.75	1.00
11	VI	43.92	2334	48.3	11	V	4.67	0.68	0.70	0.86
12	IV	45.30	2355	40.6	12	VI	2.30	1.77	2.14	0.06
13	IV	42.13	2311	55.9						
14	II	50.26	2403	76.2						
15	I	25.99	2311	61.0						

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PERFORMANCE OF EPOXY-COATED REBARS: A REVIEW OF CRSI RESEARCH STUDIES

Donald W. Pfeifer, Robert Landgren, and Paul Krauss*

This is a summary of an evaluation of corrosion research sponsored by the Concrete Reinforcing Steel Institute. This research consisted of three studies by Kenneth C. Clear, Inc. (KCC) of the corrosion resistance of epoxy-coated reinforcing bars and a review of these studies by Wiss, Janney, Elstner Associates, Inc. after some additional laboratory work. The most controversial portion of the KCC work was the accelerated corrosion study involving straight and bent epoxy-coated bars from eight different suppliers. A review of the KCC data and testing of remnant materials indicated that the frequency and size of holidays in the bar coatings was the dominant factor governing epoxy-coated bar corrosion resistance. Bars with fewer than 6 holidays per m (1.8/ft) generally performed well in these rigorous tests. Bars with higher holiday counts, thin film regions, bending damage or large areas of defective coating generally did not perform as well after the prolonged period of tap water soaking. These same bars with reduced corrosion protection qualities during the prolonged tap water ponding generally provided very good corrosion protection during the 70 week cyclic testing, i.e., prior to the prolonged tap water ponding. These same factors, defining the performance of some epoxy-coated bars, are essentially the same as those found in the first comprehensive U.S. study of epoxy-coated bars performed by the National Bureau of Standards (NBS) in 1974.

This is a summary of corrosion research sponsored by the Concrete Reinforcing Steel Institute (CRSI) from 1982 to 1992. These efforts concentrated upon the performance of epoxy-coated deformed bar reinforcement. The research consisted of three studies performed by Kenneth C. Clear, Inc. (KCC), and a review of these KCC studies and subsequent laboratory work by Wiss, Janney, Elstner Associates, Inc. (WJE) in 1991 to 1992. The reader may refer to References 1 and 5 for detailed discussion of these efforts.

HISTORY OF CRSI FUNDED CORROSION STUDIES

Long-Term Outdoor Exposure Evaluation

The initial study, funded by CRSI and performed by KCC (*1*), started in 1982. It was a long-term evaluation of small reinforced concrete slabs stored outside. Research variables were epoxy-coated, galvanized and black bars in both upper and lower reinforcing mats, and epoxy-coated and galvanized bars in the top mat with black bars in the bottom mat. The as-produced epoxy-coated bars had holiday and cut area counts averaging 10 to 13/m (3 to 4/ft), with a maximum of 23 /m (7/ft). Three percent NaCl solution was ponded three days a week on the tops of the slabs and then removed for four days a week for a total test period of about 3 years. Cyclic saltwater ponding was discontinued when chloride contents at the top mat bars exceeded 5.9 kg/m^3 (10 lbs/yd^3) of concrete.

After approximately 3 years of cyclic saltwater ponding and air drying, and an additional 5½ years of outside exposure, KCC found that "the only salted slabs... which did not crack... were the slabs with epoxy-coated reinforcing steel (top mat only and both mats coated) (*1*)." About 6 months later, the same researchers reported "Surveys in the fall of 1991 identified hairline cracking varying from 5 to 30 cm (2 to 12 in) long... on all six salted epoxy-coated rebar slabs" (*2*). These were the same slabs reported on previously. No reasons were given for the deterioration of the six slabs during the summer of 1991. One of the slabs was dismantled after 9 years of outdoor exposure and it was reported that the epoxy-coated bar beneath the crack had "significant corrosion" with "blistered and cracked areas" of epoxy coating.

Evaluation of Bent and Straight Epoxy-Coated Rebars from Eight Suppliers

In 1988 eight suppliers of epoxy-coated reinforcement furnished bars for the 3 year CRSI research effort at KCC (*1*) that involved 40 slabs cycled indoors according to the *NCHRP Report 244* Southern Exposure (SE) test method (*3*). This involved weekly soaking of the top

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of the slab with 15% sodium chloride solution for 4 days, followed by 3 days of drying at 38°C (100°F) and exposure to ultraviolet light. Each slab had two mats of reinforcement. One top mat segment consisted of two straight bars and the other top mat was a single bent bar. The bottom mat consisted of uncorroding black bars electrically connected to each top mat segment. Cover for the top bars was 25 mm (1 in) of concrete with a water-cement (w/c) ratio of 0.47.

After 70 weekly SE cycles, corrosion and cracking had occurred on the uncoated bars in the companion slabs. Chloride content levels at the top mat averaged 12.7 kg/m^3 (21.4 lbs/yd^3). With regard to the 36 slabs with epoxy-coated bars, the researchers concluded after 70 SE cycles that:

- Overall, 53 of the 72 epoxy-coated specimens exhibited negligible macrocell corrosion.
- Seventeen exhibited very low macrocell corrosion.
- Two exhibited moderate macrocell corrosion.
- Straight bars performed slightly better than bent bars.
- Mat-to-mat AC resistance measurements indicated there was no degradation of the epoxy coatings on either the bent or straight bars.
- Slab demolition and autopsies on 11 slabs with epoxy-coated bars indicated only minor corrosion on the epoxy-coated bent and straight bars.

Following the 70 SE cycles, the 25 remaining slabs with epoxy-coated bars were ponded continuously with tap water for either 4.5 or 10.5 months, and then stored outside for an additional 9.5 months. The researcher's finding on corrosion performance was "During the continuous ponding, a majority of the epoxy-coated bent and straight rebar specimens underwent a significant change. Mat-to-mat resistances were reduced many fold and macrocell corrosion currents increased significantly to levels commonly seen on uncoated rebars. Almost complete failure of the corrosion protective properties of many of the coated rebars was indicated." Two significant conclusions from the study were 1) "The only variable which had a significant effect on performance was SOURCE" and 2) "... the deterioration is probably the result of the continuously wet environment."

Field (Bridge Deck) Performance of Epoxy-Coated Reinforcing Steel

A total of 85 cores were taken through epoxy-coated reinforcement from 13 bridge decks in the eastern U.S. during the third KCC study (*1*) for CRSI. These bridge decks ranged in age from 9 to 16 years. As indicated by the researchers, "Overall, 87 percent of the top-mat epoxy-coated rebars were essentially corrosion free, and all of the 13 percent exhibiting significant corrosion were from cores with cracks to the rebar level." A cautionary note based on a small sampling of chloride contents indicated that half the cores, particularly those in uncracked areas, contained reinforcement in an environment without sufficient chloride to start corrosion. The other half of the decks had water-soluble chloride contents between 0.6 and 4.7 kg/m^3 (1 and 8 lbs/yd^3 at the rebar level.

WJE Review

In August 1991, WJE was requested by CRSI to review the June 1991 draft KCC report (*4*) entitled "Effectiveness of Epoxy-Coated Reinforcing Steel," which discussed the three CRSI-funded corrosion studies. The following documents and reports were reviewed: data gathered by KCC, findings noted in the KCC reports, and reports and documents from the Federal Highway Administration (FHWA), KCC and CRSI that related to corrosion behavior of epoxy-coated bars. A preliminary review was completed and a brief report was issued to CRSI in October 1991. The report concluded that the high mat-to-mat AC resistance of the embedded epoxy-coated bars in concrete slabs was the dominant factor in determining corrosion performance during the 70 cycles of the SE testing and the subsequent tap water ponding.

Based on this preliminary review, further investigation of the various slab specimens from the 9 year outdoor exposure study and the 3 year eight source straight and bent bar study, was proposed. This investigation consisted of a series of tests to determine the factors contributing to the corrosion performance. Eleven corrosion-tested slabs, 11 companion slabs not salted but stored outdoors for 3 years, and 54 untested companion, retained bare and epoxy-coated bars (as received from the 8 coated bar sources) were obtained from KCC for this WJE study. Four of the long-term outdoor study slabs also were obtained for the WJE study.

WJE CORROSION STUDY

Initial Considerations

During this study, the physical properties of epoxy-coated bars that might affect bar corrosion durability had to be addressed. Some of the more important properties of the bar coatings are:

- **Coating holidays.** A definition of a holiday is any opening in the epoxy coating capable of transmitting corrosion current. Manufacturers utilize holiday detectors that will sense electrical resistances of 80,000 Ω (ohms) or less. They must meet AASHTO/ASTM limitations as to number and types of holidays during coating operations. Bare areas, caused by damage during transport and job site conditions, must also be addressed.
- **Bar surface preparation.** Adequate blasting is necessary to remove scale and contaminants, and give the bar a proper surface profile to which the coating will adhere. Bar surfaces must meet cleanliness criteria before the coating operation.
- **Coating thickness.** The final coating thickness must meet AASHTO/ASTM limits.
- **Other plant operations.** Operations which may affect coating performance, but not easily checked by bar examination, are bar temperature at coating, powder application techniques, proper gel time, cure, etc.

Details of bar surface preparation and coating thickness are measured by quality control personnel at the coating plant. Equipment to measure the suitability of the epoxy cure, etc., is specialized and usually available only to the epoxy powder coating manufacturer. Determination of the number of holidays in bars and patching of damaged areas is a routine operation conducted by plant inspectors prior to shipment.

KCC Test Procedure

The significance of the test procedures utilized by KCC to measure the corrosion of reinforcement inside laboratory concrete slabs must be considered properly to evaluate the laboratory data.

Test measurements utilized during the CRSI corrosion research were:

- Half-cell potentials (ASTM C-876).
- Macrocell corrosion currents between top and bottom layers of reinforcement.
- Instant-off potential. The voltage between top and bottom reinforcement layers immediately after this circuit is disconnected.
- Alternating current electrical resistance between top and bottom layers of reinforcement.

Current, voltage and resistance are related by ohms law, that is the current (amperes) equals the potential (volts) divided by resistance (ohms). Corrosion potentials (I/O) are zero with no corrosion and gradually increase to a nominally constant voltage as corrosion becomes pronounced.

Resistance between bar layers is a function of bare areas of steel (holidays, etc.) in contact with cement paste and of the resistivity of the concrete between the bar layers. Concrete resistivity increases as concrete cures and decreases as electrically-conductive deicing salt permeates concrete pores. Increases in holiday area or ruptures of the epoxy coating during corrosion testing will cause the resistance to decrease.

Figure 1 illustrates the macrocell current relationships between corroding black bars and corroding epoxy-coated bars. Figure 1(a) illustrates that macrocell corrosion currents are a function of the corroding area when black bars are being tested. As shown in Fig. 1(b), this may not be the case with epoxy-coated bars. Epoxy coatings are insulators. Consequently, all macrocell ionic charges are initially funneled to bare steel in contact with cement paste through holiday openings in the coating. Should corrosion occur over significant areas of steel beneath the coating, as in Figure 1(b), measured macrocell currents may not increase significantly until, and if, corrosion pressures break new holes or cracks through the epoxy layer. So long as the area of bare steel at holidays and the I/O potentials at the holidays remain constant, macrocell currents may remain constant also.

In summary, corrosion measurement by electrical testing of slabs containing corroding epoxy-coated bars is not nearly as straightforward as is the testing of slabs with corroding uncoated black bars. Of the three procedures discussed above, the WJE authors believe that the resistance measurements give better information concerning the bar corrosion condition than do I/O potential or corrosion currents. This is because initial resistance measurements give a good indication of the areas of holidays at the start of the test. Changes in resistance are then due either to changes in concrete resistivity or holiday and bare area effects. Since concrete resistivity changes in black-bar companion slabs should be similar to those in slabs with epoxy-coated

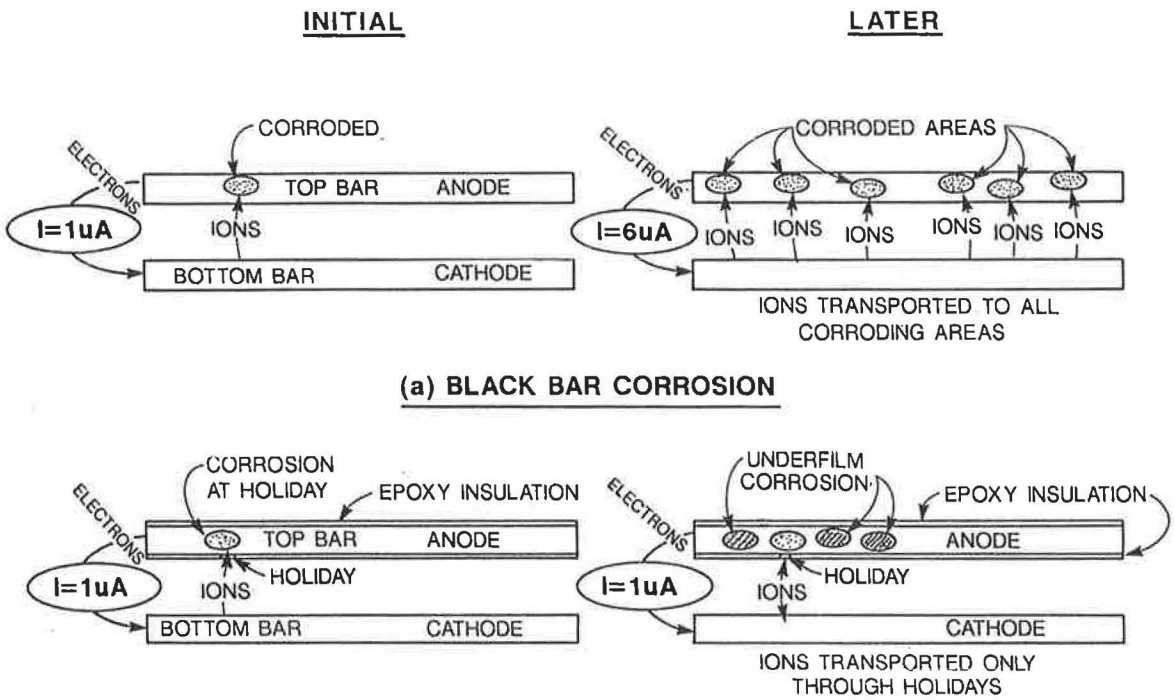

Figure 1 Macrocell current "I" may indicate greater corrosion for black bar than for epoxy coated bar.

bars, changes in resistivity of coated bar slabs can be estimated for a test series.

CRSI Long-Term Outdoor Exposure Evaluation

Four of the 9-year old CRSI slabs from the long-term outdoor exposure evaluation were requested by WJE for examination and testing. Two of these slabs (FBCA 48 and 50) had never been salted. Two slabs (FBCA 1 and 5) had been cyclicly ponded with salt water for 3.1 years and then subjected to outdoor exposure for an additional 5½ years; both FBCA 1 and 5 exhibited hairline cracks above some of the bars when received in November 1991. The four slabs were trimmed at each end and then cut in half in a direction transverse to the reinforcement to obtain 20 cm (8 in) long concrete specimens. Electrical connections between bars were reestablished. Then one-half of each original slab was ponded continuously with 15% salt solution, while the other half was ponded with tap water. This conditioning is continuing with most slab sections (October 1992).

The coated bar beneath the hairline crack in the half of slab FBCA 5 that was ponded at WJE with salt solution eventually developed significant rust exudations. After 175 days of continuous salt solution ponding, final electrical measurements were made using the original electrical hookup (with the top reinforcement mat as the anode and the bottom mat as the cathode). Finally, connections were revised so macrocell currents and A.C. resistance could be measured between individual coated bars and the bottom reinforcement mat. After these measurements, the coated bars were removed from the slab, inspected visually and tested for holidays. Attempts were made to pry or peel the coatings loose from the steel substrate with an "X-Acto" knife (No. 11 fine blade). The steel surface condition was categorized as:

- **Intact.** When it was impossible to dislodge the coating without leaving some coating residue adhering to the steel and otherwise disrupting the remaining coating.
- **Poor bond.** When the coating could be pried loose from the steel without leaving residue on the bright-steel surface.
- **Corroded.** When the steel surface was discolored by corrosion products and there was a general loss of bond between the epoxy coating and steel.

Table I shows the information obtained from the FBCA-5 slab. This slab was previously reported (*1*) to contain 31 holidays and cut areas in the four 61 cm (24 in.) long coated bars used in the top mat, i.e., 12.8/m (3.9/ft) -- about average for the other five corrosion tested FBCA slabs.

Bar B was beneath the longitudinal hairline crack. This bar segment had a high macrocell corrosion current and a low resistance of 210 Ω. When this bar was removed from the concrete, about half the coating area was intact while the remainder was blistered and cracked, making it impossible to do a meaningful holiday survey. All the coating on this corroded bar could be dislodged from corrosion stained substrate steel.

Bar A was adjacent to a surface crack. This bar had a high macrocell corrosion current and low AC resistance at 810 Ω. Ten large "areas" located over corroded steel responded to the holiday detector. Approximately two-thirds of the coating overlaid a steel surface discolored by corrosion. One-third of the coating area was bonded to substrate steel. A minor area (4% estimate) of coating was poorly bonded to bright substrate steel.

Bars C and D were cathodic to the lower steel bar mat and had high resistances of 5,300 and 39,000 Ω, respectively. Eight and five holidays, respectively, were detected for the 20 cm (8 in) long segments C and D. As shown in Table I, areas of corroded steel surface were found on these segments, with the largest areas of corrosion in regions where holidays were close together. Minor areas of disbonded coating over bright steel occurred near a few holidays. Those bars maintaining high resistance provided the best corrosion performance. Testing of the remaining slabs is continuing at WJE (October 1992).

CRSI Evaluation of Bent and Straight Epoxy-Coated Rebars from Eight Suppliers

Eleven corrosion-tested slabs, 11 companion slabs not salted but stored outdoors for 3 years, and 54 untested bare and epoxy-coated bars (as received from the 8 sources) were obtained from KCC. The 11 corrosion-tested slabs had two straight bars (one condition) and one bent bar (another condition). Six conditions had essentially no macrocell corrosion current, two had low current, four had moderate current, and 10 had high corrosion currents when received in November 1991. Numerous tests were performed on these 22 reinforced concrete slab test conditions and their companion, untested epoxy-coated and bare bars.

The details of the extensive WJE study are discussed in the June 1992 report (*5*). The observations and conclusions from this investigation are described below.

The corrosion studies conducted at FHWA and WJE during the past 12 years and those conducted at KCC each used a different corrosion test method. The severity of these test methods, as judged by the corrosion current density measured on the black bar samples, varied widely. At the conclusion of the 70 SE weekly cycles (prior to tap water ponding), the average black bar currents were 2.10 $\mu A/cm^2$ (1.95 mA/ft^2). After the subsequent three to 10.5 months of tap water ponding, the average black bar currents were 3.79 $\mu A/cm^2$ (3.52 mA/ft^2). These values are much higher than the 1.6 $\mu A/cm^2$ (1.50 mA/ft^2) found at the conclusion of the 1-year FHWA "non-specification" study (*6*), the 1.57 $\mu A/cm^2$ (1.46 mA/ft^2) at the conclusion of the 3.1 year ponding period in the 9-year long-term study (*7*), and the 0.96 $\mu A/cm^2$ (0.89 mA/ft^2) at the conclusion of a 48-week SE cycle study (*8*). Therefore, the 70-week SE cycles and tap water ponding conditions were more severe than previous accelerated corrosion studies undertaken on epoxy-coated bars.

Even with such severe test conditions and the fact that numerous coated bars had excessive holidays, the 36 corrosion-tested slabs (with 72 coated bar test conditions) exhibited the following corrosion current density distributions at the completion of the 70 SE cycles:

Corrosion current density less than	% of 72 test conditions
2.10 $\mu A/cm^{2,*}$ (1.95 $mA/ft^{2,*}$)	100
1.08 $\mu A/cm^2$ (1.00 mA/ft^2)	100
0.108 $\mu A/cm^2$ (0.10 mA/ft^2)	96
0.0108 $\mu A/cm^2$ (0.01 mA/ft^2)	74
0.00108 $\mu A/cm^2$ (0.001 mA/ft^2)	44

* Average of black bar companion slabs

These data show that the coated bent and straight bars exhibited low corrosion current densities when compared to the average of 2.10 $\mu A/cm^2$ (1.95 mA/ft^2) for the

Table I PROPERTIES OF EPOXY-COATED BARS FROM FBCA-5 CRSI SLAB AFTER TESTING AND AUTOPSY IN JUNE 1992

Bar Segment	Macrocell Current (μA)	A.C. Resistance (Ω)	Coating-Steel Interface (%)			Holidays Detected per 20 cm (8 in) bar
			Intact	Poor Bond	Corroded	
A	-63	810	30	4	66	10
B	-430	210	--	-	100	Too many to determine
C	+4	5,300	91	3	8	8
D	+1	39,000	89	5	6	5

black bar companion specimens at the end of the 70 SE cycles. These data indicate that 96 percent of the test conditions experienced 20 times less macrocell corrosion than the black bar, 74 percent experienced 200 times less, and 44 percent experienced at least 2000 times less. These 72 test conditions did not exhibit a rust stain or crack at the end of the 70 SE cycles. The four uncoated black bar slabs were rust stained and cracked at this age.

The 72 test conditions exhibited the following corrosion current density distributions after the completion of the subsequent tap water ponding:

Corrosion current density less than	% of 72 test conditions
3.79 $\mu A/cm^{2,*}$ (3.52 $mA/ft^{2,*}$)	99
1.08 $\mu A/cm^2$ (1.00 mA/ft^2)	85
0.108 $\mu A/cm^2$ (0.10 mA/ft^2)	49
0.0108 $\mu A/cm^2$ (0.01 mA/ft^2)	26
0.00108 $\mu A/cm^2$ (0.001 mA/ft^2)	18

* Average of black bar companion slabs

These data show that after tap water ponding, the average macrocell corrosion currents in the straight and bent black bar companions increased by 80 percent, from 2.10 to 3.79 $\mu A/cm^2$ (1.95 to 3.52 mA/ft^2). The data show also that the tap water ponding increased the corrosion currents in some of the epoxy-coated bars. These data indicate that 49 percent of the test conditions experience at least 35 times less macrocell corrosion than the black bar, 26 percent experience at least 350 times less, and 18 percent experience at least 3500 times less.

As discussed previously, direct comparisons of macrocell currents between black bar companion slabs and slabs reinforced with bars coated with an electrical insulator might misrepresent the magnitude of corrosion occurring within the two systems. The relationship between maccocell currents and general corrosion conditions in two slabs is comparable if both slabs have epoxy-coated reinforcement. However, because of this consideration, the most significant information in the preceding tables is that there was about three orders of magnitude difference between the macrocell corrosion currents of epoxy-coated bars after 70 SE cycles and about four orders of magnitude between the currents in the same slabs after ponding with tap water. In addition, those slabs with the lowest macrocell corrosion currents were found to contain essentially corrosion-free epoxy-coated bars.

The KCC data (*1*) show that only 16 of these 72 test conditions developed cracks during the tap water ponding. These 16 conditions did not sustain a high electrical resistance during the tap water ponding period. They ended up with low "final resistance ratios" compared to black bar specimens (*5*). This KCC study contained three series. Series I slabs had three months of continuous tap water ponding, Series II had 10½ months and Series III had 4½ months. The 56 uncracked specimens following the Series I, II and III ponding tests had "final resistance ratios" averaging 62, 150, and 205 for straight bars, respectively, and 33, 105

and 11 for bent bars (*5*). The 56 uncracked coating conditions were supplied from all eight sources. The 16 cracked coating conditions were from sources 1, 2, 3, and 7.

The corrosion performance is related to the holiday count (that is, any hole or defect in the coating that permits current to pass between the bare steel and liquids) and electrical resistance qualities, and not based on the bar source as reported in reference (*1*). High, sustained electrical resistance properties depend upon proper film thickness, good surface preparation and low holiday counts. These same factors were identified for FHWA by NBS in 1974 (*9*). A total of 20 of the 72 epoxy-coated bar test conditions provided these properties in the KCC study following tap water ponding. Seven of the bent bar configurations (4 with patches) and 13 of the straight bar configurations had final corrosion current densities less than about 0.011 $\mu A/cm^2$ (0.01 mA/ft^2), averaging 0.0027 $\mu A/cm^2$ (0.0025 mA/ft^2). These test configurations were supplied by sources 1, 3, 4, 5 and 6. The 13 straight bar specimens maintained an average resistance of about 327,000 Ω and the 7 bent bar specimens maintained an average resistance of about 14,000 Ω following the tap water ponding. The following observations can be made based on the KCC data and studies at WJE.

- **Holidays were the dominant factor in determining the corrosion performance of the tested specimens.** After the 70 weeks of SE cycling, 30 of the 38 retained coated bars with less than 98 holidays/m (30/ft) had companion corrosion tested slabs with current densities less than 0.011 $\mu A/cm^2$ (0.01 mA/ft^2). However, after tap water ponding, slabs with companion bars with 7 to 98 holidays/m (2 to 30/ft) exhibited poorer corrosion performance. Retained bars with 98 holidays/m (30/ft) had companion slabs that provided poor performance during both test phases. Eight retained bars had 3 to 7 holidays/m (1 to 2/ft). Their companion tested slabs performed well in the 70 weeks of SE exposure but had performance varying from excellent to poor when subjected to tap water ponding. Eight retained bars of the 38 with less than 2 holidays/m (1/ft) had companion slabs that exhibited excellent corrosion performance at the conclusion of the 70 weeks of SE cycling and the tap water ponding.
- **Coating films are consistently thinner at the edge of deformations than in the areas between the deformations.** This difference averaged about 0.9 mils with straight bars and about 2.2 mils with bent bars. The edge of the deformation was often times found to be a point of corrosion weakness. Microscopic measurements of coating thicknesses suggest thin films influence holiday formation and can contribute to poor corrosion performance. Microscopic examination found numerous corrosion spots that relate to thin film (2 mils) at regions that contained or developed holidays and corrosion. Undesirable thin films can be identified by laboratory microscopic measurements of bar cross-sections that would not be detected by currently specified testing techniques (that is, magnetic gage measurements taken in the flat areas between the deformation).
- **Electrical resistance, particularly at the start of testing, is a good measure of holidays and the potential corrosion protection qualities of epoxy-coated rebar.** There appears to be a good relationship between the percentage of metal exposed and the electrical resistance properties of a coated bar. The KCC researchers in the 70 week SE cycle study (*1*) measured a wide range in the initial mat-to-mat electrical resistance of the coated bar slabs. These differences correlated with numbers of holidays and electrical resistance properties measured in the follow-up study on companion retained bars (*5*). The follow-up study tested 15 retained coated bars. Resistance ranged from 8 Ω to 450,000 Ω. The 8 Ω coated bar had several large, uncoated areas. Other coated bars having low resistances (30 to 100 Ω) had 33 to 105 holidays/m (10 to 32/ft). Resistance tests on retained coated bars and review of the initial mat-to-mat resistance of the test slabs show that bars with high holiday counts or large bare areas were used in the bent and straight bar study. The resistance test is a good laboratory procedure to determine holiday and potential corrosion protection qualities based on the good correlation between the initial resistance and final corrosion current test results.
- **The 1974 NBS feasibility study (*9*) and the 1980 to 1983 FHWA non-specification bar study (*6*) showed the same type of correlation of initial resistance to corrosion performance.** Review of the straight bar initial mat-to-mat resistance data from KCC showed that the corrosion performance can be estimated based on this initial resistance.
- **Corrosion by-products are indicative of filiform corrosion.** Filiform corrosion results in hair-like corrosion tracks that occur beneath coatings of steel or other metals when exposed to a humid environment. The corrosion originates at a break in the coating. The filiform track is composed of a head and a tail. Corrosion takes place at the anodic head with the tail section being primarily cathodic. The separation between the anodic head and cathodic tail areas allows the cell to move beneath the coating in one direction. Anions

migrate to the head, due to the solution potential difference and the polarity difference between the head and tail. Oxygen and water are supplied to the head through the break in the coating and through the porous tail. Oxygen and water are required for filiform corrosion to propagate. Generally, the higher the humidity, the more rapid growth of this corrosion. When a low relative humidity is reached, the saturated solution cannot be maintained in the head, and the corrosion cell will dry out and stop. The lower relative humidity limit for filiform corrosion on steel is approximately 60 percent (*10*). Filiform corrosion is relatively insensitive to the type of coating and does not correlate to the permeability of the coating. This aspect of the corrosion process of epoxy-coated bars needs further study.

- **Limited scanning electron microscopic (SEM) studies on the cross-section of epoxy chips removed from bars after corrosion slab testing indicated that chloride did not penetrate through the epoxy coating.** Chloride ions were present along the exterior surface and along the steel interface in areas of corrosion beneath the coating, but not within the epoxy coating. This suggests that the chloride did not penetrate the coating, but entered through a break in the coating and traveled along the steel surface.
- **While most bent bars exhibited poorer performance than their companion straight bars, the use of 100 percent patching of bending-induced holidays produced corrosion-free conditions with two bent bar test conditions.** This illustrates the positive effect that patches can have on corrosion performance.
- **The role that osmotic pressure played during the tap water ponding in creating corrosion is still unknown.**

The following are items in the eight-bar source report (*1*) where additional information was needed to effectively evaluate the corrosion performance of coated bars:

- The as-received epoxy film thicknesses and thickness distributions for the 72 test conditions.
- The as-received holiday and bare area counts per meter for these 72 test conditions.
- The as-received backside contamination characteristics of the epoxy films.

Testing by the authors provided data which resulted in the following observations:

- Essentially none of the tested bars achieved the investigation's targeted film thicknesses of 6, 9 and 12 mils.
- Review of the film thicknesses found that seven of the 22 slabs contained bars having film thicknesses less than 5 mils. All of these slabs developed corrosion activity, except for CR9-SE1 where 26 patches were applied to the bent bar holidays.
- The holiday counts performed on 48 coated retained, untested bars were found to be higher than the KCC count data on companion bars.
- The holiday counts on 48 retained, untested coated bars varied from less than 3 to 115 holidays per m (1 to 35/ft). Therefore, numerous as-received coated bars were of "non-specification" quality.
- No correlation was observed using the SEM techniques between the performance of companion specimens in the slab corrosion tests and the presence of contaminates on the backside of the chips removed from retained bars. Contamination of chloride ions was not found in large quantities on the backside of the chips removed from the retained bars.

Factors that did not have a significant influence on the variability of test results were: differences of clear cover over the bars, concrete water absorption, epoxy water absorption, surface roughness of blasted bars prior to coating, backside contamination on the epoxy film, and curing of the epoxy coatings.

Other factors which need to be considered in testing and long-term structure durability are:

- The corrosion research studies during the period 1975 to 1990 (*1,6,7,8,11,12,13,14*) on unprotected black rebar and epoxy-coated rebar have utilized clear cover of 19 to 25 mm (¾ to 1 in) and w/c ratios between 0.47 and 0.53. These test conditions were selected by the researchers to produce "worst case experiments" as noted in the 1983 report (*6*). These conditions result in early initiation of corrosion of unprotected, black rebar and early cracking of concrete over these black bars with minimal cover. The 3-year FHWA-sponsored corrosion study (*8*) with 0.50 w/c ratio concrete showed that the time-to-corrosion for black bar with 25 mm (1 in) cover occurred after six weeks of SE cyclic testing. With 50 and 76 mm (2 and 3 in) clear cover, there was no corrosion with the same black bar and a 0.50 w/c ratio concrete after 48 weeks of SE cyclic testing. The time-to-corrosion for black bar was eight times longer when 50 or 76 mm (2 or 3 in) clear cover was used when compared to 25 mm (1 in).
- Another significant consideration is the time prior to cracking. Concrete test specimens with 19 to 25 mm (¾ to 1 in) clear cover will crack earlier and with less corrosion-induced pressure than specimens with

corroding bars with 50 or 76 mm (2 or 3 in) of cover. AASHTO clear cover specification requirements for the top mat were changed from 25 mm (1 in) to 50 mm (2 in) in 1974 and the AASHTO w/c ratio was changed in 1974 to a maximum of 0.44. Thus, bridge decks constructed in the last 18 years have 50 to 76 mm (2 to 3 in) of cover and higher strength concrete. Projections of serviceability life for black or epoxy-coated rebars, based upon corrosion research utilizing 19 or 25 mm (¾ or 1 in.) clear cover and high w/c ratios, will not reflect the actual serviceability conditions of black or epoxy-coated rebars embedded deeper in more crack resistant concrete.

SUMMARY

To summarize, excellent corrosion protection was achieved under the severe eight-bar source corrosion study when slabs reinforced with straight and bent bars with original retained coated bars having less that 3 to 7 holidays per m (1 to 2/ft) were tested. Properly patched bent bars with bending-induced damage and bent bars that were not patched, and which had the same low holiday counts, were essentially corrosion free. Detained coated bars with 7 to 115 holidays per m (2 to 35/ft), very thin film regions, or large areas of defective coating were associated with slabs which provided less corrosion protection and became corroded during the prolonged tap water ponding. This reduction in corrosion protection should be anticipated since every "holiday" can pass corrosion current when subjected to rigorous corrosive environments, such as were imposed upon the 8-bar source specimens. These same bars with reduced corrosion protection qualities during the prolonged tap water ponding generally provided very good corrosion protection during the 70 SE cycles, i.e., prior to the prolonged tap water ponding. Field cores by KCC (*1*) from 13 bridge decks ranging in age from 9 to 16 years showed that 87 percent of the top-mat epoxy-coated bars were essentially corrosion-free.

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APPENDIX A

EFFECTIVENESS OF EPOXY-COATED REINFORCING STEEL (C-SHRP REPORT: EXECUTIVE SUMMARY)

Reprinted with written permission of C-SHRP. The full C-SHRP report may be obtained by contacting the Canadian Strategic Highway Research Program, Transportation Association of Canada, 2323 St. Laurent Boulevard, Ottawa, Canada K1G 4K6, TEL: 613/736-1350, or FAX: 613/736-1395.

Kenneth C. Clear*

This Canadian Strategic Highway Research Program (C-SHRP) project, initiated in early 1990, was aimed at determining the effectiveness and long-term (50 years or more) performance of fusion-bonded epoxy coatings in preventing the corrosion of reinforcement in highway structures exposed to environments representative of Canadian conditions. Such conditions include deicing salt, freeze-thaw and cold and moderate temperature marine exposures.

Phase I of the project, completed in August 1990, involved the definition of the current state of knowledge, the status of usage of epoxy coated reinforcing steel (ECR) in Canada, and a return to fundamentals involving definition of the characteristics and tests utilized in the pipeline, rebar and other fields to define quality and project future performance. Phase II involved the acquisition and testing of epoxy coated reinforcing bars from 12 Canadian and U.S. coaters, 7 Canadian and U.S. jobsites and 19 field structures constructed in Canada and the northern U.S. between 1974 and 1988. Phase II also included six months of environmental exposure of ECR's in Toronto to simulate jobsite storage prior to concrete placement and an update of the state of knowledge.

The ECR's from U.S. coaters and the northern U.S. field structure cores were originally obtained and partially evaluated in a previous study by the author for the Concrete Reinforcing Steel Institute [C-SHRP Report Reference 11]. Work in that study also contributed to the fundamentals and Phase I state of the art report and to the development of new test methods for evaluating epoxy coated reinforcing bars. A third ongoing effort, National Cooperative Highway Research Program Project 10-37, provided significant input into the state of the art update, and studies by the University of South Florida and the Florida Department of Transportation in the area of failure mechanisms were most helpful.

The coated rebars were tested by visual examination for corrosion and visible coating flaws, determination of holidays (pinholes not visible to the eye), microscopic examination for underfilm contamination and foam in the coating, anchor pattern on the steel substrate, coating hardness, coating adhesion, and the electrical insulating properties of the coating. They were then subjected to two tests to project future performance: a chemical immersion test involving immersion for 45 days in saturated limewater with sodium chloride and an accelerated corrosion test involving similar immersion for 7 days and the application of a two volt DC external voltage. Both these tests are similar to tests used during the original development of ECR and the tests presently specified in the specification ANNEX [*ASTM Specification A 775/90*] for qualifying new epoxy powders. A total of 131 cores, containing 157 epoxy coated rebars, were obtained from the field structures and analyzed for concrete properties as well as properties of the ECR. Overall, more than 3,000 individual measurements were made on 317 epoxy coated rebars, 173 cores and 93 ECR concrete specimens under laboratory and outdoor exposure.

The state of the art evaluations and the field and laboratory testing suggest that fusion-bonded epoxy coatings will not be effective in providing long-term (50 years or more) corrosion protection to reinforcement in salt-contaminated concrete. An unexpected epoxy coated rebar failure mechanism involving progressive loss of coating adhesion and underfilm corrosion has

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been identified as active in northern and southern field structures and in very high quality coated rebars in laboratory and outdoor exposure specimens. The author believes that the data indicate that the increase in life of epoxy coated rebar structures over those constructed with uncoated rebar in northern U.S. and Canadian environments (marine and deicing salt) will be in the range of only 3 to 6 years in most instances; rather than the more than 40 years previously estimated.

Because means of overcoming the unexpected failure mechanism have not yet been devised and confirmed, there is insufficient knowledge today to prepare improved specifications which will ensure long-term (50 years or more) corrosion protection when only ECR and conventional concrete are used in severe chloride environments. Thus, the author believes that present and proposed specifications (even if tightly enforced and modified to require close examination and patching of all bare areas; covered, above-ground bar storage prior to use; increased minimum coating thickness (to 7.0 mils (180 micrometers)) and increased severity of the bend test) will not provide assurance of long-term (50 years or more) performance in salt-contaminated concrete. Based upon the findings of outdoor exposure studies in northern environments, these modifications may, however, increase the time to deterioration by another 5 years or so (i.e., to a total of about 8 to 11 years more than that for an equal structure constructed with uncoated rebar) if they can be truly implemented in the "real world". If longer low maintenance lives are desired, other protective systems will be required.

Future efforts should be aimed at obtaining a better understanding of the failure mechanisms of ECR and the coating properties which control performance; the development of new quality control tests; the modification of coatings, production and construction practices to obtain better performance; and should include long-term studies at both the macroscopic and microscopic levels and in various environments. Structural concerns involving the bond and creep characteristics of concrete members with epoxy coated rebars which have experienced loss of coating adhesion should be addressed. European and Japanese epoxy powders and coated rebar technologies should be studied, and efforts should be undertaken to define economical means of prolonging the low maintenance life of existing epoxy coated rebar structures.

APPENDIX B

SELECTED BIBLIOGRAPHY OF EPOXY-COATED REBAR LITERATURE

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