EPOXY COATED
REINFORCEMENT STUDY

Final Report

STATE RESEARCH PROJECT #527

by

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for

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16. Abstract

This report evaluates the use of Scotchlite 213 epoxy coated reinforcement in Oregon coastal environments. There is an extensive body of knowledge documenting epoxy coated reinforcement research in North America in the last 20 years. The research has produced mix results. However, recent studies conducted by Clear and others for the National Cooperative Highway Research Program, by Kessler and others for the Florida Department of Transportation and by Weyers and others in Virginia, provide evidence of poor performance of epoxy coated reinforcement in coastal bridge structures.

In 1989, the Oregon Department of Transportation removed a concrete test beam reinforced with Scotchlite 213 epoxy coated reinforcement after nine years of service from Yaquina Bay in Newport, Oregon. Results of the testing and evaluation showed that there was adhesion loss of the coating attributed to low blast profile of the steel and low coating thickness. There was observed corrosion along the longitudinal bars and hoop reinforcement that were located within the tidal zone.

Another concrete beam reinforced with Scotchlite 213 epoxy coated reinforcement was removed from Yaquina Bay in 1998 after eighteen years of exposure. The testing and evaluation showed that:

1. Half-cell potential measurements within the beam's tidal zone exceeded the 90% probability threshold (-0.35 V) for corrosion to occur.
2. The chloride concentrations were significantly elevated within the beam's tidal zone.
3. The adhesion loss was greatest within the tidal zone and in some areas, there was total loss of adhesion.
4. Most of the observed corrosion of the Scotchlite 213 epoxy coated reinforcement was within the tidal zone.

Based on the literature documenting previous studies and ODOT's testing and evaluation conducted in 1989 and in 1998, the use Scotchlite 213 epoxy coated reinforcement for long term protection against corrosion in coastal bridges is not recommended.

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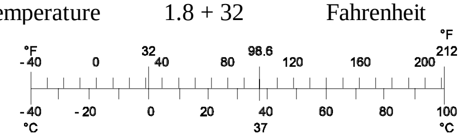
SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
In	Inches	25.4	Millimeters	mm
Ft	Feet	0.305	Meters	M
Yd	Yards	0.914	Meters	M
Mi	Miles	1.61	Kilometers	Km
		AREA		
in^2	square inches	645.2	millimeters squared	mm^2
ft^2	square feet	0.093	meters squared	M^2
yd^2	square yards	0.836	meters squared	M^2
Ac	Acres	0.405	Hectares	Ha
mi^2	square miles	2.59	kilometers squared	Km^2
		VOLUME		
fl oz	fluid ounces	29.57	Milliliters	mL
Gal	Gallons	3.785	Liters	L
ft^3	cubic feet	0.028	meters cubed	m^3
yd^3	cubic yards	0.765	meters cubed	m^3
NOTE: Volumes greater than 1000 L shall be shown in m^3.				
		MASS		
Oz	Ounces	28.35	Grams	g
Lb	Pounds	0.454	Kilograms	Kg
T	short tons (2000 lb)	0.907	Megagrams	Mg
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5(F-32)/9	Celsius temperature	°C

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	Millimeters	0.039	inches	in
m	Meters	3.28	feet	ft
m	Meters	1.09	yards	yd
km	Kilometers	0.621	miles	mi
		AREA		
mm^2	millimeters squared	0.0016	square inches	in^2
m^2	meters squared	10.764	square feet	ft^2
ha	Hectares	2.47	acres	ac
km^2	kilometers squared	0.386	square miles	mi^2
		VOLUME		
mL	Milliliters	0.034	fluid ounces	fl oz
L	Liters	0.264	gallons	gal
m^3	meters cubed	35.315	cubic feet	ft^3
m^3	meters cubed	1.308	cubic yards	yd^3
		MASS		
g	Grams	0.035	ounces	oz
kg	Kilograms	2.205	pounds	lb
Mg	Megagrams	1.102	short tons (2000 lb)	T
		TEMPERATURE (exact)		
°C	Celsius temperature	1.8 + 32	Fahrenheit	°F



* SI is the symbol for the International System of Measurement

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FINAL REPORT

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1.0 INTRODUCTION

1.1 BACKGROUND

A major concern for transportation agencies is the chloride-induced corrosion of reinforcing steel in concrete bridges located in coastal environments. One measure taken to counteract the onset of corrosion in structural concrete is the use of epoxy coated reinforcement. In the last twenty years, epoxy coated reinforcement has been used extensively in bridge decks and substructures to protect against corrosion brought on by de-icing salts or marine environments. In Oregon, there is a considerable concern about the effects of corrosion in the coastal environment. Chloride induced corrosion is the primary contributor to the deterioration of Oregon's coastal bridges *(McGill et al. 1999).*

In the 1970s and 1980s, it was a commonly accepted view that epoxy coatings would provide superior corrosion protection for reinforcing steel in concrete. For that reason, epoxy coated reinforcement was routinely used in North American bridge decks and substructures. In addition to epoxy coated reinforcement, cathodic protection systems have also been used to protect bridge substructures. In Oregon, major historic coastal bridges along Highway 101, such as Cape Creek Bridge, Yaquina Bay Bridge and Depoe Bay Bridge employ cathodic protection systems. Additionally, stainless steel reinforcement was recently used as a corrosion protection measure in the construction of Brush Creek Bridge on the southern Oregon Coast.

This research project focused exclusively on epoxy coated reinforcement. The other methods of corrosion protection were not addressed in the research. To better understand the basis for using epoxy coated reinforcement, basic principles of steel corrosion are reviewed in the next section.

1.2 CORROSION PRINCIPLES

The epoxy coating is intended to serve as a physical barrier to protect the reinforcing steel. Concrete normally provides a passive, electrically neutral film around the steel because of the high pH of the concrete. However, when chlorides in the presence of water and oxygen, penetrate the concrete, the pH is reduced and the passive film is destroyed. This allows the corrosion reaction to start at the steel surface. As diagrammed in Figure 1.1, the corrosion reaction consists of a simultaneous cathodic reaction and anodic reaction. The steel acts as an electrode that couples the two reactions.

<u>Anodic Reaction</u>

In the anodic reaction, iron atoms lose electrons (oxidation) because the passive film has been destroyed.

$$\mathbf{Fe \rightarrow Fe^{2+} + 2e^-}$$

The iron ions further react with the hydroxides to produce ferrous hydroxide.

$$\mathbf{Fe^{2+} + 2OH^- \rightarrow Fe(OH)_2}$$

Ferrous hydroxide reacts in the presence of the diffused oxygen and this results in the production of rust, the final corrosion product at the anodic site.

$$\mathbf{4\,Fe(OH)_2 + O_2 \rightarrow 2Fe_2O_3\ (rust) + 4H_2O}$$

<u>Cathodic Reaction</u>

Electrons flow through the steel where they react with water and oxygen to form hydroxide ions.

$$\mathbf{4e^- + 2H_2O + O_2 \rightarrow 4OH^-}$$

In the cathodic reaction, oxygen atoms are reduced and form hydroxide ions that further sustain the anodic reaction.

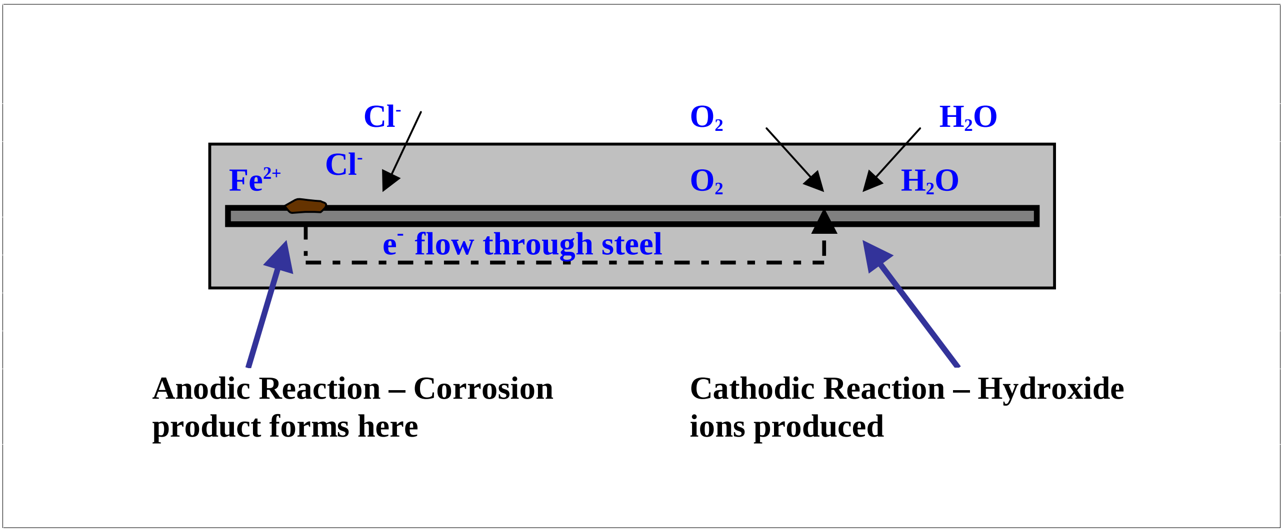

Figure 1.1: Corrosion Reaction at the Cathodic and Anodic Sites

The corrosion process is a natural electrochemical process where electron flow occurs in the steel and at the same time, hydroxide ions are conducted through an electrolyte, which in this case, is the pore water in the concrete. This completes an electric circuit. The completed circuit is known as a corrosion cell.

Two theories about the protection capability of epoxy coated reinforcement have been advanced:

- Physical Barrier Theory – The epoxy coating acts as a barrier, preventing chloride ions and other aggressive matter from coming into contact with the steel surface.

- Electrochemical Barrier Theory – The epoxy coating acts as a high resistance coating, reducing corrosion by increasing electrical resistance between neighboring coated steel locations where the cathodic reactions can take place *(Weyers et al. 1998)*.

These two theories offer sound reasoning for the use of epoxy coated reinforcement as a protective measure for structures located in marine environments. The Oregon Department of Transportation (ODOT) recognized the significance of epoxy coated reinforcement as a potential long-term solution to protect its coastal bridges from corrosion and felt it was important to determine its effectiveness in this environment.

1.3 STUDY OBJECTIVES

In 1980, ODOT placed six concrete test beams in Yaquina Bay adjacent to pier #3 of the Yaquina Bay Bridge in Newport, Oregon. The six beams were bolted onto the legs of an existing concrete dolphin protecting the bridge pier. Figure 1.2 shows the location of the dolphins alongside the bridge pier. Figure 1.3 shows a closer view of the concrete beams attached to the dolphin.

Figure 1.2: Overall View Showing Concrete Dolphins in Yaquina Bay

As seen in Figure 1.3, there are junction boxes at the top of each of the beams. The beams were pre-wired for taking half-cell potential measurements. The beam dimensions are 200 mm x 200 mm x 6.1 m. They were attached to the dolphin at a location where part of the beam was always in the water, another section was always exposed to the air and the middle section was in the tidal zone between the low and high tide levels of Yaquina Bay.

Figure 1.3: Concrete Beams Attached to the Dolphin in the Middle of the Picture

The beams contained four epoxy coated longitudinal bars (#5) and epoxy coated hoops (#3) spaced at 300 mm. The design compressive strength of the concrete was 41,370 MPa. The reinforcement was coated with Scotchlite 213 by Dura Coatings, Inc. of Springfield, Oregon and inspected at the coating plant by ODOT prior to casting. Figure 1.4 shows the plan and section views of the beam design.

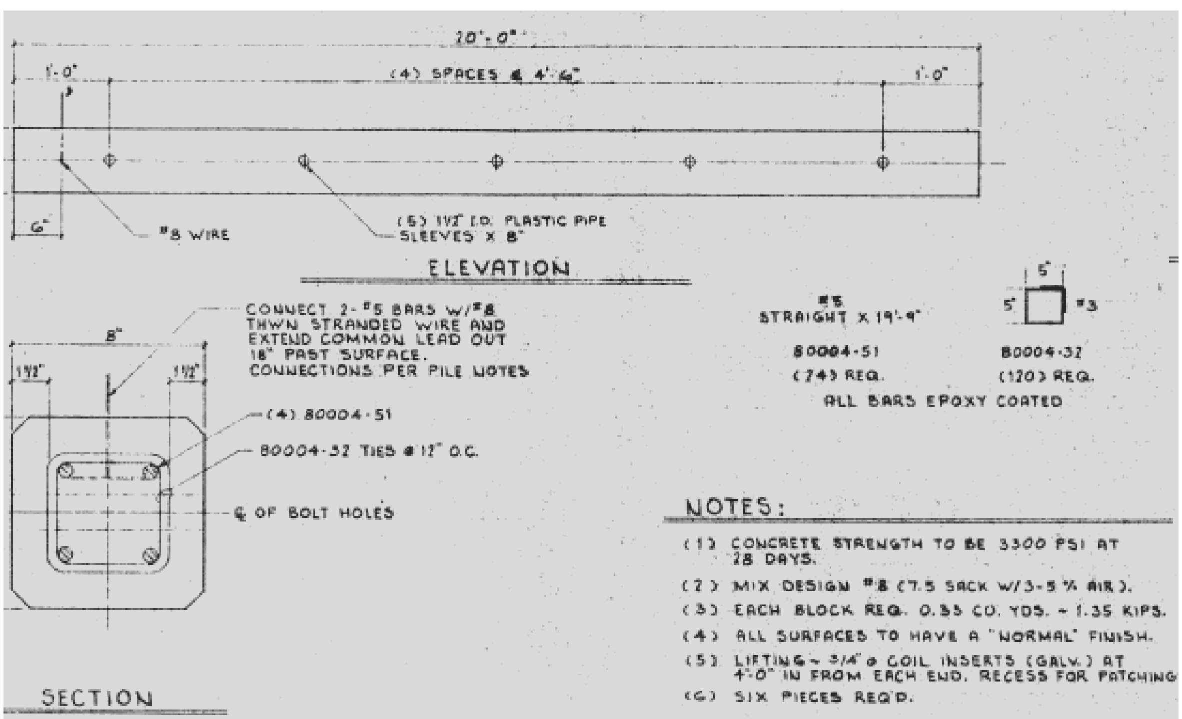

Figure 1.4: Concrete Beam Details

The concept of coating steel reinforcement for corrosion protection seems to be a valid practice. However, in the last decade, studies have shown that epoxy coated reinforcement does not provide the long-term protection (40-50 years) that had been originally forecasted. To further validate or refute the findings from previous research, this study was conducted to examine and assess the effectiveness of the Scotchlite 213 epoxy coated reinforcement used on one of the test beams in Yaquina Bay after 18 years of service. It should be clear to the reader that this study is based on the performance of Scotchlite 213 epoxy coated reinforcement and is not intended to provide broad conclusions about epoxy coated reinforcement or other protective coatings.

The specific objectives of this study included:

- Review the results from a 1989 ODOT study on the Scotchlite 213 epoxy coated reinforcement from one of the six test beams removed from Yaquina Bay after nine years of service.

- Review the previous studies conducted in North America to determine the current state of the practice with epoxy coated reinforcement.

- Visually inspect the concrete test beam to establish the character of cracking and surface defects in the concrete.

- Test the concrete and Scotchlite 213 epoxy coated reinforcement for characteristics related to corrosion, i.e., adhesion loss, half-cell potential, and chloride content.

- Analyze the results of the testing and compare findings with the previous 1989 ODOT study and the current body of information.

- Develop conclusions about the effectiveness of Scotchlite 213 epoxy coated reinforcement for use in Oregon coastal environments.

In the next chapter, epoxy coating disbondment mechanics and failure mechanisms will be addressed in greater detail as previous studies of epoxy coated reinforcement in North America are reviewed and summarized.

2.0 LITERATURE REVIEW

The performance of epoxy coated reinforcement has been studied extensively for the last twenty years. Some studies have shown favorable results, while other research has documented poor performance.

Clear and Sohanghpurwala *(1990)* studied the corrosion characteristics of straight and bent epoxy coated reinforcing steel on a total of 40 small scale concrete slabs. Their results indicated that the epoxy coating on the straight and bent bars improves resistance to chloride induced corrosion.

Clear has conducted other notable research on the effectiveness of epoxy coated reinforcement. His 1992 study, for the Canadian Strategic Highway Research Program (C-SHRP), evaluated epoxy coated reinforcement on 19 structures from 8 states and provinces in the United States and Canada. He found that epoxy coated reinforcement typically outperformed uncoated rebar, but the increased performance was not long term. Clear determined that on epoxy coated rebar samples extracted from the 19 sites where chloride contents were high, the epoxy coated reinforcement showed signs of corrosion after 8 to 16 years of field service.

Clear further hypothesized that the increase in life of epoxy coated reinforcement in northern U.S. and Canadian environments would be in the range of only 3 to 6 years in most instances, rather than the more than 40 years previously estimated. This was attributed to the failure of the epoxy coating due to progressive loss of coating adhesion and under-film corrosion *(Clear 1992)*.

Clear and others *(1995)* performed further laboratory and field research to evaluate the performance of epoxy coated reinforcement. In this study for the National Cooperative Highway Research Program (NCHRP), an extensive literature review was conducted, noting the previous results of good and poor performance of epoxy coated reinforcement in laboratory and field conditions. The authors also examined production plant procedures, job site practices and quality control methods. Field-testing was performed at bridge construction sites.

Coating failure mechanisms were identified as under-film corrosion in association with either conductive pathways through the coating, wet adhesion loss, cathodic disbondment or corrosion at coating defects, or combinations of these.

The following describes each of the failure mechanisms:

- <u>Under-film corrosion</u> results from the migration of reactants (water, oxygen and chlorides) through the concrete to the steel substrate *(Clear et al. 1995)*.

- <u>Wet adhesion loss</u> occurs when water migrates through the coating. As the water molecules migrate through the coating, they exchange with the substrate's polar adhesion sites. The water causes separation of the coating from the substrate due to the buildup of water molecules at the coating/steel interface *(Riemenschneider 1989)*.

- Cathodic disbondment takes place when coating defects (holidays and bare areas) are present. Cathodic disbondment occurs in association with anodic activity at the defect once a critical chloride concentration is reached. When water and oxygen diffuse in the concrete to the steel, hydroxide ions are produced at the cathodic site. The hydroxide ions occupy space between the coating and the steel and results in disbondment of the coating from the steel. There, the bond strength of the coating to the steel substrate immediately surrounding the anodic sites is reduced so that the coating can be easily removed from the steel *(Martin et al. 1995)*.

Clear and others *(1995)* recommended that for the epoxy coating to effectively protect reinforcement from corrosion, "improvements in practices, procedures, and quality control for each step in the process from bar fabrication to concrete placement are necessary."

In contrast to Clear's work, other research indicates good performance of epoxy coated reinforcement. A 1993 study by Erdogdu and Bremner, "Field and Laboratory Testing of Epoxy-Coated Reinforcing Bars in Concrete," documented the performance of epoxy coated reinforcement in concrete that was exposed to simulated marine environments in Maine. Concrete slab samples with damaged epoxy coated reinforcement (1% to 2% of the coating removed prior to casting) and undamaged epoxy coated bars were evaluated in laboratory and field conditions over a two year study period. The laboratory results showed no measurable corrosion on the undamaged bars for the entire test period. The damaged bars exhibited some corrosion. In the field tests, both the damaged epoxy coated bars and the undamaged epoxy coated bars were in good condition with only small pits detected on the damaged bars.

In a field evaluation of Indiana bridge decks, Hasan and others *(1995)* examined epoxy coated reinforcement extracted from bridge deck cores taken from six bridges exposed to chlorides from de-icing. They found no indication of rusting or debonding on any of the bars. Furthermore, a visual inspection of the samples from which the coating was mechanically stripped showed no sign of under-film corrosion.

More recent research supports the findings from Clear's later studies. In Virginia, Weyers and others *(1998)* carried out a field investigation of the effectiveness of epoxy coated reinforcement in three bridge decks subject to de-icing salts and three bridges located in coastal environments. They also conducted laboratory testing and analysis on test samples from various commercial coaters. The laboratory specimens were submerged in a concrete pore water solution with added sodium chloride and saturated in oxygen.

The authors found in the laboratory testing that corrosion occurred at damaged areas where bare steel was exposed and at thinly coated areas. In the areas of thin coating, the corrosion process started with the formation of blisters that eventually grew and cracked.

In their field investigation of the piles supporting the three coastal bridges, Weyers and others found that the coating on all of the cored specimens from two of the bridges had completely debonded. The specimens from the third bridge were in various stages of disbondment. The piles were seven to eight years old. The authors found that in Virginia, the epoxy coating will lose bond in less than seven years. They presumed that regardless of the type of epoxy used to coat the steel, wet adhesion loss in marine environments would occur. Once the coating separates, the steel is subject to under-film corrosion as the chlorides migrate to the steel.

Probably the most compelling evidence of inadequate performance of epoxy coated reinforcement has been with the Florida Key Bridge failures *(Kessler et al. 1993)*. Inspections in the late 1980s revealed that four of five major bridge substructures showed significant corrosion. The bridges were relatively new; all had been in service for less than 10 years. The corroding epoxy coated reinforcement (straight and bent bars) was located in the tidal zone and subjected to relatively high airborne salt spray and cycles of wetting and drying. High air and water temperatures also contributed to the adverse environment. Inspection of samples revealed coating disbondment from the steel.

Investigation of construction practices also revealed that disbondment occurred on bars that were coated before, as well as after, fabrication. Furthermore, the coated reinforcement from the epoxy coating plant was in excellent condition when leaving the plant. The reinforcement examined at the construction sites was also within specification tolerances for damages to the coating (2% per 30.5 cm). In a subsequent inspection of 14 other Florida bridges, all but one showed disbondment of the coating from the steel. Disbondment occurred even in the absence of chlorides. As a result of these findings, the Florida Department of Transportation no longer uses epoxy coated reinforcement in their bridge construction.

Many other studies have evaluated the performance of epoxy coated reinforcement. A large number of them are summarized in the NCHRP Report 370, "Performance of Epoxy-Coated Reinforcing Steel in Highway Bridges" *(Clear 1995).*

Oregon's research has focused on in-service evaluations of the test beams in Yaquina Bay. The first such evaluation was conducted in 1989 after the beams had been in Yaquina Bay for nine years. After another nine years of exposure, a second beam was removed from the Bay in 1998, then examined and evaluated.

The next chapter discusses the 1989 test beam in-service evaluation. Chapter 4 provides an extensive review of the 1998 evaluation.

3.0 1989 OREGON DEPARTMENT OF TRANSPORTATION STUDY

In June, 1989 one of the test beams was removed from Yaquina Bay to examine the condition of the epoxy coated reinforcement. The beam was removed from the Bay and lifted onto the Yaquina Bay Bridge. It was then wrapped in burlap (soaked in seawater) and plastic, then transported to ODOT's Materials Laboratory in Salem, Oregon for testing.

At the Laboratory, half-cell potential measurements were taken to develop a potential profile for the beam. Figure 3.1 provides the half-cell testing results. The tidal zone on the beam was estimated to be between 1.0 to 3.35 m, as measured from the bottom end of the beam.

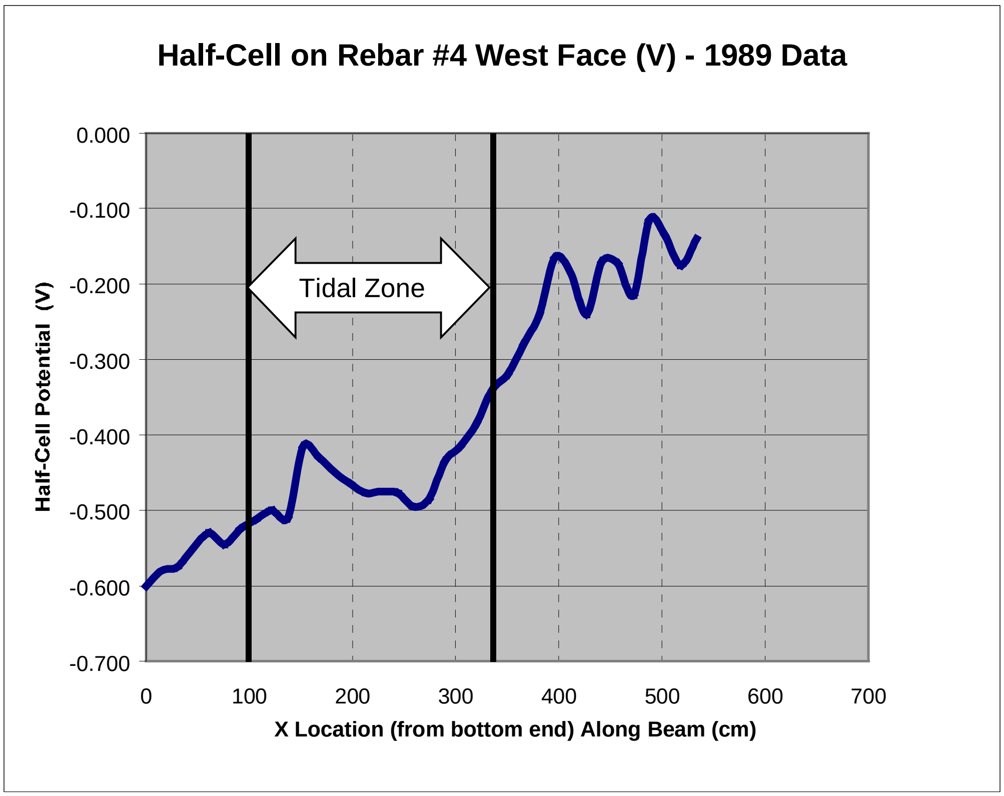

Figure 3.1: 1989 Half Cell Potential Profile for the Removed Test Beam

ASTM Standard Test Method (C 876) was used to interpret the half-cell measurements. It states:

- If potentials are more positive than -0.20 V, then there is a 90% probability that no corrosion is occurring.

- If potentials are in the range of -0.20 V to -0.35 V, then the corrosion activity is uncertain.

- If potentials are more negative than -0.35 V, then there is a 90% probability that corrosion is occurring.

As can be seen in Figure 3.1, the potential for corrosion within the tidal zone was more negative than the -0.35 V threshold, indicating a 90% probability that corrosion was occurring.

In the 1989 evaluation, the tape measurement shown in the figures documenting the observed corrosion used the top end of the beam as a reference point. In the 1998 evaluation and in this report, the bottom end of the beam is used as the reference point. Therefore, in the discussion in this chapter, the locations are referenced from the top end of the beam and are followed by a conversion that references the same location from the bottom end of the beam.

After the half-cell potential tests were completed, the concrete was removed and the epoxy coated reinforcement exposed for visual examination. Corrosion areas were observed on two of the longitudinal #5 bars as well as several #3 hoops. Figures 3.2 through 3.4 illustrate the corrosion on the longitudinal bars measured from the top end of the beam. The locations were within the beam's tidal zone, 3.0 to 4.4 m from top end of the beam (1.3 to 3.1 m from the bottom end).

Figure 3.2: Corrosion Along the Longitudinal Bar

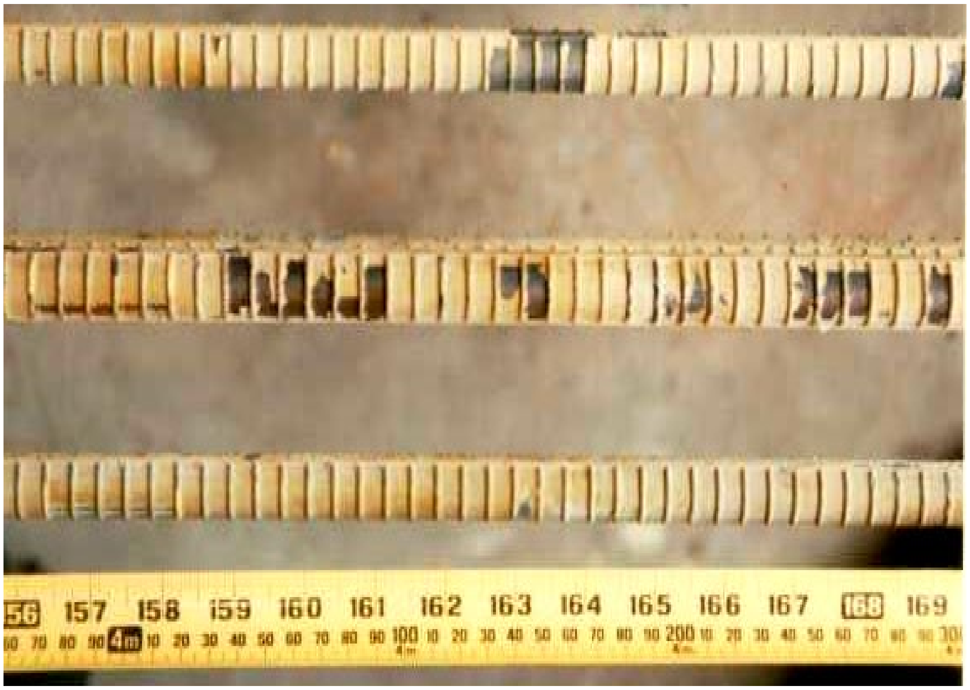

Figure 3.3: Corrosion Along the Longitudinal Bar

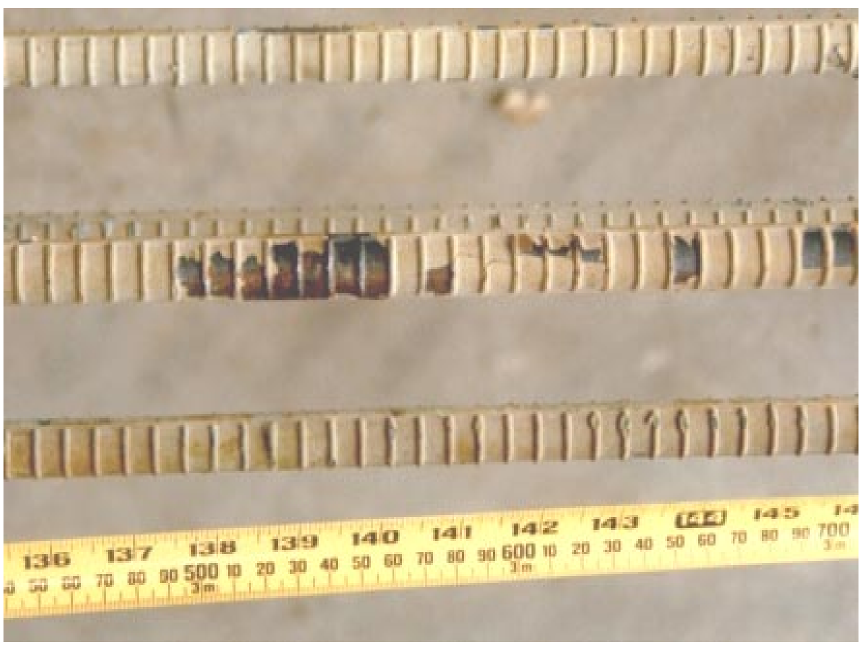

Figure 3.4: Corrosion Along the Longitudinal Bar

There was also corrosion noted on hoop reinforcement located between 3.5 and 4.0 m measured from the top end of the beam (2.1 to 2.6 m from bottom end). Figures 3.5 and 3.6 show corrosion, also occurring in the tidal zone.

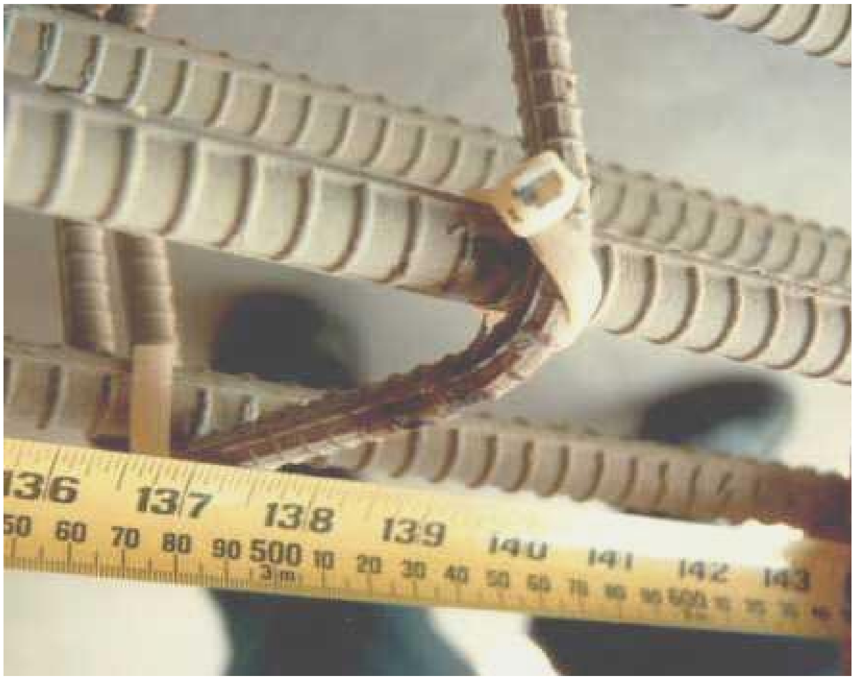

Figure 3.5: Corrosion Along a #3 Hoop

Figure 3.6: Corrosion Along a #3 Hoop

Following the visual examination, three samples of the Scotchlite 213 epoxy coated reinforcement were given to the 3M Company in September 1989, for further testing at the 3M laboratory in Austin, Texas.

There, the following equipment was used to provide a more extensive evaluation:

- Surface Profilometer – Measured the blast profile and roughness depth of the reinforcing steel.

- Differential Scanning Calorimeter (DSC) – Measured the glass transition temperature and amount of reaction remaining in an epoxy powder or cured film to provide information on how well the coating was cured.

- Scanning Electron Microscope (SEM) – Provided an enlarged view of the coating and steel surface in order to determine the amounts of chloride, sodium, calcium and iron deposits on the sample.

Results of the testing revealed that there was considerable adhesion reduction on two of the three samples. Factors that contributed to the reduction were:

- Low blast profile. The blast profile measurements for the three samples were recorded as 0.8 mils (0.02 mm), 0.9 mils (0.02 mm) and 1.0 mils (0.03 mm). The samples were lower than normal specified range of 1.5 to 4.0 mils (0.04 to 0.10 mm).

- Low coating thickness on two of the three samples (4 to 6 mils (1.0 to 2.0 mm)).

- Permeability of water through the coating.

Another factor helping bring about the adhesion loss was attributed to cathodic disbondment. As noted earlier, when water and oxygen diffuse in the concrete to the steel, hydroxide ions are produced at the cathodic site. The hydroxide ions occupy space between the coating and the steel and results in disbondment of the coating from the steel.

After the tests were completed at the 3M Laboratory, ODOT opted to leave the remaining five beams in Yaquina Bay for an extended period. They were accessible and could easily be removed when further laboratory examination and testing were required.

In 1998, ODOT decided to remove another test beam from Yaquina Bay to evaluate the long-term effectiveness of Scotchlite 213 epoxy coated reinforcement over a more extended period (18 years). The next chapter describes that effort and the results of the testing and evaluation.

4.0 1998 OREGON DEPARTMENT OF TRANSPORTATION STUDY

In September 1998, a second test beam was removed from Yaquina Bay. This time, the beam was removed and offloaded directly onto a barge (Figure 4.1) where it was transferred to shore. The beam was wrapped in soaked burlap and plastic and transported by truck to the ODOT Materials Laboratory in Salem, Oregon.

Figure 4.1: The Test Beam being Lifted onto the Barge.

After removing the barnacles and surface buildup along the beam's exterior, the beam was examined for surface cracking. Considering the long-term exposure to the harsh environment of the Oregon Coast, the beam appeared to be in relatively good condition.

4.1 VISUAL EXAMINATION OF THE CONCRETE BEAM

As seen in Figures 4.2 and 4.3, transverse cracking was noted on the beam, particularly in the upper half. The spacing of the cracks in this section occurred about every 200 to 250 mm, which mirrored the spacing of the hoop reinforcement. There are several possible factors that could have contributed to the cracking. These include:

- Reduced depth of cover of concrete at the hoop locations (50 to 64 mm);

- Expansion and contraction of the hoop reinforcement; and
- Corrosion of the hoop reinforcement with subsequent disruption and cracking of the concrete.

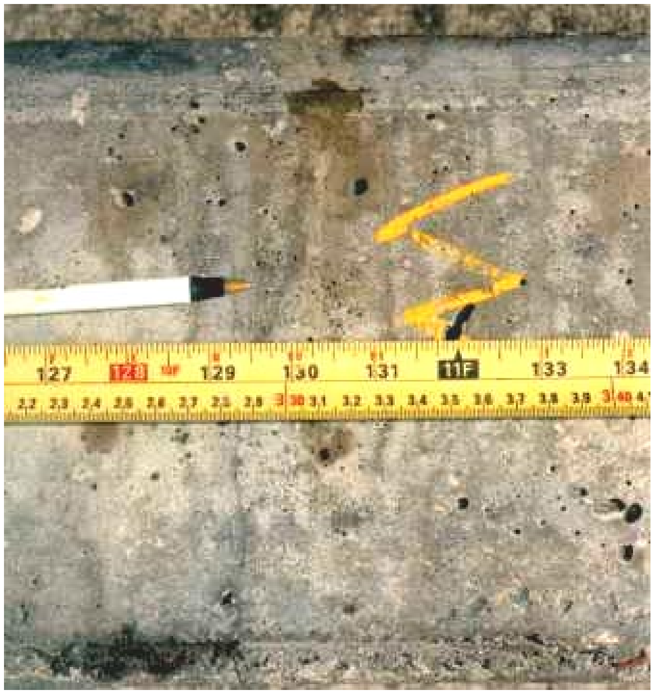

Figure 4.2: Transverse Cracking Propagated from the Reinforcing Hoop

Figure 4.3: Transverse Cracking Propagated from the Reinforcing Hoop.

There was also some random cracking noted in the tidal and submerged zones of the beam (Figure 4.4). These were hairline type cracks with no apparent pattern. Cracks were hard to identify in these areas because of the surface discoloration caused by the barnacles.

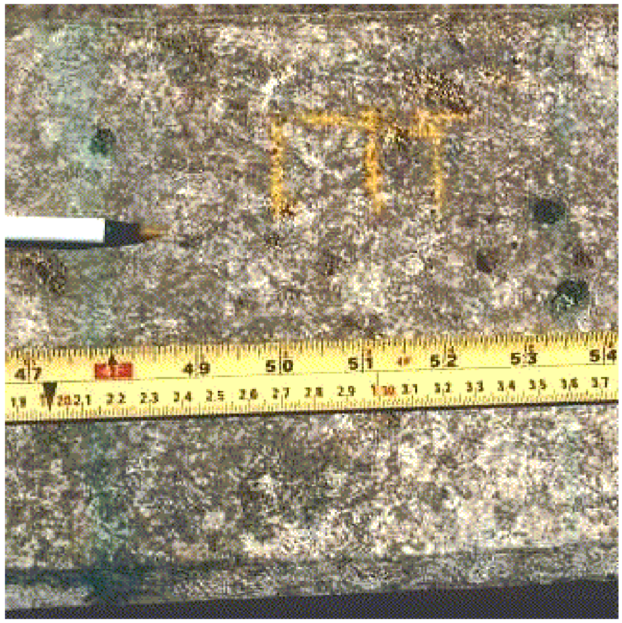

Figure 4.4: Hairline Cracks Noted in the Submerged Zone

Of the four exposed faces of the beam, the west face had the most surface defects, especially in the submerged zone. Figure 4.5 shows some of the pockmarks that were found on this face of the beam.

Figure 4.5: West Face of the Beam with Pockmarked Areas

4.2 HALF-CELL POTENTIAL TESTING

After the crack locations were recorded, half-cell potential measurements were taken. The beam had been cast with built in junction boxes and electrical connections to the reinforcing steel to facilitate the half-cell testing. The testing was done for each exposed face of the beam from the bottom to the top at approximately 150 mm intervals (Figure 4.6). The north face was not considered since this face abutted the concrete dolphin in Yaquina Bay. The half-cell measurements were made using a battery operated voltmeter and a copper-copper sulfate reference electrode.

Figure 4.6: Half-Cell Potential Measurements Using Copper-Copper Sulfate Electrode

Four separate groups of half-cell measurements were taken. They included:

- Half-Cell on Rebar #3 South Face
- Half-Cell on Rebar #4 South Face
- Half-Cell on Rebar #3 East Face
- Half-Cell on Rebar #4 West Face

For reference, Figure 4.7 shows the cross section of the beam and the rebar numbering layout for the rebar relative to the top end of the beam.

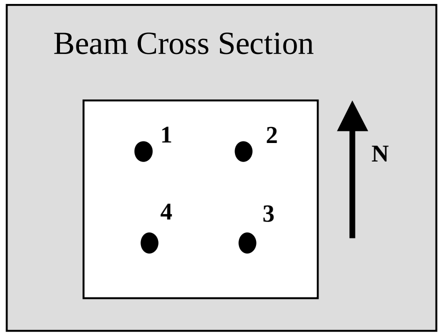

Figure 4.7: Cross Section of the Test Beam Relative to the Top End of the Beam

The test results are included in Appendix A. The information is also presented graphically in Figure 4.8, where the half-cell potentials for each of the four groups are plotted versus the horizontal position along the beam face. As described in Chapter 3, the ASTM Standard Test Method (C 876) was used to interpret the half-cell voltages.

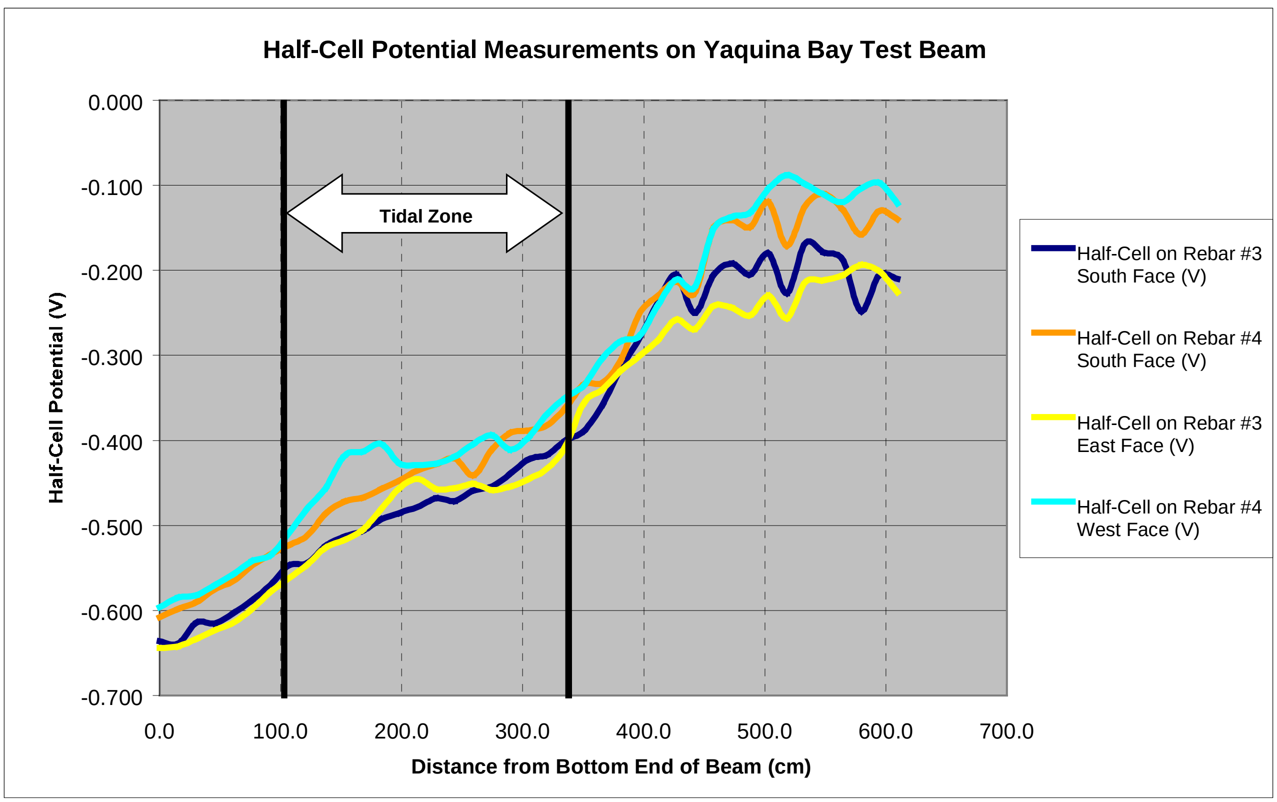

Figure 4.8: Half-Cell Potential Measurements Along Each Exposed Face of the Beam

Figure 4.8 shows there were no wide variations in the readings for each group. In the submerged zone (0 to 1.0 m), the potential levels are more negative than "-0.50 V". The potential for corrosion is therefore much higher in this zone. However, because of the lack of available oxygen in the seawater, the corrosion process is very slow.

In the tidal zone, oxygen is readily available for diffusion into the concrete and the corrosion rate in the presence of chlorides is much higher. Within this zone, the half-cell potential measurements were more negative than the “-0.35 V” threshold, indicating a 90% probability that corrosion could occur.

The numeric values of the half-cell potential measurements decreased as the distance along the beam increased. Within the dry zone, the readings were in the “-0.40 V to -0.35 V” range at the tidal zone/dry zone interface, but became more positive towards the top end of the beam. Based on the half-cell potential readings, the probability of corrosion occurring in the higher portions of the dry zone was low.

Figure 4.9 shows a comparison of the half-cell measurements taken from the 1989 test beam to the half-cell measurements taken in 1998. The two graphs are reasonably close and there appears to be little difference in the two sets of half-cell potential measurements.

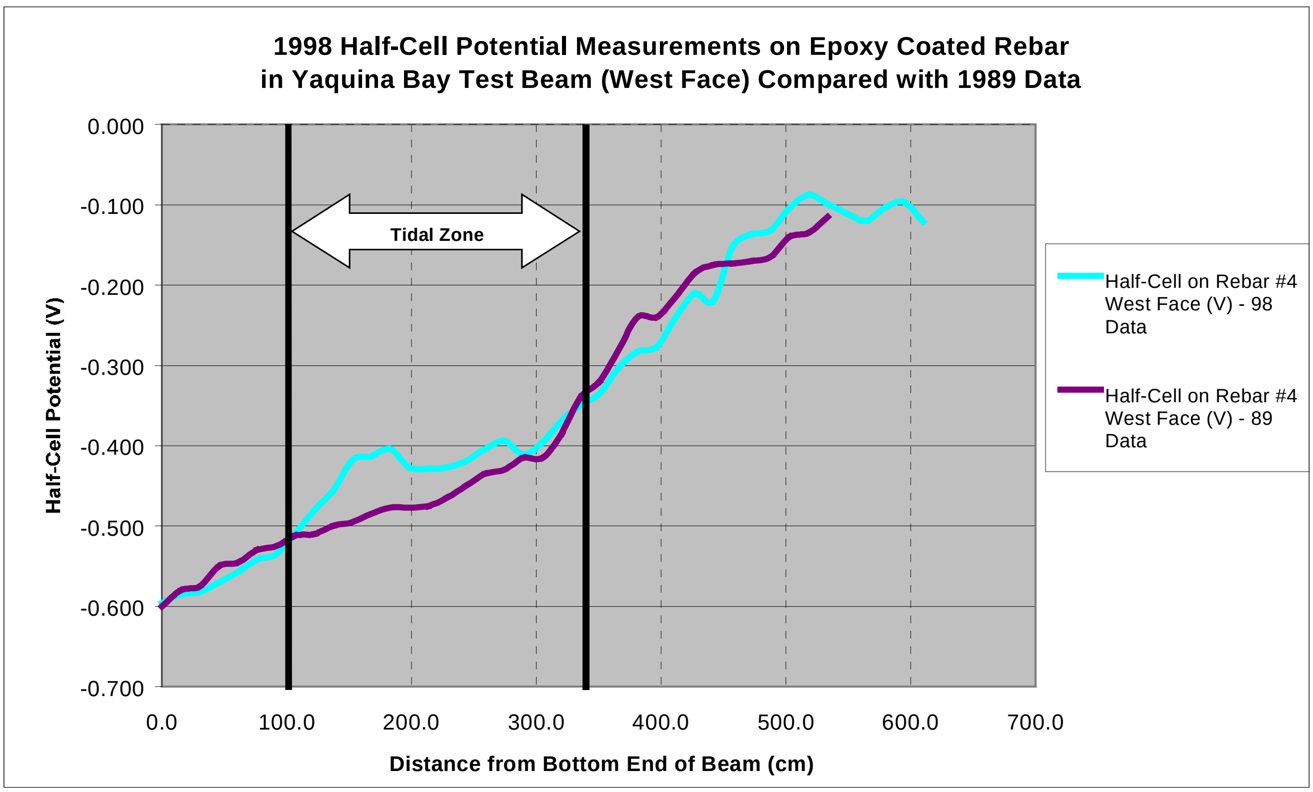

Figure 4.9: Comparisons of Half-Cell Potential Measurements for 1998 and 1989 Along the West Face for Bar #3

4.3 CHLORIDE PROFILE ALONG THE BEAM

4.3.1 Sampling and Testing Along Length of the Beam

Chloride samples were taken along the west face at 200 mm intervals for the entire length of the beam. Sampling was done in accordance with AASHTO T 260-95, “Sampling and Testing for Total Chloride Ion in Concrete and Concrete Raw Materials, Method 1.” Each sample was comprised of material taken from the beam's surface to a depth of 100 mm. Chloride concentration was then determined by testing for total chlorides in the sample and total chloride concentration by cement. The latter test was done to reduce errors occurring as a result of variable levels of aggregate in the powder sample. The results of the testing are shown in Figure 4.10.

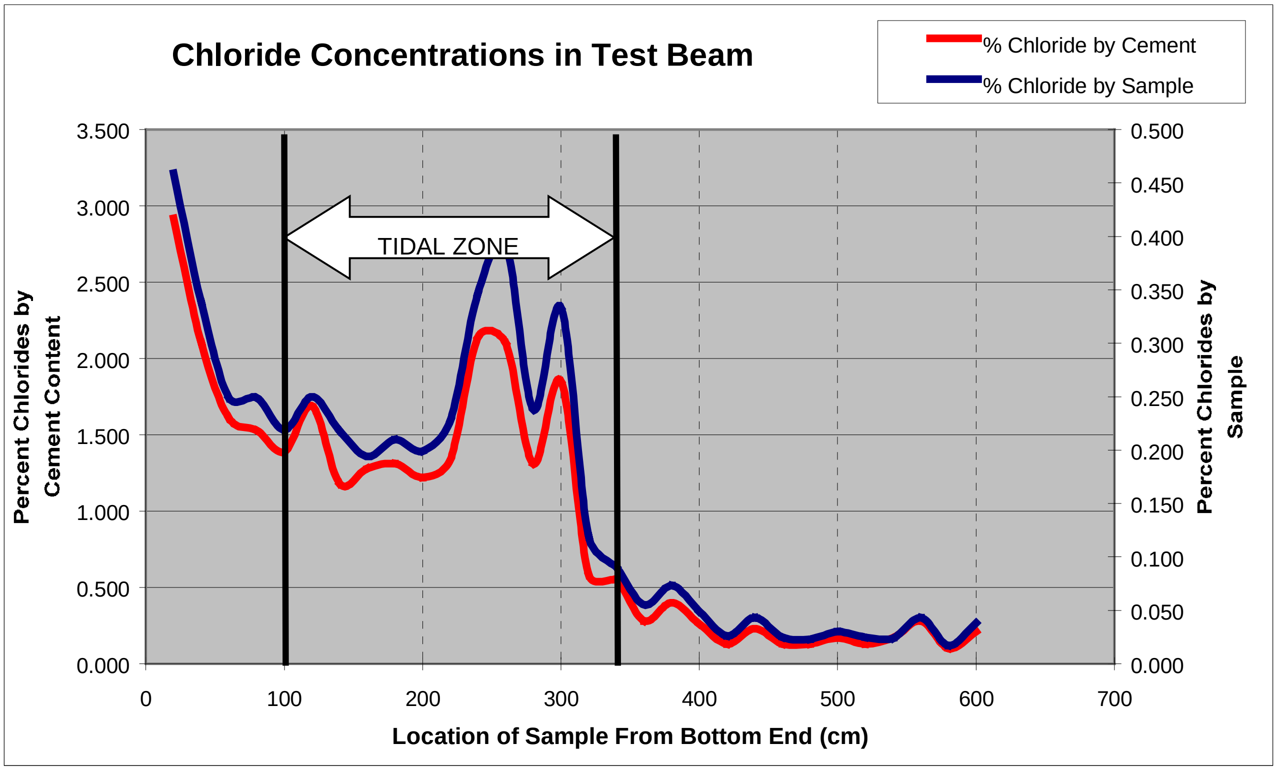

Figure 4.10: Chloride Concentrations Along the West Face From the Bottom End of the Beam

4.3.2 Chloride Sampling and Testing at Increasing Depth Intervals

A chloride profile by depth was also determined. For this, samples were taken on the east face of the beam. Eight samples in each of the three zones (submerged, tidal, and dry) were prepared. A sample consisted of a composite of material taken at ten locations within each zone for a specific depth interval. The samples were taken at depth intervals of 12.5 mm, starting at 0 to12.5 mm (zero being the surface of the beam) and continuing to a depth interval of 87.5 to 100.0 mm. This resulted in samples at eight depth intervals per zone. The results of the testing are shown in Figures 4.11 to 4.14. The data points shown in each graph represent the midpoint of the depth interval.

In the dry zone (Figure 4.11), the graphs show that the chloride concentration decreases with increasing depth to a depth of 62.5 mm where the concentration is negligible. At greater depths, the chloride concentration increases slightly. There is no apparent explanation for this other than possible sampling or testing error at these low concentrations.

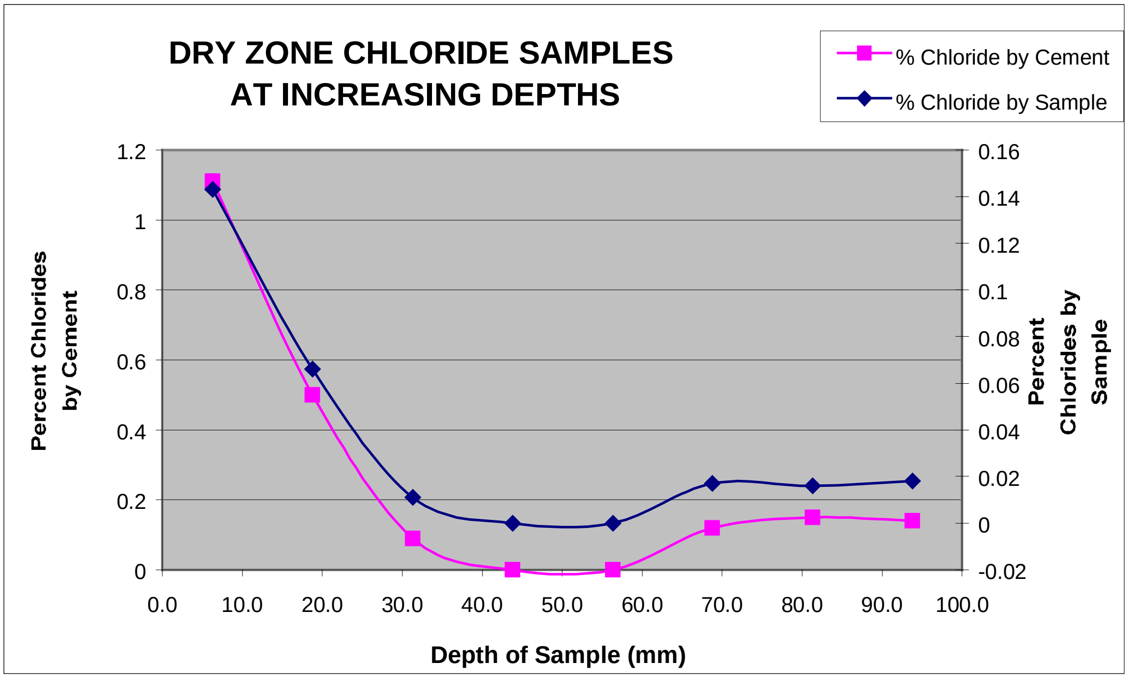

Figure 4.11: Chloride Testing on Samples Taken at Varying Depth Levels in the Dry Zone

In the tidal zone (Figure 4.12), chloride concentration decreased as the depth where the sample was taken increased. At the interval, 87.5 to 100 mm however, the concentrations did appear to increase slightly, which again could be the result of sampling or testing error.

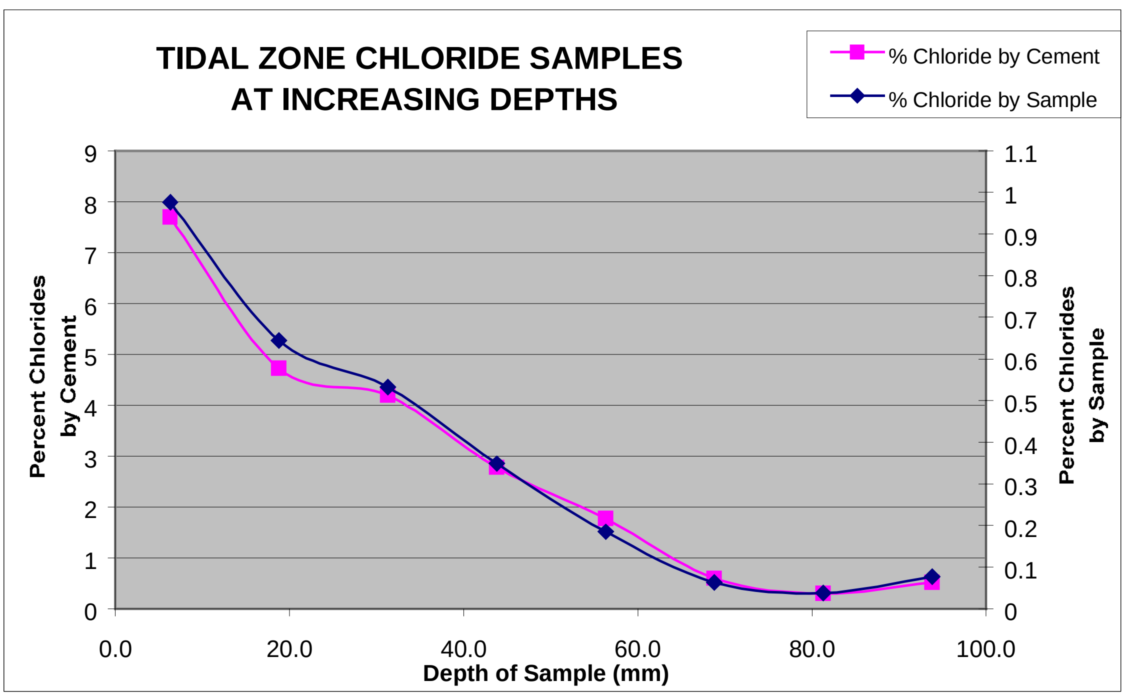

Figure 4:12: Chloride Testing on Samples Taken At Varying Depth Levels in the Tidal Zone

In the submerged zone (Figure 4.13), the concentrations did not follow the classical pattern recorded for sampling in the other two zones. As depth increased, chloride concentration also increased for three of the eight samples (12.5 to 25.0 mm, 37.5 to 50.0 mm, and 62.5 to 75.0 mm). As a result of this irregular pattern, a second group of chloride samples were taken for the submerged zone (Figure 4.14).

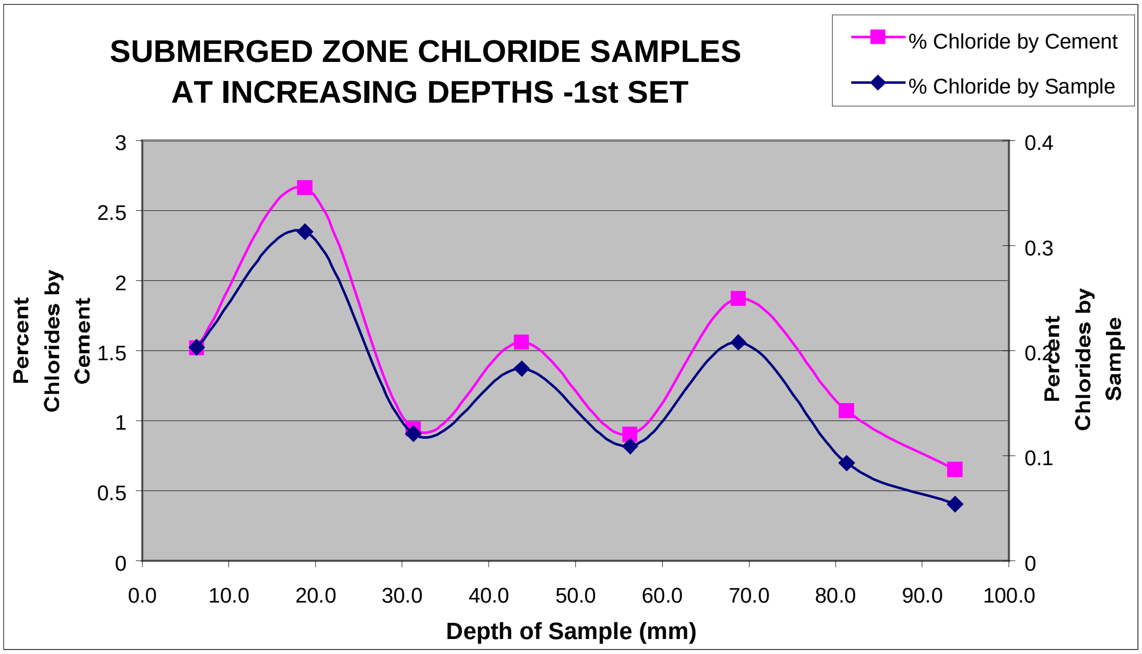

Figure 4.13: Chloride Testing on the First Set of Samples Taken at Varying Depth Levels in the Submerged Zone

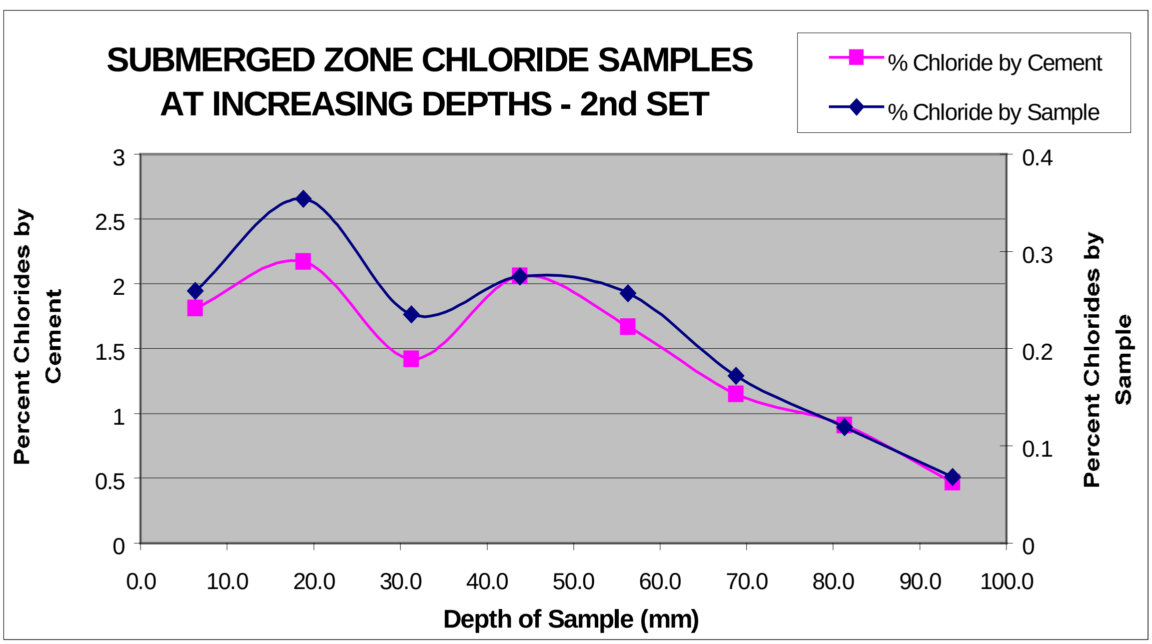

Figure 4.14: Chloride Testing on the Second Set of Samples Taken at Varying Depth Levels in the Submerged Zone

As seen in Figure 4.14, the chloride concentrations did increase at two of the same depth intervals (12.5 to 25.0 mm and 37.5 to 50.0 mm) as previously measured in the first set of samples for the dry zone. There is no apparent explanation for the irregular pattern in the two groups of dry zone samples.

Note that with the four groups of samples, the chloride concentrations were greater in the tidal zone (Figure 4-12).

4.4 VISUAL EXAMINATION OF THE EPOXY COATED REINFORCING STEEL

Upon completion of the chloride sampling and testing, the beam was cut into three equal sections for ease in handling and then the concrete was removed from the reinforcing steel. Care was taken not to damage the epoxy coating when removing the concrete. Once the concrete was removed, the Scotchlite 213 epoxy coated reinforcement was further examined. This included:

- Adhesion testing to establish the degree of disbondment.
- Blast profile measurements to determine the adequacy of the blast cleaning on the exposed rebar surface.
- Visual inspection of the Scotchlite 213 epoxy coated reinforcement to note and record corrosion activity.

4.4.1 Adhesion Testing

Adhesion tests were conducted at 200 mm intervals along the surface of two of the longitudinal bars. A scale of 1 to 5 was used to rate the adhesion of the epoxy to the bar. A value of “1” indicated the coating could not be pried from the rebar with a knife. A value of “5” represented total adhesion loss, where the coating had visibly separated from the reinforcing steel. The results of the adhesion tests are presented in Table 4.1.

Reduced adhesion was observed along most of the reinforcing steel surface. The most significant loss appeared to be in the tidal zone where, in some locations, the coating had completely disbonded. In the dry zone, the coating had better adhesion.

Adhesion ratings in the submerged zone ranged from 2 to 5. On bar #3, the coating had completely separated from the surface at 200 mm from the bottom end and there was visible evidence of corrosion on the surface.

Table 4.1: Testing Results for the Epoxy Coating Adhesion

Adhesion Test Results on Epoxy Coating				
East Face-Rebar #3			West Face-Rebar #1	
Distance from Bottom End	Adhesion Rating		Distance from Bottom End	Adhesion Rating
200	5		200	3
400	2		400	4
600	2		600	2
800	3		800	2
1000	3		1000	3
1200	3		1200	3
1400	3		1400	3
1600	3		1600	3
1800	4		1800	3
2000	4		2000	3
2200	4		2200	3
2400	4		2400	3
2600	3		2600	4
2800	2		2800	5
3000	4		3000	5
3200	4		3200	4
3400	3		3400	4
3600	3		3600	3
3800	3		3800	3
4000	3		4000	2
4200	3		4200	2
4400	3		4400	4
4600	3		4600	3
4800	3		4800	3
5000	3		5000	3
5200	1		5200	3
5400	2		5400	3
5600	1		5600	3
5800	2		5800	3
6000	1		6000	2

5 – Visible disbondment
4 – Flakes off with finger
3 – Disbonds with square cut
2 – Piece of coating disbonds with square cut
1 – Slight tearing
0 – No disbondment

– Tidal Zone

4.4.2 Blast Profile of the Reinforcing Steel Surface

AASHTO A775-97, "Standard Specification for Epoxy Coated Reinforcing Bars", states:

> "Average blast profile maximum roughness depth readings of 1.5 to 4.0 mils (0.04 to 0.10 mm), as determined by replica tape measurements using NACE RP-287-87, shall be considered suitable as an anchor pattern."

The blast profile on the reinforcing steel was measured at locations along the longitudinal bars within the tidal zone. The blast profile testing was accomplished by replica tape impressions ("PRESS-O-FILM™") measured with a micrometer. A total of 150 profile measurements were taken. The mean blast profile for longitudinal bars in the tidal zone was 3.00 mils (0.08 mm), with a 95% confidence interval of +/- 0.27 mils (0.007 mm).

Based on the AASHTO A775-97 specification, the bar blast profile of the reinforcing steel was adequate.

This is in contrast to the 1989 testing conducted by the 3M Company. As noted in Chapter 3, the blast profile average on three samples was measured below the normal specification range. The blast profile measurements for the three samples were recorded as 0.8 mils (0.02 mm), 0.9 mils (0.02 mm) and 1.0 mils (0.03 mm). Why the blast profile seemed better on the reinforcement tested in 1998 is unknown. The logical response is because the bars tested in 1998 did in fact have a better blast profile. Alternatively, the variation could be attributed to the different test methods, since the 1989 measurements were made using a surface profilometer.

4.4.2 Visual Inspection of the Reinforcing Steel

The Scotchlite 213 epoxy coated reinforcement was closely examined in each of the three zones. Photographic images were taken to document the effects of adhesion loss and corrosion on the reinforcing steel surface.

4.4.2.1 Submerged Zone

Corrosion was observed at approximately 200 mm and 950 mm from the bottom end along rebar #3 (Figure 4.15). The other bars appeared to have no visible signs of corrosion. Some corrosion was found on each of the hoops (Figure 4.16).

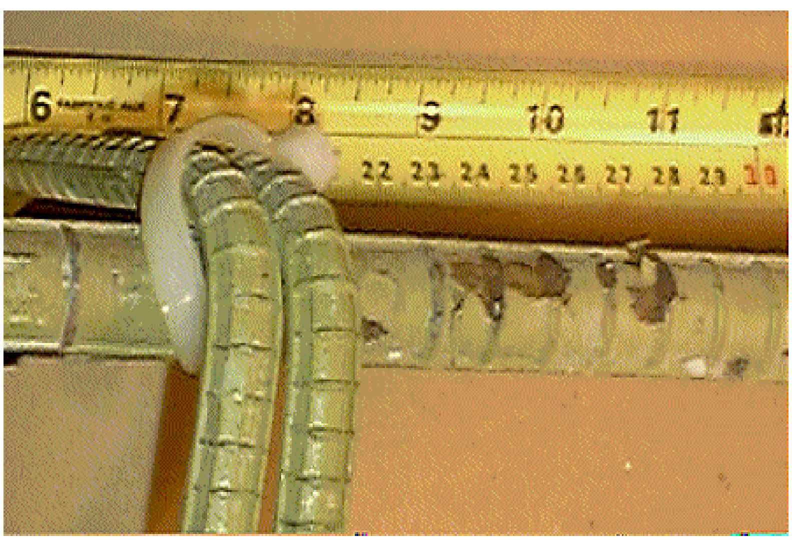

Figure 4.15: Observed Corrosion of Longitudinal Rebar in the Submerged Zone

Figure 4.16: Observed Corrosion of Hoop in the Submerged Zone

4.4.2.2 Tidal Zone

Corrosion was observed in many areas on the Scotchlite 213 epoxy coated reinforcement along rebar # 1 and the hoops. There was extensive corrosion between 2.6 and 3 m as measured from the bottom end of the beam along rebar #1. The other bars had no visible signs of corrosion. Figures 4.17 and 4.18 show two areas of corrosion along rebar #1.

Figure 4.17: Corrosion Along Rebar #1 (2.8 to 2.95 m from bottom end of beam)

Figure 4.18: Corrosion Along Rebar #1 (2.75 m from bottom end of the beam)

There was corrosion observed in all but one of the hoops in the tidal zone. The hoop that did not show signs of corrosion, was positioned in the beam at the high end of the tidal zone. Figures 4.19. and 4.20 illustrate the degree of corrosion appearing on the hoop reinforcement.

Figure 4.19: Hoop Corrosion in the Tidal Zone

Figure 4.20: Hoop Corrosion in the Tidal Zone

As noted earlier, there was significant adhesion loss of the coating within the tidal zone. An example of disbondment of the coating from the steel is shown in Figure 4.21.

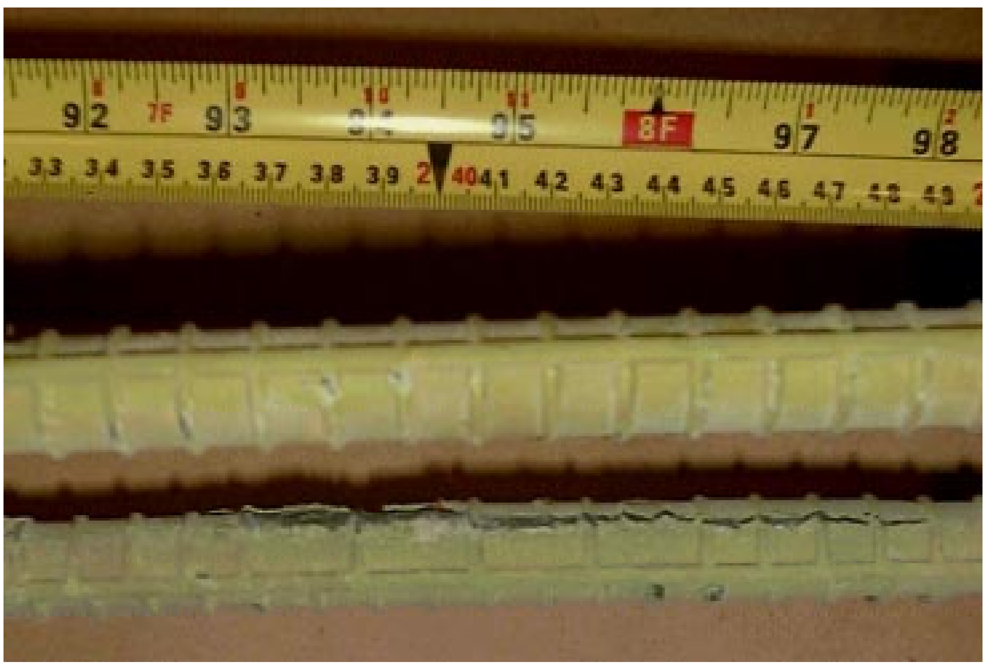

Figure 4.21: Disbondment of the Epoxy Coating in the Tidal Zone

4.4.2.3 Dry Zone

Within the dry zone, most of the Scotchlite 213 epoxy coated reinforcement appeared to be in good condition. Visible corrosion was not apparent in this area. There was some disbondment of coating closer to the tidal zone boundary (Figure 4.22).

Figure 4.22: Disbondment of the Coating at the Tidal and Dry Zone Interface

5.0 CONCLUSIONS

There has been widespread use of epoxy coated reinforcement throughout North America in the last 25 years. The literature documents a large number of research projects investigating the use of epoxy coated reinforcement, with some of the research showing favorable performance. However, recent studies conducted for the National Cooperative Highway Research Program, for the Florida Department of Transportation and in Virginia, document poor performance of epoxy coated reinforcement in bridge structures located in coastal environments *(Clear et al. 1995; Kessler et al. 1993; Weyers et al. 1998)*. Failure mechanisms causing the poor performance included:

- Under-film corrosion because of the migration of water, oxygen and chlorides through the concrete and epoxy to the steel surface;

- Wet adhesion loss resulting in the separation of the coating from the substrate; and

- Cathodic disbondment of the epoxy coating from the reinforcing steel which starts at coating defects.

In 1989, ODOT removed a concrete test beam reinforced with Scotchlite 213 epoxy coated reinforcement after nine years of service from Yaquina Bay in Newport, Oregon. The beam and the Scotchlite 213 epoxy coated reinforcement were tested and evaluated at the ODOT Materials Laboratory and at the 3M Company's laboratory in Austin, Texas. Results of the testing and evaluation showed:

- Half-cell potential measurements within the beam's tidal zone that exceeded the 90% probability threshold (-0.35 V) for the occurrence of corrosion.

- Corrosion on the longitudinal bars and hoop reinforcement at locations within the tidal zone.

- Adhesion loss of the epoxy coating on the longitudinal bars and hoop reinforcement. This was attributed to a low blast profile of the steel surface (1.0 mils (0.03 mm)) and low coating thickness (4 to 6 mils (0.1 to 0.2 mm)).

Another concrete beam reinforced with Scotchlite 213 epoxy coated reinforcement was removed from Yaquina Bay in 1998 after eighteen years of exposure. The beam and the Scotchlite 213 epoxy coated reinforcement were tested and evaluated at the ODOT Materials Laboratory. The following emerged as findings from the testing and evaluation:

- Transverse cracks were present on each exterior face of the beam at locations where the hoop reinforcement was located. Pockmarks and other surface defects were also prevalent on the west face of the beam (the windward side).

- Four groups of half-cell potential measurements within the beam's tidal zone exceeded the 90% probability threshold (-0.35 V) for the occurrence of corrosion.

- Chloride concentrations were significantly elevated within the beam's tidal zone. Also within the tidal zone, the chloride concentrations generally decreased as the depth where the sample was taken increased.

- Some loss of adhesion was measured along the steel surface for most of the reinforcement. The adhesion loss was greatest within the tidal zone.

- Average blast profile of the reinforcing steel surface in the tidal zone was 3.0 mils (0.08 mm) which is within the specification range of 1.5 to 4.0 mils (0.04 to 0.10 mm).

- The majority of observed corrosion along the longitudinal bars and hoop reinforcement was within the tidal zone.

In comparing the results of the testing done on the Scotchlite 213 epoxy coated reinforcement in 1989 and 1998, the half-cell measurements closely correlated with each other. In both cases, based on ASTM standards, there was a 90% probability that corrosion was occurring in the tidal zone. Adhesion loss detected in the 1989 and 1998 testing and evaluation, was also a factor contributing to corrosion. In both periods when the Scotchlite 213 epoxy coated reinforcement was visually examined, corrosion was observed along the longitudinal bars and hoop reinforcement within the tidal zone.

The severity of the corrosion observed in 1998 was very much like the severity of corrosion observed in the 1989 testing. Additional deterioration of the reinforcing steel in the nine years since 1989 has been slowed. This may or may not be attributed to the apparent difference in surface preparation of the reinforcing steel noted in the blast profile testing.

However, notwithstanding the decrease in the rate of corrosion, there was still considerable corrosion observed in 1998 in the tidal zone. The results of ODOT's testing and evaluation on Scotchlite 213 epoxy coated reinforcement and previous studies of epoxy coated reinforcement demonstrate its limitations. Therefore, the continued use of this type of reinforcement in coastal structures in Oregon is not recommended. It is recommended that existing coastal structures and structures subject to de-icing chemicals that are reinforced with epoxy coated steel be placed on a frequent inspection program.

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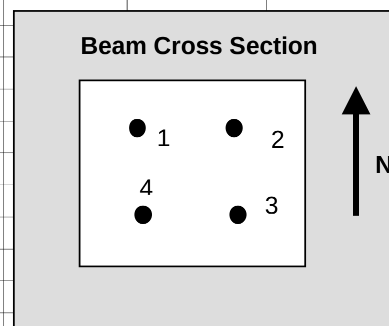
APPENDIX

Electric Half-Cell Potential Measurements

ELECTRIC HALF-CELL POTENTIAL MEASUREMENTS

Location on the Beam from Bottom Face (cm.)	Location on the Beam from Bottom Face (in.)	Half-Cell on Rebar #3 South Face (V)	Half-Cell on Rebar #4 South Face (V)	Half-Cell on Rebar #3 East Face (V)	Half-Cell on Rebar #4 West Face (V)
0.0	0.0	-0.636	-0.608	-0.644	-0.596
15.2	6.0	-0.639	-0.598	-0.642	-0.585
30.5	12.0	-0.614	-0.590	-0.633	-0.582
45.7	18.0	-0.615	-0.575	-0.623	-0.570
61.0	24.0	-0.603	-0.565	-0.614	-0.557
76.2	30.0	-0.588	-0.547	-0.598	-0.542
91.4	36.0	-0.570	-0.534	-0.578	-0.535
106.7	42.0	-0.547	-0.523	-0.562	-0.509
121.9	48.0	-0.544	-0.512	-0.546	-0.480
137.2	54.0	-0.524	-0.486	-0.526	-0.456
152.4	60.0	-0.513	-0.472	-0.517	-0.418
167.6	66.0	-0.506	-0.467	-0.505	-0.413
182.9	72.0	-0.493	-0.457	-0.481	-0.404
198.1	78.0	-0.485	-0.447	-0.456	-0.427
213.4	84.0	-0.478	-0.435	-0.445	-0.429
228.6	90.0	-0.468	-0.428	-0.457	-0.427
243.8	96.0	-0.471	-0.421	-0.456	-0.419
259.1	102.0	-0.459	-0.441	-0.451	-0.404
274.3	108.0	-0.454	-0.411	-0.458	-0.394
289.6	114.0	-0.439	-0.391	-0.454	-0.411
304.8	120.0	-0.422	-0.388	-0.445	-0.396
320.0	126.0	-0.417	-0.382	-0.433	-0.369
335.3	132.0	-0.400	-0.362	-0.407	-0.350
350.5	138.0	-0.389	-0.334	-0.356	-0.335
365.8	144.0	-0.358	-0.332	-0.340	-0.303
381.0	150.0	-0.315	-0.305	-0.318	-0.283
396.2	156.0	-0.280	-0.251	-0.301	-0.277
411.5	162.0	-0.238	-0.230	-0.282	-0.241
426.7	168.0	-0.205	-0.213	-0.258	-0.211
442.0	174.0	-0.250	-0.227	-0.269	-0.220
457.2	180.0	-0.206	-0.151	-0.242	-0.152
472.4	186.0	-0.192	-0.141	-0.244	-0.137
487.7	192.0	-0.205	-0.149	-0.253	-0.132
502.9	198.0	-0.180	-0.120	-0.230	-0.104
518.2	204.0	-0.227	-0.171	-0.256	-0.088
533.4	210.0	-0.168	-0.124	-0.214	-0.099
548.6	216.0	-0.179	-0.110	-0.212	-0.111
563.9	222.0	-0.187	-0.128	-0.206	-0.120
579.1	228.0	-0.248	-0.158	-0.194	-0.104
594.4	234.0	-0.206	-0.130	-0.201	-0.097
609.6	240.0	-0.210	-0.140	-0.226	-0.121



Half-Cell Potential Readings using a Copper-Copper Sulfate Reference Electrode