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# Cathodic Protection Field Trials on Prestressed Concrete Components



Research and Development
Turner-Fairbank Highway Research Center
6300 Georgetown Pike
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## FOREWORD

This report documents a study to demonstrate the feasibility of using cathodic protection on concrete bridge structures containing prestressed steel. This interim report describes the installation and start-up of five different cathodic protection systems on three structures in different climate zones. Following 2½ years of monitoring the cathodic protection system, bridge components will be evaluated and tests will be conducted to determine the effects on the bond between concrete and prestressing steel, and structural properties of the prestressing steel.

This report will be of interest to bridge engineers and designers of prestressed concrete structures. The investigation will also be of interest to owners, inspectors, design firms, and construction contractors who are involved with prestressed concrete bridges.

Charles J. Nemmers, P.E.
Director, Office of Engineering
and Highway Operations
Research and Development

## NOTICE

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16. Abstract

This is the Interim Report in a study to demonstrate the feasibility of using cathodic protection (CP) on concrete bridge structures containing prestressed steel. Past laboratory and test yard studies had indicated that overprotection could result in the evolution of atomic hydrogen and the embrittlement of prestressing steel. Systems utilizing catalyzed titanium mesh, conductive rubber, and arc-sprayed zinc anodes were installed on prestressed pilings and girders of the Howard Frankland Bridge in Tampa, Florida; and systems using flame-sprayed zinc and conductive paint anodes were installed on the soffit of prestressed box beams of the Abbey Road and West 130th Street bridges near Cleveland, Ohio. The installation of all systems went well, with two exceptions. A strong October storm caused damage to the substructure and CP systems installed on the Howard Frankland Bridge, and leaking joints on the Ohio bridges caused construction delays and additional work.

The three zones using zinc anodes were all started up in sacrificial mode, but after a few months on-line, polarization of the steel in these zones was inadequate, and operation will be switched to impressed current in the near future. The conductive rubber anode used on seawater pilings initially leaked a large amount of current to the seawater, but this has moderated as steel below water has become polarized. The conductive paint anode is showing signs of early disbondment, and may not be well suited for service in this environment.

A review of literature and early data indicate that constant voltage, with a current limit, may be the optimal mode of control for structures containing prestressed steel, and this will be studied in the monitoring phase of this contract. Following 2½ years of monitoring the CP systems, components will be evaluated and tests will be conducted to determine effects on the bond and structural properties of the steel.

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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 | square millimeters | $mm^2$ |
| $ft^2$ | square feet | 0.093 | square meters | $m^2$ |
| $yd^2$ | square yards | 0.836 | square meters | $m^2$ |
| ac | acres | 0.405 | hectares | ha |
| $mi^2$ | square miles | 2.59 | square kilometers | $km^2$ |
| | | **VOLUME** | | |
| fl oz | fluid ounces | 29.57 | milliliters | mL |
| gal | gallons | 3.785 | liters | L |
| $ft^3$ | cubic feet | 0.028 | cubic meters | $m^3$ |
| $yd^3$ | cubic yards | 0.765 | cubic meters | $m^3$ |
| 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 (or "metric ton") | Mg (or "t") |
| | | **TEMPERATURE (exact)** | | |
| °F | Fahrenheit temperature | 5(F-32)/9 or (F-32)/1.8 | Celcius temperature | °C |
| | | **ILLUMINATION** | | |
| fc | foot-candles | 10.76 | lux | lx |
| fl | foot-Lamberts | 3.426 | candela/$m^2$ | cd/$m^2$ |
| | | **FORCE and PRESSURE or STRESS** | | |
| lbf | poundforce | 4.45 | newtons | N |
| lbf/$in^2$ | poundforce per square inch | 6.89 | kilopascals | kPa |

## 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** | | |
| $mm^2$ | square millimeters | 0.0016 | square inches | $in^2$ |
| $m^2$ | square meters | 10.764 | square feet | $ft^2$ |
| $m^2$ | square meters | 1.195 | square yards | $yd^2$ |
| ha | hectares | 2.47 | acres | ac |
| $km^2$ | square kilometers | 0.386 | square miles | $mi^2$ |
| | | **VOLUME** | | |
| mL | milliliters | 0.034 | fluid ounces | fl oz |
| L | liters | 0.264 | gallons | gal |
| $m^3$ | cubic meters | 35.71 | cubic feet | $ft^3$ |
| $m^3$ | cubic meters | 1.307 | cubic yards | $yd^3$ |
| | | **MASS** | | |
| g | grams | 0.035 | ounces | oz |
| kg | kilograms | 2.202 | pounds | lb |
| Mg (or "t") | megagrams (or "metric ton") | 1.103 | short tons (2000 lb) | T |
| | | **TEMPERATURE (exact)** | | |
| °C | Celcius temperature | 1.8C + 32 | Fahrenheit temperature | °F |
| | | **ILLUMINATION** | | |
| lx | lux | 0.0929 | foot-candles | fc |
| cd/$m^2$ | candela/$m^2$ | 0.2919 | foot-Lamberts | fl |
| | | **FORCE and PRESSURE or STRESS** | | |
| N | newtons | 0.225 | poundforce | lbf |
| kPa | kilopascals | 0.145 | poundforce per square inch | lbf/$in^2$ |

* SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380.

(Revised September 1993)

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# CHAPTER 1. INTRODUCTION

The objectives of this study were to install durable cost-effective cathodic protection (CP) systems on prestressed concrete (PS/C) bridge components, to evaluate the effectiveness of the applied CP systems in combating corrosion, and to identify all limitations of the systems and potential risks to the structures.

The work plan called for the installation of CP systems on three structures in each of three different climate zones. The systems are to be monitored for a period of 3 years and evaluated for their effectiveness in this application. The long-term effectiveness of embedded reference electrodes and hydrogen probes are also to be studied during the monitoring phase of the contract. Methods of operation, appropriate for cathodic protection of PS/C components, are to be tested and evaluated. Near the conclusion of the study, steel and concrete specimens will be evaluated to determine whether operation of the CP systems results in loss of bond strength or hydrogen embrittlement of the steel.

Prior to this study, there had been a great deal of concern regarding the practice of using cathodic protection to mitigate corrosion in bridge components containing prestressed steel. This concern stems from the fact that high-strength prestressing steel is particularly susceptible to environmental cracking in the form of hydrogen embrittlement. If a cathodic protection system is installed and operated in such a way that the magnitude of polarization is excessive, then atomic hydrogen may be generated at the surface of the steel and embrittlement may occur.

This possibility has been investigated by many authors in the past, and it has been confirmed that excessive polarization is likely to result in embrittlement of high-strength prestressing steel. The thermodynamic potential at which hydrogen is generated in concrete with pH 12.5 is -1.055 V vs. copper/copper sulfate reference electrode (CSE) or -0.981 V vs. saturated calomel reference electrode (SCE).(1) Under a recent Federal Highway Administration (FHWA) study, Wagner and co-workers found that hydrogen was generated at the surface of highly stressed steel members embedded in concrete at potential levels consistent with thermodynamic considerations.(2) That is, the potential at which hydrogen is generated can be predicted from the pH of the environment by using Pourbaix diagrams. Hydrogen flow from cement-coated steel at a pH of 12.4 was first detected at a potential of -0.974 V vs. SCE. Hartt conducted constant extension rate and slow strain rate testing of prestressing strands, and concluded that embrittlement cracking did not occur at potentials less negative than -0.900 V vs. SCE.(3) Hope and Poland reported the onset of hydrogen evolution at an apparent voltage of -0.940 V vs. CSE, but this was the result of using an unfiltered power supply with peak voltage excursions up to -1.37 V vs. CSE.(4)

Only rarely have authors suggested that operation at potentials at or less negative than -0.900 V vs. SCE may result in hydrogen embrittlement of prestressing steel. In a recent paper, Hartt has reported a reduction in fracture load at a potential of -0.900 V vs. SCE and at a pH of 12.5 for certain grades of prestressed steel.(5) These results are unexpected since this potential is nearly 100 mV too positive to allow hydrogen evolution. No satisfactory explanation has been proposed for this anomaly, but these results have led Hartt to suggest that -0.900 V vs. SCE may not necessarily be an appropriate lower potential limit for cathodic protection of prestressing steel. Wagner and co-authors suggested that the IR-free potential of any prestressed steel member should not be permitted to become more negative than -0.800 V vs. CSE (-0.726 V vs. SCE).(2) This recommendation is based on the conclusion that lower pH values surrounding steel embedded in concrete are possible where carbonation has occurred, at cracks in concrete, and beneath corrosion products resulting from chloride ion penetration. Lower pH results in the possibility of embrittlement at less negative potentials than those required in concrete of normal alkalinity. But this rationale fails to account for the effects of cathodic current, which tend to raise the pH at all protected steel surfaces, primarily as a result of oxygen reduction. Mathematical modeling, conducted in a Strategic Highway Research Program (SHRP) study, demonstrated that even small cathodic protection currents will quickly raise pH at any active steel surface to values greater than a pH of 13.(6)

Based on the previous reported work and the technical considerations discussed above, it seems highly unlikely that significant hydrogen embrittlement will occur at steel potentials less negative than -0.900 V vs. SCE. The significance of embrittlement at potentials more negative than the threshold for hydrogen evolution is more difficult to assess. Most authors have agreed that hydrogen embrittlement and brittle fracture are likely to occur if the thermodynamic potential for hydrogen evolution is exceeded.(2,3,4) But conditions have been identified under which significant embrittlement may not occur, regardless of potential. Hartt has reported that if the prestressing strand is smooth (i.e., unnotched, uncorroded), then the potential dependence of hydrogen embrittlement is modest, and the system would be a good candidate for cathodic protection.(7) In this case, smooth specimens at potentials more negative than -1.200 V vs. SCE failed to experience significant loss of fracture load. This led Hartt to propose electrochemical proof testing (ECPT) as a means of qualifying a bridge member for cathodic protection. The proposed ECPT test involves the installation of a temporary impressed current system at a location of maximum corrosion damage and polarization of one or more tendons to a potential of at least -1.200 V vs. SCE. The absence of failure within a predetermined time period (approximately 2 days) would qualify the structure for cathodic protection. Passing this test implies that excessive polarization will not cause failure due to embrittlement. ECPT has not been fieldtested, so the procedures for detecting failure and the consequences of failure are not well defined.

Hartt has also suggested that the chromium content of steels may be used to qualify structures for cathodic protection.(8) In his work, Hartt found that relatively low strength and ductility were recorded for steel specimens prepared from material with relatively high chromium content (0.24 compared to 0.02 weight percent). In the same study, Hartt found that the loss of steel cross section due to corrosion may also be used to qualify a structure for CP. The data suggest that steel with less than 13 percent loss of cross section will not experience significant hydrogen embrittlement due to cathodic polarization, regardless of potential.

Considering the work discussed above, the extent and significance of embrittlement that can occur at potentials beyond the threshold for hydrogen evolution cannot be easily predicted. But the possibility and significance of brittle fracture still requires that cathodic protection systems be operated in such a manner as to preclude potentials that make evolution of atomic hydrogen possible.

System control is clearly important to avoid conditions that may lead to hydrogen embrittlement of prestressed steel. The most logical approach is to control the steel potential to a predetermined value, thus precluding potentials beyond the threshold for hydrogen evolution. But studies in the 1980's by Stratfull, and by Schell and Manning, demonstrated that potential control was difficult to achieve and often resulted in excessive current.[9,10] A more recent study by Wagner and co-workers also indicated that control of a cathodic protection system to a preset potential is difficult, and possibly impractical.[2]

Two different methods of CP control have been proposed for the case where prestressed components are involved: constant current and constant voltage. Wagner and co-workers have recommended constant-current operation with a potential limit.[2] Hartt has concluded that constant-current CP systems may not be appropriate for prestressed steel concrete structures unless a current-off potential limitation corresponding to a threshold of -0.900 V vs. SCE is included.[5] But, as discussed above, the study by Wagner and co-workers also determined potential control to be impractical. It is difficult to accept that potential measurements are unreliable for system control, yet acceptable as a control limit.

Bazzoni and Lazzari recommended a maximum operating voltage (constant voltage) technique.[11] The authors of this study developed a mathematical relationship, assuming a uniform anode potential and an overprotection parameter, which defines a safe operating voltage at which hydrogen evolution is impossible. A major assumption made in this treatment is that the anode potential is uniform and equal at all points in the cathodic protection system. While it is likely that this is a good assumption for highly catalytic anodes, such as mixed-metal-oxide-coated titanium, it may not be appropriate for other anode types.

# CHAPTER 2. BRIDGE SELECTION

## SOLICITATION FOR CANDIDATE STRUCTURES

Task A of this contract called for the selection of three bridges with prestressed steel; one bridge was to be selected in each of three climate zones—tropical marine, temperate marine, and temperate non-coastal. Each bridge was to have prestressed concrete members totaling at least 465 $m^2$ (5000 $ft^2$) of surface area, with the total of all three bridges having at least 2090 $m^2$ (22,500 $ft^2$). The presence of active corrosion on at least a portion of the prestressed steel was considered imperative. It was also considered important that the candidate structures have prestressed strand regions that could be destructively sampled without reducing the structural capacity of the members. Also considered, but of lesser importance, were the cooperation of State and local department of transportation (DOT) personnel, and accessibility of structures for testing and installation.

The first step in the selection process was the solicitation of State DOT's in each of the three climate zones. These States included Oregon, Washington, Texas, Florida, Ohio, Indiana, Pennsylvania, New York, Vermont, Missouri, and New Jersey. Once a list of candidate structures had been compiled, telephone contact was made with each State to gather additional information.

## SCREENING OF CANDIDATE STRUCTURES

Fourteen bridges were then selected for on-site screening. These included three bridges in tropical marine, six bridges in temperate marine, and five bridges in temperate non-coastal environments. Complete surveys and analyses were sometimes limited during the screening phase of the contract by accessibility of prestressed members. The screening tests and observations conducted at each site were:

- Accessibility of prestressed members.
- Traffic conditions.
- Availability of power and telephone lines.
- Assessment of ability to sample prestressing steel without structural consequences.
- Inspection for signs of corrosion, including cracks and delaminations.
- Spot survey of steel half-cell potentials (where possible).
- Spot analysis of concrete for chloride content (when possible).

The evaluation and selection of candidate bridges were carried out by Webster Engineering Associates, Inc., from December 1991 to September 1992. The following bridges were visited and evaluated as candidates, and comments about the most relevant test results are presented. Greater detail is contained in unpublished reports available from the contractor.

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1. Howard Frankland Bridge, Interstate 275 over Tampa Bay between St. Petersburg and Tampa, Florida. (Tropical Marine)

   This bridge is a 3.2-km (2-mi) long structure and spans Tampa Bay. It was constructed in 1959 with pretensioned/prestressed beams and prestressed square pilings. Bent spacing is about 14.4 m (48 ft) in the low spans, which comprise most of the bridge. The total width of the bridge is about 18.6 m (62 ft) with 10 simply supported prestressed beams running lengthwise from bent to bent under each section. The girders have pretensioned strands and draped post-tensioned strands in sheaths. The prestressed piles are 60-cm (24-in) square members with eight piles per bent in the low portions of the bridge. The soffits of the girders are about 2.7 m (9 ft) above the water.

   Rust staining, cracking, and spalling due to corrosion were evident throughout the structure. On the prestressed beams, corrosion appeared to be limited to the soffit and upper edges of the bottom flange. Corrosion damage on the prestressed piles was scatted throughout the length of the bridge. Although half-cell potentials and chloride analysis could not be conducted during this screening visit due to limited accessibility, the Florida DOT had conducted extensive inspection and testing on many members. They had recorded elevated chloride levels and potentials that indicate the presence of active corrosion.

   Accessibility is a problem for this structure, since most of the bridge can be accessed only by boat or barge, and the bay is often rough in adverse weather conditions. Power is readily available throughout the length of the bridge, but phone lines are not. The interest and assistance of the Florida DOT was an important advantage for this structure.

2. Interstate 10 bridge over Lake Pontchartrain, between New Orleans and Slidell, Louisiana. (Tropical Marine)

   This structure is actually two parallel two-lane bridges totaling about 17.4 km (10.8 mi) in length. There are 433 low spans of 19.8 m (65 ft) each. The prestressed girders are simply supported on pile caps, which are, in turn, supported by four 1.4-m (54-in) diameter prestressed hollow piles. The wall thickness of the piles is about 13 cm (5 in). The tops of the piles are about 2.4 m (8 ft) above water level, and the soffits of the girders are about 3 m (10 ft) above water level.

   There was no evidence of corrosion activity on the deck and girders. About 15 percent of the piles showed some evidence of corrosion activity in the form of moderate cracking and rust staining. Half-cell potentials and chloride analysis could not be taken during this visit due to limited accessibility.

   Access to the bridge requires a work boat or barge, and these would be subject to weather problems since the shallow water of the lake tends to be very rough.

There are both power and phone lines near the ends of the bridge, but running lines onto the bridge may be costly.

This bridge was judged to be unsatisfactory due to accessibility problems and the general lack of corrosion activity.

3. Ocean Drive bridge over Cayo de Oso Bay at the west entrance to Corpus Christi Naval Air Station, Corpus Christi, Texas. (Tropical Marine)

    This bridge is about 220 m (720 ft) long and 17.7 m (58 ft) wide. The bridge has 12.2-m (40-ft) long precast beams simply supported on pile caps that span 10 piles each. The piles are 46-cm (18-in) square pretensioned/prestressed concrete. The tops of the piles are approximately 5 m (16 ft) above the mean water level.

    There was significant evidence of corrosion activity in the form of cracks, spalling, and reduction of tendon cross section on 10 percent of the piles. Half-cell potentials and chloride analysis could not be taken during this visit due to limited accessibility. Chloride contents at the 5-cm (2-in) level, in a 1991 survey conducted by the Texas DOT, averaged 2.6 $kg/m^3$ (7.5 $lb/yd^3$).

    Accessibility would be difficult from a boat or small barge due to the height above the water and the usually rough condition of the water in the Cayo de Oso Bay. There are no power or phone lines close to the bridge.

    This bridge was judged to be unsatisfactory due to the accessibility problems and the lack of electrical power.

4. Route 10 (business) bridge over Cypress Creek, County Isle of Wight, Smithfield Station, Virginia. (Temperate Marine)

    This bridge structure is about 458 m (1500 ft) long and 11 m (36 ft) wide. The structure has 24 spans of 16.9 m (55.5 ft) each. Near the center, there is a minimum 3.7-m (12-ft) clearance to high water level. The structure has six longitudinal pretensioned/prestressed beams supported on pile caps.

    Inspection and testing showed corrosion activity to be limited to only two spans, where limited spalling had occurred on the bottom flanges of the prestressed beams. Half-cell potential readings in this section varied from -100 to -300 mV vs. copper/copper sulfate (CSE). Concrete powder samples taken from 0 to 2.5 cm (0 to 1 in) and from 2.5 to 5 cm (1 to 2 in) contained 1.0 and 0.2 $kg/m^3$ (2.9 and 0.6 $lb/yd^3$) of chloride, respectively.

    This structure did not have sufficient corrosion activity to qualify for this project.

5. Route 306 bridge over Rappahannock River, at Tappahannock, Essex County, Virginia. (Temperate Marine)

   This bridge is 1709 m (5605 ft) long with 101 simply supported spans. Each span has four pretensioned/prestressed concrete beams, either 15.3 or 21.4 m (50 or 70 ft) long. The bridge rises from existing grade at the ends to a 15.5-m (50-ft) clearance above high tide at the center span. Each bent is supported by four square prestressed concrete piles.

   There were many areas where corrosion damage was visible. Half-cell potential survey and sampling for chloride analysis could not be done because of the height of the bridge above water. Corrosion damage, in the form of spalling and exposed reinforcing steel, was commonly seen within 1.2 m (4 ft) of the ends of the beams. Several other areas were observed away from the ends of the beams where the bottom outside tendon had corroded and caused a spall.

   There are no power or phone lines on the bridge, but both are located near the west end. Accessibility is a significant problem due to the height of the bridge. The bridge beams are only accessible with a snooper truck from the deck of the bridge, and traffic control and a flagman would be required for all work.

   This bridge was considered undesirable because of accessibility problems and lack of power and phone lines.

6. Richmond-San Raphael bridge, Interstate 580 over San Pablo Strait, Richmond, California. (Temperate Marine)

   This bridge is about 9 km (5.6 mi) long, and has seventy-two 15.3-m (50-ft) long spans that are supported on beam-driven piles. The bridge consists of two parallel structures, approximately 13 m (42 ft) wide. There are seven pretensioned/ prestressed concrete girders running lengthwise under the eastbound and westbound parallel bridges. Beam soffits are about 1.5 m (5 ft) above mean sea level.

   The overall condition of the structure is good due to an active maintenance program. A limited visual inspection conducted during the screening revealed several corrosion-related spalls or repaired spalls on the bottom flanges of the beams. A 1990 California Department of Transportation (Caltrans) study reported that over 40 percent of the girders had evidence of spalling, cracking, rusting, or repaired spalls, probably all corrosion-related. The chloride content of two concrete samples taken adjacent to reinforcing steel was 0.5 and 0.6 kg/m$^3$ (1.4 and 1.7 lb/yd$^3$). The beams have been painted with an aluminum flake paint in an effort to control chloride ingress, but the paint is of questionable effectiveness.

The maintenance department has installed a moveable underslung access stage that could be used for testing and installation work. Power and phone lines are readily available.

This structure was judged to be a good structure, primarily because of relatively good accessibility, presence of active corrosion, and cooperation of local and State highway agencies. Tendon sampling was judged to be very feasible.

7. Bridges at Devil's Elbow State Park, Oregon Coastal Highway (mile point 178.75), north of Florence, Oregon. (Temperate Marine)

   The Devil's Elbow bridges are an in-line series of two bridges, accommodating two traffic lanes plus small sidewalk shoulders. The bridges are both about 11 m (35 ft) wide. The south bridge has a 31-m (103-ft) span with 11 side-by-side pretensioned/prestressed box beams. The northern bridge consists of two spans, simply supported, with the northern box beams spanning 17 m (48 ft) and the southern beams spanning 29 m (95 ft).

   Corrosion-related rust stains and probable incipient spalls were observed on the underside of the box beams. Several reinforcing steel bars were exposed by spalling. Two concrete samples taken between 0 to 2.5 cm (0 to 1 in) had chloride contents of 1.1 and 4.2 $kg/m^3$ (3 and 12.1 $lb/yd^3$), and two samples taken between 2.5 to 5 cm (1 to 2 in) had a chloride content below the detection limit. In a recent inspection, the Oregon DOT found half-cell potentials as negative as -199 mV vs. CSE.

   Access to the underside of these bridges is a serious problem, as the ravines drop steeply down. The height on the ocean side may be intimidating to technicians. Testing and installation could be done only with a snooper truck, and traffic control could be a problem on the two-lane bridge. There are no power or phone lines near the bridge.

   These structures were unacceptable for work under this contract due to inaccessibility and lack of alternating current (AC) power.

8. Bridge over Youngs Bay, Oregon Coastal Highway (mile point 4.91), Astoria, Oregon. (Temperate Marine)

   This bridge is 1283 m (4208 ft) long with a width of about 11 m (35 ft). The bridge has 48 spans, 24 m (80 ft) long each, with 4 simply supported pretensioned/prestressed beams running lengthwise under each span. The beams are supported by pile-cap bents, which are, in turn, supported by two 1.4-m (54-in) diameter post-tensioned/prestressed concrete piles.

Extensive corrosion is occurring at a large number of steel support chairs left in place when the beams were cast. There was no evidence of corrosion activity specifically related to the stressed tendons in the concrete. Two concrete samples taken from 0 to 2.5 cm (0 to 1 in) deep had chloride contents of 0.2 and 0.4 kg/m$^3$ (0.6 and 1.2 lb/yd$^3$), and two samples taken from 2.5 to 5 cm (1 to 2 in) deep had chloride contents of 0.6 and less than 0.04 kg/m$^3$ (1.8 and less than 0.1 lb/yd$^3$). Power and phone lines were available only at the ends of the bridge. This structure was rejected for this study due to the lack of corrosion activity on the stressed strands and the very large number of exposed chairs, which represent potential shorts.

9. Columbia River bridge, U.S. Highway 101, between Astoria, Oregon, and Point Ellice, Washington. (Temperate Marine)

   The Columbia River bridge is 6.6 km (4.1 mi) long and 10 m (33 ft) wide. There are 140 spans, 24.4 m (80 ft) long each, with 4 simply supported pretensioned/prestressed beams running lengthwise under each span.

   The prestressed beams have experienced some corrosion and cracking at their ends, and have been repaired. There was little evidence of active corrosion at the time of the screening. There was no access to the underside of the bridge, so half-cell potentials and samples for chloride analysis could not be taken. Chloride analysis of 100 samples taken in 1988 showed an average of 0.42 kg/m$^3$ (1.2 lb/yd$^3$) for samples taken 0 to 2.5 cm (0 to 1 in) deep, and 0.07 kg/m$^3$ (0.2 lb/yd$^3$) for samples taken 2.5 to 5 cm (1 to 2 in) deep.

   Access to this bridge for testing or installation could only be by a snooper truck or boat. Boat access is further complicated by very shallow water at low tide. No power or phone lines are available; alternate power sources, such as solar panels, would have to be utilized.

   This structure was unacceptable because of lack of corrosion activity, lack of AC power, and accessibility problems.

10. Sandusky Bay bridge, State Route 2, Bay View, Ohio. (Temperate Non-Coastal)

   The Sandusky Bay bridge is a 23-span structure, 625 m (2050 ft) long by 21.4 m (70 ft) wide. The spans are typically 27.5 m (90 ft) long with five parallel prestressed concrete girders.

   The soffits of several girders are spalling, and tendons are exposed and corroding as a result of deicing salt. Salt deposits exist on the sides and soffits of some of the girders. Access is limited to a snooper truck from the top of the bridge because of the height of the bridge above the water. Traffic control could be a problem, since the bridge is heavily traveled. Power is readily available.

This bridge was judged to be an acceptable candidate for this study, but accessibility was considered a serious disadvantage.

11. State Route 718 bridge over Interstate 81, Harrisonburg, Virginia. (Temperate Non-Coastal)

    This bridge is a four-span structure, with each span comprised of five parallel prestressed concrete girders supporting a composite deck. Each span is about 24.4 m (80 ft) long by 7.3 m (24 ft) wide.

    The structure is generally in good condition, although one girder shows visible signs of distress due to corrosion. Half-cell potential readings taken on girder soffits ranged from -19 to -107 mV vs. CSE. Powder samples of concrete taken at the level of the reinforcement were found to have chloride concentrations below detectable limits. Power and phone lines are readily available and accessibility is good.

    This structure was rejected for this study due to the general lack of corrosion activity.

12. U.S. Route 224 bridge over the eastbound entrance to the Loral Airdocks, Akron Municipal Airport, Akron, Ohio. (Temperate Non-Coastal)

    This bridge is a 4-span structure with 17 side-by-side prestressed box beams on each span. The structure spans a railroad track and a two-lane secondary road. The bridge is 59 m (194 ft) long by 20.7 m (68 ft) wide.

    A very limited amount of corrosion damage has occurred due to leakage through joints between box beams. This area has spalling and rust staining, and half-cell potential readings ranging from -290 to -440 mV vs. CSE. Half-cell readings in the rest of the structure, where there was no evidence of leakage, ranged from -69 to -82 mV vs. CSE. Chloride content of samples taken from 0 to 3.8 cm (0 to 1.5 in) beneath the surface averaged 0.5 $kg/m^3$ (1.4 $lb/yd^3$). Power and phone lines are readily available, and accessibility is good.

    This structure was rejected due to the lack of general corrosion activity.

13. U.S. Route 224 bridge over South Arlington Road, Akron, Ohio. (Temperate Non-Coastal)

    This bridge is a three-span structure, 37 m (120 ft) long by 21 m (68 ft) wide. Each span consists of 17 simply supported prestressed/pretensioned beams running parallel to the length of the bridge.

Although the deck is showing signs of distress due to corrosion, there were no visible signs of corrosion on the beams. Half-cell potential readings on the soffits of the beams ranged from -6 to -290 mV vs. CSE. Chloride content of concrete samples taken at the level of the reinforcement ranged from 0.03 to 0.72 $kg/m^3$ (0.8 to 2.07 $lb/yd^3$). Power and phone lines are readily available. Accessibility is good, although traffic control could pose a problem.

This structure was rejected due to the lack of general corrosion activity.

14. Interstate 80 bridge over U.S. Route 15, Milton, Pennsylvania. (Temperate Non-Coastal)

There are two identical bridges, each 61 m (200 ft) long by approximately 14 m (47 ft) wide. Each bridge has four spans with eight simply supported pretensioned/prestressed box beams. The beams support a composite deck with metal deck forms. There is cracking, which was reported to be corrosion-related, near the ends of the box beams. The cracking was not related to corrosion, and no spalling or rust staining was observed.

This bridge was rejected due to the lack of sufficient corrosion activity.

## DETAILED TESTING OF TOP CANDIDATE STRUCTURES

Following the screening of the 14 candidate structures described above, 3 structures were selected for more detailed testing. These included the Howard Frankland bridge in Tampa, Florida (tropical marine), the Richmond-San Raphael bridge in Richmond, California (temperate marine), and the Sandusky Bay bridge in Sandusky, Ohio (temperate non-coastal). It was assumed that these would be the structures selected for this study.

The work on the Howard Frankland bridge in Tampa was given first priority, since this structure was identified as an excellent candidate, meeting all of the selection criteria. Plans and specifications were prepared, and bids were received on June 14, 1993. All bids received were well over the budget estimate used in the original proposal due to the complexity of the structure and the difficulty of working over tidewater far from shore. This resulted in a major change of plan in the contract. The Richmond-San Raphael bridge was rejected for work because it would also be an expensive installation, and because it duplicated many elements of the Howard Frankland bridge. The Sandusky Bay bridge was rejected because of very difficult access problems and general lack of corrosion activity.

Additional screening was then conducted on several bridges in the Cleveland, Ohio, area and two prestressed box-beam structures were selected that met the acceptance criteria. The three candidate bridges which were then confirmed for work under this contract were the Howard Frankland bridge in Tampa, Florida (tropical marine); the Abbey Road bridge in Cuyahoga County, Ohio (temperate non-coastal); and the West 130th Street bridge in Cuyahoga County, Ohio (temperate non-coastal). The detailed testing of these structures is summarized below.

1. Howard Frankland Bridge, Interstate 275 over Tampa Bay between St. Petersburg and Tampa, Florida. (Tropical Marine)

   A. Carbonation and pH Testing.

      A sample of concrete approximately 2.5 cm (1 in) thick was removed from the top of the lower flange of beam 8, span 252. The pH of the concrete was between 11 and 13 throughout the entire depth of the sample. Based on these results, it was concluded that the concrete was not carbonated.

   B. Chloride Concentration Testing.

      Nine cores and six powder samples were obtained from various piles and beams. Each sample was analyzed for chloride content in accordance with American Association of State Highway and Transportation Officials (AASHTO) T 260-84, "Sampling and Testing for Total Chloride Ion in Concrete and Concrete Raw Materials." Chloride concentrations in the piles were exceptionally high, ranging from 1.24 percent total chloride [17 $kg/m^3$ (48.5 $lb/yd^3$)] near the concrete surface to an average of 0.14 percent total chloride [1.9 $kg/m^3$ (5.4 $lb/yd^3$)] at the depth of the steel, 60 to 64 mm (2 to 2.5 in). Chloride concentrations in the beams were moderate, but still exceeded the corrosion threshold in most samples. Chloride concentrations in the beams at the depth of the reinforcing steel ranged from below detection to 0.17 percent total chloride [2.3 $kg/m^3$ (6.7 $lb/yd^3$)], and averaged 0.08 percent total chloride [1.0 $kg/m^3$ (2.9 $lb/yd^3$)]. These data can be compared to a generally accepted value of threshold for corrosion of about 0.03 percent total chloride [0.4 $kg/m^3$ (1.2 $lb/yd^3$)].

   C. Concrete Resistivity Testing.

      Concrete resistivity was measured using a four-pin technique similar to that described in American Society for Testing and Materials (ASTM) G57. Readings on the piles ranged from 15.9 kΩ-cm at the high-tide elevation to 52 kΩ-cm at the top of the piles. Concrete resistivity increased relative to height above seawater. Readings on the beams ranged randomly between 62.9 and 124.8 kΩ-cm. Concrete resistivity, although somewhat high for the beams, was considered acceptable for cathodic protection.

   D. Half-Cell Potential Testing.

      Half-cell potential readings were taken using a standard copper/copper sulfate reference electrode in accordance with ASTM C876-87. Potentials taken on the piles indicate the presence of very active corrosion at the time of measurement, with readings ranging from -269 to -553 mV and averaging -433 mV. Potential readings taken on the beams show a general lack of corrosion occurring at the time of measurement, ranging from +18 to -152 mV, but other observations confirm the presence of corrosion activity.

E. Delamination Inspection.

Numerous pilings have vertical cracks and delaminations as a result of corrosion of the reinforcing steel. Many pilings have already been repaired and have gunnite jackets, which make the installation of cathodic protection on these members difficult. Minimal delaminations and cracks were found on the beams. Some spalling was found on the top chamfer of the bottom flanges of some of the prestressed girders. Most of these spalls were no larger than 0.09 $m^2$ (1 $ft^2$), with an average depth of approximately 2.5 cm (1 in) to the prestressing tendons. Corrosion is present near the ends of the beams, but is not as active as that in the pilings.

2. Abbey Road bridge over Baldwin Creek, Cuyahoga County, Ohio. (Temperate Non-Coastal)

A. Physical Description.

The Abbey Road bridge was constructed in 1970 with non-composite prestressed box sections and an asphaltic overlay. The deck cross section consists of eight 1.2-m (4-ft) wide and two 0.9-m (3-ft) wide prestressed box beams. The 0.9-m (3-ft) beams have 22 tendons and the 1.2-m (4-ft) beams have 28 tendons. The tendons are 0.95-cm (0.375-in) diameter stress-relieved strands running straight and horizontally along the sections. The total width of the bridge is 12.3 m (40.25 ft), and the total soffit surface area is 109 $m^2$ (1170 $ft^2$). The bridge is superelevated and slopes at a rate of 4.2 cm/m (0.5 in/ft). The average annual precipitation is 88 cm (34.5 in), with an average annual snowfall of 137 cm (54 in). Temperatures normally fall below freezing 124 days per year, and salting is heavy during winter months. Some of the joints between box beams are leaking, and the elevation has allowed saltwater to run along the underside of some of the beams.

B. Chloride Concentration Testing.

Two cores and two powder samples were obtained from various beams and were analyzed for total acid-soluble chloride ion content in accordance with AASHTO T 260-84. Chloride ion content in the samples was low to moderate, from below detection to 0.11 percent total chloride [1.5 $kg/m^3$ (4.2 $lb/yd^3$)]; but it is clear that the beams are very non-homogeneous with regard to chloride content and that active corrosion is taking place in portions of the structure.

C. Resistance Testing.

Resistance testing on this structure was conducted by pressing a mesh anode with a lightly soaked sponge, wetted with a solution of wetting agent and distilled water, against the concrete surface. Resistance was then measured, using an AC soil resistivity meter (Nilsson Model 400), between the surface anode and the prestressing steel. The contact area of the anode was about 0.01 $m^2$ (0.10 $ft^2$). Experience has shown that concrete having a resistance reading of less than 3000 $\Omega$ is acceptable for cathodic protection. Resistance measured at 14 locations on each of the 5 beams ranged from 190 to 3000 $\Omega$, and averaged 1106 $\Omega$. This test again showed the beams to be non-homogeneous.

D. Half-Cell Potential Testing.

A complete half-cell potential survey was conducted on the soffit side of the beams using a standard CSE in accordance with ASTM C876-87. Potentials varied widely from +43 to -625 mV. Potential readings less negative than -350 mV, indicating active corrosion at the time of measurement, comprised 37 percent of the total readings. Potential readings more positive than -200 mV, indicating absence of corrosion at the time of measurement, comprised 28 percent of the total readings. The most positive readings (least corrosive) were associated with beams 1 and 10, the outside beams. The most negative readings (most corrosive) were associated with leaking joints, and occurred on beams 2, 3, 5, 7, and 9. Potential readings were consistent in lines parallel to the joints, but varied sharply, sometimes as much as 300 mV, across joints.

E. Delamination Testing.

Despite the presence of very active corrosion, no delaminations or cracks were found on the prestressed box members.

3. West 130th Street bridge over Baldwin Creek, Cuyahoga County, Ohio. (Temperate, Non-Coastal)

A. Physical Description.

The West 130th Street bridge over Baldwin Creek was constructed in 1961 with non-composite, prestressed box sections and an asphaltic overlay. The deck cross section consists of fifteen 1.2-m (4-ft) wide prestressed box beams. Each box beam has 35 pretensioned, 9.5-mm (0.375-in) diameter stress-relieved strands that run straight and horizontally along the sections. The total width of the bridge is 18.6 m (61 ft), and about 223 $m^2$ (2400 $ft^2$) of soffit were available for cathodic protection. The average annual precipitation is 88 cm (34.5 in), and the average annual snowfall is 137 cm (54 in). Temperatures normally fall below freezing 124 days per year, and the bridge is heavily salted during winter months. Some of the joints between box beams showed signs of leaking, and several areas underneath the bridge were covered with salt deposits.

B. Chloride Concentration Testing.

Three cores were obtained from different box beams and were analyzed for total acid-soluble chloride ion content in accordance with AASHTO T 260-84. Cores were extracted from the center of the beam soffits. Chloride ion content in the samples was low to moderate, ranging from 0.02 percent total chloride [0.25 kg/m$^3$ (0.7 lb/yd$^3$)] at the design level of the prestressing strands, to 0.07 percent total chloride [0.95 kg/m$^3$ (2.7 lb/yd$^3$)] at a depth of 12 to 25 mm (0.5 to 1 in). It is clear that chloride ion content in this structure was very non-homogeneous, and samples taken near the joints would likely have been more contaminated.

C. Resistance Testing.

Resistance testing on this structure was conducted by pressing a mesh anode with a lightly soaked sponge, wetted with a solution of wetting agent and distilled water, against the concrete surface. Resistance was then measured, using an AC soil resistivity meter (Nilsson Model 400), between the surface anode and prestressing steel. The contact area of the anode is about 0.01 m$^2$ (0.10 ft$^2$). Experience has shown that concrete having a resistance reading of less than 3000 Ω is acceptable for cathodic protection. Resistance measurements varied greatly, ranging from 1050 to 7700 Ω, illustrating the non-homogeneous nature of the structure. Resistance readings were generally lower in regions of active corrosion, as determined from potential readings.

Resistance readings taken between individual prestressing tendons indicated that tendons within the box beams were often electrically discontinuous. In all cases, adjacent box beams were discontinuous.

D. Half-Cell Potential Testing.

A partial half-cell potential survey was conducted on the soffit side of a representative number of beams using a standard CSE in accordance with ASTM C876-87. Potentials varied widely from +88 to -481 mV. Potential readings less negative than -350 mV, indicating active corrosion at the time of measurement, comprised 21 percent of the total readings. Potential readings more positive than -200 mV, indicating absence of corrosion at the time of measurement, comprised 40 percent of the total readings. Of those beams tested, the most negative (most corrosive) readings were recorded on beams 2, 3, and 5. Potential readings were consistent in lines parallel to the joints, but varied sharply, sometimes as much as 250 mV, across joints.

E. Delamination Testing.

Despite the presence of very active corrosion, no delaminations or cracks were found on the prestressed box members.

# CHAPTER 3. CATHODIC PROTECTION SYSTEM SELECTION

An independent consultant was retained to select the most appropriate cathodic protection systems for each of the bridge structures chosen. These are his recommendations, made on October 5, 1992, after reviewing information available in the literature and on the candidate structures.

1. Interstate 275, Howard Frankland Bridge over Tampa Bay, Tampa, Florida.

   For the prestressed beams, thermally sprayed zinc was recommended as both a sacrificial system and an impressed current system. It was not clear from available information whether zinc would supply enough current sacrificially to adequately protect the prestressing steel. It was therefore recommended to install two zinc zones, one to operate in impressed current mode, and one to operate in sacrificial mode, but which could be converted to impressed mode if polarization proved to be inadequate.

   For the prestressed pilings, impressed current systems using titanium mesh anode and conductive rubber anode were recommended. It was also recommended to monitor the zinc "penny sheet" sacrificial system being installed by the Florida DOT for comparison to work conducted under this contract. It was suggested to install bulk zinc anodes below water level to minimize current bleed to footings and pile portions below low tide.

2. Interstate 580, Richmond-San Raphael bridge, Richmond, California.

   The beams were judged to be too close to the water for viable shotcreting, eliminating titanium mesh systems. Zones of both thermally sprayed zinc and water-based conductive paint were therefore recommended.

3. Abbey Road bridge over Baldwin Creek, Cuyahoga County, Ohio.

   Thermally sprayed zinc was recommended for the underside of the Abbey Road bridge. Conductive paint was not recommended for this structure, since it was felt that the superelevation would cause water to run on the underside of the beams and compromise the durability of that anode. Systems requiring shotcrete were not chosen for this structure since little mechanical interlock was available, and because shotcrete would crack or need to be formed at every joint.

4. West 130th Street bridge over Baldwin Creek, Cuyahoga County, Ohio.

   Conductive paint anode was recommended for the underside of the West 130th Street bridge. Again, systems requiring shotcrete were not chosen since little mechanical interlock was available, and because shotcrete would require forming at every joint.

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Of the four candidates selected, the Howard Frankland Bridge was considered to be the most advantageous. The structure offered both prestressed pilings in the splash and tidal zone, and prestressed beams between bents. All members were undergoing active corrosion, but were not extensively delaminated. Also, the Howard Frankland Bridge had been the subject of several studies in the past, and other CP systems were already installed and functional. For these reasons, the design and bid packages were completed for the Howard Frankland Bridge as quickly as possible.

When the bids for the Howard Frankland Bridge were reviewed, it was obvious that the expected budget for the contract would be exceeded. This was largely a result of the CP systems selected for installation and the difficulty of working over tidewater on this particular structure. It was then decided, after consultation with the FHWA Contracting Officer and the Contracting Officer's Technical Representative, to reduce the scope of the contract rather than exceed the contract budget. This decision resulted in deletion of the Richmond-San Rafael bridge as a candidate structure, and plans proceeded for installation of CP systems on the three remaining candidates: the Howard Frankland, Abbey Road, and West 130th Street bridges.

# CHAPTER 4. SYSTEM INSTALLATION

## HOWARD FRANKLAND BRIDGE

### Metallized Zinc Anode System

It was decided to cathodically protect the 10 bottom prestressing strands in each girder. Zinc coating was not required on the upper portions of the girders due to a lack of corrosion activity in this area. The two zinc anode zones, shown in figure 1, each had an area of 359 $m^2$ (3865 $ft^2$), totaling 718 $m^2$ (7730 $ft^2$) for both zones. Zinc was arc sprayed to the bottom portion of 20 girders in each zone, as shown in figure 2. A thickness of 0.25 to 0.38 mm (10 to 15 mil) of 99.9-percent pure zinc was specified. During quality control monitoring of the installation, zinc thicknesses were measured between 0.28 and 0.36 mm (11 and 14 mil).

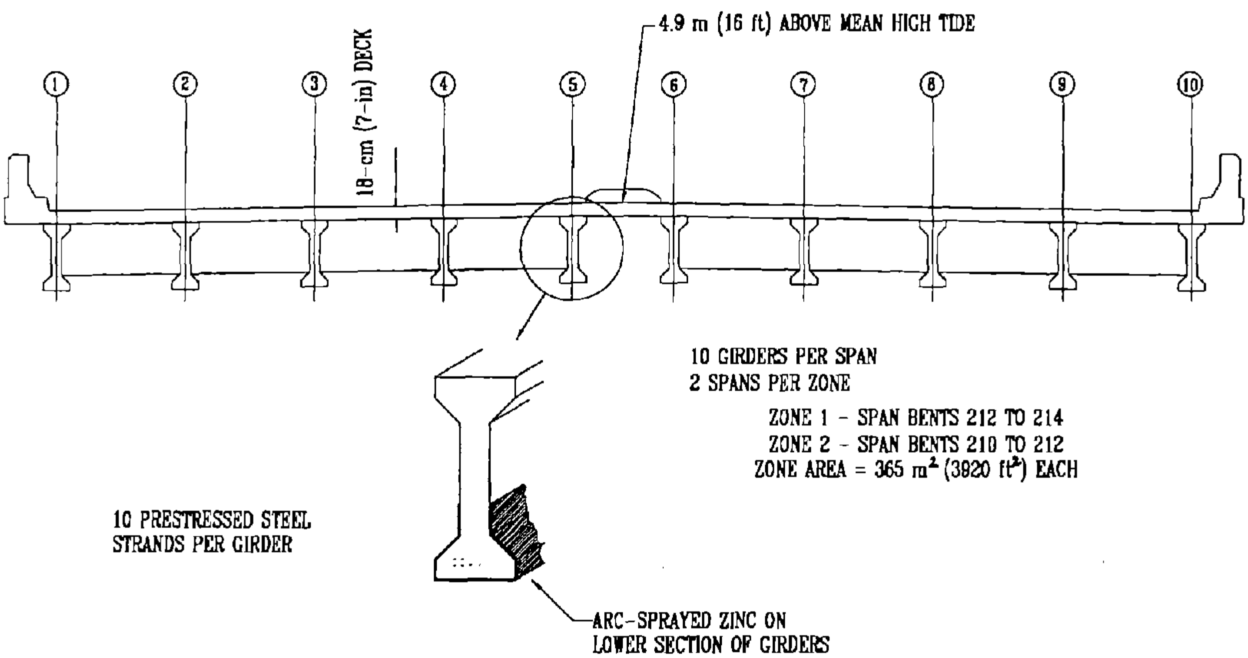


Figure 1. Howard Frankland Bridge - zinc beam zones section view.

Adhesion of the applied zinc coating was tested at one spot per girder using an Elcometer Adhesion Tester with a minimum specified pull-off strength of 690 kPa (100 $lbf/in^2$). All 40 adhesion tests conducted were above the minimum strength and ranged from 690 to 2760 kPa (100 to 400 $lbf/in^2$).

The primary anode connection to the zinc coating was made by 15.2-cm (6-in) long, 1.27-cm (1/2-in) wide titanium strips, which were mechanically connected to the surface after the application of zinc. The same connectors were used to establish electrical contact across bents between zinc anodes of the same zone, as shown in figure 3. Wiring for the zinc anode zone is shown in figure 4. Contact between each titanium strip and the zinc anode was established by three stainless steel screws, which were inserted through 6.4-mm (0.25-in) diameter holes drilled in the strip. Plastic anchors were used to secure the titanium strip tightly to the zinc, ensuring good electrical contact.

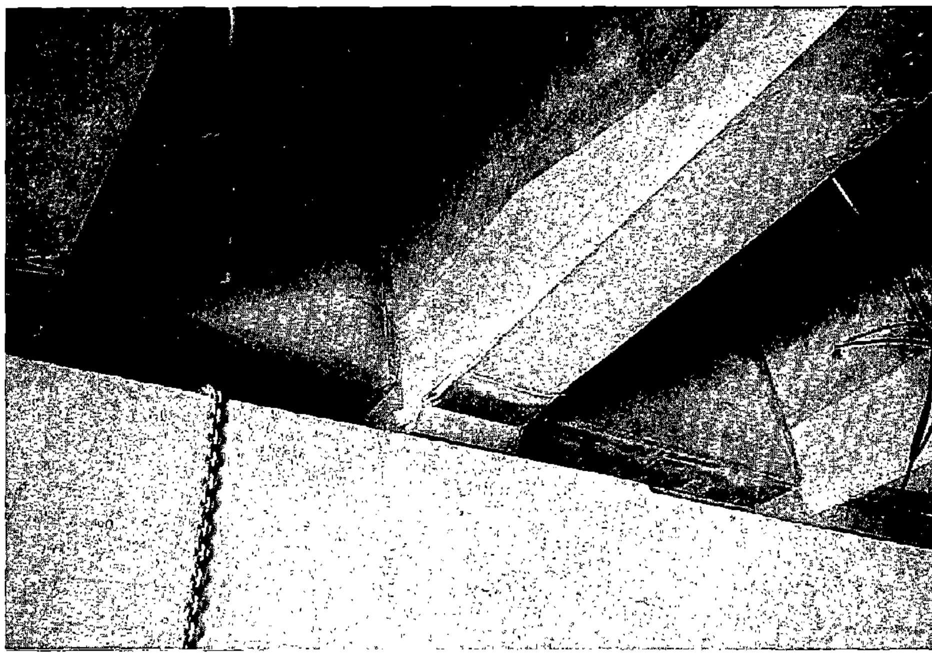

Figure 2. Completed zinc anode on bottom of girder.

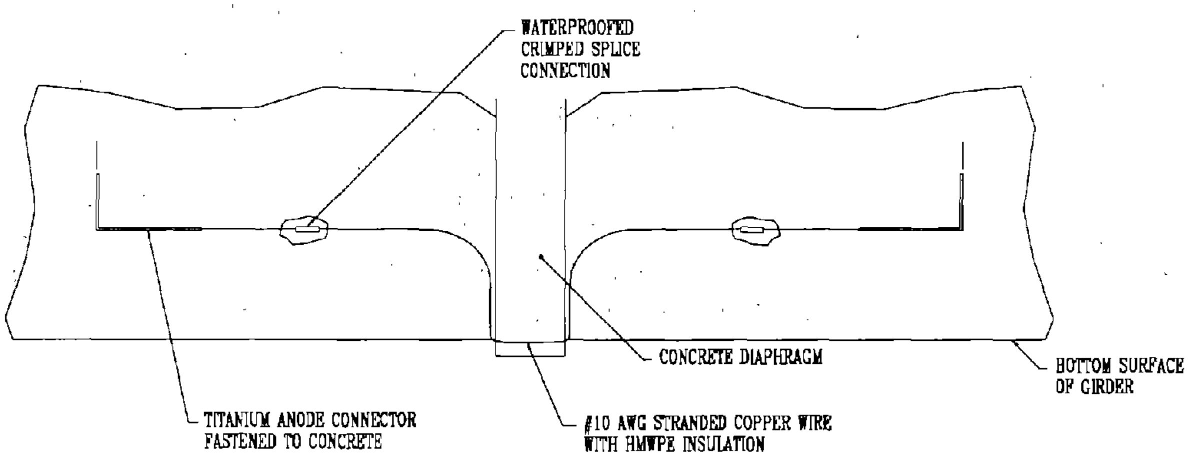


Figure 3. Howard Frankland Bridge - end of girder connection detail.

Figure 4. Wiring installation of zinc anode zone.

Fiberglass Jacket/Titanium Mesh Anode System

A fiberglass jacket/titanium mesh anode system zone was installed on the 60-cm (24-in) square prestressed piles of three bents, as shown in figure 5. The fiber-reinforced plastic (FRP) jacket consisted of two sections, each 1.22 m (4 ft) long with a male coupling on one side and a female coupling on the other side of the half-rectangle sections. When mated together, the two sections formed a rectangular jacket surrounding the pile with a 5.1-cm (2-in) annular space. The FRP jacket was fabricated to hold the activated titanium anode mesh and to retain the encapsulating concrete inside the jacket, as shown in figure 6. The outside dimension of the jacket, shown in figure 7, was 71 cm (28 in) wide, with the inner dimension matching the width of the pile. Four horizontal rows of fiberglass standoffs were integrally cast into the jacket to maintain the 5.1-cm (2-in) standoff and to provide fastening points for the activated titanium anode mesh in the center of the gap between the inner face of the jacket and the outer surface of the pile. The FRP jackets were supplied by Master Builders Technologies, Cleveland, Ohio. The titanium mesh anode used was ELGARD 300, supplied by ELGARD Corp., Chardon, Ohio. This anode is capable of delivering a current of 32.2 mA/m$^2$ (3.0 mA/ft$^2$) of concrete.

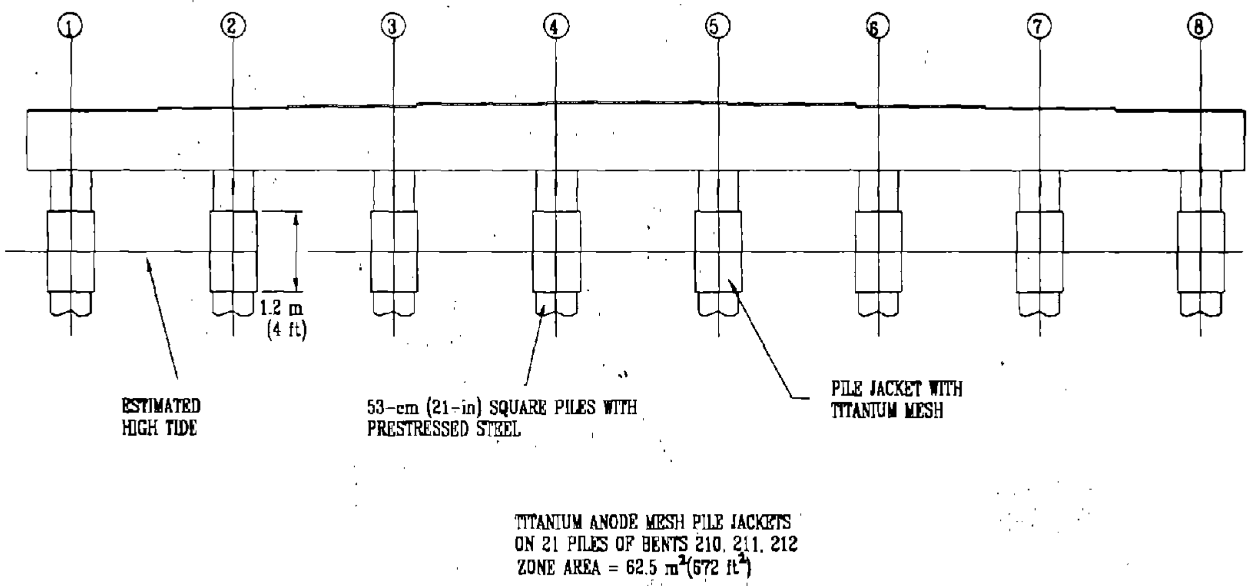


Figure 5. Howard Frankland Bridge - mesh pile jacket zone section view.

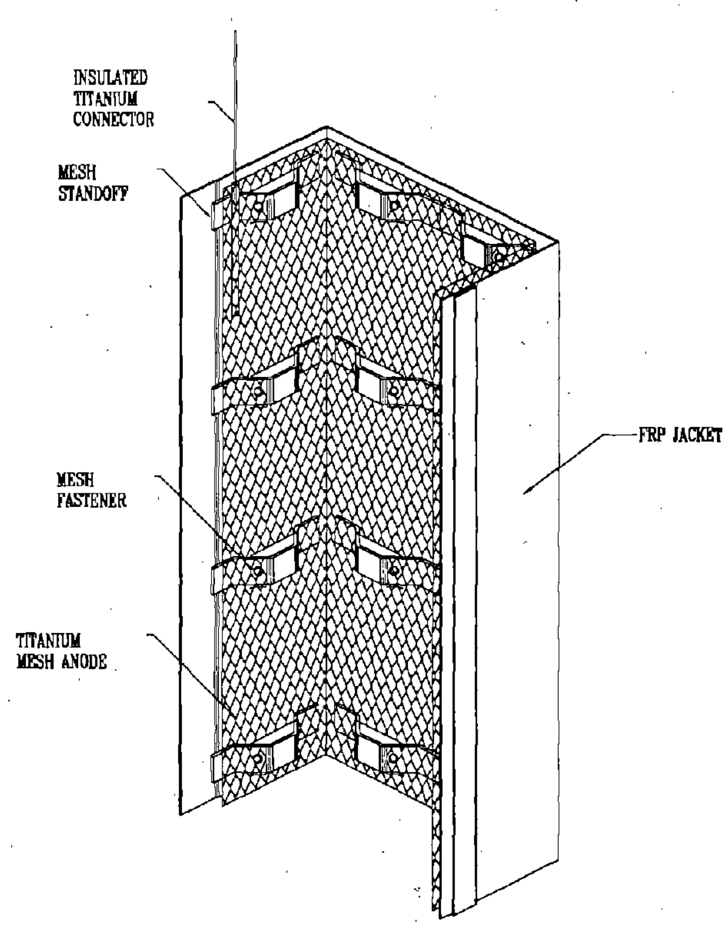


Figure 6. Howard Frankland Bridge - integral mesh anode pile jacket half assembly.

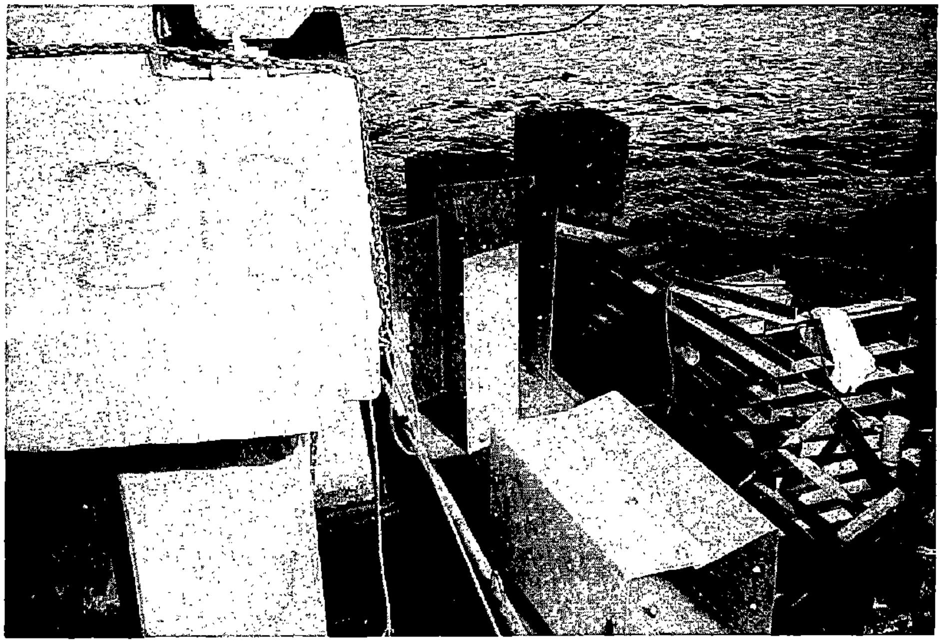

Figure 7. Titanium mesh anode and two-part fiberglass jacket.

The electrical connections to the titanium mesh anodes, partially shown in figure 8, were made using split-bolt electrical connectors, which were then covered with nonconductive epoxy and heat-shrink tubing. These connections were made in a junction box placed just above the jacket on the pile.

Figure 8. Conductor strips for connection of mesh anode to wiring.

After mounting the jackets to the pile, the procedure for placement of grout into the jackets was as follows:

(1) Wooden forms were used to hold the jackets at the proper elevation. Five temporary aluminum frames were used to keep the jackets from expanding when being filled.

(2) The jackets were filled in two stages. Each jacket was nearly half-filled, as shown in figure 9, while the exterior of the jacket was tapped with a hammer to remove air voids. After this procedure was completed on all the jackets, the second stage was pumped to fill each jacket. All the piles in one bent were filled at the same time.

(3) After the jackets had been completely filled, additional grout was applied to the top of the jacket and beveled to slope downwards from the pile to the outside of the jacket.

Figure 9. Pumping grout into installed pile jacket with horizontal bracing to prevent jacket deflection.

No voids were visible after the jackets were filled, and the grout appeared to have completely covered the titanium mesh. Pile jackets during the grout curing phase and completed pile jackets are shown in figures 10 and 11, respectively. Finished wiring for the fiberglass jacket/titanium mesh anode system is shown in figure 12.

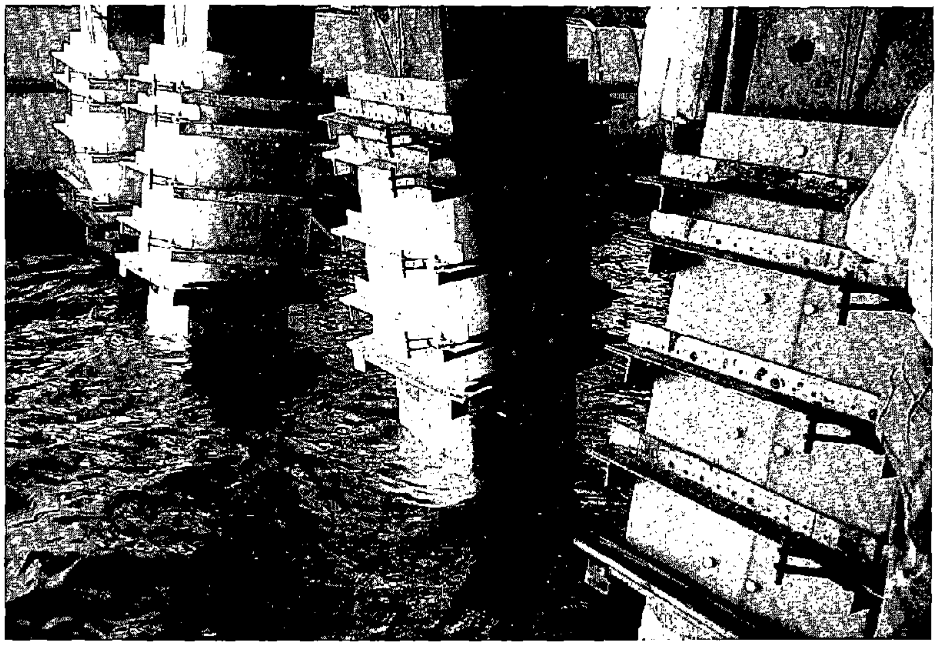

Figure 10. Grout curing phase of installed pile jackets.

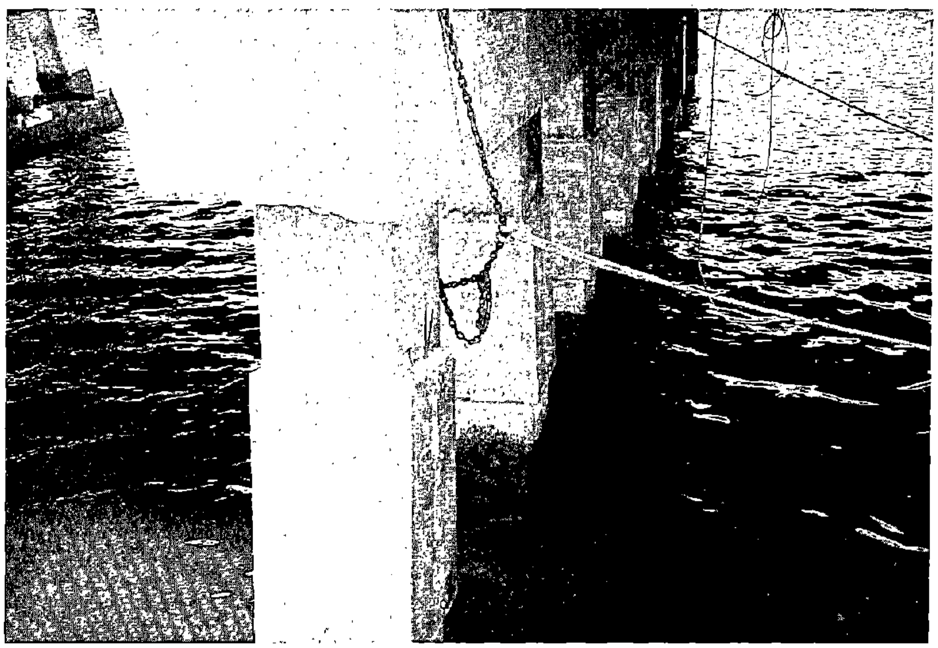

Figure 11. Completed pile jackets.

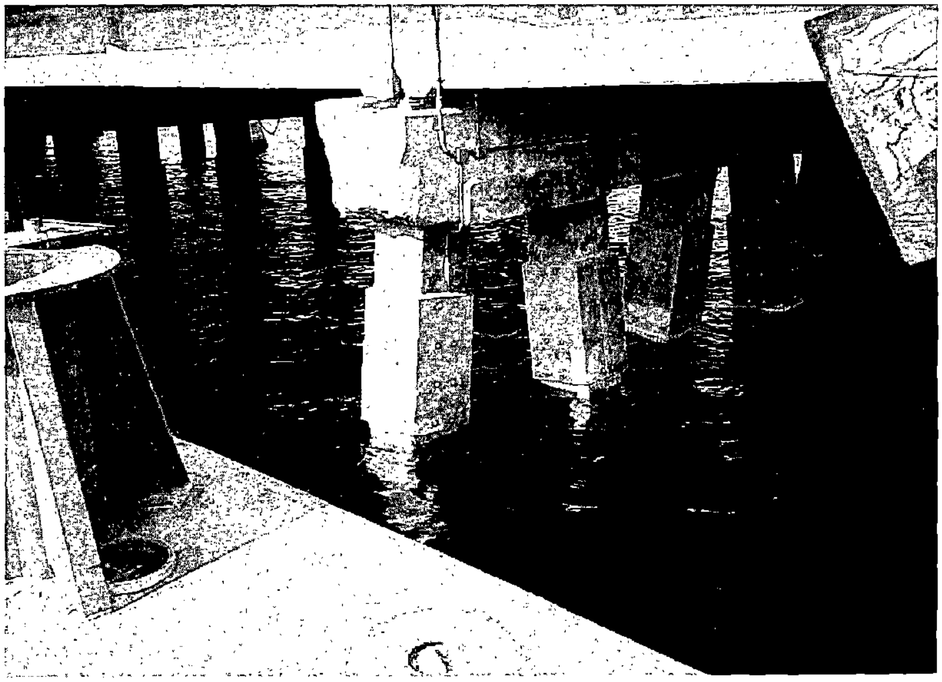

Figure 12. Wiring on pile jacket/titanium mesh anode zone.

A layer of epoxy was then applied at the bottom of the jackets after the jackets were installed and the grout was cured. This layer covered a portion of the jacket, the exposed grout, and the adjacent pile area. The epoxy coating was applied to prevent leakage of cathodic protection current to seawater.

Conductive Rubber Anode System

The conductive rubber anode system was installed on eight prestressed concrete piles as shown in figure 13. The anodes for each pile were manufactured by Lauren Rubber, New Philadelphia, Ohio, and supplied by the Florida DOT. The anodes consisted of flat sheets of carbon-loaded ethylene propylene diene monomer (EPDM) conductive rubber. Anode sheets were held in place on the flat surfaces of the pile by four fiberglass panels, which also had a layer of nonconductive rubber adjacent to the fiberglass panels. One panel was mounted on each face of the pile. Five permanent stainless steel straps, which were wrapped around all four jacket segments, were used to press the jackets against the piles. Conductive rubber anode details are shown in figure 14.

The electrical connection to the anode, shown in figure 15, was obtained by using a small portion of titanium mesh, which was pressed against the anode at the top of the jacket. This mesh completely encircled the pile, contacting all four anode sheets. A 12.5-mm (0.5-in) wide titanium distributor bar, which was resistance-welded to the mesh, provided contact between the titanium mesh and anode system wiring.

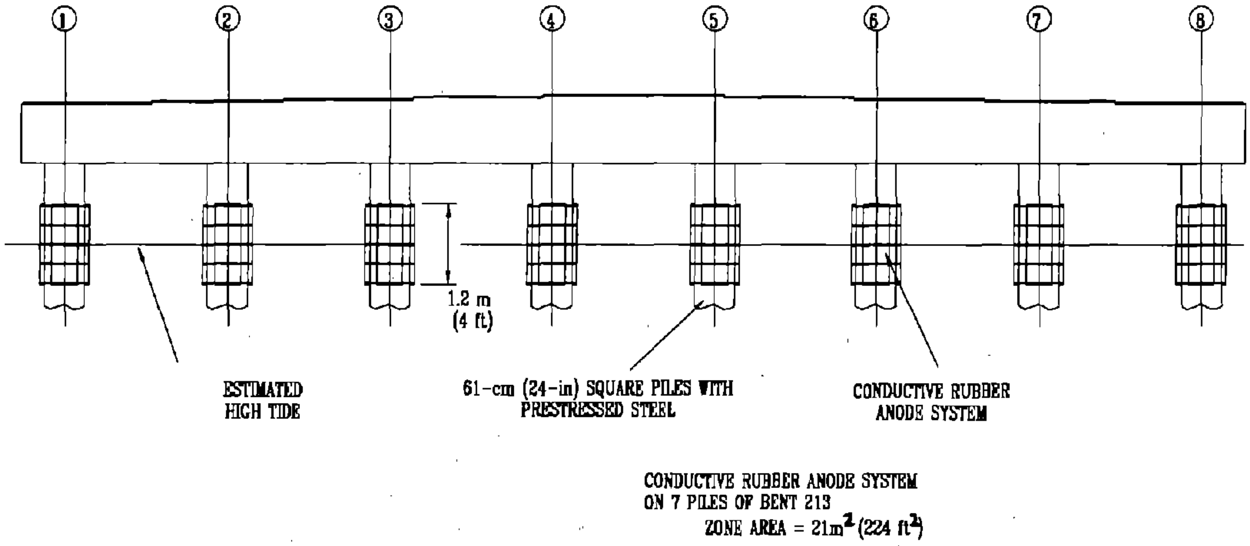


Figure 13. Howard Frankland Bridge - conductive rubber pile zone section view.

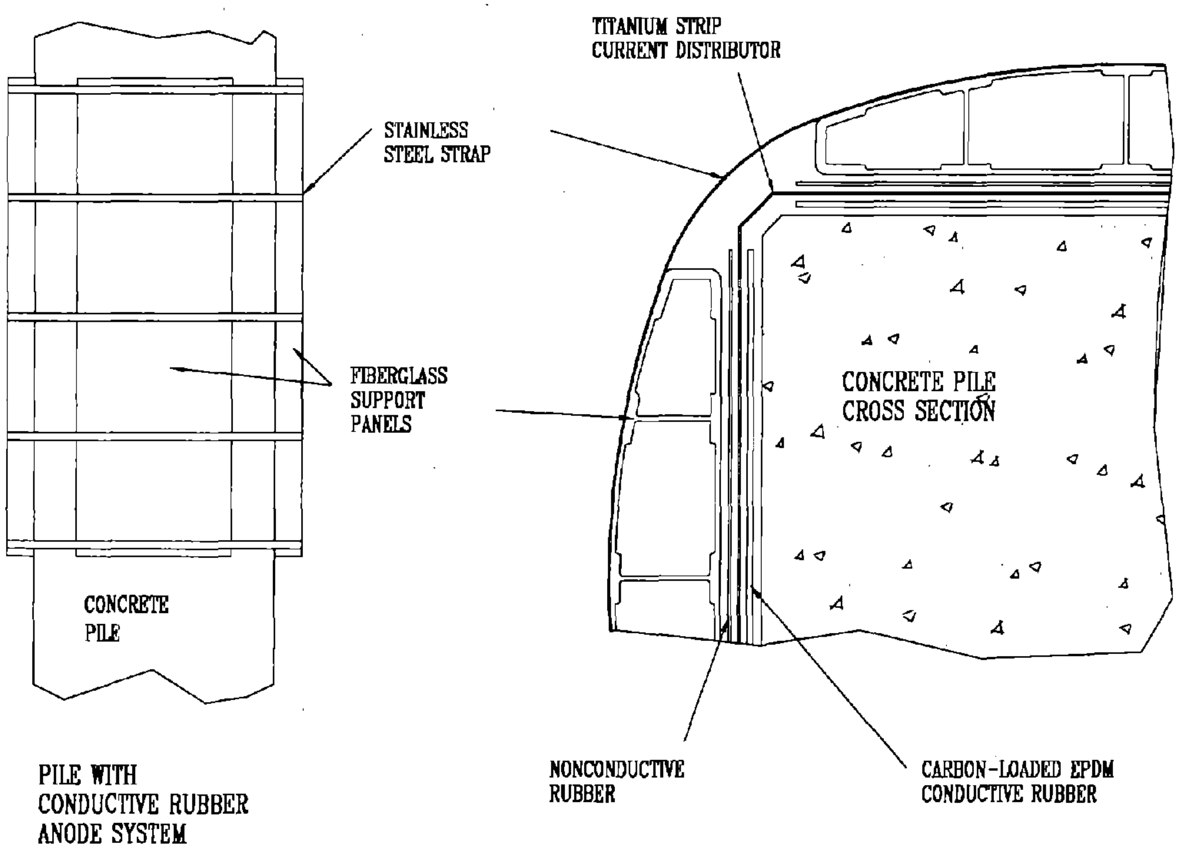


Figure 14. Howard Frankland Bridge - conductive rubber anode system and cross section.

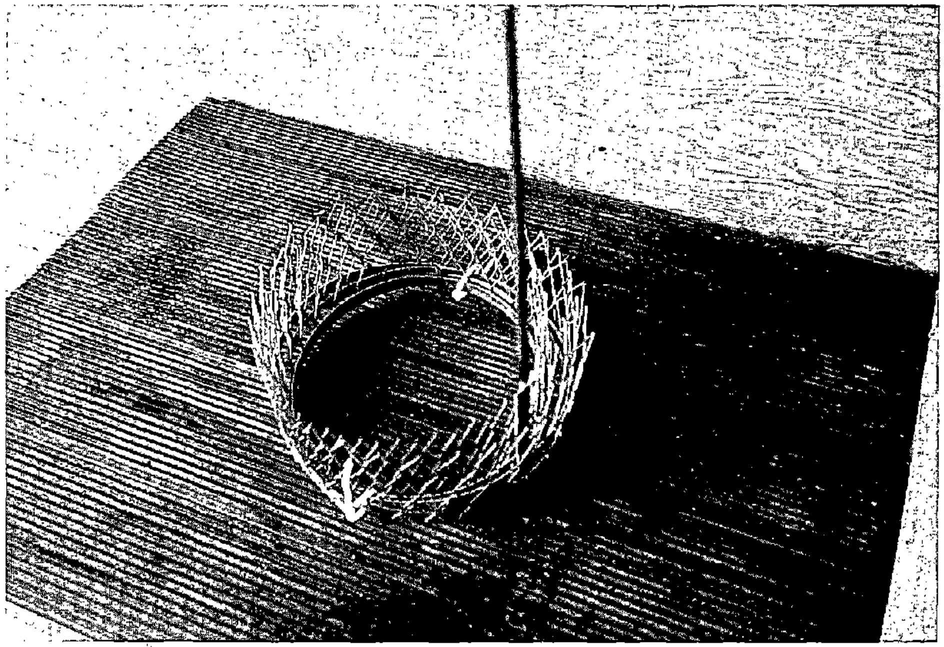

Figure 15. Titanium mesh conductor strip for current connection to conductive rubber.

The electrical connections to the titanium bar, which distributed current to the conductive rubber anode, was made with a butt splice. These connections were covered in nonconductive epoxy and heat-shrink tubing. Connections were made in a watertight electrical junction box located on the pile just above the jacket. Completed conductive rubber anodes are shown in figures 16 and 17.

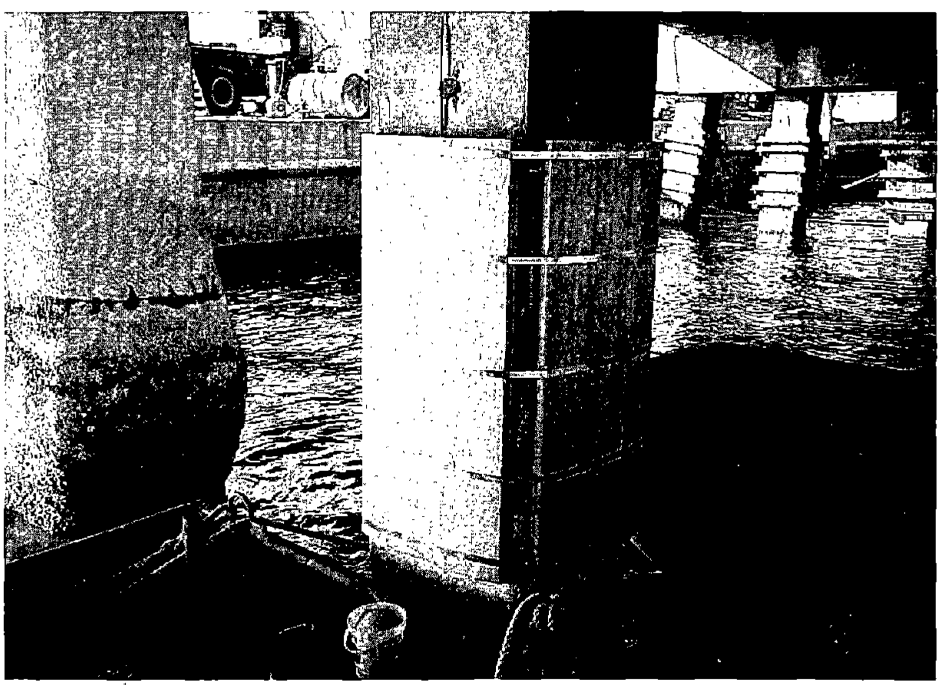

Figure 16. Installed conductive rubber anode.

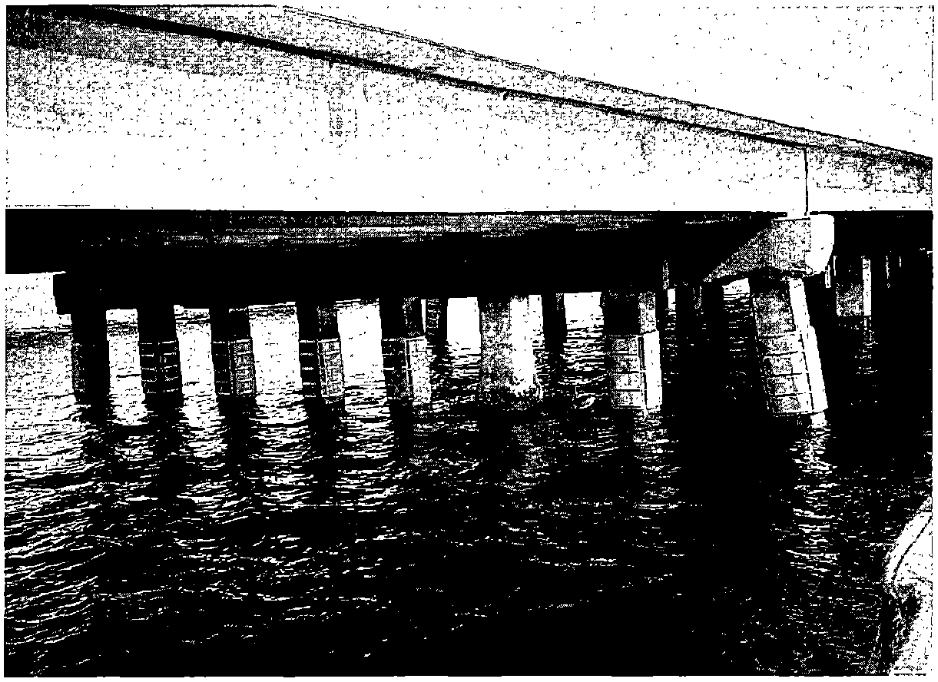

Figure 17. Bent with installed conductive rubber anodes.

Rectifier, Remote Monitoring Unit, and Wiring

The power supply used on the Howard Frankland bridge was a rectifier manufactured by Goodall Electric, Ft. Collins, Colorado. The power supply was designed with four circuits capable of independently controlling the four cathodic protection zones. Each circuit was capable of operating in either constant current or constant voltage mode, with a maximum current of 6 A and a maximum compliance voltage of 24 V. Since the AC power available on the bridge was 480 V, a disconnect box was installed on the AC power supply together with a stepdown transformer.

A remote monitoring unit (RMU) was installed to remotely monitor the status of zone current, zone voltage, reference electrode voltage, and hydrogen cell readings for each anode zone. The RMU also had the capability of performing remote depolarization tests using the embedded reference electrodes. The system was designed to locally store, in battery-backed memory, 31 days of current, voltage, and potential readings on each cathodic protection zone. Since there were no phone lines on the bridge, a cellular phone was installed to access the RMU. The monitoring system installed at this site was a CORD-2 RMU supplied by COREXCO, Dorval, Quebec, Canada.

Both the power supply and the RMU were installed on the outside of the bridge retaining wall west of the bent. A permanent inspection platform, shown in figure 18, was erected to provide access from the bridge and by boat. Normal access is by boat, since the Howard

Frankland bridge does not have a lane for stopping or parking. Neither the platform nor the electrical equipment is visible to vehicular traffic on the bridge.

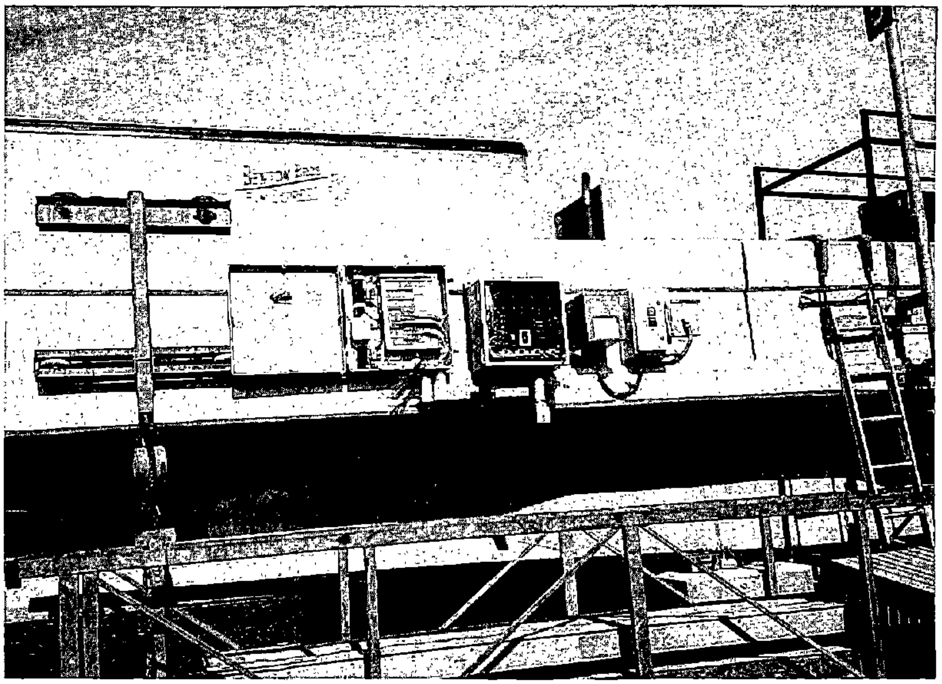

Figure 18. Installed rectifier and remote monitoring unit.

Electrical continuity of the prestressing strands was not assumed, and provisions were made to excavate the girders to achieve full electrical continuity. Of particular concern were the strands in the second layer. At the time of plan preparation it was not known which of several types of girders had been used during initial construction of the bridge. An x-ray procedure was specified to identify the girder design and confirm location of the prestressing strands. However, due to the bevels on the bottom flange of each girder, the x-ray procedure was not effective. A metal locator was then used to approximate the strand location, and exploratory excavations were made to find the strands. Exposed prestressed strands are shown in the beams and pilings in figures 19 and 20, respectively.

The original specifications called for soft-solder connections to the prestressing strands for both the system negative and reference ground connections, but it was not possible to maintain a temperature below the prescribed limit and still make a viable connection. The method of connection was then changed to utilize stainless steel hose clamps and a 5.3-mm$^2$ (#10 AWG THHN) wire, as shown in figure 21. These connections were coated with nonconductive epoxy after installation, as shown in figure 22, and excavations were filled with grout. This method was used for both system negative and reference ground connections, as it provided a good mechanical and electrical connection without damage to the strands.

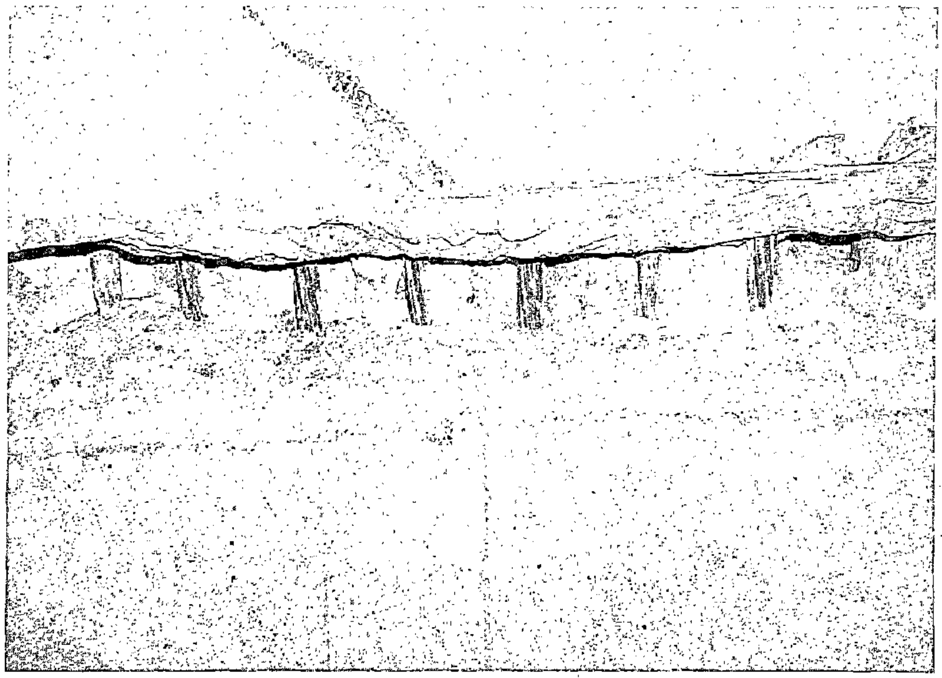

Figure 19. Exposed prestressing strands on bottom of girder prior to continuity bonding.

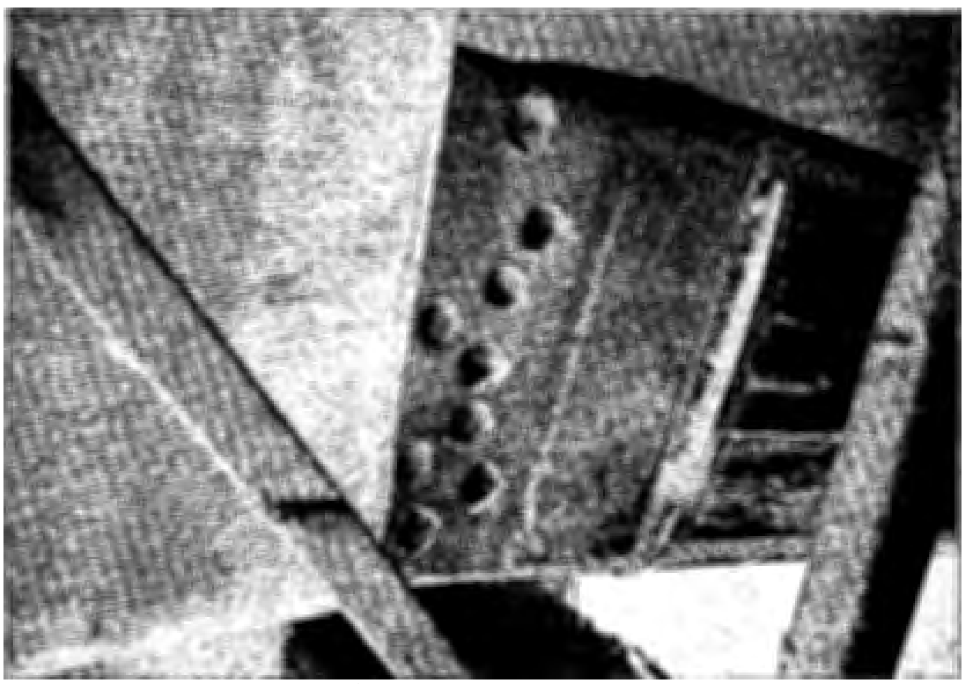

Figure 20. Excavation on piling for prestressing strand location and continuity bonding.

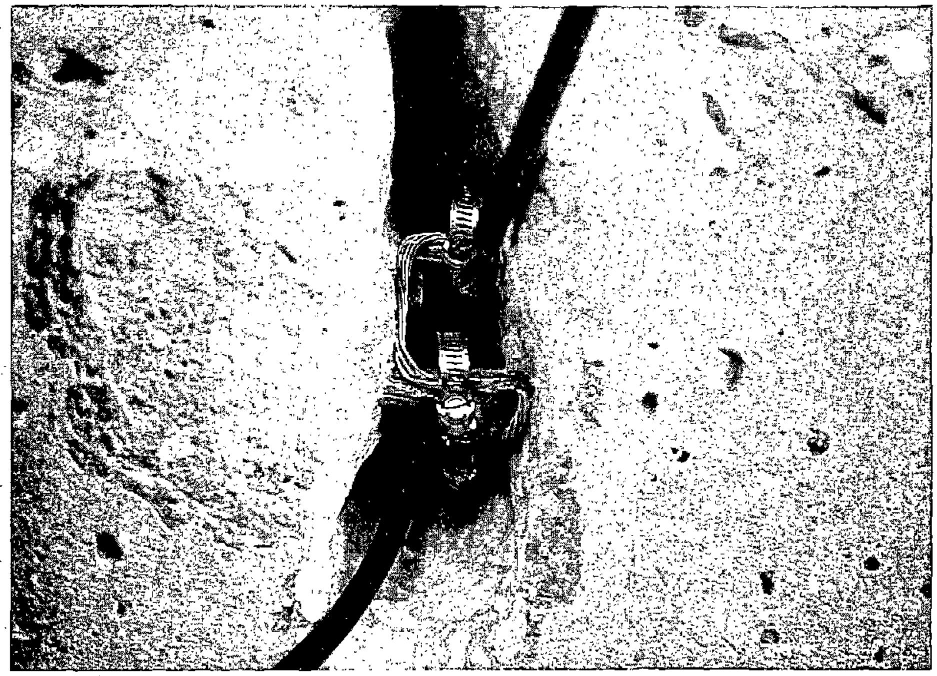

Figure 21. Hose clamps and wire connecting prestressing strands for continuity.

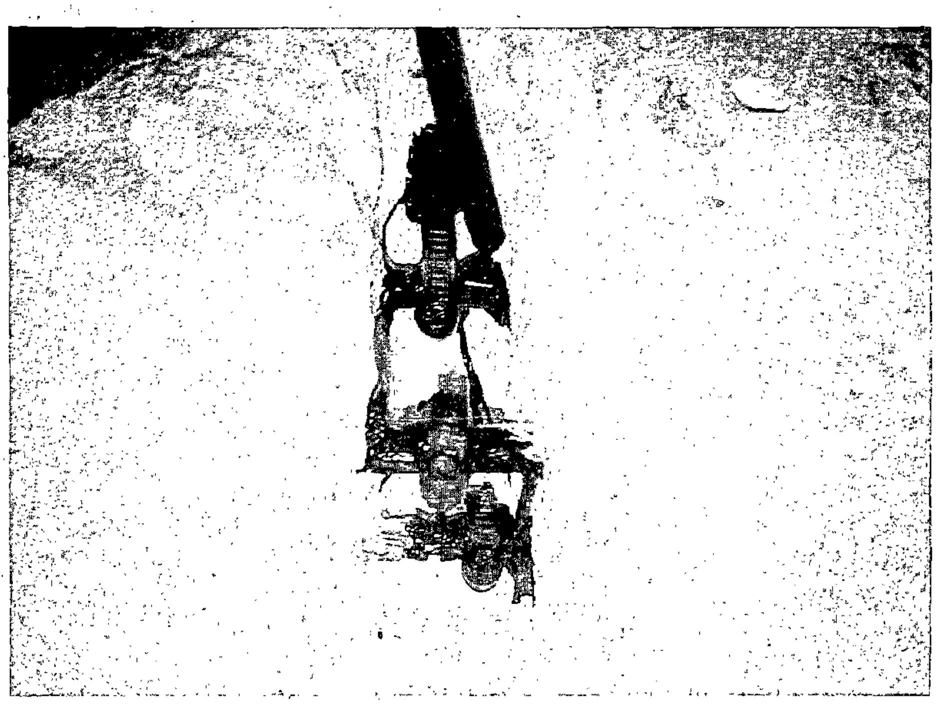

Figure 22. Coated wire and prestressing strand connections.

The beams contained 10 strands of prestressed steel in two rows. The lower layer contained six strands and the upper layer contained four strands. It was found that the upper layer of strands were not continuous with the lower strands. Excavations were therefore made on both sides of the beams to expose the strands. Both of the upper strands located on either side of the beams were connected to the outermost strand in the lower level. After connections were made, the excavations were filled with grout.

Two silver/silver chloride reference electrodes were installed in each electrical zone. Each reference electrode was installed in an area of sound concrete where the most negative (corrosive) readings were recorded on the half-cell potential survey. The half-cell potential survey was conducted in accordance with ASTM C876-87. At each reference electrode site, a slot was excavated between reinforcing bars or prestressing strands without exposing the surrounding steel. The reference electrode was then installed in the slot and was backfilled with patching concrete with salt added to match the salt content of the surrounding concrete. A separate ground wire was installed for each reference electrode at least 0.6 to 1.5 m (2 to 5 ft) from the reference, and that excavation was backfilled with salt-free concrete. The connections to the reference electrode and reference electrode ground wires were made with silicone-filled King connectors. These connections were made in a watertight electrical junction box placed near the location of the cells. The wires running from this connection to the rectifier were continuous without any splices.

One embedded hydrogen cell was also installed in each electrical zone. The hydrogen cell was a sealed Devanathan probe with a palladium membrane capable of transmitting hydrogen. This membrane is connected to the system negative (steel), and if hydrogen is generated, the cell internally converts the flux of hydrogen into an electrical current. The current, if generated, is sensed as a millivolt signal and recorded by the RMU. The location and installation procedures for the hydrogen cell were similar to that used for the reference electrodes, except that backfill was made with salt-free patching concrete. Connections to the hydrogen cell leads were made with butt splices, which were coated with nonconductive epoxy and covered with heat-shrink tubing. As with the reference cells, a continuous shielded cable was used to connect the cell to the rectifier. A hydrogen probe and reference electrode are shown in an excavation in figure 23.

## Barge Accident and Subsequent Repairs

### *Barge Accident*

On October 10, 1993, a storm with 65- to 80-km/h (40- to 50-mi/h) winds blew in from the southwest. The anchor for the 45.8-m (150-ft) long work barge failed, allowing the barge to drift into the structure. Damage to the southernmost piles of three bents occurred. Due to the high tide and direction of the wind, damage also occurred on the pile caps.

Pile 8 of bent 211 sustained major damage, as shown in figure 24. The concrete was completely gone for a distance of about 60 cm (2 ft) from the bottom of the pile cap, leaving only the prestressed strands. Part of the southern face of the pile was worn down due to abrasion by the barge. Florida DOT divers inspected the pile and found a crack in the pile 46 cm (18 in) below the mud line. The titanium mesh jacket on this pile was destroyed, with only part of it remaining on the other sides of the pile. Part of the concrete on the eastern side of the face of the pile cap was knocked off, leaving exposed bar. Several cracks were also present on the cap.

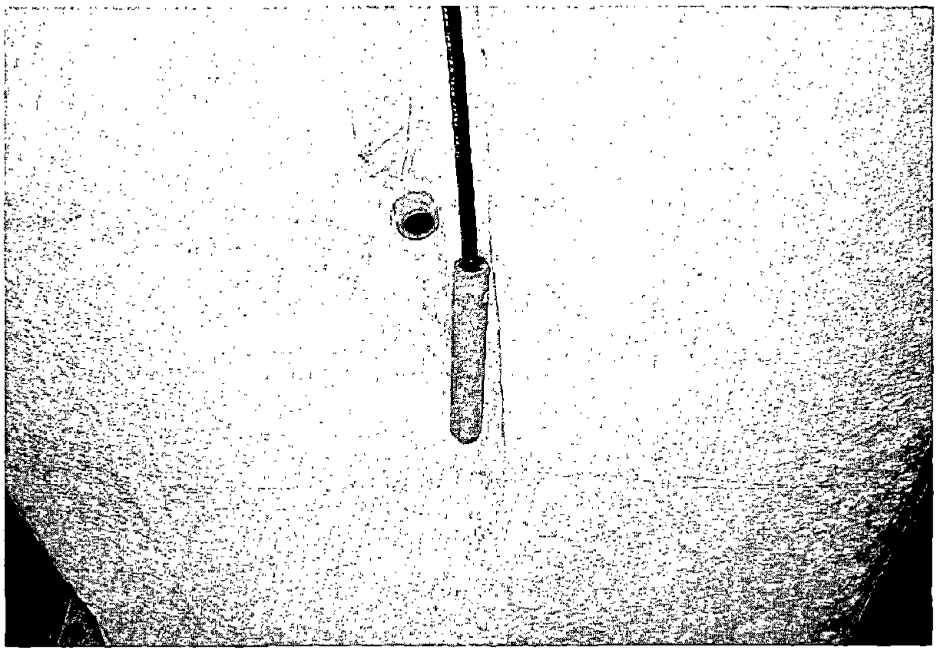

Figure 23. Hydrogen probe and reference electrode in excavation on piling.

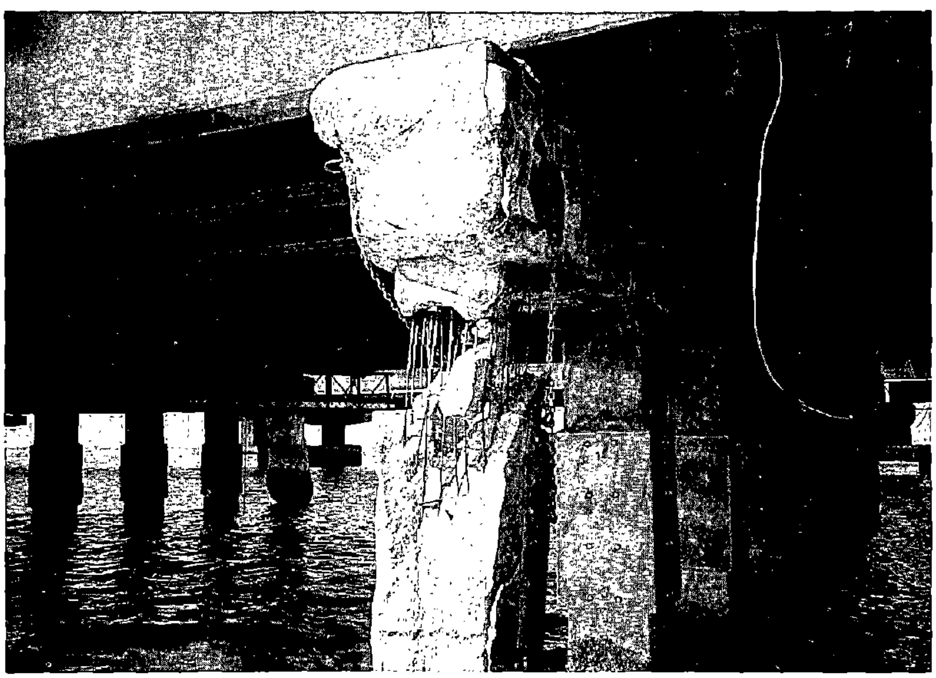

Figure 24. Barge damage to bridge structure.

The southernmost pile of bent 212 sustained only minor damage, as shown in figure 25. The lower half of the fiberglass jacket was broken off, but the titanium mesh was still in place. The concrete on both the pile and cap was abraded. No cracks were observed by Florida DOT divers.

Figure 25. Barge damage to pile jacket.

The southwest corner of pile 3 of bent 212 and the northwest corner of pile 4 of this bent were damaged. In addition, some small punctures were observed in the west face of the jacket on pile 4. This damage was caused by a small barge that was used by the metallizing crew.

Pile 8 of bent 213 suffered only minor abrasions, and the conductive rubber jacket was damaged. Two of the panels were salvaged; one of them was cracked and the other was reusable.

*Repairs to CP System*

The damaged fiberglass jackets of bent 211 and bent 212 were replaced with one-piece jackets. New titanium mesh and plastic fasteners were provided to attach the mesh to the jackets. The same spacing and procedures were used as was used for the original installation. The damaged fiberglass jackets on piles 3 and 4 of bent 212 were repaired with fiberglass mat and resin.

The Florida DOT Materials Department provided four conductive rubber panels to replace those that were damaged or lost. These were mounted in the same fashion as for the original installation.

*Repairs to Structure*

The pile caps of bents 212 and 213 were sandblasted, and zinc was reapplied (zinc had been applied to these members prior to this contract). A sand cement mix was used to reface the piles directly below these caps to their original profile.

Major repairs were required on pile 8 of bent 211. An excavation was made to a distance of 0.92 m (3 ft) below the mud line around the base of the pile. Florida DOT divers inspected this area to ensure that no further cracks were present. Since no other cracks were observed, the repairs were completed as follows.

The existing pile was cut off at a distance of 5.34 m (17.5 ft) below the pile cap. A 1.07-m (42-in) diameter, 3.36-m (11-ft) long, corrugated metal pipe (CMP) was placed around the pile such that it extended 0.92 m (3 ft) into the bottom of the bay. This pipe extended vertically for another 2.4 m (8 ft). The CMP extended 30 cm (1 ft) above the remainder of the pile. A steel section, which was zinc metallized, was placed such that it extended 60 cm (2 ft) into the hollow core portion of the pile. This beam was centered with the central axis of the pile. A rebar cage consisting of eight #8 bars ran the entire length of the CMP. Circular hoops made from #4 bars were placed on 30-cm (1-ft) centers. This stage of the repair was filled with a Class V concrete mix, which was approved by the Florida DOT. The concrete was pumped into the CMP from the top, with the hose extending all the way to the bottom of the pipe. The hose was gradually removed as the pipe was filled. Florida DOT divers were present at the site during this operation.

The second stage of the pile repair consisted of placing a new 60-cm by 60-cm (24-in by 24-in) section from the top of the stage 1 repair to the pile cap. A square rebar cage consisting of eight #8 bars with #4 bar hoops placed on 30-cm (1-ft) centers was used during this stage of the repairs. Holes were drilled in the top surface of the first-stage repair and the cage was fitted into these holes. The pile was attached to the cap using eight #8 bars as dowels. These were placed in holes drilled into the bottom of the cap and were epoxied in place. The rebar cage was tied to these dowels using tie wires. The same concrete mix design used on the first-stage repair was pumped from the bottom of the mold. Two small holes were drilled in the sides of the cap extending into the area of the pile repair. Concrete was observed in these holes when it reached the bottom of the pile cap.

The cracks in the end of the pile cap were either injected or covered with epoxy as per Florida DOT repair procedure. Any shotcrete in the repair area was removed. A rebar cage consisting of #4 bars was placed in the bottom of the west pile face, and this was tied to the exposed rebar in that area. A wooden form was placed around the end and sides of the pile cap, and Sikatop 123 poly-modified portland cement mortar was used to resurface the pile.

ABBEY ROAD BRIDGE

Metallized Zinc Anode System

A single metallized zinc anode zone was installed on the soffit of the bridge, which was comprised of 10 prestressed box beams totaling an area of 106 $m^2$ (1140 $ft^2$). A layout and section view of the CP system are shown in figure 26. Construction began on the Abbey Road bridge on December 13, 1993, with a careful inspection and sounding of the soffit of the bridge for delaminations. Consistent with the results of tests conducted during the selection process, no delaminations were found. Exposed chair feet were located on beams 3, 5, 6, 7, and 10, and the exposed metal surfaces were covered with epoxy resin.

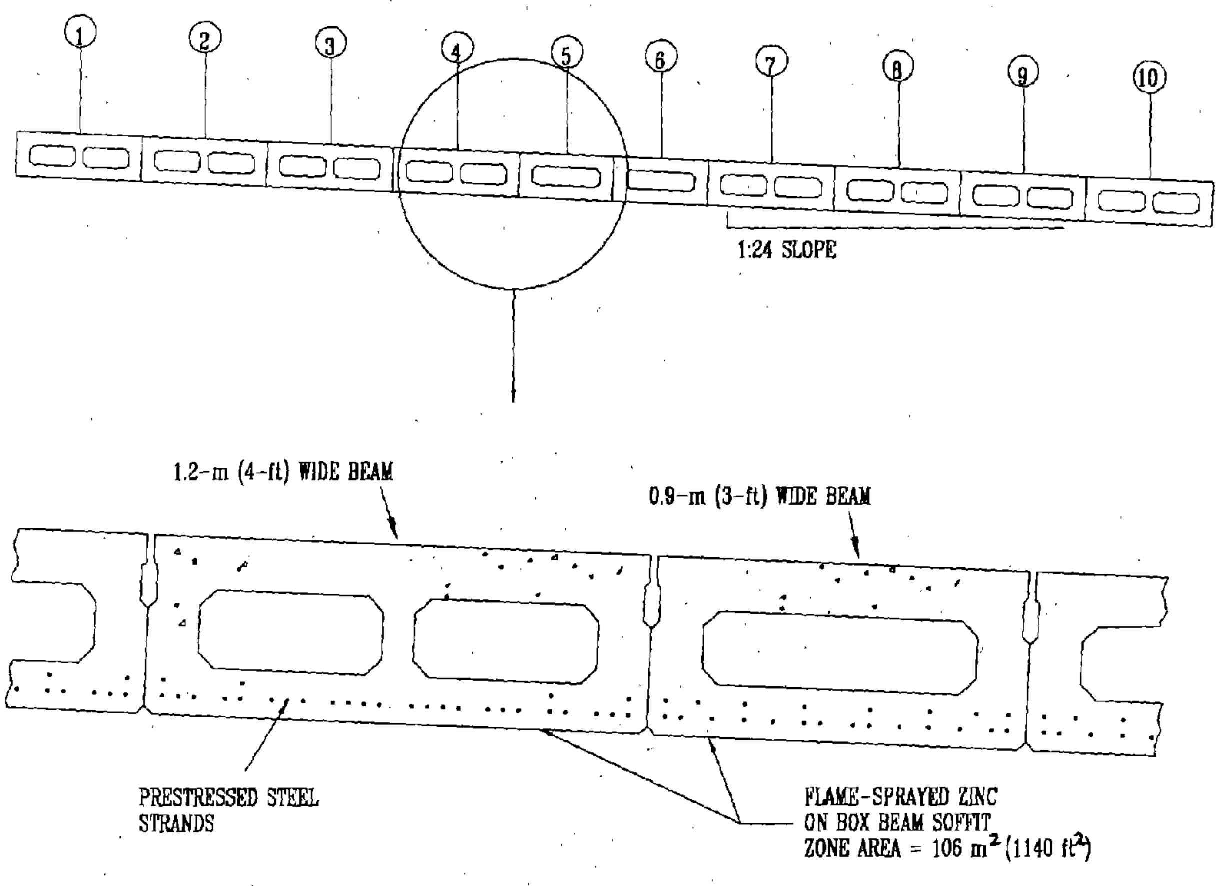


Figure 26. Abbey Road Bridge - box beam section view.

Since construction took place during the winter months, the weather caused several delays and problems. Ambient temperatures were as low as -29 °C (-20 °F), which caused delays because the flame-spray equipment would not function at very low temperatures. Plastic tarps were hung across the opening of each end of the bridge, and propane heaters were used to heat the spray area and soffit surface of the bridge to improve bond strength.

Melting snow leaked through at several joints between the box beams and, because of the superelevation of the bridge, ran across the underside of the beams. Several attempts to dry the underside of the bridge were unsuccessful. A number of options were then tested to prevent the water from wetting the soffit side of the beams. These included caulking along the edges of the joints to form a dike to stop water movement, placing geotextile fabric in the joints to keep water confined to the joints and to act as a drip edge, and sawcutting a drip line along the edges of each box beam. The last option, sawcutting a drip edge, proved to be the most effective, and this is shown in figure 27.

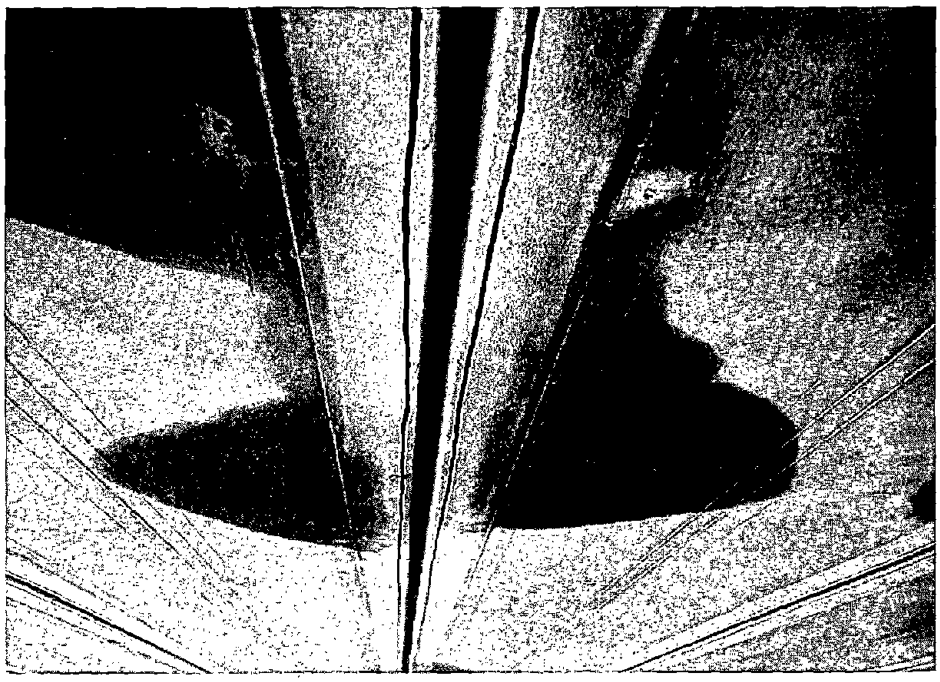

Figure 27. Finished grooves on each side of the joint between adjacent box beams.

The sawcuts were made with a dry, diamond-edged, 15.2-cm (6-in) diameter saw blade in an angle-head grinder. The 6.4-mm (0.25-in) wide cuts were made 1.27 cm (0.5 in) away from the edge of the beam chamfers. The depth of the cut was 6.4 mm (0.25 in). Once the sawcuts were completed, the soffit of the bridge could be dried with propane torches, and the thermal spraying of the zinc anode proceeded without difficulty.

The zinc thermal spraying, shown in figure 28, was done by Metallizing Masters of Alliance, Ohio. The concrete surface was sandblasted prior to the flame-spray application of the zinc coating. A 0.25- to 0.38-mm (10- to 15-mil) thickness was specified. Tape coupons were attached to the soffit before spraying, and these coupons were removed after spraying for thickness measurement. Zinc thickness, as measured with a micrometer, ranged from 0.38 to 0.79 mm (15 to 31 mil).

Figure 28. Flame spraying of zinc onto prepared surface.

The system was continuously monitored for shorts as the flame spraying progressed. Several shorts occurred at chair feet that were previously coated with epoxy resin. These areas were cleaned and epoxy was reapplied. When the system was retested, no shorts were found.

Adhesion of the applied zinc coating was tested at one spot per box beam using an Elcometer Adhesion Tester. All 10 tests conducted were above the minimum bond strength of 690 kPa (100 lbf/in$^2$), ranging from 1035 to 3280 kPa (150 to 475 lbf/in$^2$).

The zinc anode of each box beam was electrically isolated from the rest, so a single, continuous 1.3-cm (0.5-in) wide by 0.889-mm (0.035-in) thick titanium current distributor strip was held against the soffit with plastic anchors to electrically connect each box-beam anode. A layer of zinc was sprayed over the length of the conductor strip to achieve a more intimate contact. Although this looked adequate, testing revealed that several beam anodes remained electrically isolated. This was corrected at each beam by placing a nickel reticulate contact across the conductor strip and against the zinc anode with a titanium compression plate over the nickel contact. Four stainless steel screws were tightened into plastic anchors in the beam for compression, which provided a reliable connection. The edges of this fixture were sealed from moisture intrusion with silicone caulk. The completed soffit wiring with connections to the anodes is shown in figure 29.

Figure 29. Soffit wiring with connections to anode and reference electrode.

Rectifier, Remote Monitoring Unit, and Wiring

The power supply was a rectifier manufactured by Goodall Electric, Ft. Collins, Colorado. The supply was designed for operation in either constant-current or constant-voltage mode, with a maximum current of 6 A and a maximum compliance voltage of 24 V.

As with the Howard Frankland Bridge, an RMU was installed to remotely monitor the status of the zone current, zone voltage, reference electrode voltage, and hydrogen cell readings. The RMU also had the capability of performing remote depolarization tests using the embedded reference electrodes. The system was designed to locally store, in battery-backed memory, 31 days of current, voltage, and potential readings. A telephone line was available at the Abbey Road bridge site for connection to the RMU. The RMU installed was again a CORD-2 RMU supplied by COREXCO, Dorval, Quebec, Canada.

The mounting location for the rectifier and RMU was on the west side of the south abutment wingwall, as shown in figures 30 and 31. This location was thought to be safe from vandalism, salt spray, and traffic damage. The mounting height was chosen so that the rectifier and RMU would not be submerged when Baldwin Creek flooded. Power was obtained from a nearby power line, and a main power cut-off switch was placed before all other electrical components. A system ground wire was connected to the grounding lug on the rectifier cabinet and was connected to a copper rod driven into the ground.

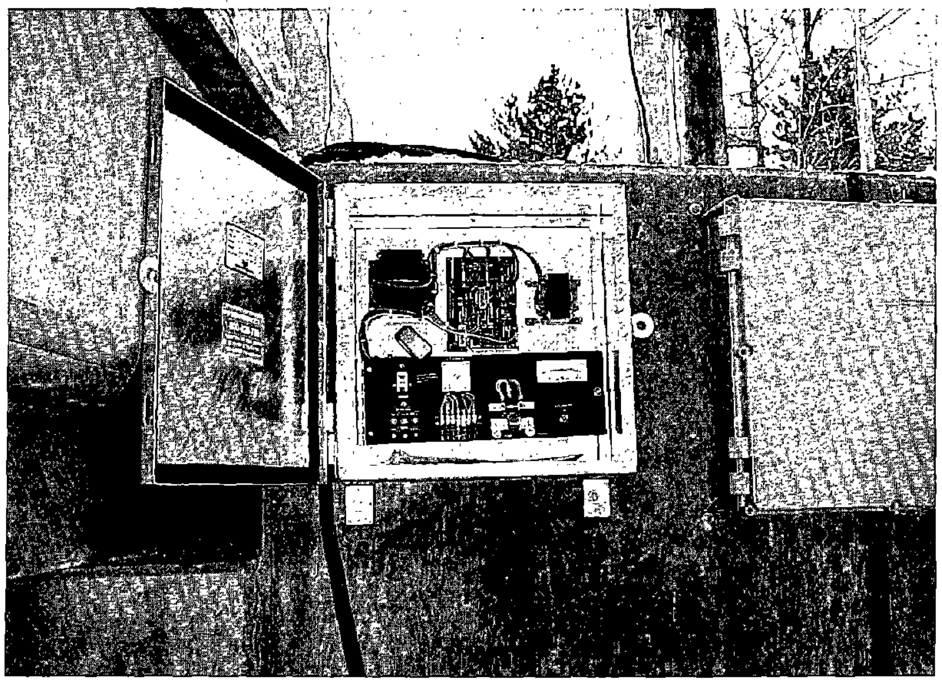

Figure 30. Rectifier mounted on abutment wingwall.

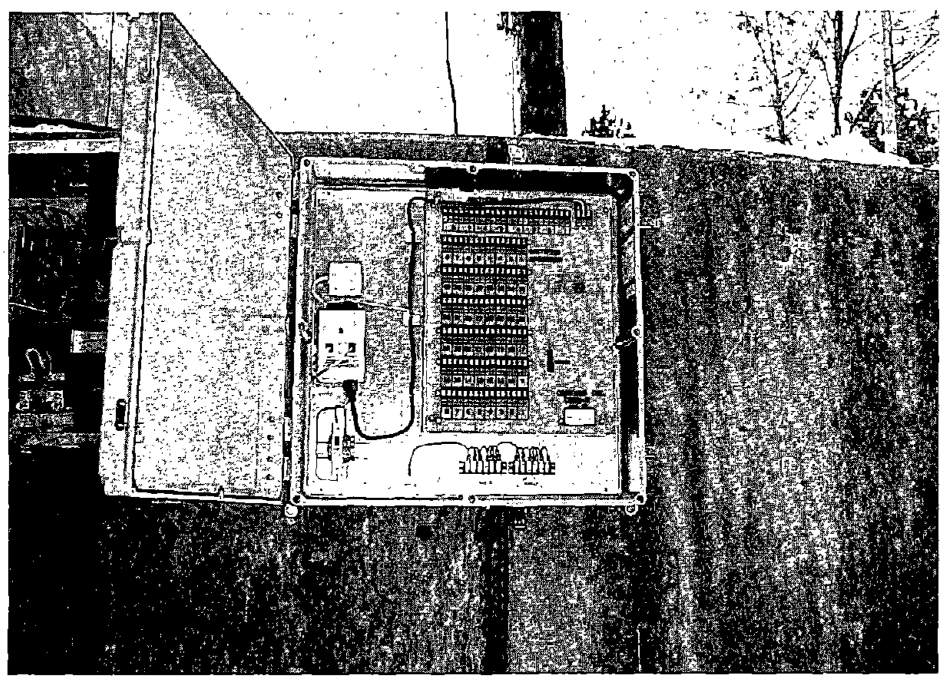

Figure 31. Remote monitoring unit mounted next to rectifier on abutment wingwall.

All prestressed strands were discontinuous. The upper row of strands in each beam was not bonded to the lower row since chloride levels at the upper strands were too low to induce corrosion. Three powder samples obtained at depths ranging from 32 to 44 mm (1.25 to 1.75 in) all contained 0.02-percent total chlorides by weight [0.27 kg/m$^3$ (0.77 lb/yd$^3$)], whereas three powder samples taken at depths ranging from 12 to 25 mm (0.5 to 1 in) averaged 0.07-percent total chlorides by weight [0.95 kg/m$^3$ (2.7 lb/yd$^3$)]. Excavations for continuity bonding, shown in figure 32, were made across the bottom of the beams to expose the bottom row of prestressing strands. An insulated, 5.3-mm$^2$ (#10 AWG) copper wire was placed in each excavation. A short section of insulation at each end of the wire was removed, and a stainless steel hose clamp was used to mechanically connect the wire to the prestressing strand for electrical continuity. The connection was then coated with liquid plastic. After connections were made, the excavations were filled with grout.

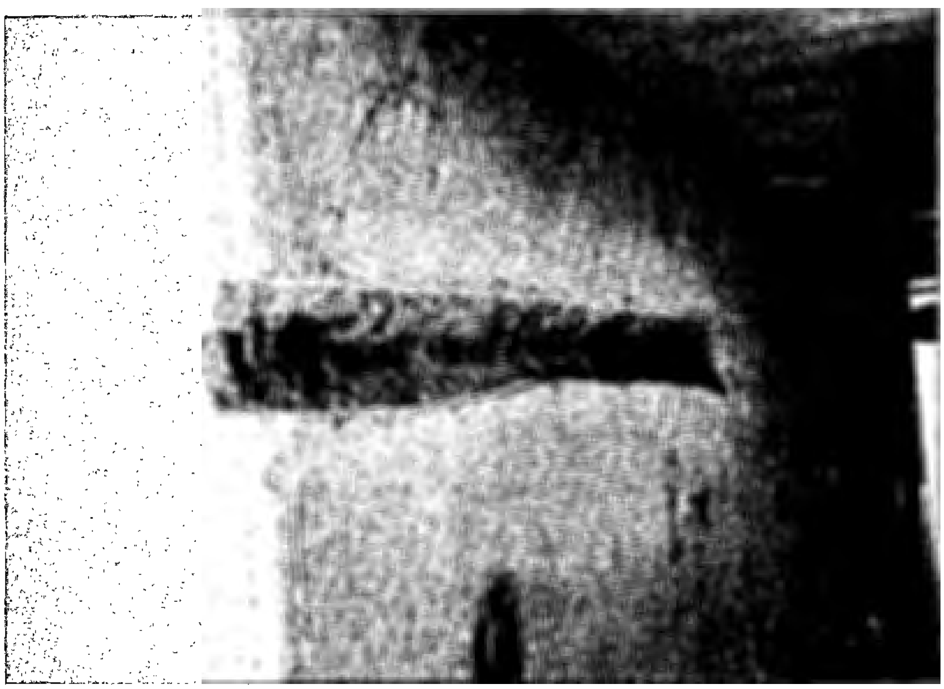

Figure 32. Excavated strands for continuity bonding.

One prestressing strand was found to be severed by corrosion. It was repaired with a Grabb-it splice and retensioned to provide a tensile force of 89 000 N (20,000 lbf). The Grabb-it splice, shown in figure 33, was supplied by Prestress Supply Inc., Lakeland, Florida.

The method of making connections to the prestressing strands for both the system negative and reference ground connections was accomplished in the same manner as on the Howard Frankland Bridge trial in Florida. These connections were coated with nonconductive epoxy after installation, and excavations were filled with grout. This method was used for both system negative and reference ground connections, as it provided a good mechanical and electrical connection without damaging the strands.

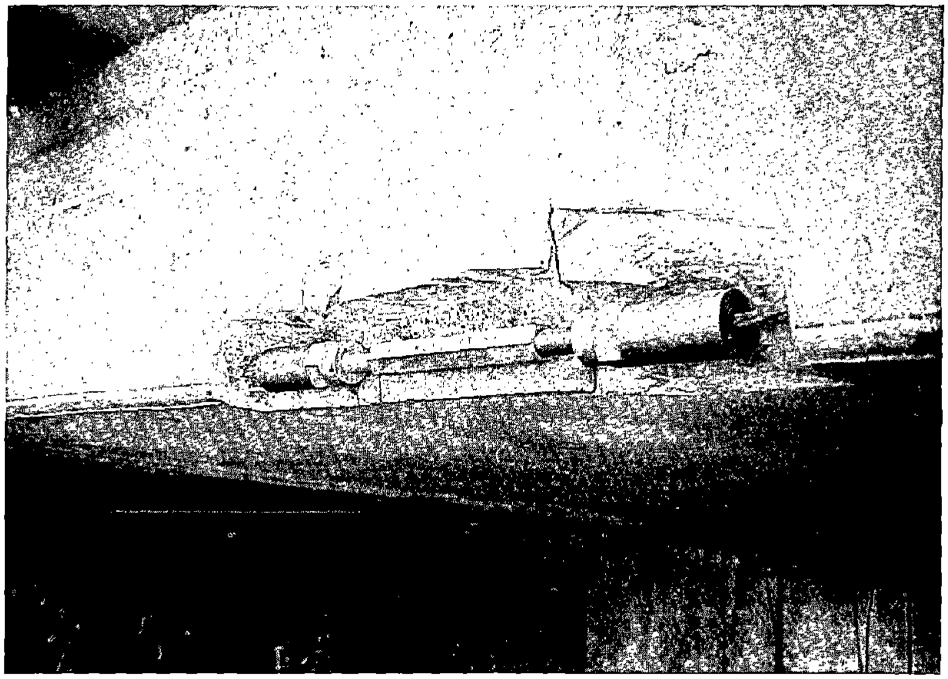

Figure 33. Grabb-it splice connector installed at site where a prestressing strand sample was removed.

As with the Howard Frankland Bridge in Florida, two silver/silver chloride reference electrodes and one Devanathan hydrogen cell were installed in the most negative (corrosive) locations in the zone. The procedures for installation of reference electrodes and hydrogen cell were also the same as previously reported. The reference cell, reference cell ground, and hydrogen cell lead connections were all made in the same manner as on the Howard Frankland Bridge trial.

The wiring system utilized 3.8-cm (1.5-in) diameter polyvinyl chloride (PVC) conduit. One main conduit ran the full length of the abutment and contained all of the system negative, reference electrode, and hydrogen probe wires. The system negatives for each beam were relocated to the end closest to the abutment. The reference electrode and hydrogen probe conduits ran directly to the main conduit. A small junction box was placed over the wires where they exited the concrete.

## WEST 130$^{th}$ STREET BRIDGE

### Conductive Coating Anode System

The single conductive paint anode system zone was installed on the soffit of the bridge, which was comprised of 15 prestressed box beams totaling 223 m$^2$ (2400 ft$^2$), as shown in figure 34. Work began on the West 130$^{th}$ Street bridge on December 13, 1993, and proceeded in parallel with construction on the Abbey Road bridge. Sounding of the soffit of the bridge confirmed that there was no loose or delaminated concrete to repair. A sample of prestressing strand was removed from beam 15 for petrographic analysis. The strand in the beam was repaired with a Grabb-it splice.

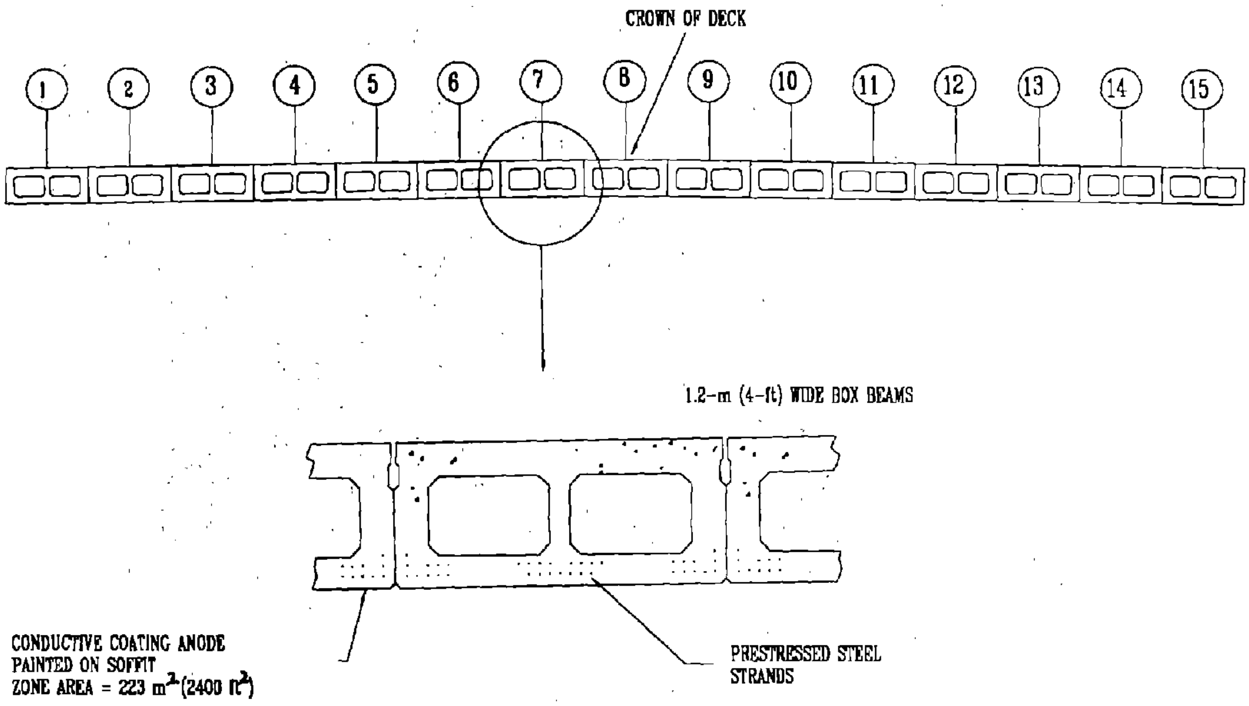


Figure 34. West 130th Street Bridge - box beam section view.

The single CP zone was instrumented with two Ag/AgCl reference electrodes, one in beam 2 and one in beam 15. One hydrogen probe was placed in beam 15, which had the most corrosive half-cell potential readings.

As with the Abbey Road bridge, the cold temperatures and leaking of water through joints between the box beams presented major problems. The leaking joints were of particular concern since a conductive paint was selected for application to the soffit of this bridge. The manufacturer of the conductive coating indicated that the coating could be successfully applied only if the soffit was dry prior to and during the coating application and curing. The soffit was therefore prepared with sawcut drip edges, as described for the Abbey Road bridge, in an effort to keep the concrete dry during conductive coating application and for several consecutive days afterward for curing. Drying of the soffit 2 days after a storm is shown in figure 35. Even with these measures, it was not possible to adequately dry the concrete during the winter months, and the work was stopped from mid-January until April 1994. Application of the conductive coating then proceeded without difficulty.

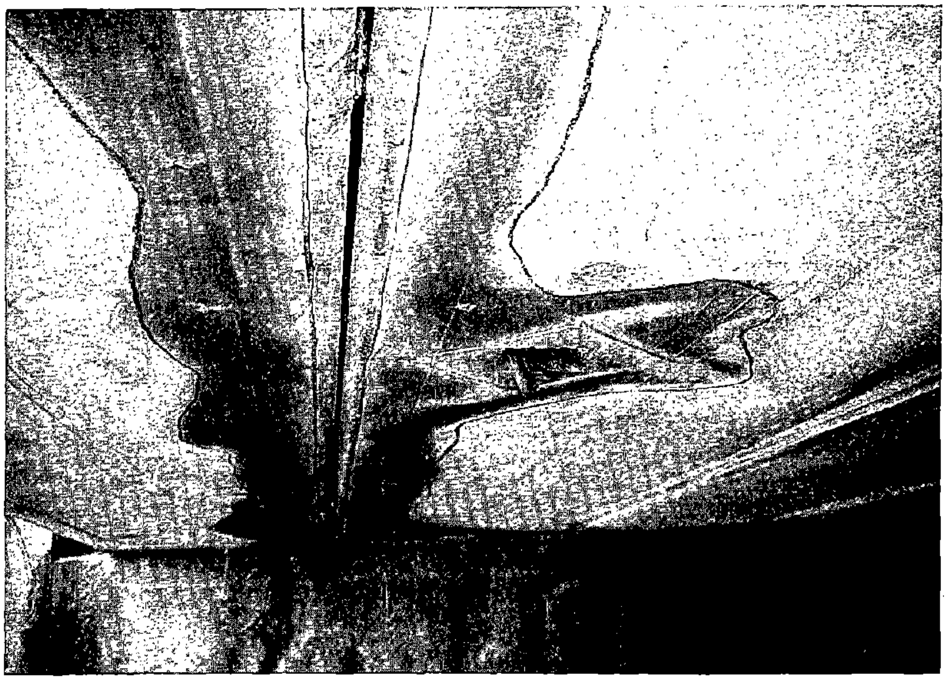

Figure 35. Drying of soffit 2 days after a storm.

The conductive coating selected was Permarock SC5, a solvent-based coating with a successful record of field installations, particularly in Europe. This paint formulation is based on inert chlorinated polymers with a carbon matrix, which makes it very resistant to both acidic and alkali environments. When fully cured, its electrical resistance is less than 40 Ω/square when applied to a thickness of 150 μm (6 mil). The paint was supplied by the ELGARD Corporation, Chardon, Ohio.

On April 20, 1994, the bridge soffit was sandblasted to remove dirt, laitance, and other materials. Two coats of conductive coating were then applied using rollers over an adhesion-promoting primer. Hooded suits, gloves, boots, goggles, and half-face respirators with organic vapor canisters were necessary safety precautions. A moveable, plastic-covered wooden frame (shown in figure 36) was placed on the ground under an area being painted to prevent drips and spills from entering Baldwin Creek. The coating thickness was measured during placement with a thickness gauge and after final application with a Tooke gauge. The specified minimum 0.25-mm (10-mil) dry-film thickness was achieved. The system was continuously monitored for shorts during the rolling application, and none was encountered. Short-circuit testing after the coating had cured revealed that there were no shorts of the anode to the steel.

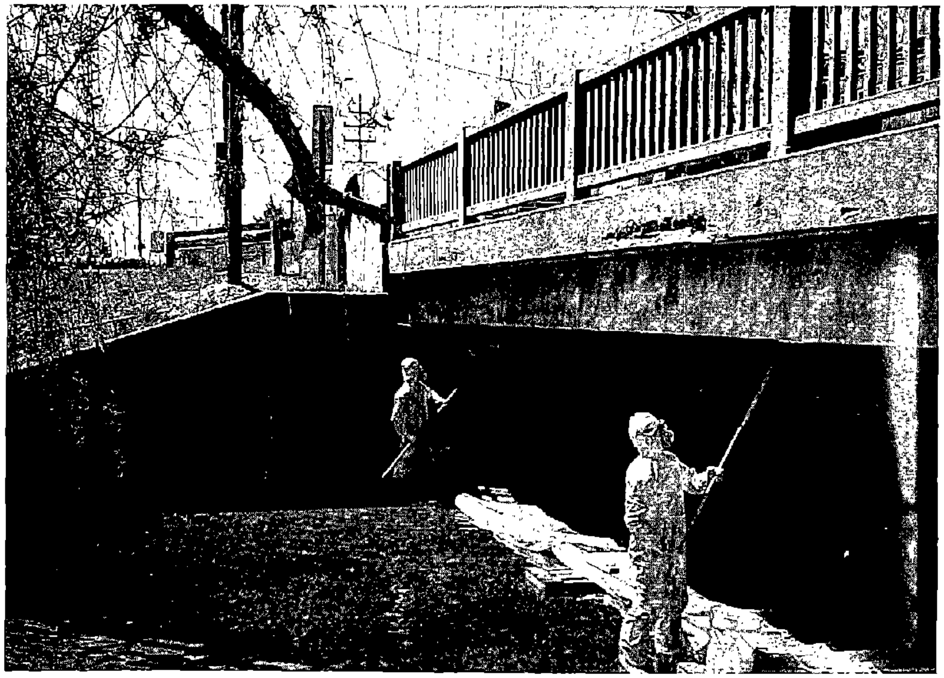

Figure 36. Roller application of conductive coating with polyethylene-covered frame used to catch coating spills and drips.

Each box beam was electrically isolated from the rest, so a continuous anode conductor was needed to supply power to all of the box beams. A 1.62-mm (0.064-in) diameter, mixed-metal-oxide-coated titanium wire was centered on the bridge soffit and fastened down with plastic anchors to make this connection. Two additional wires were placed 4 m (13 ft) to each side of this central wire, 2.1 m (7 ft) from the abutment walls, and fastened in the same manner.

Rectifier, Remote Monitoring Unit, and Wiring

The same power supply and remote monitoring unit were used for the West 130$^{th}$ Street bridge as were used for the other bridges in this contract. As with the Abbey Road bridge site, a telephone line was conventiently available and was used to access the RMU.

The rectifier and RMU were located on the east side of the south abutment wingwall, as shown in figure 37. This location was thought to be safe from vandalism, salt spray, and traffic damage. The mounting height was chosen so that the rectifier and RMU would not be submerged when Baldwin Creek flooded. Power was obtained from a nearby power line, and a main power cut-off switch was placed before all other electrical components. A system ground wire was connected to the grounding lug on the rectifier cabinet and was connected to a copper rod driven into the ground.

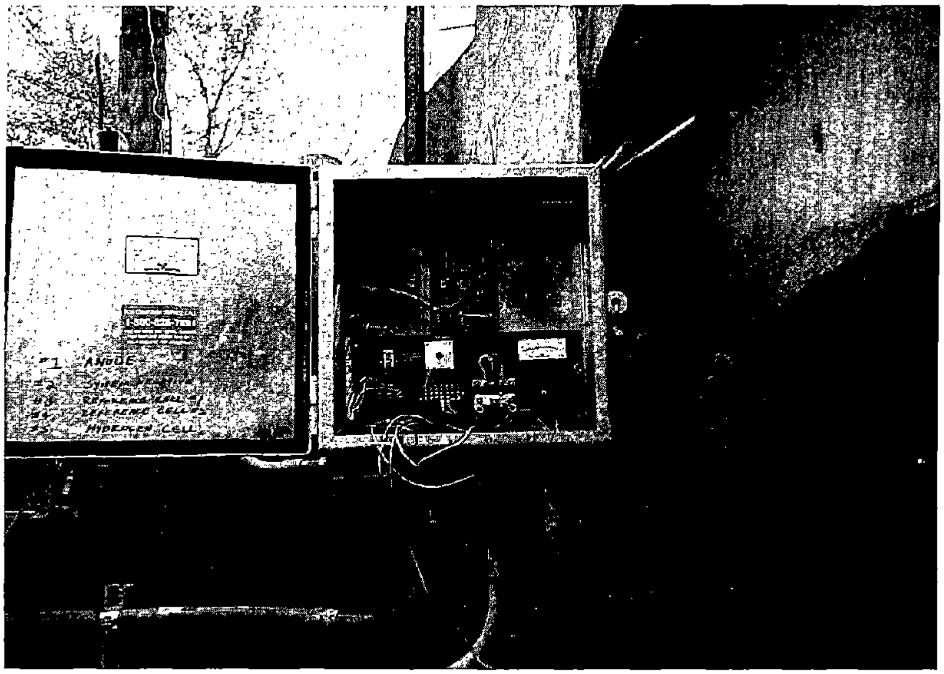

Figure 37. Partially wired rectifier on abutment wingwall.

Again, all prestressed strands were discontinuous. The procedures used at the Abbey Road installation for continuity bonding, as well as for system negative and reference ground connections, were also used at this installation. All excavations were then filled with grout.

Placement and system wiring for system negatives, reference electrodes, and hydrogen cells were done in the same manner as at the Abbey Road installation. An installed hydrogen probe is shown in figure 38. The connections between the anode conductor bars and the system wiring were made using butt connectors and were then insulated with heat-shrink tubing and nonconductive epoxy. Partially completed system wiring is shown in figure 39.

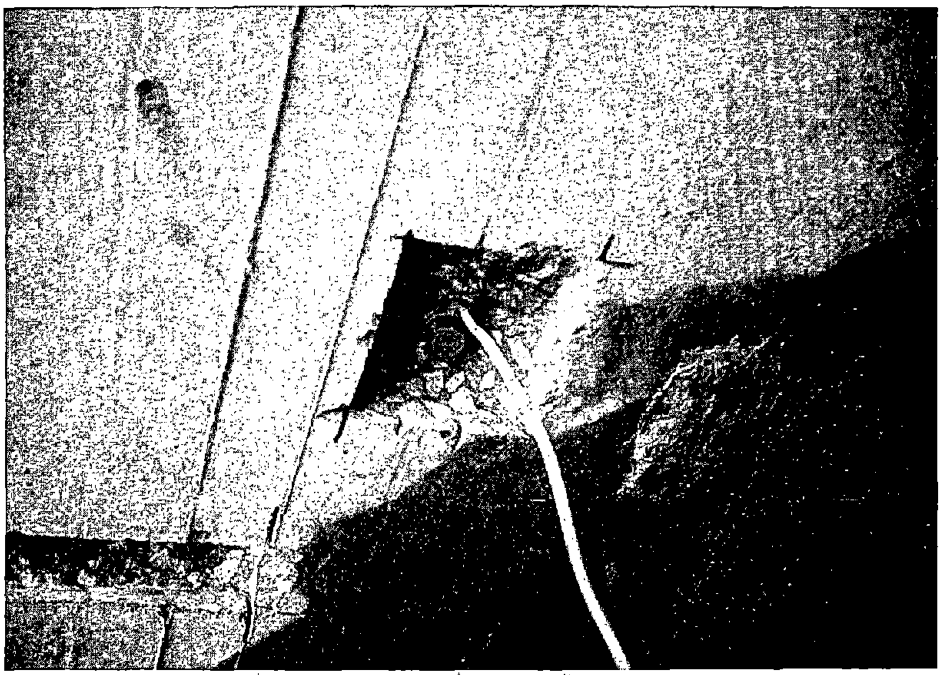

Figure 38. Installation of hydrogen probe near reference cell installed in lower left corner.

Figure 39. Partially completed system wiring after coating was completed.

# CHAPTER 5. SYSTEM START-UP

## HOWARD FRANKLAND BRIDGE

### Status Prior to Start-Up

Data taken prior to start-up is presented in table 1. All reference-to-ground and anode-to-steel resistances were acceptable, although the anode-to-steel resistances for zones 1 and 2 were higher than expected. Anode-to-steel and embedded reference cell-to-ground potentials were reasonable. Steel static potentials indicated little or no active corrosion on the beams (zones 1 and 2) at the time of measurement, and significant active corrosion occurring on the piles (zones 3 and 4). The hydrogen probe readings showed an absence of hydrogen.

Table 1. Data Prior to Start-Up of Howard Frankland Bridge.

Area of Zones:

- Zone 1, arc-sprayed zinc beams = 364 $m^2$ (3920 $ft^2$)
- Zone 2, arc-sprayed zinc beams = 364 $m^2$ (3920 $ft^2$)
- Zone 3, titanium mesh grouted in pile jackets = 62.5 $m^2$ (672 $ft^2$)
- Zone 4, conductive rubber in pile jackets = 20.8 $m^2$ (224 $ft^2$)

| Anode-to-Steel Resistance: | Reference-to-Ground Resistance: |
|---|---|
| Zone 1 = 0.52 ohms | R1-1 = 4400 ohms |
| Zone 2 = 0.68 ohms | R1-2 = 3000 ohms |
| Zone 3 = 0.37 ohms (new pile = 2.7) | R2-1 = 2200 ohms |
| Zone 4 = 2.4 ohms | R2-2 = 11 000 ohms |

Potentials:

| | Anode-Steel | Ag/AgCl Reference 1 | Ag/AgCl Reference 2 | Hydrogen Cell |
|---|---|---|---|---|
| Zone 1: | 657 mV | -245 mV | -123 mV | 0.0 mV |
| Zone 2: | 662 mV | -208 mV | -187 mV | 12.1 mV |
| Zone 3: | -625 mV | -557 mV | -378 mV | 0.6 mV |
| Zone 4: | -547 mV | -316 mV | -530 mV | 2.5 mV |

(Note: Steel always connected to voltmeter positive.)

### E-Log I Testing

E-Log I tests were conducted on January 4 and 5, 1994, using the embedded Ag/AgCl reference electrodes, and using a saturated calomel reference electrode immersed in the seawater. All potentials reported are IR-free, having been taken in instant-off mode, 0.5 to 1.0 s after current-off. E-Log I data are presented in figures 40 through 43. Each data point was taken by increasing the current in small increments and recording instant-off potentials 2 min after current increase. Current was increased until the straight-line portion of the curve became apparent, or until cathodic steel potentials exceeded -900 mV vs. Ag/AgCl.

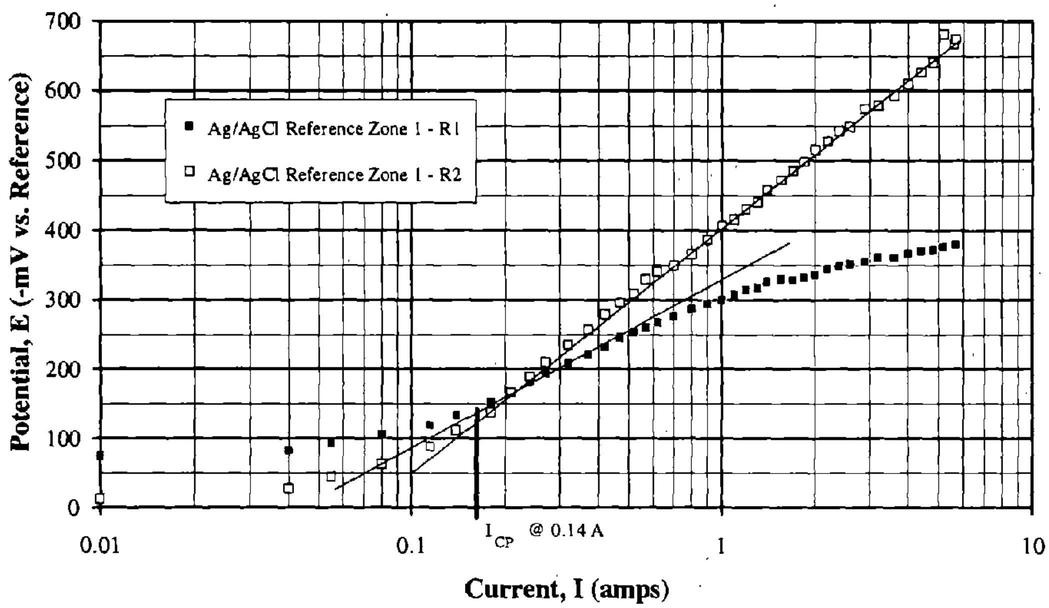


Figure 40. Zone 1 E-Log I of Howard Frankland Bridge.

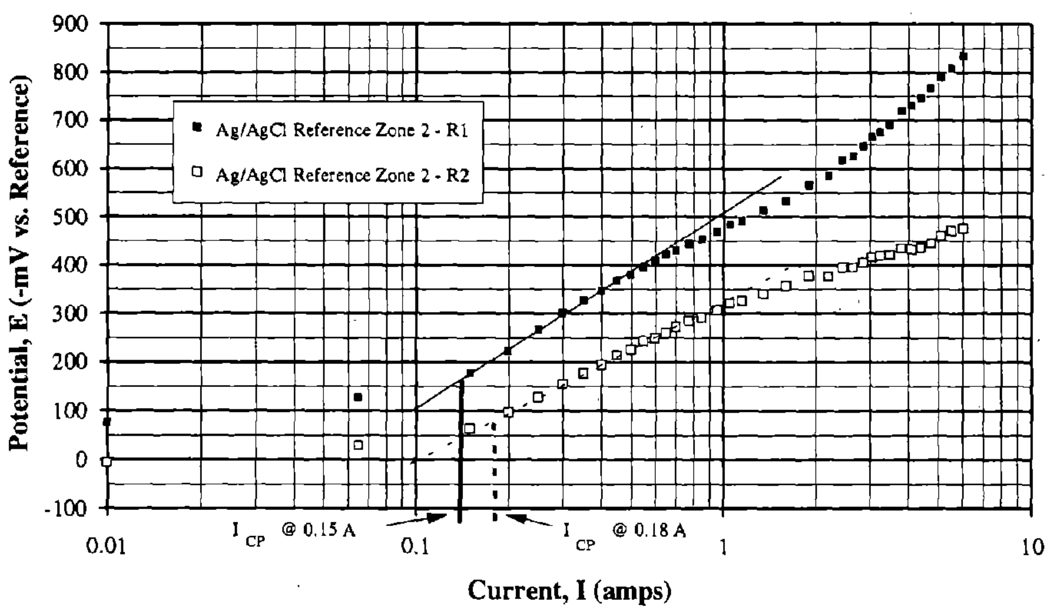


Figure 41. Zone 2 E-Log I of Howard Frankland Bridge.

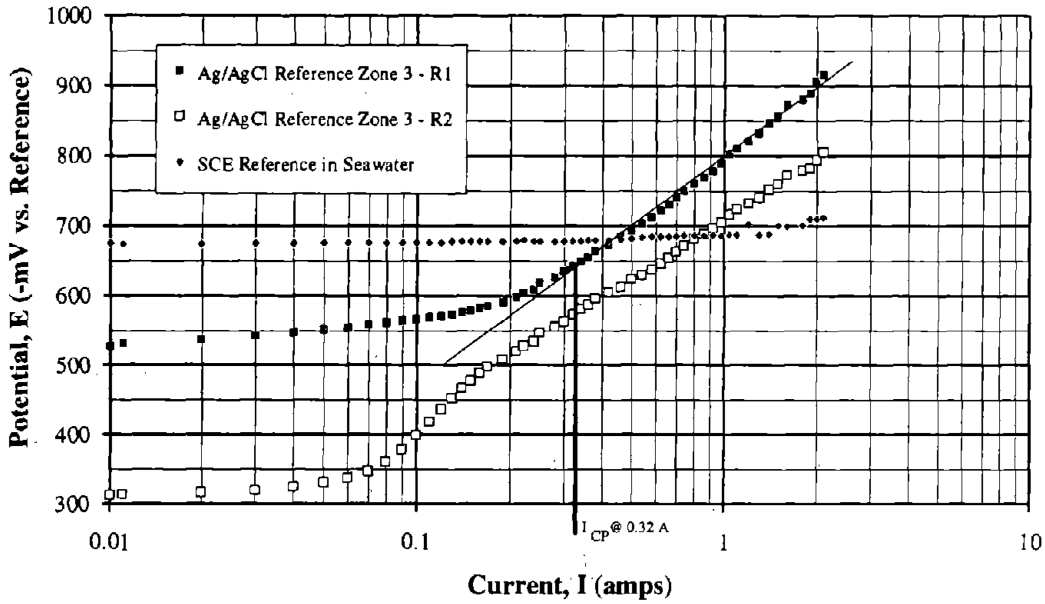


Figure 42. Zone 3 E-Log I of Howard Frankland Bridge.

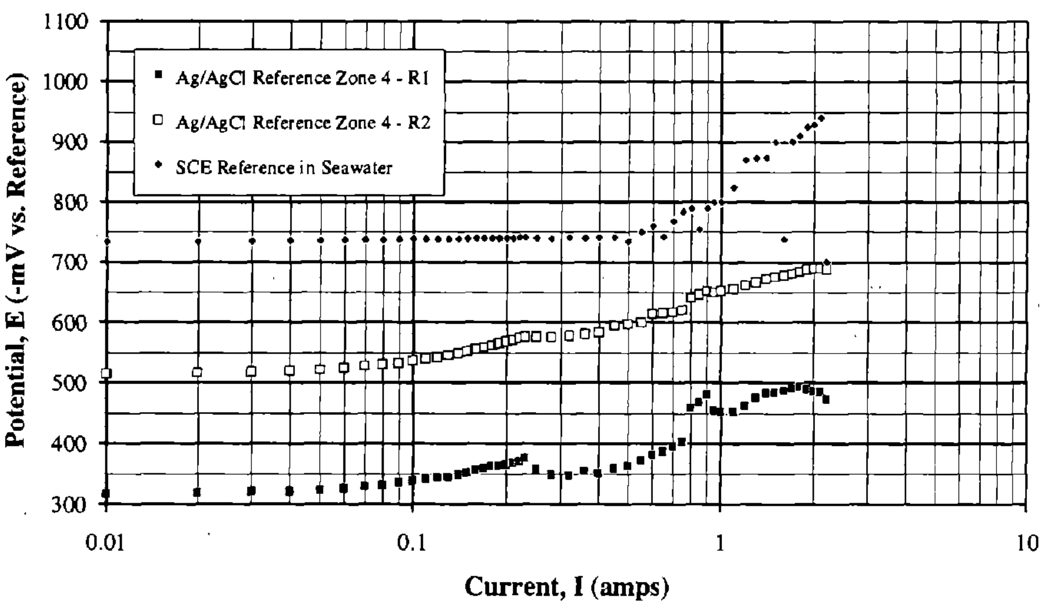


Figure 43. Zone 4 E-Log I of Howard Frankland Bridge.

The current requirements for the arc-sprayed zinc zones 1 and 2, as determined from the E-Log I curves, were very low (as shown in figures 40 and 41). Cathodic protection current ($I_{CP}$) for zone 1 was established as 0.14 A, 0.39 mA/m$^2$ (0.036 mA/ft$^2$). The cathodic protection potential ($E_{CP}$) for the steel at these points was -140 mV using R1-1 and -120 mV using R1-2. It is possible that current requirements were this low since steel potentials indicate little corrosion activity. Cathodic protection current for zone 2 is established as 0.15 to 0.18 A, 0.41 to 0.49 mA/m$^2$ (0.038 to 0.046 mA/ft$^2$). $E_{CP}$ for the steel at these points were -170 mV using R2-1 and -70 mV using R2-2. Again, current requirements were very low and steel potentials were noncorrosive.

The straight-line portion of the E-Log I curve, generated using reference electrode R3-1 of the titanium mesh pile zone 3, began at 0.32 A, 5.16 mA/m$^2$ (0.48 mA/ft$^2$), as shown in figure 42. $E_{CP}$ for the steel at this point was -640 mV. Given the corrosive state of the reinforcing steel in these piles, this seems to be a reasonable cathodic protection current. The cathodic protection potential does not risk hydrogen evolution. The E-Log I curve generated using the other embedded reference electrode of zone 3, R3-2, did not lend itself to graphical interpretation. The E-Log I curve of the steel below water level, as monitored by the reference electrode in the seawater, showed only a slight response above 1.5 A. This demonstrated that a very small amount of current was leaking from zone 3 titanium mesh anodes into the seawater.

The E-Log I curves for the conductive rubber zone 4, shown in figure 43, were not very useful. A portable Hewlett Packard power supply was used to generate the first portion of the curve since current requirements were very low. When power was switched to the installed rectifier at 0.25 A, a serious discontinuity occurred. Other unexplained discontinuities also occurred at higher currents. No meaningful cathodic protection currents or potentials could be determined from these curves. It did appear that the steel was not polarized 100 mV above static potentials until over 0.67 A, 32 mA/m$^2$ (3 mA/ft$^2$) of current had been applied. The potential of the steel in the piles below water level did show a significant response above 0.50 A. At a current of 1.5 A or 72 mA/m$^2$ (6.7 mA/ft$^2$), cathodic potentials of the steel exceeded -900 mV vs. SCE. This shows a tendency for the conductive rubber anode to leak current to the portion of the piles beneath the seawater. This result was not unexpected and it has been observed before by Florida DOT. It is expected that this tendency to leak current will decrease as the steel below water becomes polarized. This should result in better polarization of the steel above water level at that time.

The response of the hydrogen cells was also monitored during the E-Log I testing. For zones 1, 3, and 4, hydrogen cell response was minimal throughout the test. For the hydrogen cell in zone 2, a significant response was recorded. At a zone current of 6.0 A or 16.5 mA/m$^2$ (1.53 mA/ft$^2$), the hydrogen probe in zone 2 read 48.0 mV. At this point, the two reference cells recorded steel potentials of -833 and -476 mV vs. Ag/AgCl, too positive to give rise to hydrogen evolution. Indeed, the hydrogen cell first began responding at a current of 0.2 A when steel potentials were only -223 and -97 mV vs. Ag/AgCl. It appears at this time that a response from this hydrogen cell is not a true indicator for atomic hydrogen generation. Shortly after start-up, other hydrogen cells also appeared to show false responses. This issue will be examined during Task D, "Follow-Up Study," of this contract.

### Initial Settings

It appeared that operation of zones 1 and 2 in sacrificial mode would supply enough current to protect the steel in the beams. When a power supply voltage of 0.0 V was recorded (when the zone was operating sacrificially), zone 1 was supplying 0.60 A or 1.6 mA/m$^2$ (0.15 mA/ft$^2$) and zone 2 was supplying 0.50 A or 1.4 mA/m$^2$ (0.13 mA/ft$^2$). Both of these currents exceed the cathodic protection current as determined from the E-Log I curves. For this reason, it was decided to start-up both zones 1 and 2 sacrificially. This allowed remote monitoring of the current with the RMU, but does not allow monitoring of IR-free potentials. It will be determined if another method of wiring will allow monitoring of the instant-off potentials as well. Initial data for zones 1 and 2 operating in sacrificial mode are shown in table 2.

Table 2. Start-Up Data for Zones 1 and 2 in Sacrificial Mode at Howard Frankland Bridge.

| | Zone 1 | Zone 2 |
|---|---|---|
| Current | 0.76 A | 0.41 A |
| Hydrogen Cell | 0.0 mV | 19.2 mV |
| Reference 1 (Ag/AgCl) | -421 mV (on) | -500 mV (on) |
| Reference 2 (Ag/AgCl) | -463 mV (on) | -399 mV (on) |

Based on the E-Log I data, it was decided to start-up zone 3 at about 0.35 A or 5.6 mA/m$^2$ (0.52 mA/ft$^2$). This current was established at 1.4 V, and the zone was left in constant voltage mode at that voltage. Readings taken the next morning showed the IR-free potentials for zone 3 to be 110 and 132 mV more negative than their static potentials. Based on these readings, zone 3 was left in constant voltage control mode at 1.4 V.

The E-Log I data were of little help in establishing start-up conditions for zone 4. But since zone 4 current seemed to be easily polarizing steel below water level, it was decided to be more conservative than for zone 3. Based mainly on previous Florida DOT experience, zone 4 was started up in constant voltage mode at 1.0 V. Readings taken the next morning showed IR-free potentials of steel in zone 4 to be almost the same as the original static potentials. Polarization of steel adjacent to the anode was expected to increase with time. Early data also showed that zone 4 was not controlling constant voltage as intended. Although left in constant voltage mode at 1.0 V, zone voltage ranged from 1.1 to 2.0 V. This problem will be studied further under Task D of this contract.

## ABBEY ROAD BRIDGE

### Status Prior to Start-Up

Data taken prior to start-up are presented in table 3. In addition to the embedded Ag/AgCl reference cells, potential wells were installed in two beams near leaking joints. This was done to enable potential measurement of steel in wet areas where current density is likely to be highest. The potential well in beam 4 is labeled "W-Well," and the one in beam 9 is labeled "E-Well." All potentials recorded at the potential wells, as well as potentials taken from the concrete surface, were taken using a portable saturated calomel reference electrode. All anode-to-steel and embedded reference-to-ground resistances were considered acceptable, although the reference-to-ground resistance for RC-2 was higher than normal. This is not expected to impact the results or conclusions of the test. Static potentials indicate that active corrosion was occurring at the steel at the time of the measurement.

Table 3. Data Prior to Start-Up of Abbey Road Bridge.

Area of Zone: 106 $m^2$ (1140 $ft^2$)

| Anode-to-Steel Resistance: | Reference-to-Ground Resistance |
|---|---|
| 0.26 ohms | RC-1 = 2000 ohms |
| | RC-2 = 15 000 ohms |

Potentials:

| | |
|---|---|
| RC-1 | = -270 mV vs. Ag/AgCl (-364 mV vs. CSE) |
| RC-2 | = -273 mV vs. Ag/AgCl (-367 mV vs. CSE) |
| W-Well | = -370 mV vs. SCE (-444 mV vs. CSE) |
| E-Well | = -582 mV vs. SCE (-656 mV vs. CSE) |

E-Log I Testing

E-Log I tests were conducted on April 26, 1994, using the embedded Ag/AgCl reference electrodes and portable saturated calomel reference electrodes in the potential wells. All potentials are reported as IR-free potentials, and were calculated by measuring the resistance of each reference cell potential by instant-off technique (0.5 to 1.0 s after current-off) at the highest current value. These resistances were then used to calculate the IR components at each current setting, which were then subtracted from the current-on potential to obtain the IR-free potential. Each data point was taken by increasing the current in small increments and recording the potential 2 min after current increase. Current was increased until the straight-line portion of the curve became apparent.

All four E-Log I curves, shown in figure 44, exhibit a straight-line portion. The point at which the line first deviated from straight-line behavior was noted and was recorded as the desired cathodic protection current ($I_{CP}$). The cathodic protection current ranged from 1.15 to 1.50 A or 10.8 to 14.0 mA/$m^2$ (1.00 to 1.30 mA/$ft^2$). This was considered reasonable for this structure. Cathodic protection potentials ranged from -380 to -830 mV vs. Ag/AgCl (-474 to -904 mV vs. CSE). Although the $E_{CP}$ of steel at the east potential well is very negative, it is still well short of the potential needed for generation of atomic hydrogen in concrete (-1050 mV vs. CSE). Steel potential at this well will be closely monitored in the future.

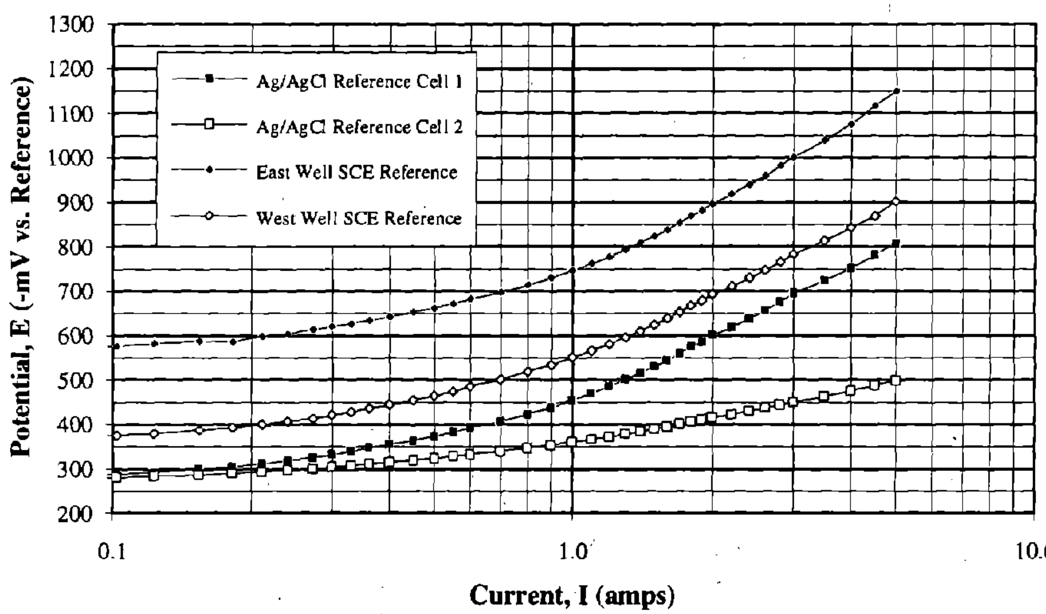


Figure 44. E-Log I of Abbey Road bridge.

Initial Start-Up in Impressed Current Mode

Based on the E-Log I test results, it was decided to start up the system in constant voltage mode at a voltage that would result in a current of 10.8 mA/m$^2$ (1 mA/ft$^2$). This proved to be impossible since system voltage was only 0.227 V, and the rectifier was incapable of control at such a low voltage. The very low rectifier voltage was a result of the additional electromotive force supplied by the zinc anode.

Since the system could not be controlled in constant voltage mode, the system was energized in constant current mode at 1.14 A or 10.8 mA/m$^2$ (1.0 mA/ft$^2$) at 2:00 p.m., April 25, 1994. Table 4 shows data taken at 2:15 p.m., 15 min after start-up.

Table 4. Impressed Current Start-Up Data for Abbey Road Bridge.

| | |
|---|---|
| System Voltage | = 0.227 V |
| Current | = 1140 mA |
| On-Potential RC-1 | = -637 mV vs. Ag/AgCl (-731 mV vs. CSE) |
| On-Potential RC-2 | = -436 mV vs. Ag/AgCl (-530 mV vs. CSE) |
| On-Potential W-Well | = -658 mV vs. SCE (-732 mV vs. CSE) |
| On-Potential E-Well | = -928 mV vs. SCE (-1002 mV vs. CSE) |
| Hydrogen Cell | = 457 mV |

The data taken from the hydrogen cell were erratic, showing significant values at potentials too positive to result in generation of atomic hydrogen. This is the same result noted during start-up testing of systems on the Howard Frankland Bridge in Tampa, Florida. There is no immediate explanation for the apparent failure of the hydrogen cells in these systems.

Start-Up in Sacrificial Mode

After further consideration, it was decided that operation of this system in constant current mode was not good practice at that time. Since the structure was subject to uneven wetting because of leaking joints between the box beams, it was considered that this could lead to high localized current, which might result in potentials sufficiently negative to generate hydrogen at the prestressing steel. Because of this concern, the system was shut off at 10:00 a.m. on April 26, 1994.

It was then decided to restart the system in sacrificial mode. It was felt that operation in sacrificial mode would limit the potentials on the protected prestressing steel and minimize the concern for hydrogen embrittlement. The mode of operation can be changed in the future, if so desired, after additional experience is gained on the operation of this system. Table 5 shows data taken during start-up in sacrificial mode.

Table 5. Sacrificial Mode Start-Up Potentials of Abbey Road Bridge.

| | Static Potentials | 30 min After Start-Up* | 1 h After Start-Up* | Instant-Off 1 h After Start-Up |
|---|---|---|---|---|
| RC-1 | -379 mV | -500 mV | -510 mV | -473 mV |
| RC-2 | -325 mV | -384 mV | -385 mV | -367 mV |
| W-Well | -455 mV | -530 mV | - | - |
| E-Well | -736 mV | -837 mV | - | - |
| H-Cell Reading | | 507 mV | 506 mV | |

* Current = 0.56 A or 5.27 mA/m$^2$ (0.49 mA/ft$^2$)

Depolarization Testing

Depolarization testing of the system on Abbey Road bridge was conducted on April 29, 1994, after about 3 days of operation in sacrificial mode. Testing was conducted at the two embedded reference electrodes, the two potential wells, and at windows in the zinc anode at the south end of each beam. Current-on and current-off potentials were first taken at each location. Current-off potential was taken by very briefly shutting current off, noting IR-free potential, turning current back on, and allowing the steel to repolarize before taking the next current-off potential. The 4-h depolarization data are presented in table 6:

Table 6. The 4-h Depolarization Data for the Abbey Road Bridge. (Potentials for RC-1 and RC-2 are -mV vs. Ag/AgCl; potentials for all others are -mV vs. SCE)

| Cell | On | Off | 15 min | 30 min | 1 h | 2 h | 4 h | Depol. |
|---|---|---|---|---|---|---|---|---|
| **RC-1** | 580 | 523 | 436 | 420 | 399 | 382 | 375 | 148 mV |
| **RC-2** | 516 | 474 | 418 | 408 | 397 | 377 | 358 | 116 mV |
| **W-Well** | 813 | 780 | 707 | 686 | 663 | 643 | 634 | 146 mV |
| **E-Well** | 896 | 818 | 759 | 737 | 694 | 697 | 708 | 110 mV |
| **Beam 1** | 348 | 318 | 260 | 236 | 203 | 175 | 131 | 187 mV |
| **Beam 2** | 327 | 293 | 237 | 229 | 212 | 189 | 177 | 116 mV |
| **Beam 3** | 607 | 569 | 502 | 485 | 447 | 396 | 363 | 206 mV |
| **Beam 4** | 349 | 332 | 291 | 286 | 269 | 249 | 231 | 101 mV |
| **Beam 5** | 600 | 520 | 431 | 403 | 381 | 337 | 310 | 210 mV |
| **Beam 6** | 825 | 722 | 623 | 606 | 583 | 549 | 420 | 232 mV |
| **Beam 7** | 612 | 549 | 454 | 434 | 402 | 368 | 332 | 217 mV |
| **Beam 8** | 490 | 443 | 353 | 338 | 315 | 297 | 278 | 165 mV |
| **Beam 9** | 683 | 608 | 523 | 497 | 490 | 480 | 429 | 179 mV |
| **Beam 10** | 282 | 252 | 199 | 191 | 165 | 141 | 114 | 138 mV |

Final values for the 4-h depolarization were greater than 100 mV at all 14 test points. The average depolarization was 162 mV. Based on the 100-mV National Association of Corrosion Engineers (NACE) criterion as stated in Standard Recommended Practice RP0290-90, the steel was being adequately protected at the time of the test. None of the IR-free potentials recorded were sufficiently negative to permit the generation of hydrogen.

It is expected that the current supplied from zinc operating in sacrificial mode will decrease with time. This system will be carefully monitored to determine when, or if, the flow of current becomes insufficient to meet accepted protection criteria.

Another major objective will be to monitor the reaction of the system to environmental changes. Since surface-applied zinc is very subject to abrupt changes in temperature and moisture content, this may have an impact on the ability of the system to properly and safely provide cathodic protection to prestressed steel. For example, rainwater leaking through the joints may cause localized concentrations of current and very negative potentials in those areas. The consequences of an event of this type may depend, in part, on the mode of operation. These issues will be studied further under Task D of this contract.

## WEST 130th STREET BRIDGE

### Status Prior to Start-Up

Data taken prior to start-up are presented in table 7. In addition to the embedded Ag/AgCl chloride reference cells, potential wells were installed in two beams at areas of significant wetness where current density is likely to be highest. The potential well in beam 2 is labeled "2W" and located near a scupper. The well in beam 12 is labeled "12W" and is located at the center feed wire.

Two more potential wells were installed in dry areas of beam 11, one (labeled "11N") near the center feed wire, and one (labeled "11F") far from the center feed wire, where voltage drop through the anode is greatest. All potentials recorded at the potential wells, as well as potentials taken from the concrete surface, were taken using a portable saturated calomel reference electrode. All anode-to-steel and embedded reference-to-ground resistances were acceptable. The static potentials taken with the embedded reference cells and the concrete surface in wet areas indicated that active corrosion was occurring at the time of the measurement.

Table 7. Data Prior to Start-Up of West 130th Street Bridge.

Area of Zone:
223 $m^2$ (2400 $ft^2$)

| Anode-to-Steel Resistance: | Reference-to-Ground Resistance: |
|---|---|
| 0.57 ohms | RC-1 = 3700 ohms |
| | RC-2 = 3100 ohms |

Potentials:

| | |
|---|---|
| RC-1 | = -371 mV vs. Ag/AgCl (-465 mV vs. CSE) |
| RC-2 | = -356 mV vs. Ag/AgCl (-450 mV vs. CSE) |
| 11N | = -146 mV vs. SCE (-220 mV vs. CSE) |
| 11F | = -114 mV vs. SCE (-188 mV vs. CSE) |
| 12W | = -244 mV vs. SCE (-318 mV vs. CSE) |
| 2W | = -330 mV vs. SCE (-404 mV vs. CSE) |

### E-Log I Testing

It is customary to conduct E-Log I tests on a cathodic protection system to determine the desired cathodic protection current. However, this system utilized a PermaRock conductive paint anode, and the manufacturer's recommendations state that the current is to be gradually increased to the desired current over several weeks. High currents needed for E-Log I testing could damage the anode. Therefore, the desired current was determined by monitoring reference cell potentials and confirming sufficient polarization with depolarization results.

Incremental Impressed Current Start-Up

Since the box beams contained prestressed steel, it was essential to control corrosion with sufficient cathodic polarization of the steel and still preclude evolution of atomic hydrogen and hydrogen embrittlement. The embedded silver/silver chloride reference cells were monitored during the current increase procedure to set a high current limit. Hydrogen evolution potentials are reached when the instant-off potential readings from these cells are approximately -961 mV vs. Ag/AgCl (-981 mV vs. SCE, -1055 mV vs. CSE).

The rectifier was incapable of operating at constant voltage control at a low system voltage, so the system was energized in constant current mode at 0.48 A or 2.15 $mA/m^2$ (0.2 $mA/ft^2$) at 11:00 a.m., May 26, 1994. After 13 days, on June 8, 1994, the system current was increased to 0.96 A or 4.3 $mA/m^2$ (0.4 $mA/ft^2$). On June 15, 1994, the system current was again increased to 1.44 A or 6.45 $mA/m^2$ (0.6 $mA/ft^2$). The operating data at these current densities are shown in table 8.

Table 8. Incremental Start-Up Data for West 130th Street Bridge.

| | 2.15 $mA/m^2$ (0.2 $mA/ft^2$) | 4.3 $mA/m^2$ (0.4 $mA/ft^2$) | 6.45 $mA/m^2$ (0.6 $mA/ft^2$) |
|---|---|---|---|
| System Voltage (V) | 1.66 | 2.05 | 2.30 |
| Current (A) | 0.48 | 0.96 | 1.44 |
| Hydrogen Cell (mV) | 1.7 | 0.0 | 0.0 |
| Potential RC-1 (on) mV vs. Ag/AgCl | -594 | -721 | -840 |
| Potential RC-2 (on) mV vs. Ag/AgCl | -600 | -778 | -930 |

The on-potentials of the embedded reference cells, as well as those potentials taken from potential wells and through windows in the paint anode, were unusually negative on August 4, 1994. This was not expected based on the initial reference cell readings taken on June 15 at the 6.45-$mA/m^2$ (0.6-$mA/ft^2$) current density. It is possible that the low steel density of 0.42 $m^2$ of steel surface/$m^2$ of concrete surface (0.42 $ft^2/ft^2$) could account for higher polarization at a relatively low current density. Depolarization testing was done at this point.

Depolarization Testing

Depolarization testing of the system on the West 130th Street Bridge was conducted on August 4, 1994. Testing was conducted at the two embedded reference electrodes, four potential wells, and at windows in the conductive paint anode at the south of each beam. Current-on and current-off potentials were first taken at each location. Each current-off potential was taken by briefly shutting the current off, noting IR-free potential, turning current back on, and allowing the steel to repolarize. The 4-h depolarization data are presented in table 9.

Table 9. The 4-h Depolarization Data for the West 130th Street Bridge.
(Potentials for RC-1 and RC-2 are -mV vs. Ag/AgCl;
potentials for all others are -mV vs. SCE)

| Cell | On | Off | 15 min | 30 min | 1 h | 2 h | 4 h | Depol. |
|---|---|---|---|---|---|---|---|---|
| **RC-1** | 1297 | 1027 | 806 | 768 | 741 | 691 | 642 | 385 mV |
| **RC-2** | 942 | 789 | 670 | 626 | 593 | 542 | 510 | 279 mV |
| **2W** | 1576 | 1116 | 923 | 897 | 879 | 720 | 811 | 305 mV |
| **12W** | 1665 | 1097 | 867 | 809 | 733 | 627 | 577 | 520 mV |
| **11N** | 2360 | 1501 | 405 | 372 | 329 | 282 | 104 | 1397 mV |
| **11F** | 2670 | 1815 | 744 | 496 | 282 | 165 | 70 | 1745 mV |
| **Beam 1** | 1260 | 863 | 533 | 542 | 595 | 454 | 422 | 441 mV |
| **Beam 2** | 1520 | 1119 | 984 | 985 | 985 | 807 | 807 | 312 mV |
| **Beam 3** | 1215 | 806 | 533 | 655 | 595 | 470 | 426 | 380 mV |
| **Beam 4** | 1842 | 1237 | 683 | 833 | 796 | 626 | 386 | 851 mV |
| **Beam 5** | 1720 | 1111 | 479 | 740 | 720 | 558 | 197 | 914 mV |
| **Beam 6** | 1620 | 1039 | 611 | 798 | 755 | 639 | 241 | 798 mV |
| **Beam 7** | 1133 | 739 | 208 | 593 | 687 | 626 | 104 | 635 mV |
| **Beam 8** | 1895 | 1225 | 190 | 699 | 673 | 665 | 326 | 899 mV |
| **Beam 9** | 1415 | 905 | 454 | 575 | 547 | 535 | 155 | 750 mV |
| **Beam 10** | 1925 | 1301 | 461 | 740 | 772 | 714 | 224 | 1077 mV |
| **Beam 11** | 2250 | 1480 | 282 | 975 | 860 | 823 | 304 | 1176 mV |
| **Beam 12** | 1895 | 1226 | 443 | 782 | 714 | 558 | 218 | 1008 mV |
| **Beam 13** | 1741 | 1098 | 215 | 560 | 723 | 548 | 204 | 894 mV |
| **Beam 14** | 1625 | 1202 | 1018 | 1032 | 1018 | 1012 | 1004 | 198 mV |
| **Beam 15** | 1380 | 938 | 474 | 739 | 578 | 563 | 318 | 620 mV |

Final values for the 4-h depolarization were greater than 100 mV at all 21 test points. The average depolarization was 742 mV. Based on the 100-mV NACE criterion as stated in Standard Recommended Practice RP0290-90, the steel was being adequately protected at the time of the test. However, most IR-free, instant-off potentials were sufficiently negative to suggest hydrogen was being generated at a current density of only 6.45 mA/m$^2$ (0.6 mA/ft$^2$).

The system current density was then adjusted down to 3.2 mA/m$^2$ (0.3 mA/ft$^2$) to allow for steel protection without reaching the hydrogen evolution potential. This system will be carefully monitored to ensure that the prestressed steel potentials remain below hydrogen evolution potentials during long-term operation.

# CHAPTER 6. FUTURE PLANS

During the 2.5-year evaluation period, data will be routinely taken by the remote monitoring units at each structure. Data will be accessed and downloaded periodically for analysis. The contract also calls for site visits to be made to each structure every 7 to 8 months during the monitoring task. Site visits will include the following as a minimum:

- Visual inspection for general condition, cracks, delaminations, spalling, etc.
- Tests for anode polarization and condition.
- Bond strength measurements of anode.
- Rectifier operation.
- Behavior of hydrogen pick-up probes.
- Depolarization tests using all embedded reference electrodes. Additional depolarization tests may be conducted from the surface through potential wells. If needed, the system may be left off for long-term depolarization before re-energizing.
- Voltage or current level adjustment to operate the system below hydrogen evolution potentials.
- Brief State personnel on all tests conducted and data collected.

At the end of the evaluation period during the final site visit, samples of concrete and prestressing steel will be taken for laboratory analysis.

There are four major objectives of the follow-up study phase of this contract, and these are discussed separately below.

## EVALUATION OF COMPONENT RELIABILITY

All system components will be evaluated for performance and reliability. These components include anodes, embedded reference electrodes, hydrogen probes, rectifiers, and remote monitoring units. Attention will focus, in particular, on the performance of these elements with regard to the cathodic protection of prestressing steel. Results and conclusions will be presented in the final report. There are some preliminary concerns about the accuracy of the remote monitoring units and the durability of the conductive paint anode.

## CP CONTROL STRATEGY FOR COMPONENTS CONTAINING PRESTRESSED STEEL

It is important to control cathodic protection systems in such a way as to minimize the possibility of hydrogen evolution in components containing prestressed steel. The two modes of control that have been recommended for this purpose are a maximum operating voltage (constant voltage) technique and a constant current with potential limit technique.[2,11]

It will be a major objective of this contract to observe the performance of the installed CP systems and to relate this performance to the mode of control. Fluctuations of potential will be observed as environmental conditions change. Potential variations will also be recorded as a function of disparity in conductivity throughout the structure, a result of uneven wetting or chloride concentration. The control mode best suited for cathodic protection of prestressed components will be recommended at the conclusion of the contract.

## BOND STRENGTH BETWEEN PRESTRESSING STEEL AND CONCRETE

Some previous studies have indicated a slight decrease in bond strength between steel and concrete with increasing ampere-hours of applied direct current, presumably arising from interface modification due to migration of ions within the concrete associated with flow of direct current, and/or accumulation of reaction products (especially acid).[12]

Previous studies that have indicated a decrease in bond strength were accelerated. That is, the studies were conducted at levels of current and total charge higher than those typically used in the field for cathodic protection of concrete structures. Under Strategic Highway Research Program (SHRP) Contract C-102A, tests were conducted to determine the effects of both cathodic protection and electrochemical chloride removal on bond strengths of conventional deformed reinforcing steel in concrete.[13] Levels of current and total applied charge typical of cathodic protection [20 $mA/m^2$ and 500 $A\text{-}h/m^2$ (2 $mA/ft^2$ and 50 $A\text{-}h/ft^2$)] had no adverse effect on loaded-end slip, free-end slip, or ultimate bond strength. Total charge was somewhat limited by the duration of the contract. Significant reduction in stress at free-end slip was recorded at the highest current density and longest treatment time [50 $mA/m^2$ and 2000 $A\text{-}h/m^2$ (5 $mA/ft^2$ and 200 $A\text{-}h/ft^2$)] that might be reached during strong chloride removal treatments. This reduction was accompanied by black powdery deposits on the surface of the steel.

It would be very difficult, if not impossible, to directly determine the actual bond strength of prestressing steel in the structures tested under this contract. First, traditional pull-out tests are designed for use with conventional deformed rebar, not prestressing strands. Extensive development tests would have been necessary to confidently measure bond strength of the specimens tested under this contract. Second, this type of test requires a large population of samples because of the variability of this test. Large numbers of samples of prestressing steel cannot be taken from field structures due to structural consequences.

We will therefore be using an indirect method of examining for loss of bond strength. As a result of the SHRP studies described above, we have found that concurrent with a loss of bond strength, certain observable changes take place in the concrete adjacent to the protected steel. We have learned that these changes, when observed, offer evidence of bond loss. These changes include softening of the cement paste and certain aggregates (measured by application of a known force to a pyramidal diamond tip followed by microscopic measurement of the resulting imprint), and an increase in porosity of the cement paste adjacent to the steel surface (measured by mercury porosimetry and SEM photomicrographs). These two tests offer indirect, but fairly conclusive, evidence for the loss of (or maintenance of) bond strength.

During the CP installation phase of this contract, four samples of prestressing strand, together with surrounding concrete, were taken from the structures. These specimens are being stored at Lankard Materials Laboratory, Inc., and will be used as control specimens. Assessments of the quality of bond will be delayed until the post-treatment samples are in hand. At that time, a side-by-side analysis will be conducted to investigate for loss of bond by studying the interface between the prestressing steel and concrete.

## HYDROGEN EMBRITTLEMENT TESTING

Many people have expressed concern that the application of cathodic protection may result in hydrogen-induced cracking or hydrogen embrittlement of prestressing steel in concrete. These concerns are discussed in the Introduction to this report.

Specimens of prestressing steel will be tested for hydrogen embrittlement after 2.5 years of cathodic protection. At least two specimens per structure will be taken for examination. It will be essential that the samples of prestressing steel be kept cold and transported as soon as possible in order that hydrogen does not permeate out before the samples can be tested. Hydrogen loss during transport is expected to be not more than 15 percent. Testing for hydrogen content and embrittlement will be conducted at Queens University in Kingston, Ontario. As soon as the specimens are received, the individual wires of each strand will be separated. Half of the wires will be tested electrochemically for hydrogen content. The remaining wires will be subjected to slow strain rate testing for signs of hydrogen embrittlement.

For determination of hydrogen content in the steel, the individual wires will be immersed in a 0.2 molar sodium hydroxide solution, and an anodic potential of +300 mV (versus the hydrogen electrode) will be applied to the steel to drive out and oxidize the dissolved hydrogen from the samples. The resulting electrochemical current flowing between the sample and the counter electrode will be determined and equations will be applied to calculate the original hydrogen content.

Fracture toughness testing of pre-notched samples is the preferred laboratory technique for determining hydrogen embrittlement. However, such methods cannot be used on the wires of prestressing strand because of the dimensions and geometry of the wires. Therefore, slow strain rate tests on unnotched strands will be performed to determine any loss of ductility due to the presence of hydrogen. It will, obviously, be necessary to test samples that are known not to contain any hydrogen. Since it is unlikely that samples of the same prestressing steel that have not been exposed to CP will be available, some of the strands will be heated to drive the hydrogen out prior to testing. These will then be used as control specimens.

# CHAPTER 7. INTERIM CONCLUSIONS AND RECOMMENDATIONS

1. The current contact specified for the arc-sprayed zinc was a titanium conductor bar mounted to the structure surface. Zinc was sprayed both underneath the bar and over the bar following mounting. This contact was unsatisfactory. Sprayed zinc did not adhere well to the smooth titanium bar and did not bridge the contact well. This necessitated the installation of additional conductor plates before energizing. Firmly anchored plates embedded to be flush with the exterior concrete surface would be an improvement.

2. Electrical continuity of the prestressing strands in the structures was not adequate. This required careful concrete excavation to expose the strands to establish continuity. Careful excavation is paramount in avoiding damage to, or severing of the strands. Improved strand location equipment is needed since it will not be unusual for construction records to be incomplete and lacking the details of strand location.

3. The movement of rain and snow meltwater through cracks between the prestressed boxes created major problems on two of the structures. It would be desirable to check for leaking water during the condition survey to identify any problems before CP installation. Leaking cracks must be repaired before installation of surface-applied anodes. This may be done by application of a membrane and/or by resurfacing of the deck riding surface.

4. The carbon-based conductive coating anode used on the West 130$^{th}$ Street bridge is showing signs of early disbondment after only a few weeks of operation. This type of anode system may be better suited for use on very dry surfaces of structures that do not have any water leakage problems.

5. The conductive rubber anode leaked large amounts of current to steel below seawater. As a result, the steel in the splash and tidal zone was polarized very little at system start-up. This is expected to decrease as steel below water is slowly polarized.

6. The response for the hydrogen cells used under this contract is not a reliable indicator for generation of atomic hydrogen. The cells often showed significant response at a potential too positive to evolve hydrogen. There is no explanation for this behavior at present, and this issue will be studied further in the monitoring phase of the contract.

7. Problems are being experienced with two of the three remote monitoring units. These units are not accurately transmitting the system data. This problem will also be studied and hopefully resolved in the monitoring phase of the contract.

8. Constant voltage, with a current limit, appears to be a satisfactory mode of operation for CP systems installed on prestressed concrete bridge members. This will be studied further during this contract. However, this mode of operation requires rectifiers with a maximum output voltage of not more than 12 V. The 24-V maximum output rectifiers used in this

contract were not capable of operating in constant voltage mode at less than 2 V. The rectifiers are being modified to allow constant voltage operation at low output voltage.

9. Thermally sprayed zinc anodes operating in sacrificial mode were not capable of providing sufficient current on a long-term basis to meet cathodic protection criteria. The two zones of arc-sprayed zinc on the Howard Frankland Bridge were well above seawater level [about 3 m (10 ft)], resulting in high concrete resistivity relative to that in the splash and tidal zone. The concrete on the soffit of the Abbey Road bridge also lacked sufficient conductivity. It is likely that these zones will be switched to impressed current in the future.

10. The installed cathodic protection systems are providing adequate protection to the steel without reaching potentials needed for hydrogen evolution.

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