

FINAL REPORT

**TRIAL APPLICATION OF ELECTROCHEMICAL CHLORIDE EXTRACTION
ON CONCRETE BRIDGE COMPONENTS IN VIRGINIA**

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(The opinions, findings, and conclusions expressed in this
report are those of the authors and not necessarily
those of the sponsoring agencies.)

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ABSTRACT

Electrochemical chloride extraction is a new technique for mitigating chloride-induced corrosion of the reinforcing bars in concrete bridges. To demonstrate the feasibility of applying this technique on full-sized concrete bridge members and to identify needed improvements to the technology, the technique was tried on two concrete deck spans and three concrete piers in Virginia.

For the two deck spans, an anode system consisting of felt-sandwiched catalyzed titanium mesh kept wet by an electrolyte was used. An electrical charge of 741 to 1077 A-hr/m² was applied between the anode and the steel bar for 57 or 58 days. Approximately 72 to 82 percent of the chloride ions was removed from the concrete at the depth of the first mat of steel bars.

For the three piers, an anode system consisting of wet cellulose fibers and steel or titanium mesh was used. An electrical charge of 249 to 382 A-hr/m² was applied between the anode and the steel bars for 72 to 77 days. The system for the piers appeared to be relatively less effective than that for the deck spans, removing approximately 13 to 53 percent of the chloride ions from the concrete near the steel bars. In addition, more problems were encountered with this treatment system.

No damage to the concrete attributable to the treatment was observed with either system, and it is likely that shorter treatment times would have sufficed. Based on potential surveys, the bars in the concrete piers were still passive at 4 years after their treatment. To facilitate the comparison between electrochemical chloride extraction and other corrosion control options, such as impressed-current and galvanic cathodic protection, preliminary information on the costs and projected service lives of all options is provided.

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INTRODUCTION

Since concrete is permeable, it is susceptible to the intrusion of chloride ions when exposed to deicing salts or surrounded by seawater. The chloride will eventually reach the depth of the reinforcing steel and accumulate in the surrounding concrete until reaching a concentration sufficient to initiate corrosion. This corrosion threshold is approximately 0.77 kg/m^3 , depending on the type of cement used and the concentration of the hydroxide ions in the concrete. The heterogeneous nature of concrete leads to uneven distributions of chloride ions, oxygen, and moisture across a concrete structure. This heterogeneity, in turn, creates electrochemical cells with different potentials along the surfaces of neighboring steel bars, or even individual bars, which serve to drive the corrosion reactions. Eventually, the stress exerted by the accumulating corrosion products causes the concrete in some areas to delaminate and then spall. Different stages of this deterioration process are typically found in a structure exposed to chloride intrusion.

Until recently, rehabilitating concrete bridge decks and piers involved only removing the damaged concrete and then patching. However, this practice was found to lead to the introduction of new electrochemical cells between the new chloride-free concrete in the patches and the surrounding old concrete that contained varying amounts of chloride. In turn, these new cells accelerated corrosion and damage of the old concrete, often within a few years of the repair. An understanding of this phenomenon led to the awareness that additional measures had to be taken to avoid the consequent wasteful cycles of repair and damage. Consequently, many transportation agencies adopted the practice of placing a concrete overlay over an entire deck after repairing the damaged concrete. This practice appeared to be beneficial in delaying the corrosion process by depriving the concrete of oxygen and moisture. However, since a large amount of chloride is still trapped in the overlaid concrete, steel corrosion will set in again. Regardless of the true long-term benefit of this practice, it is not applicable to concrete piers since overlays cannot be applied to this type of bridge member.

Since steel corrosion is an electrochemical process, once it occurs in a concrete structure only an electrochemical measure, such as cathodic protection (CP) or electrochemical chloride extraction (ECE), can stop it or slow it down to a significant extent. CP, which stops corrosion by cathodically polarizing the metal being protected, has long been applied in many industries. Now, it has been proven to be effective in halting the corrosion of steel bars in concrete and, thereby, in preventing untimely and costly bridge replacements. In practice, however, the effectiveness of a CP system on a bridge lasts only as long as the system is properly maintained. Although the type of electrical maintenance required is often simple and inexpensive, it is new to many bridge engineers. However, the present cost of CP systems and the prospect of additional maintenance burdens have probably dampened interest in CP. Therefore, the use of CP has been limited to major bridges and, in some states, mostly to concrete piers.

ECE is based on the principle that opposite electrical charges attract and like charges repel. The ECE process is as follows: (1) a suitable metal is placed or attached to the surface of a concrete structure, (2) an electrical field is applied between this metal and the embedded steel bars by the passage of a direct current through the concrete (as in CP) in such a manner that the bars become negatively charged and the metal becomes positively charged, and (3) the negatively charged chloride ions (Cl^-) in the concrete are drawn away from the steel bars and toward the surface of the concrete. The outward migration of the chloride ions accompanies the movement of other mobile ions in the concrete, each in the direction dictated by its electrical charge, contributing to the conduction of the electric current through the top layer of concrete. In fact, this electromigration of ions occurs in a cathodically protected concrete structure and is manifested noticeably during the first several months of operation of a new CP system by an increase in its circuit resistance.

The feasibility of this treatment for removing undesirable chloride from concrete was first confirmed in the late 1970s.^{1,2} Unfortunately, the use of unnecessarily high levels of current in those early studies (23 to 28 A/m^2 , at a constant voltage of 100 V in one study) revealed potential adverse effects, such as increased permeability in the concrete, a decreased concrete-to-steel bond, and cracking in the concrete. Concerns for these effects halted research on the application of ECE until the Strategic Highway Research Program (SHRP) funded studies to address them. These studies, conducted on concrete specimens and small sections of several concrete bridge members, found that by maintaining the applied current to less than 5A/m^2 , the treatment would have no adverse effect on the concrete.³⁻⁵ Further, the treatment removed 20 to 50 percent of the admixed chloride from the test concrete slabs and redistributed the remaining chloride well away from the steel bars.

The encouraging results from the SHRP studies rekindled interest in the use of ECE as a rehabilitation method. Unlike CP, it involves no anode materials or electrical components that must be regularly maintained after the treatment is completed. However, questions about ECE remain. One concerns the duration of the protection the treatment provides a concrete structure, which can be determined only by long-term monitoring of treated concrete. When this investigation was initiated, the half-cell potential and corrosion rate data for the treated section of a concrete pier in a Canadian experiment indicated that the steel bars were still passive after 6 years (Ip et al., unpublished data). Results from an ongoing follow-up investigation of the

concrete treated in the SHRP studies show the same long-term passivation of the steel bars (Islam et al., unpublished data).

Since these studies involved only concrete specimens and very small and isolated sections of several bridges, a question arises as to the practicality of treating entire concrete bridge members, such as deck spans and piers. Although ECE is similar to CP, which has been demonstrated to be feasible on full-sized concrete bridge members, with ECE, all concrete surfaces must be kept in contact with a liquid electrolyte during the several-week duration of the treatment. This may be difficult to achieve with concrete piers, which have a relatively high proportion of vertical surfaces.

In addition to the investigation of ECE, a new concept of providing long-term synergistic benefits to a reinforced concrete by combining ECE treatment with electrochemical injection of a corrosion inhibitor was explored in a separate SHRP study. Using test concrete slabs, it was shown that corrosion-inhibiting cations such as quaternary phosphoniums and ammoniums can be electrochemically injected into concrete, although at different rates.⁶ The potential of this concept warrants investigation into its feasibility with full-sized concrete members.

PURPOSE AND SCOPE

This investigation was conducted to fulfill three objectives:

1. to demonstrate the practicality of ECE treatment of full-sized concrete bridge members, specifically deck spans and piers
2. to identify any needed improvements for conducting such treatment
3. To test the feasibility of simultaneously injecting a positively charged corrosion inhibitor into concrete and removing the negatively charged chloride ions.

Tests were conducted on three concrete deck spans and two piers in Virginia.

METHODOLOGY

The methodology for this study included (1) identifying a suitable concrete deck and concrete piers on which to conduct ECE, (2) determining the pretreatment chloride contents in the deck and piers, (3) preparing the structures, (4) constructing the ECE treatment systems, (5) treating the concrete members, and (6) determining the post-treatment chloride contents and other relevant conditions of the treated members.

Identification of Deck and Piers

The concrete deck and piers used in this investigation were selected based on several factors, the most important being (1) location, (2) size of each structure, (3) general condition of each structure, and (4) whether the structure was scheduled for rehabilitation or had just been rehabilitated. Size was considered to prevent the cost from exceeding the special funds granted by the former Office of Technology Applications of the Federal Highway Administration and to use the funds for both concrete decks and piers. Since the ECE was to be used in conjunction with normal bridge rehabilitation and the funds did not include rehabilitation, it was necessary to use only structures that had just been repaired or for which rehabilitation funds had already been allocated. One of the bridge sites was selected based on the availability of a facility to hold an open house to allow highway engineers from nearby states to observe the treatment.

Bridge Deck

The selected bridge deck, approximately 28 years old at the time of the treatment, is on the 34th Street Bridge over I-395 in Arlington (Figure 1). The reinforced concrete deck is 109 m long and 17 m wide with a curb-to-curb width of 14 m. The deck consists of five simple spans supported by steel beams. A previous inspection of the deck indicated that 2 to 29 percent of the concrete in each span was delaminated because of rebar corrosion. Two spans (4 and 5) were selected for treatment. The spans had delaminations and spalls in 2 to 5 percent of the concrete.



Figure 1. 34th Street Bridge Over I-395 in Arlington

Piers

The selected concrete piers, which were 27 years old at the time of the treatment, support the southbound lanes of 5th Street (Rt. 631) over I-64 in Charlottesville (Figure 2). The piers



Figure 2. Concrete Piers of 5th Street Bridge Over I-64 in Albemarle County

have rectangular pier caps and cylindrical columns. The cross sections of the pier caps are 1.3 m by 1.5 m, and the lengths vary from 13.9 to 16.1 m. The columns have a diameter of 1.1 m and heights (above ground) ranging from 5.6 to 6.5 m. Approximately 4 years before the study, delaminated and spalled concrete (with exposed rebars), mostly in the bottoms of the pier caps and the tops of columns, was repaired. During preparation for the ECE treatment, 1.6 m² of delaminated concrete, mostly at the bottoms of caps and tops of columns, was detected and repaired.

Arrangement for Utilities and Preparation of the Structures

Electric power and water are essential during ECE treatment. Bennet et al.⁴ reported that treating a salt-contaminated concrete area would require as long as 4 to 8 weeks, depending on the initial chloride content and the current density used. Accordingly, it was estimated that treating the three concrete piers, if done simultaneously, could take as long as 8 weeks and treating the two deck spans could take as long as 18 weeks. Therefore, arrangements were made with local public utilities to have a 220-volt AC line and water provided at each bridge site.

Preparing the structures for the ECE treatment involved the same procedures typically used in preparing a structure for application of CP. The procedures included removing delaminated concrete (to a depth slightly below the steel bars), blast cleaning the exposed corroded bars, and patching with conventional portland cement concrete. It is preferable to use conventional portland cement concrete rather than concrete mixtures containing partial mineral substitutes (such as fly ash or silica fume) because the latter yields concrete with a considerably higher electrical resistance than the existing concrete in most concrete structures.

After this procedure, the surface of the concrete was inspected for cracks. Since cracks may extend well into the concrete and even close to the reinforcing steel, cracks were covered

carefully with a cementitious grout to avoid any electrical short that would jeopardize the treatment.

Finally, the electrical continuity between the steel bars was ensured so that no area in the structure would be left untreated. This was performed by exposing the steel bars at the locations intended to serve as the ground connections for each span or pier and measuring (with a multimeter) the resistance between each pair of all possible pairings between the exposed steel locations. For each span, the steel bars at four intended ground connection points and eight additional points were exposed. The measured resistances ranged from 0.3 to 0.7 ohm for span 4 and 0.3 to 1.0 ohm for span 5 and were considered indications that the steel was continuous. Similarly, the steel bars at three intended ground connection points in each pier were exposed and tested. The measured resistances ranged from 0.1 to 0.3 ohm for piers 1 and 2 and 0.1 to 0.2 ohm for pier 3. Likewise, the electrical continuity between the exposed steel bars was considered adequate. Had it not been, a copper wire of adequate gage (No. 10 AWG) would have been welded between each isolated bar and a nearby continuous bar.

Construction of ECE Treatment Systems

Deck Spans

To minimize traffic interruption, the treatment was conducted first on the north portions of the spans (the westbound lane), with all traffic routed to the eastbound lane, and then on the southern portions of the eastbound lane (Table 1). The anode system consisted of a very inert catalyzed-titanium-mesh anode, sandwiched between two layers of synthetic felt, which was surrounded by a dam and kept wet by a pool of electrolyte (Figures 3 and 4).

TABLE 1. ECE TREATMENT OF DECK SPANS

Phase	Span	Half Treated	Area (m ²)	Anode System	Electrolyte
1	4	North	174	Felt/Ti Mesh/Felt	LiBO ₃ /LiOH, H ₂ O
	5	North	183	Felt/Ti Mesh/Felt	LiBO ₃ /LiOH, (C ₆ H ₅) ₄ PCl, H ₂ O
2	4	South	180	Felt/Ti Mesh/Felt	Ca(OH) ₂ , H ₂ O
	5	South	183	Felt/Ti Mesh/Felt	Ca(OH) ₂ , H ₂ O

The system was installed on each portion of a span in accordance with the following steps:

1. *Connect an insulated copper lead wire of adequate gage (No. 6 AWG) to each of the four designated ground connections in each span.* As illustrated in Figure 4, these connections were located close to the centerline of each span so that the same connections could be used during treatment of to the remaining half of the span. Each connection was waterproofed with epoxy.

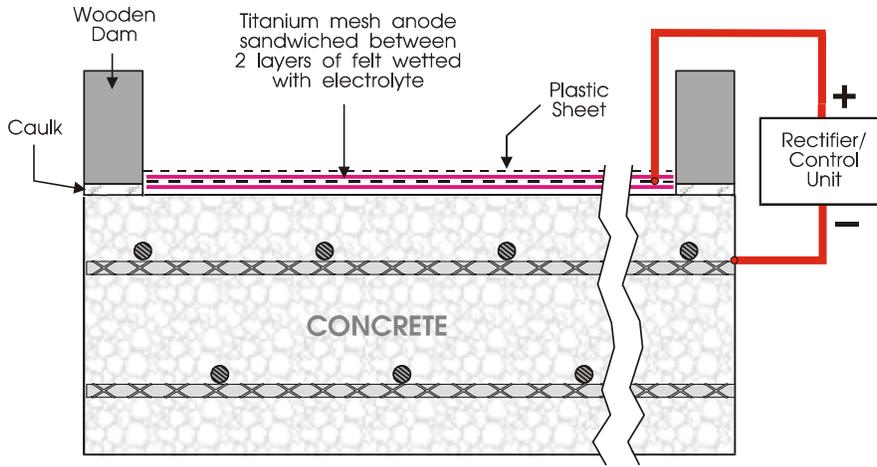


Figure 3. Anode System Used for ECE Treatment of Deck Spans

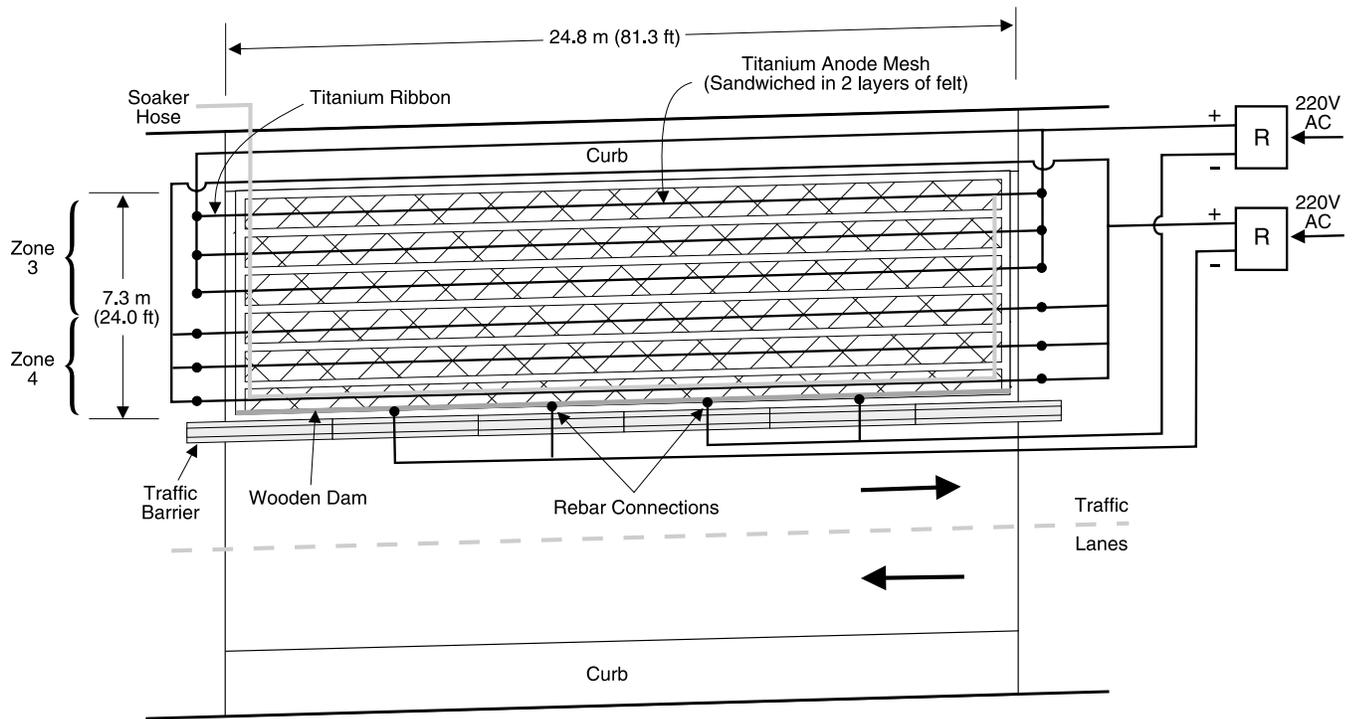


Figure 4. Layout of Setup Used for ECE Treatment of Northern Half of Span 5

2. *Construct a wooden dam around the area to be treated, stopping within 8 cm of the joints. This was accomplished by securing wood strips, 5 cm x 10 cm x 365 cm, around the perimeter of the area with anchor bolts and applying enough silicone caulk underneath and all around the base of the wood strips to seal the edges (Figure 5).*
3. *Place the first layer of 6-mm-thick synthetic felt over the entire area within the dam (Figure 6). The 1.8-m-wide felt was placed longitudinally on each span, from end to*



Figure 5. Installation of Ponding Dam with Wood Strips and Silicone Caulk



Figure 6. Placement of First Layer of Felt Within Ponding Dam on Deck Span to Be Treated

- end, before being cut to the length of the span. Adjacent pieces were allowed to overlap by 2 to 5 cm.
4. *Place a layer of titanium-mesh anode (Elgard 210) over the felt. (Any equivalent titanium anode mesh will suffice.) Similarly, place the 1.2-m-wide anode mesh longitudinally over each span, and then cut it to the length of the span. Five full-width strips and one half-width strip of the mesh were required to cover the entire 7.3-m width of each treatment area (Figure 4). The gap between adjacent strips was kept at less than 11 cm. .*
 5. *Perform electric-resistance welding of a continuous piece of 1.2-cm-wide titanium ribbon every 0.6 to 0.9 m to each strip of anode mesh (Figure 7). Each ribbon was*



Figure 7. Strips of Catalyzed Titanium Mesh Placed Over First Layer of Felt

then cut to extend approximately 0.3 m beyond each end of the anode mesh. (As described later, both ends of each ribbon will be connected to a rectifier.)

6. *Place a second layer of felt over the anode mesh.*
7. *Route a soaker water hose from the lowest corner of each treatment area along three sides of the area. A small water pump was connected to the hose to allow circulation of the electrolyte to the entire area.*
8. *Carefully introduce a sufficient amount of liquid electrolyte into each dam to cover the entire felt/mesh/felt anode system with an electrolyte. Table 1 shows the various electrolytes used in the four treated areas. For the northern halves of both spans, approximately 950 L of the electrolyte RenewTM from FMC Corporation was used. This electrolyte is an aqueous solution of 1.3 percent LiBO₃ and 1.0 percent LiOH. In the northern half of span 5, a corrosion inhibitor, tetraphenylphosphonium, (C₆H₅)₄P⁺, which was suggested by and provided free courtesy of SRI International of Menlo Park, California, was also added in the electrolyte.*
9. *Cover each treatment area with sheets of 10-mil-thick black plastic to protect the system and to minimize evaporation of the electrolytes.*

Piers

In treating the piers, traffic on I-64 (beside the piers) and the 5th Street Bridge (above the piers) was not interrupted, and only setting up standard traffic control warning signs on the shoulders of the interstate was required.

Since piers have mostly vertical surfaces and overhangs, a different anode system was used (Table 2). As illustrated in Figure 8, this consisted of an anode secured on the surface of the concrete and surrounded by wet cellulose fibers. The same titanium mesh used on the deck spans was used as the anode for the rounded ends of the pier caps and the entire portion of each column; heavy gauge steel mesh was used as the anode for the rest of each pier cap.

TABLE 2. ECE TREATMENT OF PIERS

Pier	Area (m ²)	Anode System	Electrolyte
1	177	Cellulose fiber/metal mesh	H ₂ O, Ca(OH) ₂
2	176	Cellulose fiber/metal mesh	H ₂ O, Ca(OH) ₂
3	134	Cellulose fiber/metal mesh	H ₂ O, Ca(OH) ₂

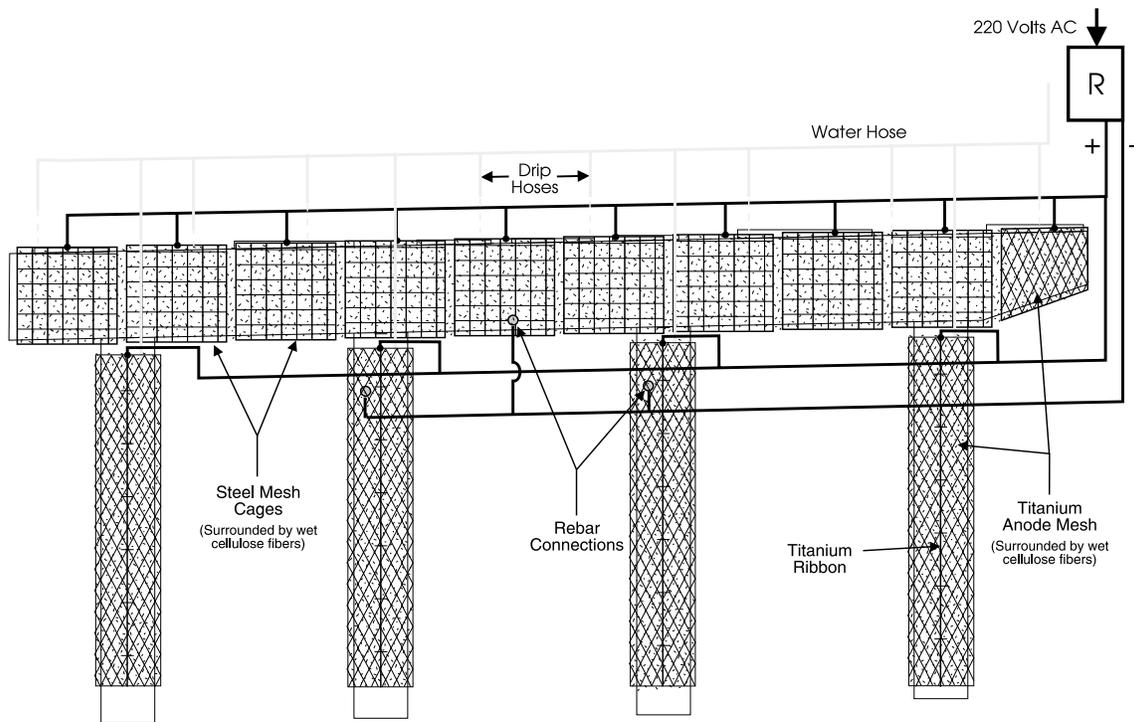


Figure 8. Anode System Used for ECE Treatment of Concrete Piers

The installation of the treatment system, which took 6 to 7 days for all three piers, proceeded on each pier in accordance with the following steps:

1. Connect an insulated copper lead wire of adequate gage (No. 6 AWG) to each of the three intended ground connections on each pier. Each connection was then waterproofed with epoxy.
2. Install wooden battens or spacers on the pier caps and column to hold the anode off the concrete surface. Along the entire length of each cap, at least two 25-mm-thick

wooden spacers were installed longitudinally along each side (except the top side). On the curved end of each cap, pieces of the wooden spacers were installed vertically over almost the entire height of these sections. Plastic anchor bolts were used to secure the wooden spacers on the concrete to prevent any electrical short during the treatment. From the top to the bottom of each column, a minimum of three vertical wooden spacers were secured on the concrete surface with plastic anchor bolts.

3. *Install the mesh anode over the wooden spacers on the pier caps, taking care not to damage the lead wires from the ground connections. Two types of mesh, titanium and 6-mm-thick steel, were used as anodes. Because the titanium mesh is more pliable than the steel mesh, it was used over the curved concrete surfaces at the rounded end of each pier cap. The same titanium mesh used on the deck spans was securely tied with plastic cable ties to the wooden spacers anchored on the concrete. Each plastic cable tie was run underneath a wooden spacer and over the mesh. For the straight section of the caps, preformed sections of the steel mesh, which had 10-cm grids, were hoisted up to the cap, wrapped around it, and tied to the cap with the plastic cable ties (Figure 9).*



Figure 9. Steel Mesh and Titanium Mesh Secured Over Straight Section and Curved End, Respectively, of Pier Cap

4. *Provide electrical connections between all sections of anode mesh on the cap and anode lead wires for later connection to a rectifier.*
5. *Cover the entire pier cap, except the bearing pads, and the columns with wet cellulose fibers by spraying the material over the concrete until the layer is at least 50 mm thick (Figure 10).*
6. *Immediately wrap the entire fiber-coated pier cap, from end to end, with a continuous sheet of black plastic to keep the fibers in place and minimize evaporation of the electrolyte (Figure 11). For the wrapping to be tight, there must be considerable*



Figure 10. Spraying of Wet Cellulose Fibers Over Pier Cap to Encapsulate Mesh Anode and Cap



Figure 11. Wrapping of Fiber-Encapsulated Pier Cap with Heavy Plastic Sheet to Keep Cellulose Fibers in Place and Wet During treatment

overlapping of the plastic between turns. The plastic sheets were then secured by banding the caps with reinforced plastic straps.

- 7. Install the anode around each pier column.* Because of the cylindrical form of the columns, it was easier to place the more pliable titanium mesh around them than the thick and rigid steel mesh. However, because the titanium mesh has diamond-shaped openings of no more than about 63 mm wide, which hinder the passage of the clumping wet cellulose fibers, a layer of this material must be applied on each column before the titanium mesh can be installed. This material was sprayed on each column, from the top to about 0.15 m above the ground or slope protection, to a thickness of approximately 25 mm. Then, the titanium mesh, which was already cut to size and had a continuous piece of 12-mm-wide titanium ribbon pre-welded to its entire length, was installed over this first layer of cellulose fibers and all around each

column. This titanium ribbon serves as the connection of the mesh to the rectifier. Finally, another 25-mm-thick layer of wet cellulose fibers was sprayed over the mesh.

8. *Connect the titanium ribbon welded to the titanium mesh on each column to an anode lead wire for later connection to the rectifier.*
9. *Immediately wrap each column tightly, from top to bottom, with a continuous piece of plastic film, such as Shrink Wrap, allowing for considerable overlapping between turns (Figure 12). Figure 13 shows an entire pier completely wrapped.*



Figure 12. Wrapping of Fiber-Encapsulated Pier Column, From Top to Bottom, with Self-Clinging Transparent Plastic Film



Figure 13. Pier After Installation of Anode System. Notice the wooden box where one of the rectifiers was housed during treatment.

10. *Install a system of drip hoses on top of each pier cap.* The drip hoses, similar to the type used in gardening, were connected through a main hose to a central water source or faucet to keep the cellulose fibers on the cap and the columns wet throughout the treatment. This main hose was used to serve all three piers. To facilitate the addition of lime to the water dripping on the piers, should it become necessary to neutralize any excessive acid generated by the titanium mesh, a mixing tank (converted from a water heater) was connected between the central water source and the common main hose.

Treatment of the Structures

In starting the ECE treatment, basic system preparations had to be made. These included completing connections between hoses to pumps, if used, or to a water source, and connecting all lead wires to proper terminals on the designated rectifiers, i.e., leads from the anodes to the positive and ground leads to the negative. For each pier, a single-phase 220 V AC rectifier with a maximum output of 150A at 40 V was used; for each deck area, two of the rectifiers were used.

After the dams built on the deck areas to be treated were checked for leaking and all electrical and water/electrolyte connections were in order, the water and pumps were turned on. Then, each rectifier was activated and operated in constant-voltage mode, with the DC output level set as high as possible without exceeding 40 V and 1 A/m² of concrete.

At the time this project was being planned, there was, and still is, no logical means to predict the duration of treatment needed for a given concrete structure to help in scheduling the entire rehabilitation project. Therefore, even at the planning stage, it was decided that each deck area and pier would be treated for approximately 8 weeks, regardless of whether the additional chloride being removed by prolonging the treatment was worthwhile or cost-effective. The decision was made to make it easy for the different contractors involved to schedule their respective jobs. For most concrete piers, treatment of such duration would not normally cause any traffic problem. However, for many bridge decks, particularly those on heavily traveled roads, even partial closure for this long (not including the time required for the repair of the concrete) might cause major inconvenience for motorists. It is likely that different structures would require different treatment times to remove the chloride that is removable. It would be beneficial to bridge engineers to be able to estimate this during the project planning stage.

During the treatment, the output voltage and current of each rectifier, the current passing through all the positive and negative lead wires, and the pH of the electrolyte were measured once every week. The dams on the deck areas were checked often for possible leakage of the electrolyte. The plastic wrapping around the pier caps and columns and the water flow to each pier was inspected regularly. During the first few weeks of the deck treatment, when the formation of acid around the titanium anodes appeared to be most pronounced, it was necessary to adjust the pH of the electrolyte by spreading lime over the treatment areas. This was also necessary with the piers; in this case, since the piers were wrapped and concealed by the plastic sheets and films, lime was added to the water in the converted water heater to raise the pH.

After the deck areas were treated, a low-permeability concrete overlay containing fly ash was applied to reduce the ingress of new chloride. In the case of the three treated piers, a water-based silane sealer was applied.

Pretreatment and Post-Treatment Tests

To facilitate an estimation of the extent of chloride removal from the deck spans and piers and to verify that ECE treatment is feasible on full-sized concrete bridge members, pulverized concrete samples were collected from each span before and after treatment and analyzed for their chloride contents. The sampling points in each span were determined by use of a sampling grid. At each point, concrete samples were taken at two depths from the surface: 0.6 to 1.9 cm and 1.9 to 3.2 cm. No samples were taken at depths below 3.2 cm because the depths of the transverse steel bars ranged from only 1.9 to 4.4 cm. To minimize the possible effect of the natural variability in the composition of concrete on the results, the pretreatment and post-treatment samples from each sampling point were collected within 1 cm of each other.

Similarly, pulverized concrete samples were taken from several designated sampling points on the cap and columns of each pier before and after treatment for chloride analyses. At each sampling point, concrete samples were taken at two depths from the surface: 0.6 to 1.9 cm and 2.5 to 3.8 cm. At a few locations on pier 1, a third sample was taken at 4.4 to 5.7 cm.

A potentiometric titration procedure described elsewhere was used to determine the total chloride contents of all concrete samples.⁷ In addition, to determine the amount of lithium and inhibitor tetraphenylphosphonium that might have migrated from the electrolytes into the concrete deck, 75 percent of the concrete samples from the deck spans, selected randomly, was subjected to capillary electrophoresis (CE) analyses.

To determine if there was any adverse effect on the concrete surface, a digital Schmidt rebound hammer was used to measure the surface hardness of the concrete at the sampling points in each span and pier before and after treatment. In conducting this test, caution was taken to avoid impacting on any visible aggregates with the hammer or plunger, which could lead to erroneous readings. In addition, several surveys of the half-cell potentials of the reinforcing steel in the three concrete piers (using a portable Cu/CuSO₄ electrode) were conducted before and a few times after treatment to help assess the long-term effect of the treatment on the steel. Since the piers were sealed with a silane after the treatment, in the post-treatment surveys, the concrete at each survey point had to be abraded first before the half-cell potential was measured. A similar survey of the treated deck spans was not possible because of the application of a concrete overlay.

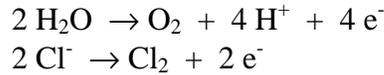
RESULTS AND DISCUSSION

Problems Encountered with Electrolytes and Treatment Systems

Decks

The wooden dams constructed over the deck areas were very effective in containing the electrolyte and keeping the concrete surface wet during treatment. However, the lack of a mechanical pumping system to circulate the electrolyte and a system to flush out and collect a portion of the electrolyte during a treatment had caused problems.

One of these problems concerned the pH of the electrolyte. Reactions that can occur at the anodes in the electrolyte include the electrolysis of water molecules to produce H^+ and the oxidation of the chloride that migrated out from the concrete to produce chlorine, or HOCl:



The first reaction is expected to be more prevalent at the early stage of the treatment. However, if the H^+ produced is allowed to accumulate, the second reaction will occur and the small amount of chlorine that evolves is rapidly hydrolyzed to make the electrolyte even more acidic:



If a portion of the electrolyte is not replaced and is not adequately neutralized or buffered, the electrolyte can become acidic at some stage of the treatment. Since the treatment system for the deck areas did not have an adequate liquid pumping system that allowed for adequate flushing of the “old” electrolyte and replenishment with fresh electrolyte, the electrolyte in some of the deck areas became very acidic during the treatment. This problem was most pronounced at about the third week of the treatment on the northern half of both deck spans despite the presence of buffer $LiBO_3$ in the electrolytes. Consequently, it was necessary to manually flush the electrolytes on the deck areas and add fresh water and lime. Because this problem was the most severe in the northern half of span 5, where the acidic inhibitor $(C_6H_5)_4P^+$ was used in the electrolyte, it appeared that this inhibitor had interfered with the buffering capacity of $LiBO_3$.

To avoid this problem during the treatment of the second (south) half of the spans, a modified setup was used. Instead of containing and recirculating the electrolyte within the dams, fresh water was allowed to flow continuously from the highest corner and drain out at the lowest corner in each span. Because of later concerns with the possible environmental impact of the lithium salts and the inhibitor draining out of the deck, lime was used instead when necessary. This considerably minimized the need to neutralize the electrolytes during the treatment of these two deck areas. However, because of a lack of a proper drainage and collection system, some of the electrolyte overflow from these deck areas dripped onto portions of the concrete piers directly below. Even though the electrolyte was not harmful, this was not desirable and pointed to the need to improve the treatment system to provide for better circulation and collection of spent electrolyte.

Piers

During the treatment of the piers, the electrolyte had also become acidic, albeit to a lesser extent. This occurred during the first several days when the pH of the electrolyte had decreased to 6.5, but never lower. This necessitated adding lime to the converted mixing tank that fed fresh tap water to the piers by gravity alone. At that time, yellowish staining of the wet cellulose fibers was observed at the bottom portions of some of the columns through the transparent plastic film wrap. It was uncertain whether this was caused by the generation of a minute amount of chlorine or rust particles coming from the steel mesh around the caps. Unfortunately, it was not possible to examine whether such staining was present in the cellulose fibers on the pier caps and whether the fibers were still adequately wet because the caps were wrapped with the black plastic sheets. This aspect of the design of the treatment system for the piers needs to be improved.

Another problem with the treatment system for the piers was sagging of the black plastic wrapping around some sections of some pier caps. The sagging resulted from an accumulation of rainwater inside the plastic, particularly near the west end of the pier caps where there was no sheltering deck overhang. Since the layer of cellulose fibers can become detached from the concrete surface if it becomes too wet and heavy, the sagging caused concern. Again, because the plastic wrapping was not transparent, it was impossible to determine the extent to which the cellulose fibers had detached from those sections of the pier caps and to apply corrective measures if necessary.

Leaking of electrolyte around the connectors between some drip hoses and the feeder hose over pier 1 was also observed. Because of the bad connectors, these leaks could not be stopped during treatment. This may have allowed some of the concrete on the cap to become dry, which would account for the relatively low chloride removal rate for that pier.

Both titanium mesh and steel mesh appeared to be suitable for use as anodes in ECE treatments. Since steel mesh is considerably more active than titanium mesh, it was expected to undergo significant mass loss during a treatment such that it would be usable only once. Examination of the steel mesh after the treatment of the piers did not show that the expected corrosion on the mesh led to any discernible loss of cross section. However, as anticipated, the steel mesh left rust stains on the concrete surface after treatment, which were removed easily by washing with a strong water jet.

Electrical Parameters During Treatments

At the beginning of the ECE treatment of each deck area and pier, the output voltages of the rectifiers were adjusted so that the total current passing through each area was less than 1 A/m². The rectifiers were then left to operate constantly at those voltages, except for two or three times during the treatment of the deck areas when the voltage settings were increased slightly.

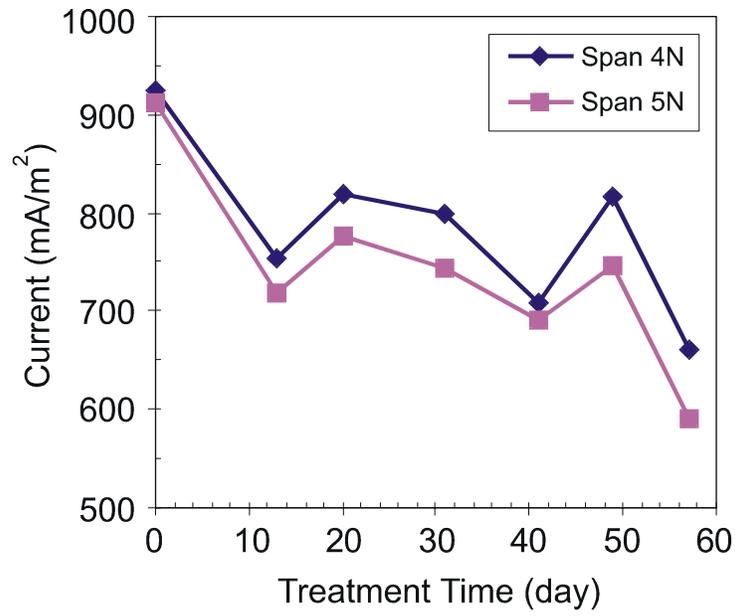
Figure 14 shows the current density passing through each deck area during treatment. As illustrated, except for occasional rises corresponding to increases in the driving voltage, the current generally decreased as the treatment progressed. This decreasing trend, which was expected in a constant-voltage mode of operation and is typically observed in CP systems operated in the same mode, resulted from an increasing trend in the effective circuit resistance of the treated areas (Figure 15). The increase in the resistance of any area during treatment is the net result of the electromigration of different ions of different charges into and out of the concrete and other auxiliary processes, such as crystallization of minerals in the pores of the concrete.

Figure 16 shows the total amount of electrical charges, in ampere-hours/square meter, estimated to have passed through each deck area at different stages of the treatment. At the end of treatment, the accumulated charges ranged from a low of 741 A-hr/m² for deck area 5S (southern half of span 5) to a high of 1,077 A-hr/m² for area 4N (northern half of span 4). This is within the range of 600 to 1500 A-hr/m² that was predicted to be typical.⁸ Table 3 summarizes the electrical parameters recorded during the treatment of the deck areas. Noticeable differences were the resistance of and the total charge passed through deck area 5S in comparison to the other areas. It is uncertain what caused the differences.

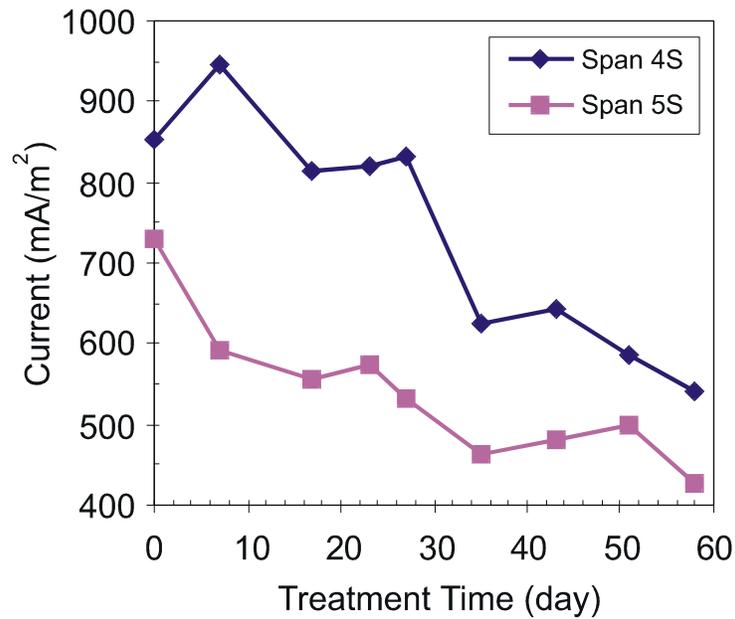
Figure 17 shows the change in the density of direct current passing through the piers during treatment. The small differences among treatment times were due only to the fact that the installation of the treatment system and the initiation of treatment were first started on pier 1, then on pier 2, and finally on pier 3. The treatments of the piers were terminated the same day. As with the deck areas, the current passing through each pier, in general, decreased with increased treatment time. Since the voltage settings on the rectifiers were kept constant more than with the deck areas, the expected exponential relationship between current and treatment time was more definitive. Figure 18 shows the changes in the total charges for these piers at different stages of treatment.

A summary of the various electrical parameters is presented in Table 3. There was a significant difference between the average electrical resistances of the deck areas and those of the concrete piers. This significant difference led to significant differences in the total amount of electrical charges that were able to pass into these concrete components, despite the fact that the piers were treated for about 2 weeks longer than the deck areas. The treatments were able to pass only 249 to 382 A-hr/m² through the three piers. A likely explanation is that the treatment system used for the piers is not as effective as that used for the deck areas in keeping the concrete reasonably wet during the treatment. This explanation is plausible, since the effective circuit resistance of any concrete area or component being treated increases with the thickness of the concrete cover over its steel bars and the resistance of that layer of concrete, the latter being considerably reduced when the concrete is kept wet.

A short was encountered during treatment startup in a circuit connected to the mesh on a section of the cap of pier 3. Because it was impossible to locate the short once the entire pier was covered and wrapped in opaque plastic film, that section of mesh was disconnected from the system. This could have affected chloride removal in that portion of pier 3, which (as discussed later) showed the lowest removal efficiency. Because electrical shorts cannot be located after the pier is wrapped, there is a need for a procedure for detecting shorts during the spraying of the

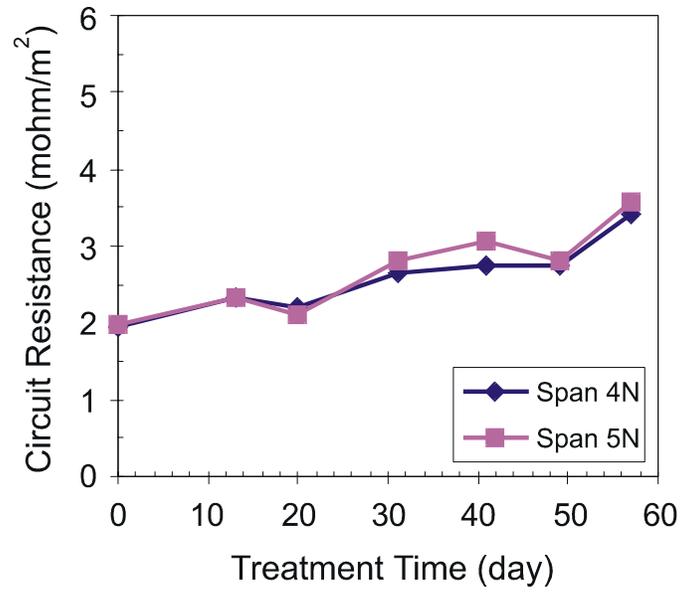


(a)

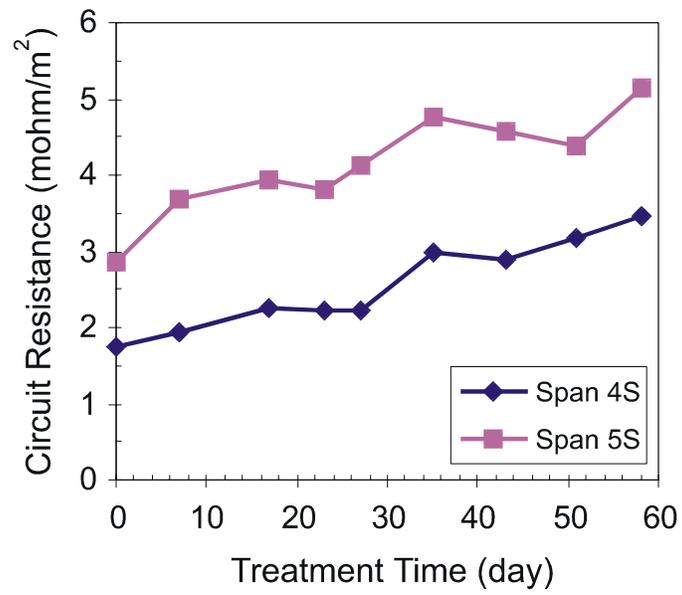


(b)

Figure 14. Treatment Currents for (a) Northern and (b) Southern Halves of Deck Spans

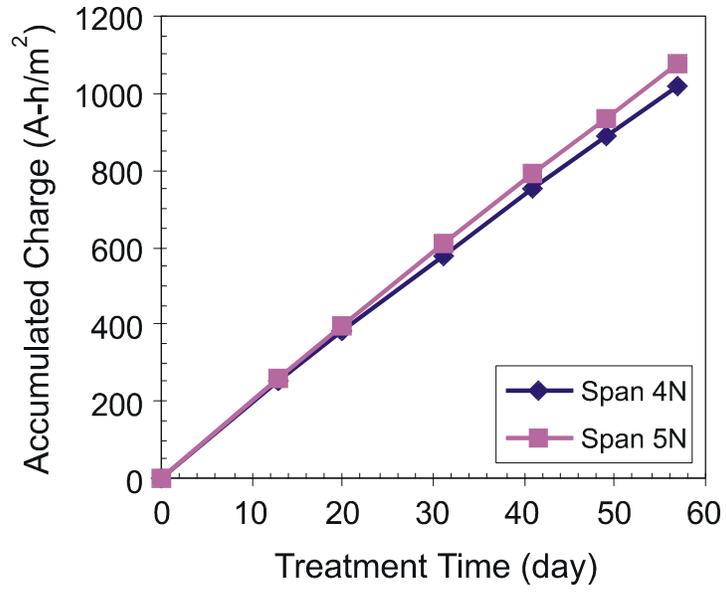


(a)

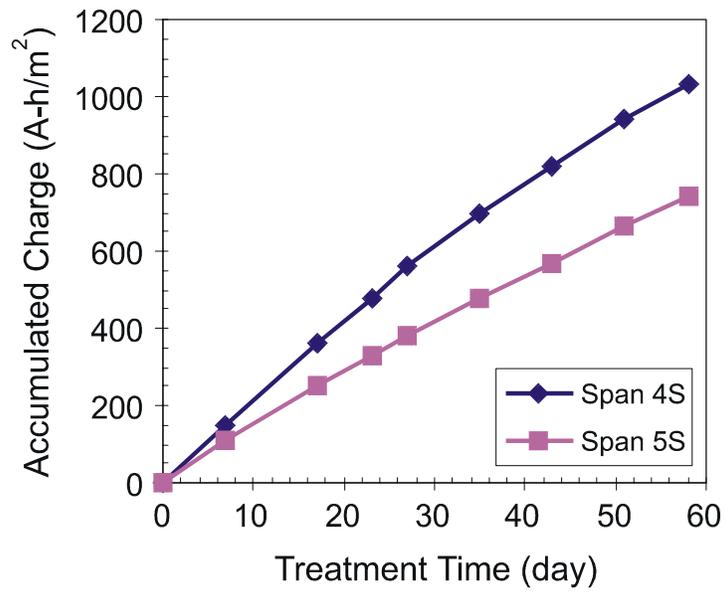


(b)

Figure 15. Circuit resistances of (a) Northern and (b) Southern Halves of Deck Spans



(a)



(b)

Figure 16. Accumulated Electrical Charges for (a) Northern and (b) Southern Halves of Spans

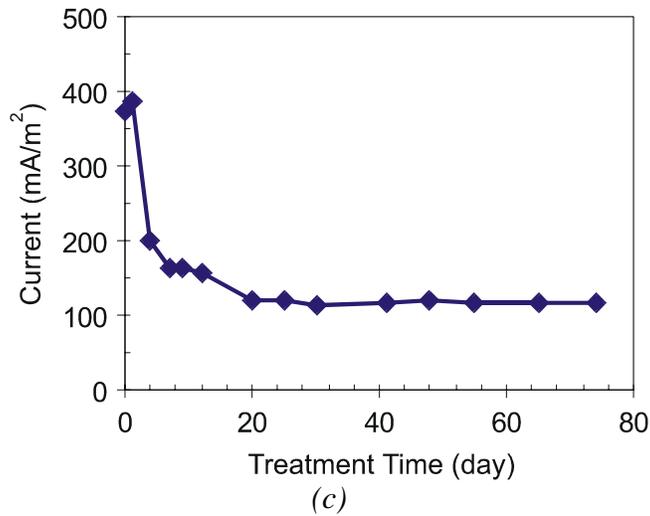
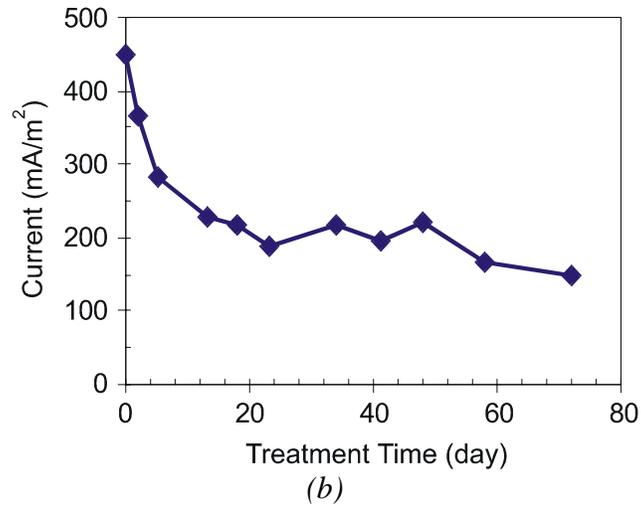
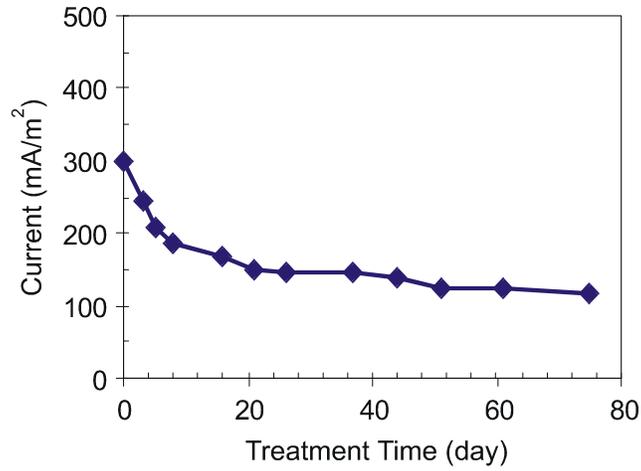


Figure 17. Treatment Current Passed into (a) Pier 1, (b) Pier 2, and (c) Pier 3

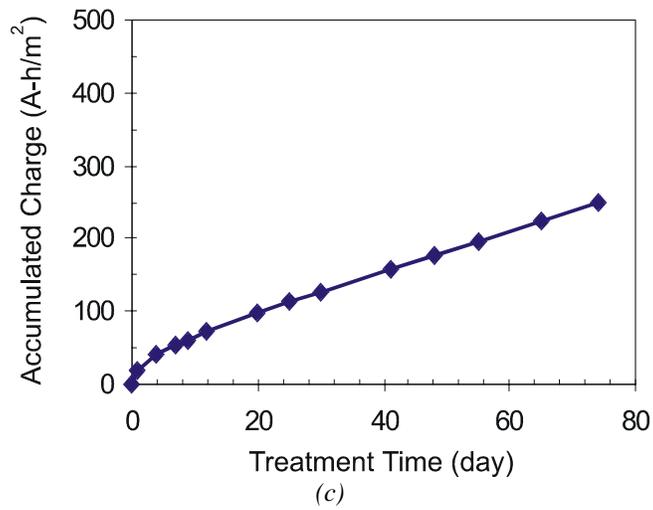
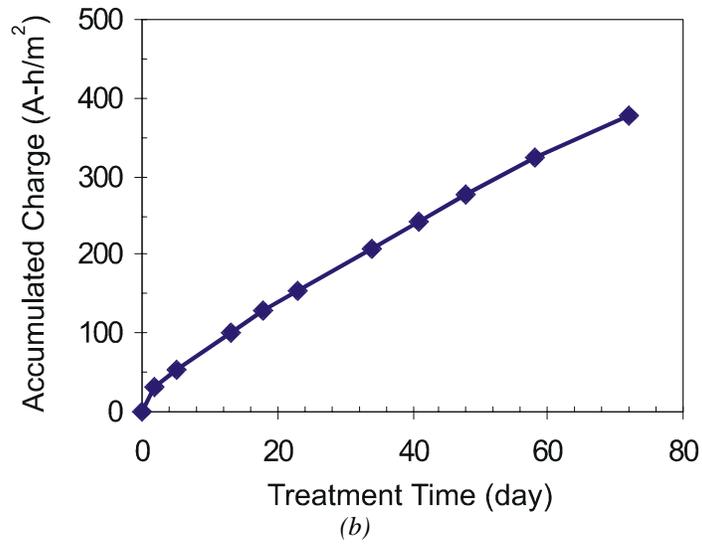
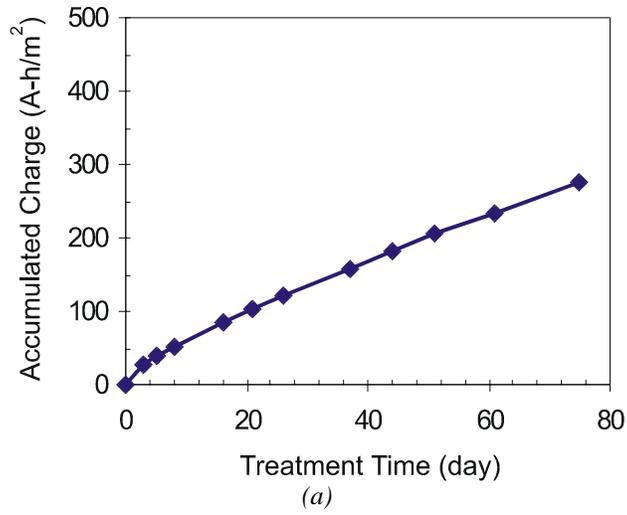


Figure 18. Accumulated Charges for (a) Pier 1, (b) Pier 2, and (c) Pier 3

TABLE 3. ELECTRICAL PARAMETERS RELATED TO VARIOUS ECE TREATMENTS

Concrete Component	Treatment Duration (day)	Average Voltage (V)	Current (mA/m ²)		Resistance (mohm/m ²)	Total Charge (A-hr/m ²)
			Initial	Final		
Deck 4N	57	30.2	925	662	1.29	1,077
Deck 5N	57	32.5	914	593	1.33	1,033
Deck 4S	58	29.3	854	541	1.27	1,019
Deck 5S	58	36.6	730	427	2.08	741
Pier 1	77	38.5	300	117	7.8	276
Pier 2	74	40.1	450	150	5.8	382
Pier 3	72	41.2	373	117	15.5	249

wet fibers. Perhaps the test procedures used during the application of anodes in the CP of concrete piers and decks to allow instantaneous detection of shorts between the anode and the rebars can be adapted for this purpose.

As noted earlier, it would be beneficial during the planning stage of a bridge rehabilitation project to be able to estimate how long a bridge needs to be treated so that lane closure can be minimized and the project can be scheduled efficiently, perhaps resulting in lower costs. Examination of the data on the amount of total charges passed through the three piers at different stages of treatment (Figure 18) offers a clue. When the derivative of the cumulative fractional charge is plotted against the treatment time, as in Figure 19, it shows that, for these piers, the gain in charge by extending the treatment diminishes significantly after 21 to 28 days of treatment. Since the amount of chloride removed at each stage is a fraction of the accumulated charge, as dictated by its transference number, this implies that the additional amount of chloride that will be removed becomes insignificant after approximately 21 to 28 days, which may be considered the “practical” treatment time for these piers. For a deck, being able to end the treatment after 21 to 28 days instead of at 56 days might mean considerably less interruption to traffic.

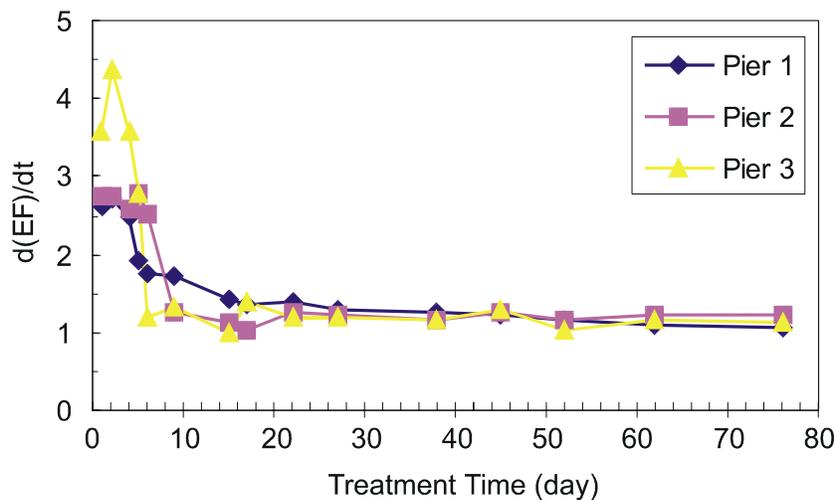


Figure 19. Increase in Accumulated Fractional Charge, as a Function of an Increase in Treatment Time

It would be beneficial to be able to predict *a priori* not only the practical treatment time for any given concrete structure but also the current density that could be passed through the structure at the start of a treatment. With these two estimates, it should be possible for an engineer to estimate how much of the chloride can be removed from that structure, once the initial chloride content in the structure is determined. In addition to allowing for efficient scheduling of the rehabilitation project, this may also allow for estimating the cost-benefit ratio of applying ECE treatment to that structure.

Effect of Treatment on Ion Concentrations in Concrete

Concentration of Chloride

When electrical current is passed through a chloride-contaminated concrete, all the mobile ions in the concrete, such as Cl^- , OH^- , Na^+ , K^+ , Ca^{+2} , etc., contribute by carrying their respective share of the current. It is this transfer of current by the Cl^- that results in its movement away from the reinforcing steel and toward the surface of the concrete structure.

To determine the amount of chloride ions removed from the treated deck spans and piers, the concentrations of chloride (at two depths) in the concrete before and after treatment were determined by chemical analysis and then compared. In the case of the decks, this comparison showed that the treatment led to significant decreases in the concentration of chloride in the concrete, at both depths, in all four deck areas (Table 4). As expected, treatment of the piers also led to a reduction in the chloride concentrations (Table 5). The extent of the reduction in chloride concentration (by percentages) in the piers was not only considerably less than those in the four deck areas, but it also varied more among piers, with standard deviations of 20.2 and 3.9 percent for the piers and the deck areas, respectively.

With regard to the percentage of samples with residual chloride in concentrations still exceeding the corrosion threshold of 0.77 kg/m^3 , the same differences are seen between the decks and the piers (Table 6). Before the treatment, 100 percent of the concrete samples taken from the depth of the steel in all four deck areas had concentrations exceeding the corrosion threshold. After treatment, only 20 to 25 percent of the samples (from the same depth and virtually the same sampling locations) had chloride concentrations exceeding the corrosion

TABLE 4. MEAN CHLORIDE CONCENTRATION AT VARIOUS DEPTHS IN DECK AREAS BEFORE AND AFTER TREATMENT (KG/M^3)

Concrete Component	At 6 to 19 mm			At 19 to 32 mm		
	Before	After	Change (%)	Before	After	Change (%)
Deck 4N	5.20	1.04	-80.0	2.68	0.59	-78.0
Deck 5N	5.92	1.06	-82.1	3.78	0.69	-81.7
Deck 4S	5.03	1.07	-78.7	3.05	0.71	-76.7
Deck 5S	4.97	1.20	-75.8	2.34	0.65	-72.2
	Average		-79.2	Average		-77.2
	SD		2.6	SD		3.9

TABLE 5. MEAN CHLORIDE CONCENTRATION AT VARIOUS DEPTHS IN PIERS BEFORE AND AFTER TREATMENT (KG/M³)

Concrete Component	At 6 to 19 mm			At 25 to 38 mm		
	Before	After	Change (%)	Before	After	Change (%)
Pier 1	1.35	0.88	-34.6	0.85	0.74	-12.6
Pier 2	1.97	0.79	-59.7	1.10	0.52	-52.9
Pier 3	1.47	1.07	-27.2	1.12	0.73	-34.8
	Average		-40.5	Average		-33.4
	SD		17.0	SD		20.2

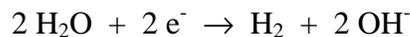
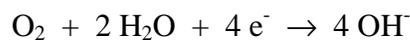
TABLE 6. PERCENTAGE OF CONCRETE SAMPLES WITH CHLORIDE EXCEEDING CORROSION THRESHOLD BEFORE AND AFTER TREATMENT

Concrete Component	Before	After	Change
Deck 4N	100	20	-80
Deck 5N	100	25	-75
Deck 4S	100	25	-75
Deck 5S	100	20	-80
Pier 1	75	12	-63
Pier 2	54	9	-45
Pier 3	80	60	-20

threshold. For the three piers, the benefits were lesser: with the percentage of concrete samples with chloride concentrations exceeding the corrosion threshold decreasing from 54 to 80 percent before treatment to 9 to 60 percent after treatment.

The differences in the results for the deck areas and piers are attributable to a combination of factors. These factors include (1) the relatively lower initial chloride concentrations in the piers, (2) the presence of vertical faces and undersides that make the piers relatively more difficult to treat, and (3) the relatively lower average concrete cover in the deck areas (33 mm) compared to that in the piers (51 mm). Regarding the last factor, as the concrete cover and, therefore, the distance between the anode and the steel increase, the strength of the electrical field pulling the chloride ions toward the anode weakens. The net result is less chloride removed. However, it is not possible at present to determine how much each factor contributed to the differences in the amount of chloride removed from the decks and the piers.

The fact that chloride remained in some locations does not imply that the steel bars did not benefit from the treatment. This is because in addition to moving significant amount of chloride ions away from the steel, beneficial reactions occurred around the steel as consequences of the treatment: the reduction of oxygen and water molecules at the surface of the steel bars to produce hydroxide ions, OH⁻:



The production of hydroxide ions on the surface of the steel bars is beneficial because it increases the alkalinity that helps prevent the return of corrosion on the steel bars. The beneficial combined result is the simultaneous reduction of chloride ions and increase of hydroxide ions, both around the steel bars. This reduces the $[Cl^-]/[OH^-]$ ratio, which, according to previous studies, influences the status of the steel bars more than $[Cl^-]$ alone.⁹ According to these studies, the chloride threshold level, i.e., the concentration of chloride that can be tolerated in the concrete before the initiation of steel corrosion, is considerably higher than 0.77 kg/m^3 and may actually be 2.5 to 6 times the hydroxide concentration in the concrete. This means that after treatment, the concrete in both the decks and the piers can tolerate more chloride ions than before without the initiation of corrosion.

Concentration of Cations Introduced in Electrolytes

As noted earlier, LiBO_3 was added to the electrolytes used in the treatment of the northern half of both deck spans, not only as a pH buffer but also as an agent for the mitigation of ASR, when the Li^+ was injected into the concrete. Since data pertinent to electrochemical injection of Li^+ into a concrete structure in conjunction with ECE treatment have never been reported, randomly selected concrete samples from these areas (taken before and after treatment) were chemically analyzed by the CE method. In addition, samples of the electrolyte used in these two deck areas were analyzed for Li^+ content by atomic absorption spectrometry.¹⁰

Table 7 shows that the concentrations of Li^+ in the electrolytes decreased by approximately 4,000 to 4,020 ppm (by weight of solution), or 95 percent, in 24 or 25 days of treatment. This decrease in the Li^+ concentration in the electrolyte, which was ponded on the surface of the concrete, resulted from electromigration of the Li^+ into the concrete, causing increases in the concentrations of Li^+ in the concrete. As Table 8 shows, the increases in Li^+ concentrations in both concrete deck areas ranged from 226 to 319 ppm (by weight of concrete) at the depth of 6 to 19 mm and 141 to 240 ppm at the depth of 19 to 32 mm. The Li^+ concentrations in the concrete before the treatment were attributed to the aggregates and the cement used in building the deck. These results provided clear evidence of the movement of the Li^+ from the electrolyte into the concrete in response to the presence of an electric field between the surface of the concrete deck and the network of reinforcing bars in the concrete. They also demonstrated the feasibility of using electrochemical injection for introducing Li^+ into an existing concrete structure to alleviate the effects of any ongoing ASR.

TABLE 7. CONCENTRATIONS OF Li^+ IN ELECTROLYTE DURING TREATMENT OF TWO DECK AREAS

Deck Area			
4N		5N	
Treatment Time (day)	$[\text{Li}^+]$ (ppm)	Treatment Time (day)	$[\text{Li}^+]$ (ppm)
0	4,200	0	4,200
5	2,147	4	2,554
17	707	16	646
25	179	24	217

TABLE 8. MEAN CONCENTRATIONS OF LI+ (PPM) AT VARIOUS DEPTHS IN DECK AREAS BEFORE AND AFTER ECE TREATMENT

Deck Area	At 6 to 19 mm			At 19 to 32 mm		
	Before	After	Change	Before	After	Change
4N	89	315	226	25	265	240
5N	24	343	319	62	203	141

However, electrochemically injecting a corrosion-inhibiting cation into the northern half of span 5 simultaneously with removing chlorides was not so successful. A total of 2.5 kg of the inhibitor tetraphenyl phosphonium chloride, $(C_6H_5)_4P^+ Cl^-$, was mixed with 120 L of water; then, the mixture was applied by a broom on the concrete surface of deck area 5N prior to the installation of the anode system. Analysis of the after-treatment samples by the CE method revealed no detectable amount of the cation $(C_6H_5)_4P^+$ in any sample. It is uncertain whether this meant that (1) the cationic inhibitor did not migrate into the concrete or (2) it did migrate, although in trace amounts undetectable by the CE method used. The latter would mean that migration of the cationic inhibitor into the concrete deck occurred in amounts less than 25 ppm (25 micrograms of inhibitor per gram of concrete), which is the estimated minimum detection limit of this inhibitor by the CE method.

A major difference between this field trial on a concrete deck and the experiments conducted at SRI International⁶ on relatively small concrete slabs was that Li^+ was not present in the electrolyte used in the latter. Although still unproven, it is possible that the Li^+ ions, even though likely to be solvated or surrounded by water molecules, were still relatively more mobile than the $(C_6H_5)_4P^+$ so that when both were present the former dominated the migration into the concrete deck. The detection of a considerable amount of Li^+ in the treated concrete (Table 8) did not eliminate this possibility.

Effects of ECE Treatment on Concrete

Previously, the concern with the pH of the electrolyte decreasing too much to become acidic during the treatment was raised. The decreased pH can lead to etching or softening of the cement paste on the surface of the concrete, especially in the deck where the surface is mostly horizontal. (Had the structures been constructed with concrete containing calcareous aggregates, etching of the aggregates would have been another concern.) To assess the extent to which softening may occur, the hardness of the concrete in the deck areas and piers was measured before and after the treatments. The results, presented in Table 9, indicated that softening of the concrete may have occurred in the deck areas, except for deck area 4S. However, if the standard deviation corresponding to the surface hardness measurements made over all the sampling points for each of the three areas is considered, only the loss in deck area 5N was statistically significant (at the 99 percent confidence level). As noted earlier, the pH of the electrolyte in this area became very acidic during the third week of the treatment.

TABLE 9. MEAN RELATIVE SURFACE HARDNESS (N/MM²) OF CONCRETE BEFORE AND AFTER TREATMENT

Concrete Component	Before Treatment		After Treatment		Change (%)
	Mean	SD (%)	Mean	SD (%)	
Deck 4N	60	7.8	56	4.5	-6.7
Deck 5N	56	5.5	51	4.1	-8.9
Deck 4S	56	2.9	57	2.6	1.8
Deck 5S	57	4.7	55	4.7	-3.5
Pier 1	53	6.0	56	6.1	5.7
Pier 2	55	6.9	54	5.0	-1.8
Pier 3	57	5.6	55	3.9	-3.5

Softening of the concrete is undesirable because it reflects the softening of the cement paste that holds the aggregate together. However, in the case of these deck areas, since the entire deck was going to be scarified before an overlay was applied, which is a common rehabilitation practice in Virginia and other states, a slight etching of the surface concrete would be of no real concern. Nevertheless, it is advisable to attain adequate control of the pH of the electrolyte during treatment. The results clearly point out the deficiency of the anode/dam system used for the treatment of the deck areas in that it did not allow for the convenient introduction of lime into the electrolyte and thorough circulation of the electrolyte through an entire treatment area. This deficiency clearly needs to be eliminated.

This softening problem was not encountered with the treatment of the concrete piers, as indicated in Table 9. The changes in the hardness ranged from -3.5 to +5.7 percent. In pier 1, the hardness of the concrete increased, and in piers 2 and 3 the hardness decreased. Again, based on the standard deviations for each set of samples, these changes appeared to be statistically insignificant. Examination of the concrete cores taken from the piers after treatment revealed that the concrete is generally in good condition. In addition to surface hardness, other aspects of the condition of the treated concrete deck areas were examined. The vertical sections of six 57-mm-diameter cores taken from the treated areas were examined by petrography, with particular attention to any signs of abnormality. Two cores each came from deck areas 4N and 5N, and one each from deck areas 4S and 5S. In all cores, the concrete was found to be of good quality, with the coarse and fine aggregates tightly bonded in a good cement paste that had a low water-cement ratio of 0.45 to 0.50 and an excellent entrained-air void system. The concrete was in good condition and free of cracks. The wearing surface of the two cores taken from the southern halves showed a slight loss of paste around the fine aggregate particles, which is likely the effect of the chemical erosion or etching that occurs when the electrolyte becomes acidic during some stages of the treatment.

Minimal ASR was observed around a few chert particles in three cores: one each from deck areas 4N and 5N and one from deck area 4S. It is believed that the ASR activity had occurred before the ECE treatment. In any event, it is still too early to conclude whether the use of LiBO₃ in the electrolyte for the northern halves helped mitigate possible ECE-induced ASR activity or was even necessary. In fact, one of the SHRP studies concluded that ECE treatment of a section of a concrete pier column containing ASR-susceptible fine aggregates without the use of LiBO₃ did not aggravate ASR activity.⁴

A similar petrographic examination of cores taken from the treated concrete piers indicated no sign of any possible adverse effects on the condition of the concrete.

Comparison of Benefits of ECE Treatment and Cathodic Protection

Both ECE treatment and CP will suppress ongoing reinforcing steel corrosion and prevent new corrosion from occurring in a concrete bridge, albeit each for a different length of time and at a different cost. As noted earlier, if the rehabilitation of a bridge includes only repair of the damaged concrete, one is risking the inescapable recurrence of steel corrosion around the repaired areas (the so-called halo effect). This will, in turn, again cause lengthy interruptions of traffic flow on the bridge during each rehabilitation, inconvenience to motorists, and possibly traffic crashes. The time interval between rehabilitations of a bridge can be increased by supplementing the repair of the damaged concrete by implementing a corrosion control method, such as ECE or CP.

The decision concerning which of these two methods to use is basically an economic consideration of (1) how long each option will suppress steel corrosion in the bridge, thereby extending the time interval between rehabilitations or the service life of the structure by at least that long, and (2) how much each option will cost.

Duration of Protection

ECE

The duration of protection offered by an ECE treatment depends on two factors: (1) how soon the intense polarizing effect that ECE imparts on the steel bars in a treated bridge fades, and (2) how soon the $[Cl^-]/[OH^-]$ ratio in the concrete shifts toward an undesirable level. The rates at which these two processes occur can be slowed considerably by sealing the treated bridge components from future ingress of oxygen, moisture, and chloride. For this purpose, a low-permeability concrete overlay can be applied on a treated deck, and a silane or a siloxane (both containing octyl or butyl alkyl group) can be applied on treated piers.

Since ECE is a new method, published data on the length of protection it provides are, not surprisingly, scarce. To help shed light on this, surveys of the half-cell potentials (with Cu/CuSO₄ electrode) were conducted on the concrete piers, at different locations across the pier caps and the columns, immediately before the treatment and at 1 and 4 years after. Table 10 shows the percentages of the measured potentials in each survey that were more negative than -350 mV. (According to ASTM C 876-91, if the potential over a concrete area is more negative than -350 mV, there is a greater than 90 percent probability that reinforcing steel corrosion is occurring at that area at the time of measurement.)

Table 10. Half-Cell Potentials of Steel in Piers

		Pier 1	Pier 2	Pier 3
Total sampling locations		8	11	13
Locations (%) with potentials ≤ -350 mV in year:	1995	37	45	31
	1996	0	0	0
	1999	0	0	0

Before the treatment in the spring of 1995, the percentages of all measurements that reflected half-cell potentials lower than -350 mV were 37, 45, and 31 for pier 1, 2, and 3, respectively. In the 1996 survey, at the same concrete locations, none of the potentials was lower than -350 mV. The subsequent survey in 1999 showed the same. These results indicate that it is probable that the steel bars in these piers are still passive and that the beneficial effect of the treatment has lasted at least 4 years. For added measure, the piers were sealed with a silane shortly after the treatment to prevent future ingress of moisture and chloride. Unpublished half-cell potential data from a concrete pier in Canada (Ontario Ministry of Transportation, unpublished data, 1999) that was partially treated in 1989 showed that the steel in the treated section was still passive after approximately 10 years. Therefore, it is perhaps valid to conclude, on a preliminary basis, that the protection offered by ECE treatment can likely last for at least 10 years and possibly 15 years. A point worth emphasizing is that the same concrete component can be treated again and again, after every 10 to 15 years or longer, to keep the benefit indefinitely.

CP

In determining the duration of protection provided to a concrete structure by application of CP, the influences of all components of a CP system must be considered. If the impressed-current method is used, the essential components are a rectifier, a wiring system, and an anode. (Although not essential in the operation of a CP system, concrete-embeddable reference electrodes are convenient for measuring the response of the steel bars to CP.) If the galvanic method is used, which is beginning to be applied in cases where the current requirements are low, the anode also assumes the role of the rectifier and provides the protection current, thereby eliminating the rectifier as a component. In either case, if the wiring system is properly installed and well protected from moisture, it should be virtually maintenance free. When a rectifier is used, if it is provided with adequate protection against transient power surges and is properly maintained, with spare parts replaced whenever necessary, it would be a negligible factor in influencing the duration of protection offered by a CP system. This leaves the life of the anode, which depends on the anode chosen, as the determining factor.

In the last 15 years or so, considerable strides have been accomplished in developing and/or identifying anodes suitable for CP of different concrete bridge components for different environments.¹¹ During this period, anodes, including catalyzed titanium mesh and the new thermally sprayed coating of Al-Zn-In and Zn/hydrogel, have been subjected to field tests for at least a few years so that reasonably reliable estimates of their life can be made.¹²⁻¹⁵

Cost

To allow bridge engineers to make rational choice between CP and ECE for bridge rehabilitation, useful information is provided in Table 11. The total cost of the treatment of the four deck areas was \$92,412 for a total concrete area of 720 m², which came to \$128/m². For the three piers, with a total concrete area of 488 m², the total cost was \$63,932, or \$131/m². Since ECE technology is new, experienced or qualified contractors were (and still are) limited, and it is uncertain to what extent this has influenced the costs of the treatments so far. As Table 11 shows, for all known ECE treatments, the costs ranged from \$128 to \$135/m² for decks and \$86 to \$321/m² for piers, with a preliminary conservative projected life of at least 10 years.

TABLE 11. INFORMATION FROM KNOWN APPLICATIONS OF CORROSION CONTROL METHODS FOR CONCRETE BRIDGES

Corrosion Control Option	Bridge Component	Anode	Amount of Area Involved (m ²)	Unit Cost (\$/m ²)	Projected Life (yr)
Impressed-Current CP	Decks	Catalyzed Ti Mesh	740-38,500	57-97	60-90
	Piers/Abutments	Conductive Paints	1,040-7,700	82-151	12-15
		Thermal-Sprayed Zn Coating	19-18,200	86-108	Up-27
		Thermal-Sprayed Ti Coating	66-280	105	20-40
Galvanic CP	Piers/Abutments	Thermal-Sprayed Zn Coating	480-981	86-108	<10 ^a
		Thermal-Sprayed Al-Zn-In Coating	42-4,180	118-160	10-15 ^a
		Zn/Hydrogel	24-8,750	26-171	10-12 ^b
ECE Treatment	Decks	Catalyzed Ti Mesh	720-1,560	128-135	> 10
	Piers/Abutments	Steel/Catalyzed Ti Mesh	89-488	86-321	> 10

^aAt 12 mil.

^bAt 10 mil.

Approaches for Particular Bridge Members

As Table 11 also shows, there are many approaches for controlling corrosion in a concrete bridge.

Decks

For concrete decks, either impressed-current CP or ECE treatment can be used, each to give a different projected length of protection at a different cost. When the two factors are weighed, it is clear that impressed-current CP is the better choice provided the rectifier of the CP system is maintained in working condition all the time, which has often not been the case. However, bridge engineers may prefer to apply ECE to a deck if (1) the treatment cost becomes

lower than that shown in Table 11, (2) the deck no longer complies with current specifications, (3) they want to use the deck for only 10 to 15 more years before replacing it, or (4) they do not want to be encumbered by maintaining a rectifier.

Piers and Abutments

Approximately 11 more ECE treatments of concrete substructures had been conducted after the treatment of the three piers in this investigation. The costs for all treatments ranged from \$86 to \$321. With a current conservative estimated life of at least 10 years (which needs to be validated), it is going to be difficult for ECE treatment to be considered favorably as an alternative to either mode of CP for corrosion control of concrete piers and abutments.

As listed in Table 11, more CP options are available for corrosion control in concrete piers and abutments. In the galvanic mode, the protection current is provided by the chemical degradation of the selected anode (as in batteries); therefore, such systems have a comparatively shorter life than when the anode is used in the impressed-current mode with a rectifier supplying the current. (Notice that the thermal-sprayed Zn coating is the only anode that is suitable for use in both galvanic and impressed-current modes.)

This general difference in service lives is reflected in Table 11, which indicates that the sprayed Zn and Ti coatings, when used in the impressed-current mode, provide comparatively longer estimated lives and would be worth more consideration than the other options. However, the 27-year life projected for the sprayed Zn coating is for only two bridge installations in the coastal area of a western state that use a comparatively low current of 22 mA/m^2 , only 25 percent of the typical level. Since Ti is chemically inactive and, therefore, stable, the thermal-sprayed Ti coating is likely to behave as a truly good long-term anode. Weighing the cost and service life together for each option, the use of thermal-sprayed Ti and, possibly, Zn coatings in the impressed-mode CP may be unsurpassable.

If engineers do not want to be encumbered by the upkeep of rectifiers and the current requirement is low, there are three options for galvanic CP of piers and abutments. The thermal-sprayed Al-Zn-In and the Zn/hydrogel are projected to perform virtually equal to or possibly slightly better than thermal-sprayed Zn. In such a case, wherein there are at least two options offering virtually the same service lives, engineers might consider specifying performance (in terms of service life) instead of the current practice of specifying a specific option or anode material, which was dictated by the lack of options and the type of information presented in Table 11. This new approach of bid solicitation will result in advantages for engineers because a new factor, the choice of materials or methods, is available for contractors to consider.

Generally, the impressed-current mode of CP is clearly a better method of controlling corrosion in reinforced concrete bridges because of the projected life of the anodes and the costs seen so far. However, concern with potential problems with rectifiers has undoubtedly hindered the wider application of this method of CP. A viable solution to this power issue is to use solar panels to generate the DC current and the use of backup batteries to store the excess current for use during nights and cloudy days, in place of rectifiers. This approach is being applied in

pipeline and other industries and should be explored in CP of concrete bridges. Data on the effectiveness of using solar panels, with and without backup batteries, need to be collected to pave the way for this approach.

CONCLUSIONS

- *ECE can be used to control corrosion in full-sized concrete bridge members.*
- *ECE is comparatively less efficient for concrete piers (at least with the treatment system used) than for deck spans.* This difference can be attributed to a combination of factors; most important is the fact that a major portion of the concrete surfaces encountered in a pier are vertical surfaces and undersides, which present formidable challenges to a treatment system in keeping the electrolyte contained and in good contact with these surfaces.
- *The ECE treatment systems for both concrete decks and piers would require improvements to allow for adequate circulation and, if the electrolyte used contained more than water and lime, proper collection of the electrolytes.*
- *Lithium ions can be electrochemically injected into concrete while chloride is being removed.* This offers a possible method for remedying ASR in existing concrete structures.
- *The positively charged corrosion inhibitor $(C_6H_5)_4P^+$ cannot be successfully injected into concrete in the presence of Li^+ in the electrolyte.*
- *ECE treatment does not adversely affect the concrete, with the exception of some etching of the concrete surface.* This can be prevented by better control of the pH of the electrolyte.
- *Considering only the projected life of the beneficial effects of a treatment and its cost, it would be difficult to compare ECE favorably with CP.*

RECOMMENDATIONS

1. Consider ECE on a project-to-project basis as a viable corrosion control method for inland concrete decks and piers only if the cost of such treatment becomes competitive and the desired life is only 10 to 15 years.
2. In future rehabilitation projects where equally performing corrosion control methods are available, specify performance in terms of projected protection life instead of a specific control method.

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