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PENNDOT

CATHODIC PROTECTION OF BRIDGES IN PENNSYLVANIA

University-Based Research, Education and Technology Transfer Program
AGREEMENT NO. 359704, WORK ORDER 24

FINAL REPORT

September 2001

By G. Sabnis

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16. Abstract

This report is a compilation of several documents relating to cathodic protection systems used in rehabilitation efforts on existing reinforced concrete bridge structures in Districts 3, 5, and 11 of Pennsylvania DOT. This summary report briefly touches on the history of the systems in Pennsylvania, the current state of the art for corrosion protection, the basic principle behind CP systems as well as other available systems, both from a installation\performance point of view. Details of projects that were executed in the various districts are also presented. These include several bridge decks and a pier structure that were rehabilitated using various CP systems. In addition details from two control bridges that did not utilize the system are also presented.

In addition, this report attempts to examine the various problems that were experienced, and to highlight the lessons learnt from these projects. In its conclusion, it offers recommendations with respect to the efficient and proper use of Cp systems in Pennsylvania.

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Commonwealth of Pennsylvania
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TABLE OF CONTENTS

ABSTRACT

This report is a summary of several documents covering cathodic protection (CP) systems used in rehabilitation of the existing reinforced concrete bridge structures in Districts 3, 5, and 11 of Pennsylvania DOT, which were made available for this document. In addition, the report presents the current information on both the installation and performance of the systems and their record in Pennsylvania.

Details of the projects that were executed in the PENNDOT districts are presented. These include several bridge decks and a pier structure that were rehabilitated using CP systems. In addition, details from two "control" bridges that did not utilize CP are also presented.

This report attempts to analyze the various problems experienced with CP system and highlight the lessons learned from these projects. Finally, some recommendations for efficient use of CP systems are presented.

1. INTRODUCTION

1.1 Scope and Objectives

This report is a summary that is based primarily on several documents issued by Pennsylvania Department of Transportation (PENNDOT) Districts 3, 5, and 11 that include the design and construction details of the cathodic protection (CP) projects. The analysis of performance of the cathodic protection systems in the rehabilitation of existing reinforced concrete bridge structures in the three districts is presented.

While certain specific details of the projects undertaken are included, it is not the intent of this report to present comprehensive design, construction, and monitoring data, which are available from the literature. This report attempts to examine the various problems that were experienced by PENNDOT and to highlight the lessons learned, as well as address economic issues. In conclusion, recommendations are offered for corrosion protection of existing bridge structures, as well as for the design and construction of new ones.

1.2 The Corrosion Problem

It has been reported that up to 40 percent of steel produced worldwide each year is used to replace corroded metal. Corrosion cost studies carried out in the U.S., Europe, and Japan have shown that, in all cases, a cost figure of 3% to 4% of the gross national product (GNP) can be attributed to direct and indirect costs of corrosion. Of this, approximately one quarter of the cost is related to corrosion of steel in reinforced concrete structures. [1,3,4,6]

Such corrosion is a problem that has been around almost as long as the reinforced concrete structures have been built. This problem, along with the subsequent deterioration of concrete, has been a major concern for bridge engineers since the early 1960's. According to NBC/COX, (all 50 states have bridges in serious distress. Ten states have at least 1,000 bridges in this category, with New York having the most, with 2,600 bridges). A total of 26,883 bridges in the country have been identified as being in serious condition. [6]

It is now common in the state of Pennsylvania, as well as across the U.S., that bridge structures that have been designed for a half-century service life are in need of major repairs within a decade after construction. Traditionally, these failures have been blamed on vehicle loads exceeding regulated design capacities, poor material selection and construction practices, and inadequate construction supervision. In more recent years, it has been demonstrated that the high concentration of salts used on the roads to melt snow, as well as those present in structures located in a marine environment, is actually a major contributor to the problem. The U.S. Secretary of Transportation Action Report to Congress has projected costs as staggering as $50 billion in deck repairs. National Bridge Inventory data collected over the past few years indicates that deterioration due to corrosion has substantially increased, and thousands of additional bridges on the nations highways will become structurally deficient and in need of intensive maintenance or rehabilitation in the near future.

While it has been indicated that as little as 1 to 2 pounds of chloride ions per cubic yard of concrete may be enough to destroy the passivity of reinforcing steel in concrete, surveys have shown that many highway structures have chloride concentrations far in excess of this critical level. Pennsylvania's "Bare Roads Policy" of the 1960's and 1970's has been responsible for introducing salt concentrations in excess of ten pounds per cubic yard of concrete on highways and bridge decks. [3]

1.3 Overview of Corrosion of Reinforcement In Concrete

Reinforcing and pre-stressing steels in concrete may be subject to corrosion due to carbonation and/or chloride ion attack. References 6 and 15 present a detailed treatment of the topic and are probably the best references on the topic. The natural protection of steel embedded in concrete is disturbed due to the following:

1. Neutralization: a reduction of pH due to the chemical reaction between the products of cement hydration and carbon dioxide, which diffuses from the atmosphere (carbonation).
2. The presence of the chloride ions in contact with the steel reinforcement surface.

<u>Carbonation</u>

The determining factor of the protective ability of concrete against corrosion is the alkaline nature of calcium hydroxide--an element of the hardened cement paste.

In freshly placed concrete, steel reinforcement is protected from corrosion by the high alkalinity of the surrounding cement paste. The alkalinity of the mix initially rises to a pH value of 12.8 or even higher due to the calcium hydroxide released during the cement hydration. The amount of the calcium hydroxide available in the concrete depends primarily upon the amount and composition of the calcium silicate phases present in the cement and degree of hydration.

A passive layer protects the surface of the steel, which is stable and adherent in this range of alkalinity, however, alkalis in concrete eventually react with acidic components of the atmosphere, particularly carbon dioxide (CO_2). As a result, the alkalinity of the concrete is progressively reduced by converting the calcium hydroxide to calcium carbonate, thus reducing the pH value of the concrete to below 10 and, consequently, its protective ability. The reaction of carbon dioxide with the products of cement hydration is called "carbonation."

Vaysburd, et al. (10) have pointed out that the reduction of the pH of the pore solution in the cement phase to less than 10 destroys the passivity of the protective layer. When steel in concrete is depassivated and the environment is acidic or mildly alkaline, corrosion occurs when moisture and oxygen gain access into the concrete. In general, both conditions are fulfilled because concrete is permeable to oxygen in the atmosphere, and even the driest concrete contains sufficient capillary water to sustain corrosion.

Carbonation of concrete leads to corrosion of steel to form corrosion products that are collectively referred to as rust. Normal air may contain carbon dioxide in relatively low concentrations (0.03 percent); however, its levels in industrial atmospheres are normally higher. The atmospheric carbon dioxide penetrates into the pores of the concrete by diffusion and reacts with the calcium hydroxide dissolved in the pore water.

The permeability and the calcium content of the concrete as well as the ambient atmospheric conditions--amount of carbon dioxide and relative humidity-mainly influence the rate of carbonation. The front of carbonation moves into the concrete from the exposed surface very slowly at both low and high relative humidity, and progresses rapidly when it reaches a relative humidity between 50 and 70 percent.

Chloride Ions

Steel reinforcement in concrete will not corrode because in an alkaline environment it is passive and unable to dissolve; steel in concrete will corrode when its surface is not rendered completely passive during the manufacturing of the structure or becomes active during service (18).

Chloride ions may enter concrete when it is being mixed. In such an instance, the ions are introduced via additives such as chloride-based accelerators or water reducers. Even though the use of such products has been significantly reduced, it is still being exercised in certain areas. In addition, some older structures, which were built with such additives, still face the problem of chloride contamination. In other instances, chlorides are introduced as a result of the use of contaminated materials.

In most of the cases, chloride ions enter the structure during its service life. The penetration of chloride ions into reinforced concrete structures is prevalent in structures located in marine environments and those exposed to de-icing salts. It is believed that the contribution of chloride ions to the corrosion of reinforcement is based on the ions becoming incorporated into the protective film, subsequently increasing its solubility and conductivity and reducing its protective character (18).

All metals have a different natural electrical potential. Whenever dissimilar metals are connected in a similar environment, current will flow, causing corrosion to occur. Under certain circumstances, this will also occur with similar metals, such as when neither the passive layer nor the steel is truly homogeneous. Oxidation or an anodic reaction occurs in areas of the reinforcing bars where the current discharges into the environment, and protection or cathodic reaction occurs where the current returns to the metal surface. Corrosion is an electrochemical process; the important factor affecting a corrosion cell is the difference in potentials along the metal surface. The driving force for current and corrosion is the potential development. Since the structure of steel and the contact layer of concrete are both heterogeneous, the requirement for potential difference between the separate portions of the metal surface (the electrochemical non-homogeneity) is always present.

Effect of Cracking

The mechanisms of the protective action of concrete to the reinforcement and corrosion need correction for cracking. Concrete cracking is listed among the principal phenomena associated with deterioration. In such a heterogeneous composite material as concrete, an invisible system of micro-cracks already exists at the cement matrix-aggregate and cement matrix-steel interfaces. Additional cracking occurs and the cracking system may further develop due to the volume changes in production and in service, freezing-thawing, and wetting-drying cycles, dynamic loads, due to alkali-aggregate reactions, etc. Cracking initiates and promotes corrosion, especially when large visible cracks become interconnected with closely spaced micro-cracks, which facilitates the transport of aggressive gases to the embedded steel (19).

Concrete is a permeable material, in which aggressive agents by diffusion (theoretically) are reaching reinforcing steel, causing its depassivation and corrosion when water and oxygen are available. This, however, is a relatively lengthy process. Concrete is a brittle material and always contains micro-cracks. When these micro-cracks combine in a network with macro-cracks, the prevailing transport mechanism is not diffusion; it is the permeation of water and aggressive agents via water through the cracks to the reinforcement. Why enter through the closed door, when an open door is nearby? (20). Cracking of cement-based materials is responsible for high permeability and lack of durability. For corrosion to occur, it is necessary that both the passivating film on the steel be destroyed and that there exists an electrochemical potential differential within the steel/concrete system. Cracking is a substantial contributor to this differential in potentials.

1.4 Overview of Current Approaches To Identify Corrosion-Vulnerable Conditions

Several methods are currently being utilized to evaluate the levels of corrosion and associated problems that a particular structure may be experiencing. Over the years, a great number of methods have been developed to evaluate the state of structures, ranging from visual inspections to semi-destructive means. In recent years, non-destructive methods has taken center stage in the evaluation of levels of corrosion, corrosion rates and associated problems in concrete structures. Following is an overview of some of the more common procedures.

Delamination

Delamination - A separation along a plane parallel to a concrete surface is currently detected by several methods. One of the most common approaches involves the use of a chain, a hammer or some other sounding device that will allow the detection of audible differences in sound frequencies. This gives an indication of the location of hollow areas that have been partially or totally separated from the main mass of the structure. When struck with a hammer, or when a chain is dragged across a surface, solid concrete will give a sharp ringing sound; delaminated concrete will give a dull, hollow sound.

Currently, several forms of mechanical energy and electromagnetic energy-based devices are being employed, including impact-echo method, pulse-echo method, ultrasonic pulse velocity, and infrared thermography. Pulse Velocity and Impact Echo are echo techniques used to detect delaminations, as well as other defects in hardened concrete.

Pulse velocity, which is used to measure uniformity, relative quality, and even estimate crack depth in concrete, consists of measuring the travel time of ultrasonic pulses of compressional waves generated by an electro-acoustical transducer through the concrete structure. There is also a reasonable correlation between the transit time and the compressive strength of concrete. Details of the standard method of testing for the Pulse Velocity are detailed in ASTM C 597-83.

The impact echo technique differs from the pulse velocity in the frequency difference of the waves and the analysis of the reflected waves. The impact echo technique considers the shape and attenuation of reflected signals.

Depth of Concrete Cover

Various forms of "pachometer" or "covermeter" have replaced destructive methods that were once prevalent for locating reinforcing bars. These equipments are typically based on the measurement of the variation in magnetic flux caused by the presence of steel in concrete. While some level of refining is still desirable, the various apparatus that are available offer reasonably accurate estimations on the cover depth, size, and location of rebars. There are currently no standards on this technique.

Chloride Content

It has been generally accepted that chloride concentrations of above 0.03% by weight of concrete present a potential risk of corrosion of steel in concrete. In addition to concentration levels, chloride profiles or concentration versus depth is also an important consideration.

Both chloride concentrations and profiles can be determined by analyzing concrete samples. The Standard method for sampling and testing conceptually involves the collection of samples from various locations and depths using a hammer drill or by extracting cores that are later crushed. The results are obtained in the lab typically using a wet chemical analysis technique and are reported in chloride by weight of concrete, chloride parts per million, percent chloride by weight of cement, or weight of chloride per volume of concrete. Some developments have been made in the area of field test kits; however, they are typically not as accurate as the lab results.

The AASHTO standard test method T 260-94 provides for three methods in determining chloride ion content. Procedures A and B are laboratory methods that utilize potentiometric titration and atomic absorbsion respectively. Test Procedure C determines total ions using a Specific Ion Probe.

Carbonation Testing

Carbonation is typically a slow process, approximately about 0.04 in. per annum, that can be performed during petrographic analysis or utilizing core samples. Depth of carbonation is usually determined using phenolphthalein pH indicator solution sprayed onto freshly fractured concrete surface. Indications on the change of color tints indicate carbonated concrete. Photographic documentation is typically taken. There are no current standards for this technique [6].

Corrosion Rate Measurement

Corrosion rate measurement is basically used to estimate rates of corrosion of reinforcing bars in concrete. While several variations exist in the testing equipment, many are based on the principle of applying a small voltage or current perturbation to the reinforcement, and the corresponding current or voltage response is measured. This data is then mathematically manipulated to estimate the rate of corrosion.

1.5 Overview of Available Protection Systems

The material manufacturers offer a tiered choice of corrosion protective systems for concrete repair with varying degrees of cost. The systems include the use of various protective coatings, cathodic protection, corrosion-inhibiting admixtures, etc. In some cases, combinations of protection systems have been recommended. Several theoretical justifications have been proposed for different protective systems, but all contain elements of speculation, and not one of them has been demonstrated effectiveness beyond a doubt. Current protection theories usually resolve themselves into one of three mechanisms or their combinations:

1. Electrochemical barrier between reinforcement and cementitious material to minimize reactive sites (barrier coatings).
2. Chemical stabilization of the steel surface.
3. Cathodic protection in the form of sacrificial anode.

The following sections present discussion on some of the protective systems.

Rebar Coatings

To inhibit the corrosion process, one approach is to isolate the reinforcement from its environment by applying a physical barrier. This is achieved by coating rebars with epoxies and polymers. Factory applied coatings are often more successful than site applied coatings because they are applied under controlled conditions to rebars. Rehabilitation works often employ the use of coatings that are applied on site typically to partially corroded bars. Difficulties in achieving full coverage, especially at the backside of the rebar and poor adhesion caused by improper surface preparation, are often problems associated with site-applied coatings.

Corrosion Inhibitors

A corrosion-inhibiting admixture in concrete can be defined as a chemical compound that effectively checks, decreases, or prevents the reaction of steel with the surrounding environment when added in a small concentrations to concrete. Corrosion-inhibiting admixtures, depending on how they affect the corrosion process, can be grouped into three broad classes: (a) anodic, (b) cathodic, and (c) mixed, depending on whether they interfere with the corrosion reaction preferentially at the anodic or cathodic sites or whether both are involved. Corrosion inhibitors are also classified into two groups: inorganic and organic.

Anodic inhibitors are materials that function as inhibitors due to their ability to accept electrons. They exert their action by stifling the reaction at the anode. Most of the admixtures in this group are effective only when present in sufficiently high concentrations. The concentration required is often determined by the level of chloride to which the steel will be exposed. When insufficient quantities are used, corrosion occurs, intensity being localized, causing severe pitting.

Cathodic inhibitors act either by slowing the cathodic reaction or selectively precipitating cathodic sites. Materials in this group are strong proton acceptors, and their action in contrast to anodic inhibitors is usually indirect.

Mixed inhibitors may simultaneously affect both anodic and cathodic processes. A mixed inhibitor is usually more desirable because its effect is all encompassing, covering corrosion resulting from chloride attack as well as that due to micro-cells on the metal surface. Since microscopic distances separating anodic and cathodic areas characterize micro-cell corrosion, it is impossible to locate either the anodic or cathodic sites on the reinforcement.

In the 1960's, calcium nitrite, an inorganic inhibitor, became available for use as an anodic type of corrosion inhibitor and presently is the one most used. It had been proposed that calcium nitrite inhibits corrosion by reacting with ferrous ions to form a film of ferric oxide around the anode.

One of the serious drawbacks in the use of anodic inhibitors is that the admixtures are effective in maintaining passivity only if present in sufficient concentrations. It is generally agreed that the calcium nitrite corrosion-inhibiting effect is degenerative in nature. The unbound nitrite ions diminish in concentration as they stabilize the passivation layer of the steel reinforcement. Due to the relatively small amounts in concrete, the admixture tends to be dispersed in the mass rather then at concrete/steel interface.

These inhibitors, however, may act as an accelerator. In addition, they tend to promote slump loss. (6) The organic products include water-based organic amine and an oxygenated hydrocarbon. These are effective in that they provide a blocking action on the penetration of chloride ions through the concrete matrix. In addition, they form a thin protective film on the steel surface that prevents chloride from coming into contact with the steel. On the other hand, organic-based inhibitors have been reported to be relatively ineffective in concrete with low w/c ratio. In addition, its use may cause a slight decrease in compressive strength.

1.6 Cathodic Protection Systems

Cathodic protection (CP) systems have been used successfully as early as the 1800's to protect ship hulls from deterioration due to corrosion. The system has been applied to bridge decks in the 1970's on U.S. Highways. With the first CP system developed for use on a California bridge deck in 1974. Currently, it is commonly used in underground pipelines, offshore structures, bridges, and parking facilities. The system is basically an electrochemical corrosion protection technique whereby direct current is introduced into a structure such that the vulnerable metal is protected by forcing it to become a cathode. This is accomplished by introducing an alternative anode, which assumes the environment for the oxidation process. Anodes may be sacrificial when utilized as the external current source, or they may be relatively inert when the impressed current system is used. Most of the early efforts in CP systems utilized sacrificial anodes.

The sacrificial method consists of the forming of a galvanic couple with the use of dissimilar metals with significant natural voltage differences, which produces the flow of electrical current. The sacrificial anode system was found to be limited in its potential for protection simply because the amount of current that could be generated was limited by the potential difference between the anode being used and the metal being protected. While the approach did not require any external power source, its disadvantages, particularly its power generation limitation and invariability, as well as the fact that the sacrificial anodes periodically required replacement, influenced the industry to lean towards the impressed current approach.

The impressed current approach consists of the introduction of an external power source, typically an alternating current source. This is converted to direct current and regulated with the use of a rectifier. While this approach to CP is often more elaborate in the design, construction, and maintenance phases of a project, the ability to vary the protection power, as well as advantages gained in remote monitoring techniques associated with the approach, have proven very useful. Materials utilized for impressed current anodes (ICA) are generally those with low consumption rates and good electrical and mechanical properties. Some of these include titanium, steel, graphite, and platinum, among, others.

In bridge decks, CP systems are applied as non-overlay slotted, non-conductive overlay, and conductive overlay systems. Bridge decks are typically broken down into sections that are referred to as zones. This is done in order to achieve more control over smaller sections of the deck. Each zone is an isolated system. A zone usually consists of two lead wires that are connected to the reinforcing steel in that section and an external power source. For the purpose of reference monitoring, two independently grounded reference cells are also installed in each zone. The most popular reference cells are silver-silver chloride. The cells are installed at the bottoms of slots and covered with salt-free patching materials.

2. CP SYSTEMS IN PENNSYLVANIA

2.1 Background of CP Systems In Pennsylvania[1, 2, 4, 15]

Prior to 1986, PENNDOT installed two cathodic protection systems (CP) on the deck of U.S. Route 15 near its junction with Interstate Route 80. The first system was installed on a bridge over White Deer Hole Creek at the Village of Allenwood. This project was constructed in 1977 at a cost of $5 per square foot. The system utilized the impressed current approach with a two-inch layer of conductive coke-asphalt and a two-inch asphalt overlay-wearing surface. The rectifier is controlled by zinc reference cells, which were implanted in eight zones. Each zone has four high silicone-cast iron anodes to transmit the current to the coke-breeze. Four corrosion meter probes were also installed to monitor the effectiveness of the CP system. Subsequently, the wiring system on the deck was rehabilitated in 1983 as a result of breaks in the wires.

The first slotted system was installed in Pennsylvania in 1982 at a cost of $11 per square foot. The bridge is located on the southbound of the New Columbia Interchange, just south of I-80 in Union County. It consisted of half-inch-wide by three-quarter-inch-deep longitudinal diamond cut slots in the deck at two-foot spacings. A 0.031" diameter palatalized niobium copper wire anode was located in each slot prior to back filling with the conductive polymer grout. The deck is separated into six zones. Four corrosion meter probes were installed, as in the first system.

The second slotted system was installed in 1983 at a cost of $12.75 per square foot. This was done on the northbound lane structure at the New Columbia site. The system is a constant current mode system where the total deck is one zone and the current is the same throughout. Continuous longitudinal slots, three-quarter-inch by three-quarter-inch were placed at one foot on centers on the deck. Two high purity carbon strands were located in each slot prior to back filling with FHWA's conductive polymer grout. Three lateral primary anodes consisting of a palatalized niobium copper wire and two carbon strands in conductive grout back-filled slots energize them. Electrical continuity between the primary and the secondary anodes is achieved by physical contact of the anode strands and wires through the conductive grout itself.

2.2 Identification and Objectives of Projects

Bridge Identification and Objectives of Projects- District 3[2]

Rehabilitation of six miles of Interstate Route 80 (I-80) in Columbia County, Pennsylvania, east of the North Branch of the Susquehanna River to the Luzerne County line was carried out in 1983. The project was designated as Columbia County, Legislative route 1009, Section 014 and included the rehabilitation of four bridges (3-A through 3-D). These bridges were originally constructed in 1964 as part of a major construction project for this section of Interstate Route 80. Two of the CP bridges span Traffic Route 339 at the Mifflinville Interchange (Exit No. 37). The other two CP bridges carry westbound traffic over S/R/ 2028 (LR 19023) at two separate locations east of the Mifflinville Interchange.

Under a separate contract completed in 1983, two bridges (3-E, 3-F) that carry the eastbound lanes of I-80 over S.R. 2028 (LR 19023) were rehabilitated without the use of CP. Work on these bridges included patching of the reinforced concrete decks, modifying the parapets to meet current safety standards, and providing a $1^{1}/_{4}$-inch Latex Modified concrete-wearing surface. These two bridges were rehabilitated previously in 1979 at Stations +779 and +865 (twin bridges to the +780 and +867 WB structures). The following table shows some details of the four CP bridges as well as the two bridges, which were rehabilitated around the same time. ***Figure 2.1*** shows the locations of these bridges.

Table 2.1 PROJECTS IN DISTRICT 3 [1,4]

ITEM	BRIDGE 3A	BRIDGE 3B	BRIDGE 3C	BRIDGE 3D	BRIDGE 3E (Control)	BRIDGE 3F (Control)
L.R.	1009	1009	1009	1009	19023	19023
STATION	724+79	725+09 WBL	780+01	867+20	865+00	779+00
Traffic Direction	EBL	190080-	WBL	WBL	EBL	EBL
Structure Identification Number	190080-24141267	24151185	190080-24251816	190080-24450000	190080-24440000	190080-24241911
BRIDGE OVER	L.R. 19020	L.R. 19020	L.R. 19023	L.R. 19023	-	-
BRIDGE UNDER	-	-	-	-	SR 2028 (LR 19023)	SR 2028 (LR19023)
DATE OF CONSTRUCTION	1964	1964	1964	1964	na	na
BRIDGE WIDTH (curb to curb)		40'				
Original Construction	40' 0"	0" 40'	40' 0"	40' 0"	40' 0"	40' 0"
After Rehabilitation	40' 8"	8"	40' 8"	40' 8"	40' 8"	40' 8"
TOTAL STRUCTURAL LENGTH	114' 8"	144' 8"	117' 6"	110' 10"	110' 10"	117' 6"
Span No. 1	33' 3.5"	33' 3.5"	69' 0"	33' 4"	33' 4"	69' 0"
Span No. 2	46' 7"	46' 7"	46' 0"	41' 8"	41' 8"	46' 0"
Span No. 3	33' 3.5"	33' 3.5"		33' 4"	33' 4"	
TOTAL DECK AREA	4,663 sf	4,663 sf	4,778 sf	4,507 sf	4,778 sf	4,507 sf
DECK THICKNESS	7.5"	7.5"	7.5"	7.5"	7.5"	7.5"
LONGITUDINAL REINFORCEMENT						
Top Bars	# 4 @ 15"	# 4 @ 15"	# 4 @ 16"	# 4 @ 14"	# 4 @ 14"	# 4 @ 16"
Bottom Bars	#4 @ 11"	#4 @ 11"	#4 @ 9"	#4 @ 9"	#4 @ 9"	#4 @ 9"
TRANSVERSE REINFORCEMENT (cc)	# 5@ 6.5"	# 5 @ 6.5"	# 5 @ 6"	# 5 @ 6"	# 5 @ 6"	# 5 @ 6"
DECK REINFORCEMENT COVER						
Average	1.875"	1.8125"	1.688"	1.719"	NA	NA
Range	1.13"- 2.65"	1.25 "- .38"	1.13"- 2.13"	1.00 "- 2.25"	NA	NA
AVERAGE CL CONCENTRATION (lbs/cy)	8.17 lbs / cy	6.6 lbs /c y	4.66 lbs / cy	3.17 lbs / cy	NA	NA
Cu-Cu SULFATE HALF CELL POTENTIAL						
Average	-0.4.5 V	-0.317V	-0.322 V	-0.277V	NA	NA
Range	-0.144 to -0.748V	-0.119V to - .602V	-0.081 to - 0.597V	-0.005V to – 0.570V	NA	NA
Continuous Bridge with Pre-stressed Concrete Spread Box Beams.	YES	YES	NO	NO	na	na
Continuous with Steel I Beams and Conc. Decks.	NO	NO	YES	YES	na	na

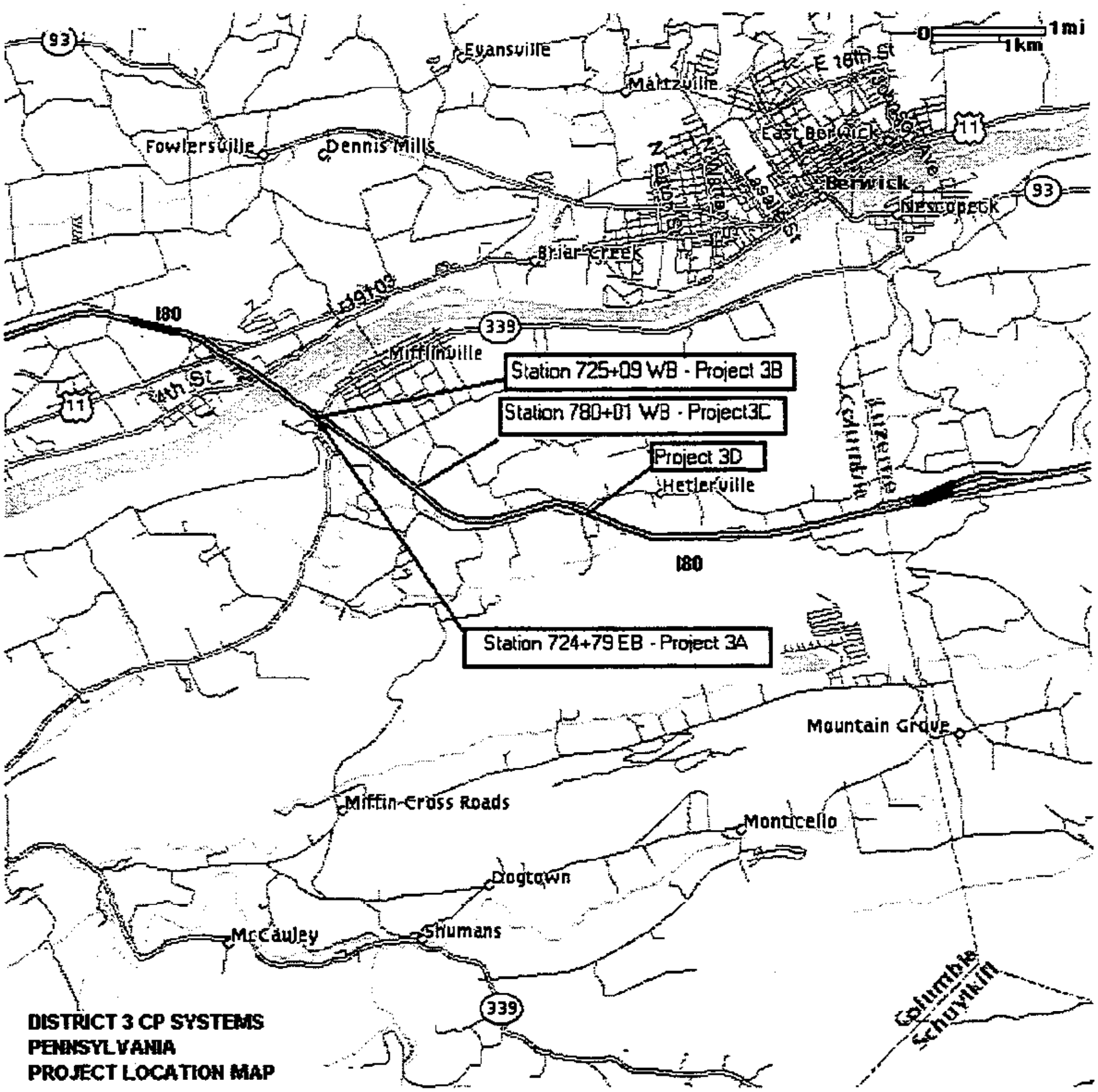


FIGURE 2.1 Locations of Various Bridges with CP Installation in District 3

The objectives of the bridge deck rehabilitation work were to provide a structurally adequate and long-term improvement to the structures. Construction economy, low maintenance requirements, and good riding quality for an extended period of time were also considerations. The use of CP in conjunction with a latex-modified concrete overlay was found to satisfy the objectives.

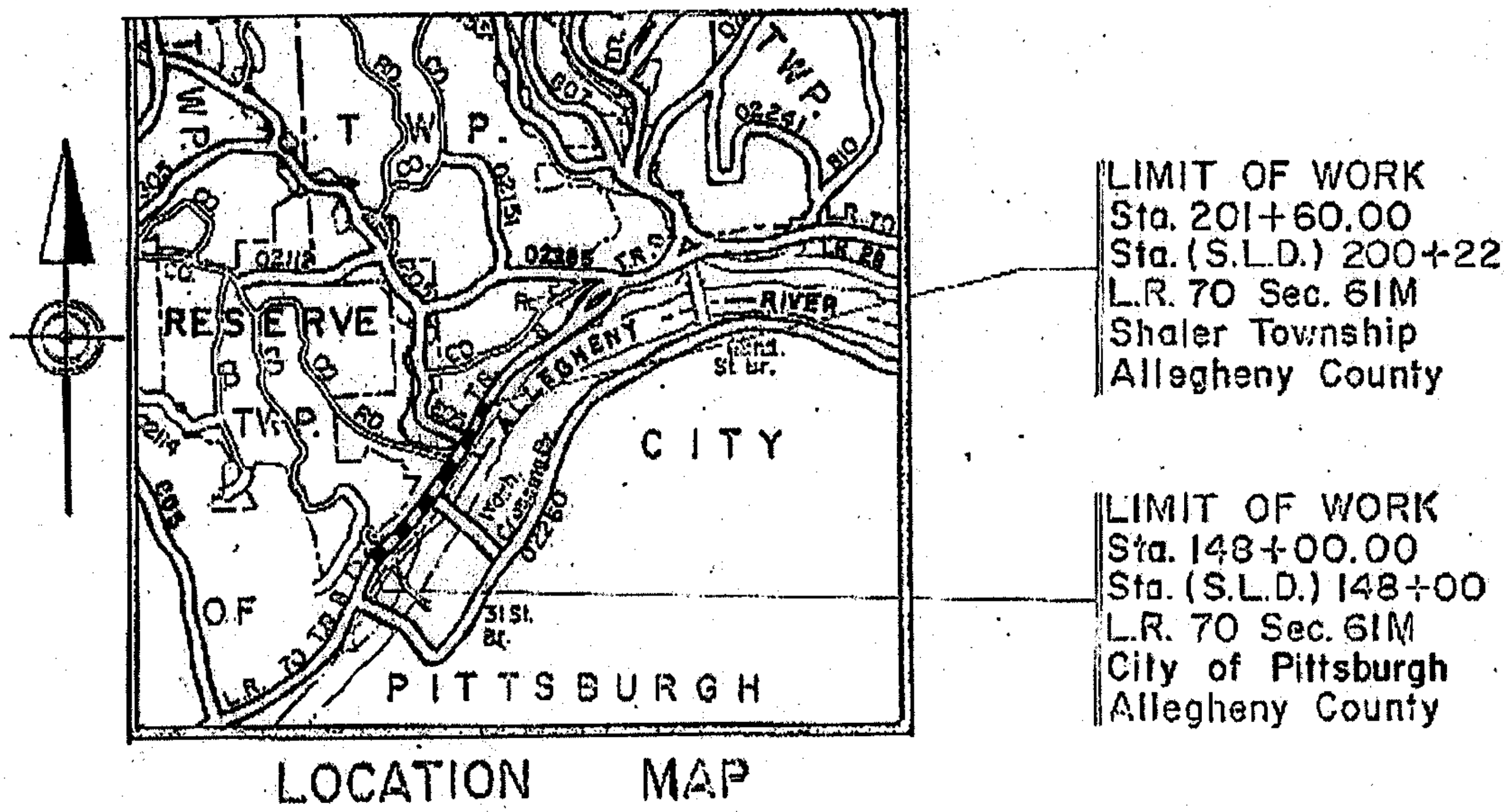
Identification and Objectives of Project – District 5[5]

CP was chosen to extend the life of Pier No. 2 of the I-80 bridge over the Lehigh River (Project 5A), Conrail, and SR 1005 in Carbon and Luzerne Counties. ***Figures 2.2 - 2.4*** show the location of the bridge. The objective of the project was to evaluate the Elgard Anode Mesh System, or equal on a pier application. The intention was to design, install, and have the supplier and his consultants test the system in the field. The study was aimed at noting its ease of application on pier 3 determining its effectiveness in reducing corrosion rates and determining the optimum design aspects in terms of current density hardware. In addition, it was intended for the study to address the types of titanium mesh available, their costs, and effectiveness.

The proposed study include the following:

1. Review and establish a design procedure for the system and the type of application.
2. Evaluate the constructability and report the construction details.
3. Performance study to include its effectiveness as a CP system.
4. Evaluate the Remote Recording System.
5. Cost effectiveness of the system.

Corrosion protection by the titanium anode mesh system is believed to be the state-of the-art system. This system is new, however; once successfully tested in the field, it has the potential to provide an effective and economical protection method for structures. It is anticipated that this project will provide the Department and others with valuable experience in the installation, design, testing, and specification preparation for future use. No follow-up information was available for additional discussion.



LEGEND

CONSTRUCTION
STATE HIGHWAYS
COUNTY ROADS
TOWNSHIP ROADS

FIGURE 2.2

DISTRICT 5 CP SYSTEMS
ALLEGHENY COUNTY
PROJECT LOCATION MAP

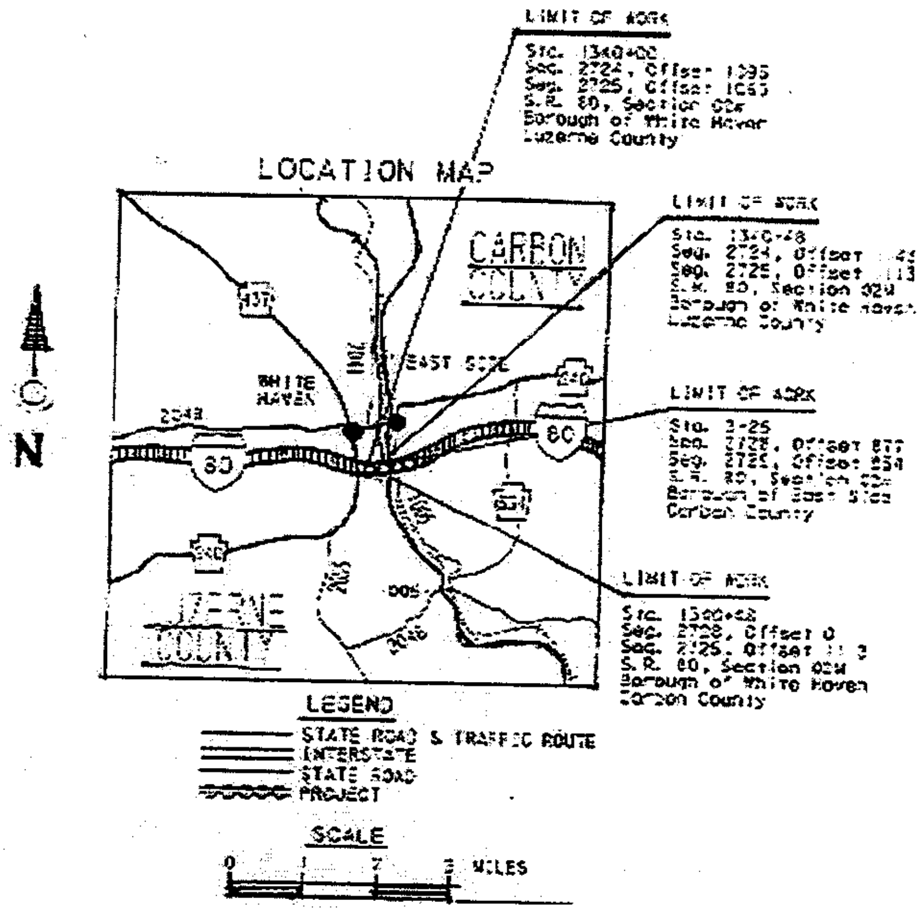


Figure 2.3 District 5 CP Systems: Project Location Map
Luzerne and Carbon Counties, PA.

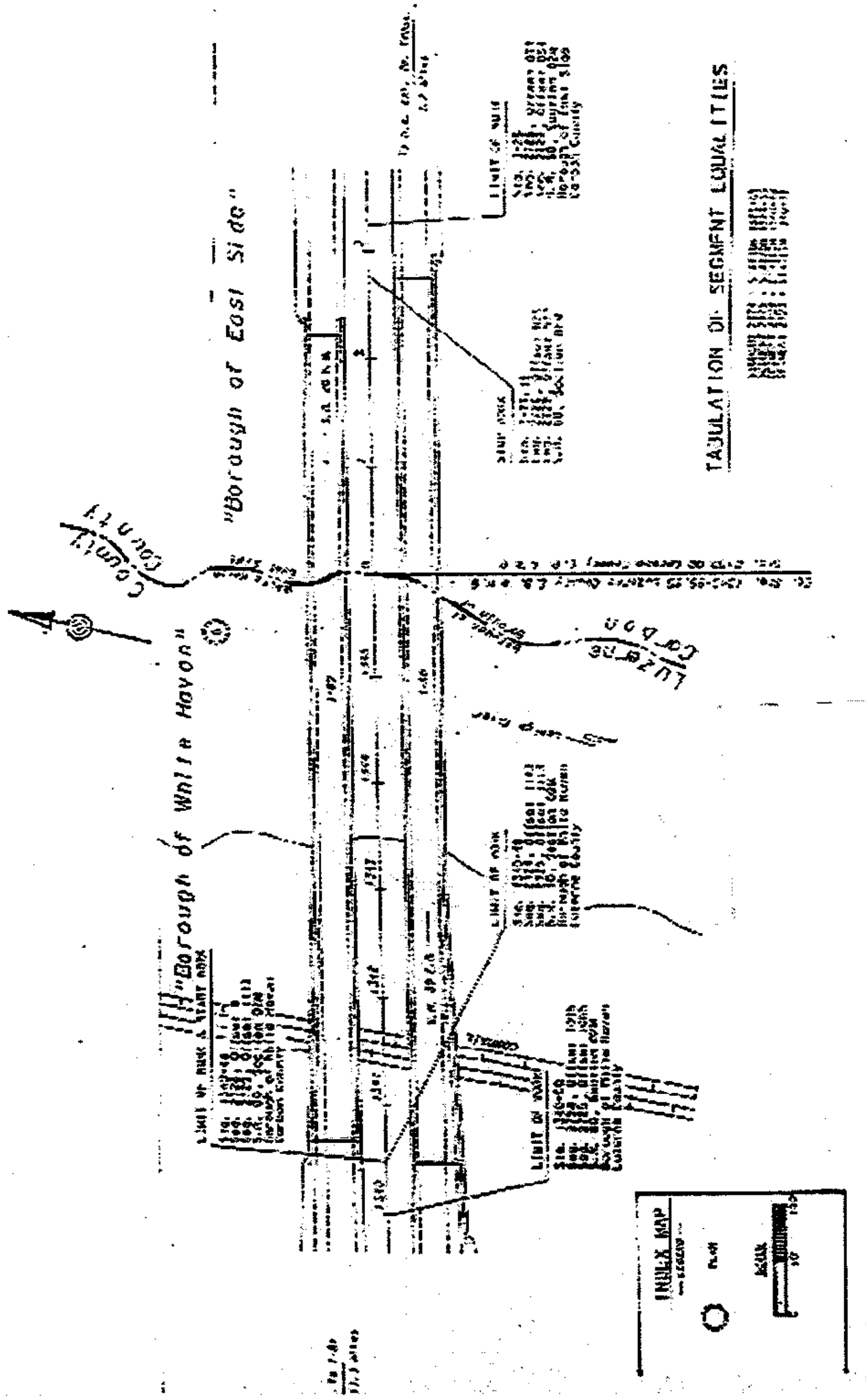


Figure 2.4 District 5 CP Systems, General Site Plan
Luzerne and Carbon Counties, PA.

Identification and Objectives of Project - District 11

Project 11A is located on SR 0028 Ramp A over Ramp B at station 83+66.52. This structure is a single-span concrete bridge with steel W girders. ***Table 2.2*** shows some basic details of this bridge.

Table 2.2 Project in District 11

Width (outside to outside)	29' 6"
Span	45' to 64'
Roadway Area	1500 sf
Deck Thickness	+/- 8"
Cover	1.5" minimum

The structure is a single-span, concrete bridge with steel "I" girders, which was constructed in 1964. The objectives of the bridge deck rehabilitation work were to provide a structurally adequate and long-term improvement that would last at least 25 years. Construction economy, low maintenance requirements, and good riding quality for an extended period were other considerations. CP in conjunction with a latex-modified concrete overlay satisfied most of the objectives.

3. DETAILS OF VARIOUS PROJECTS

3.1 State of the Projects Before Application Of CP

Deck Condition[1, 2,3] - District 3

All the decks of bridges 3A to 3D experienced the expected wear and tear of nearly 20 years of interstate traffic and numerous applications of deicing chemicals. The chloride ion content averaged well above concentrations of 2 pounds per cubic yard of concrete, which is considered critical to promote the corrosive environment. The half-cell potential readings demonstrated that active corrosion was undoubtedly widespread, and the extent of the delaminations confirmed this. Open spalls and bituminous patches existed heavily at random locations throughout all four decks. In addition, light efflorescence was exuding from the hairline cracks on the underside of the decks. The deck expansion joints were also in poor condition and leaked heavily. The concrete edges of sections of the neoprene sponge-rubberized sealer-type joints were broken away.

Prior to 1986, three separate inspections were carried out for each of the decks in 1977, 1980, and 1984. In 1984, inspections were completed while the construction work was underway and provided up-to-date data on the decks at the time of the rehabilitation. Tests on decks 3C and 3D were completed after the contractor removed the remaining portions of the synthetic resin "Pavebrite" wearing surface that was placed in 1968.

The three separate detailed deck inspections that were performed on the bridges between 1977 and 1984 in District 3 included the following:

a) Sub-surface concrete delamination test - A chain drag was run over the entire deck surface to detect sub-surface fractures planes at the level of the top mat of reinforcing bars. The delaminations were measured, located, and then plotted on the plan of the deck ***(See Figures 3.1 - 3.4 Deck Map Of Concrete patches – Chain Drag.)***

b) Depth of concrete cover test - A pachometer was used at random locations through the deck to determine the actual depth from the concrete surface to the top reinforcement layer. Similar test locations were used for the chloride ion test.

c) Chloride ion penetration test- Test holes were drilled to within 1/2 an inch of the top layer of rebars. Concrete dust samples were recovered from the last 1/2 an inch of concrete over the bars. The samples were forwarded to the department's Materials and Testing Division for determination of the chloride ion content ***(See Figures 3.5 – 3.6).***

d) Copper-Copper Sulfate half-cell potential - The test was conducted in 1984, separate from the deck inspections. It was performed using a three-foot grid. Readings were recorded at each test point and plotted on the deck plan. The extent and probable severity of corrosion was determined as shown by the results in ***Tables 3.1 and 3.2 below.***

Table 3.1 - Deck Test Data

Bridge	Test 1[a] 1977	Test 1[a] 1980	Test 1[a] 1984	Test 2[b] 1977	Test 2[b] 1980	Test 2[b] 1984	Test 3[c] 1977	Test 3[c] 1980	Test 3[c] 1984
3A	7.49	7.07	41.3	5.84	8.42	8.17	1.55	1.76	1.88
3B	1.66	3.64	18.9	3.41	9.20	6.63	1.62	1.32	1.81
3C	0.11[d]	10.0[d]	32.9	4.37	5.76	4.66	1.29	1.65	1.69
3D	2.7[d]	10.0[d]	56.5	3.33	3.84	3.17	1.48	1.90	1.69

a - Deck Deterioration (%)
b - Average Cl Ion (lb/cy)
c - Average Rebar Cover (in.)
d - True Deck Deterioration could not be determined due to presence of Synthetic Resin Overlay. 1984 test performed after removal.
Notes
Rebar covers are pachometer measurements
Some half-cell readings were very low possibly due to excessive delamination under the test points.

Table 3.2 – Half-Cell Readings - 1984 test only (Volts)

Bridge	High	Low	Average
3A	-.748	-.144	-.405
3B	-.602	-.119	-.317
3C	-.597	-.018	-.322
3D	-.570	-.005	-.277

The variability and apparent inconsistencies in the chloride ion readings and reinforcing bar covering data from one inspection to the next are apparently due to the random selection of test locations and the need to conduct the tests in areas of reasonably sound concrete. Some of the half-cell readings did not indicate a high probability of corrosion in several areas where it was apparent that corrosion was indeed occurring. As a result of the poor condition of the surface of the existing decks, a new wearing surface was deemed necessary for several bridges in District 3. The decks exhibited moderately heavy surface wear, poor riding quality, and minimal reinforcement cover. In addition, two specific bridges were previously provided with overlays in an attempt to seal cracks that developed shortly after construction.

<u>Decks 3A and 3B</u>

Practically the entire deck surfaces for these structures showed evidence of severe spalling to the extent that rebars were visible. As much as 41% of the deck surface of Bridge 3A and 19% of Bridge 3B were delaminated. Several large concrete patches were found on the underside of the decks. In addition, hairline cracks and mild efflorescence were found.

The expansion joints for the structures were constructed of pre-molded filler, which were found to be in poor condition to the extent that they were leaking. It was determined that the leaking was contributing to the deterioration that was being experienced in the sub-structure.

<u>Decks 3C and 3D</u>

The existing synthetic resin deck overlay for these bridges was found to be in poor condition with severe wear and peeling throughout out the surfaces. Upon removal of the overlay, 32% and 52% of the reinforced concrete deck surface was found to be delaminated for Bridges 3C and 3D, respectively. The underside of the decks were in a similar state as Bridges 3A and 3B with efflorescence extruding from hairline cracks. In addition, several full depth patches were found in these two structures.

The expansion joints were similarly constructed as with the previous bridges with pre-molded filler. Those located at the abutments were found to be in poor condition to the extent that they were leaking. Apparently, this has been a contributor to the deterioration of the abutment.

The use of a "mound cathodic system" was determined to be the best approach for rehabilitation since it posed no significant problems for the sites, and it was economical. The system consists of patching and scarifying the deck and placing the CP system on top. The approach minimized the possibility of electrical short circuits from forming as a result of lack of reinforcement cover. In addition, it eliminated the costly cutting of slots in the deck and was consistent with the need to provide a new wearing surface.

The Contract Special Provisions for the project were derived from those used for the Department's second slotted system; these specifications were developed by Stratfull and Clear as part of their contract with the FHWA.[6]

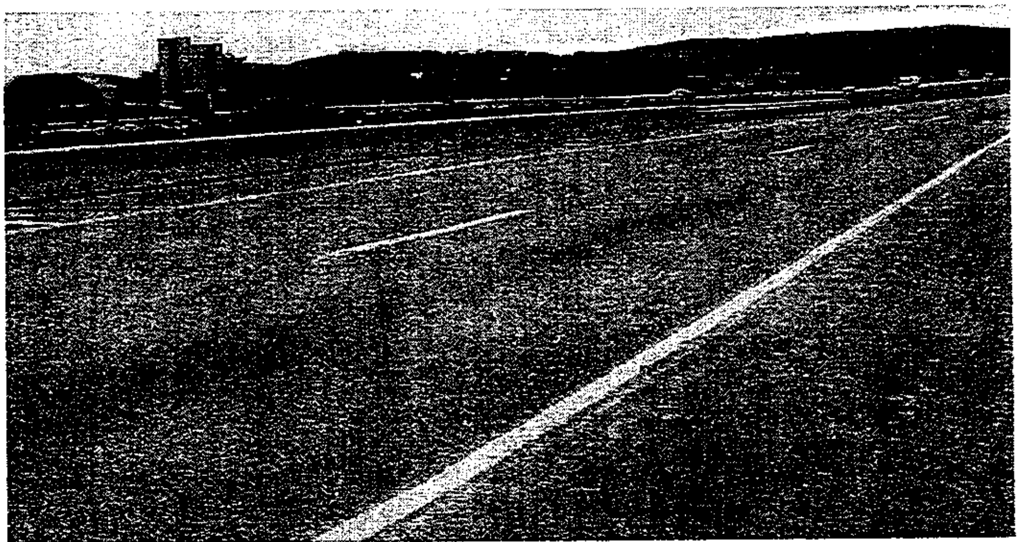


FIGURE 3.1 PROJECT 3A - CP BRIDGE DECK: OVERVIEW

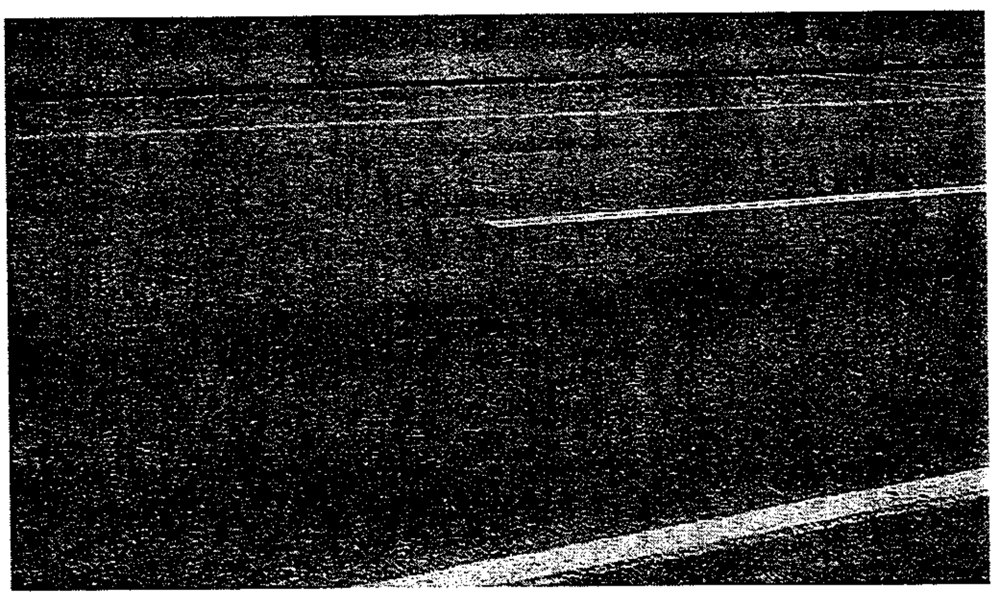


FIGURE 3.2 PROJECT 3B – CP BRIDGE DECK: LONGITUDINAL CRACKING IN DECK

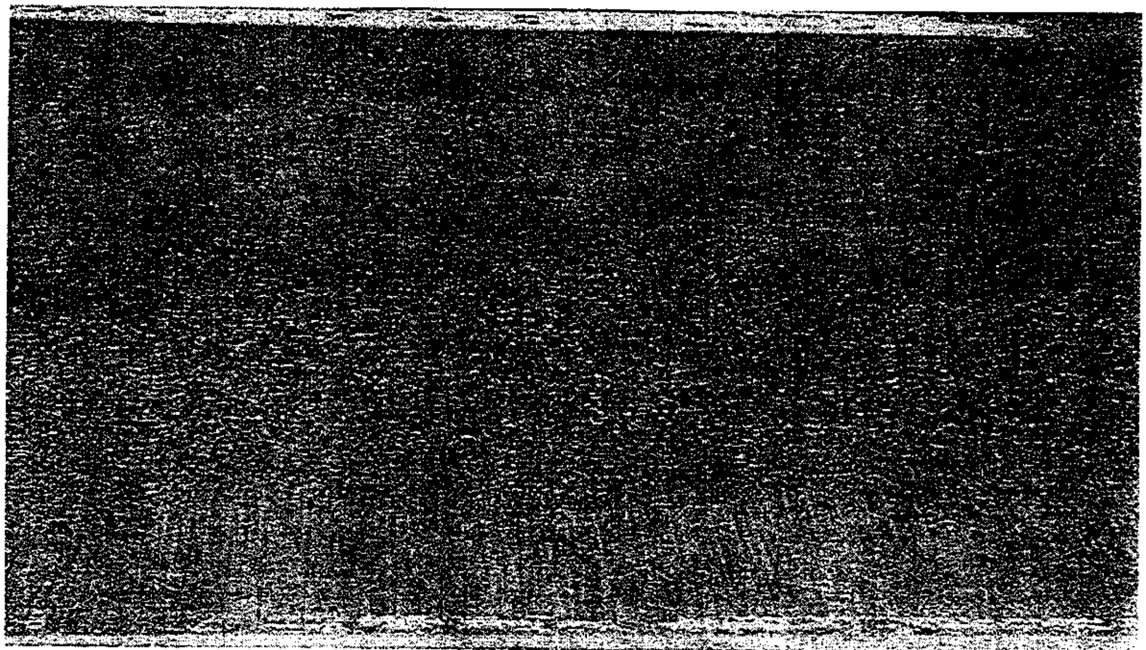


FIGURE 3.3
PROJECT 3C – CP BRIDGE DECK: SMALL TRAVERSE CRACK

FIGURE 3.4
PROJECT 3D – CP BRIDGE DECK: LONGITUDINAL CRACKS

Control Decks

The two control decks (3E, 3F) were rehabilitated with latex-modified overlays. These bridges experienced the same level of wear and tear as the other four decks; however, as a result of the sealing characteristics of the overlays, the chloride content was below the threshold of 2 pounds per cubic yard of concrete, and amount of total spalls and or delaminations was also well below that of the other decks.

Bituminous patches existed at random locations on both decks, with one open spall on the 3E deck. The epoxy overlays were heavily worn in the travel lanes. Deck 3E was heavily spalled, and the deck concrete surface was exposed. The undersides of the decks were cracked moderately in a transverse direction every 4 feet to 10 feet with efflorescence visible. The overall condition of both structures was found to be fair to good.

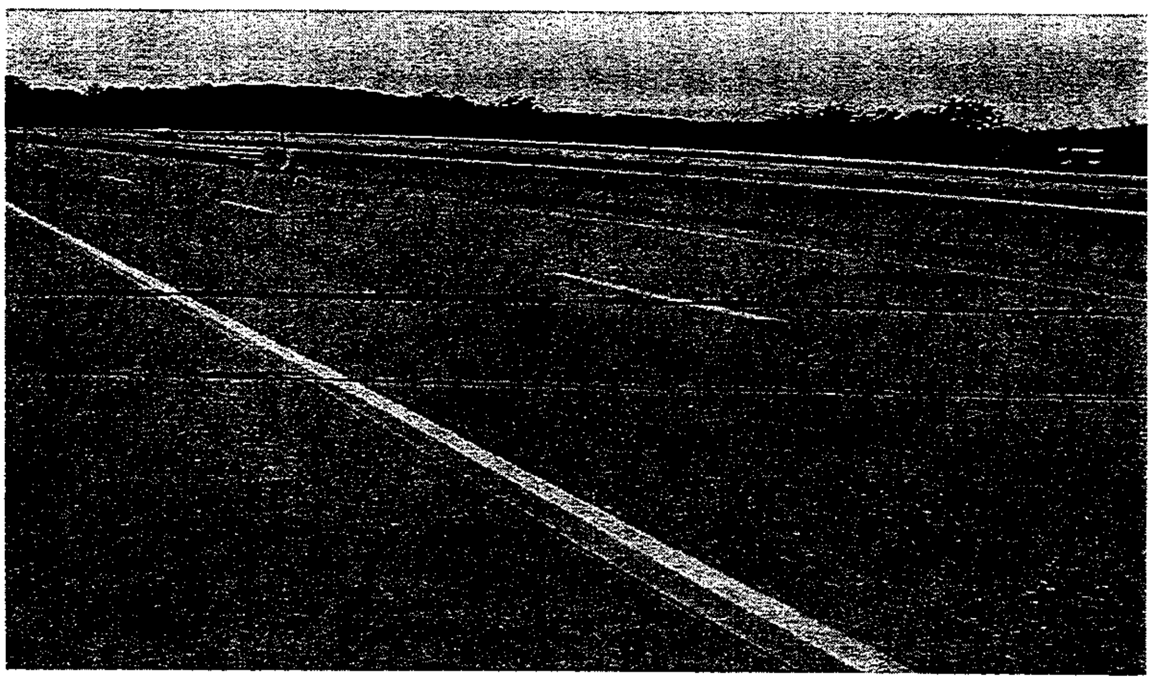


FIGURE 3.5 PROJECT 3E - NON-CP DECK: EVIDENCE OF SPALL IN DECK

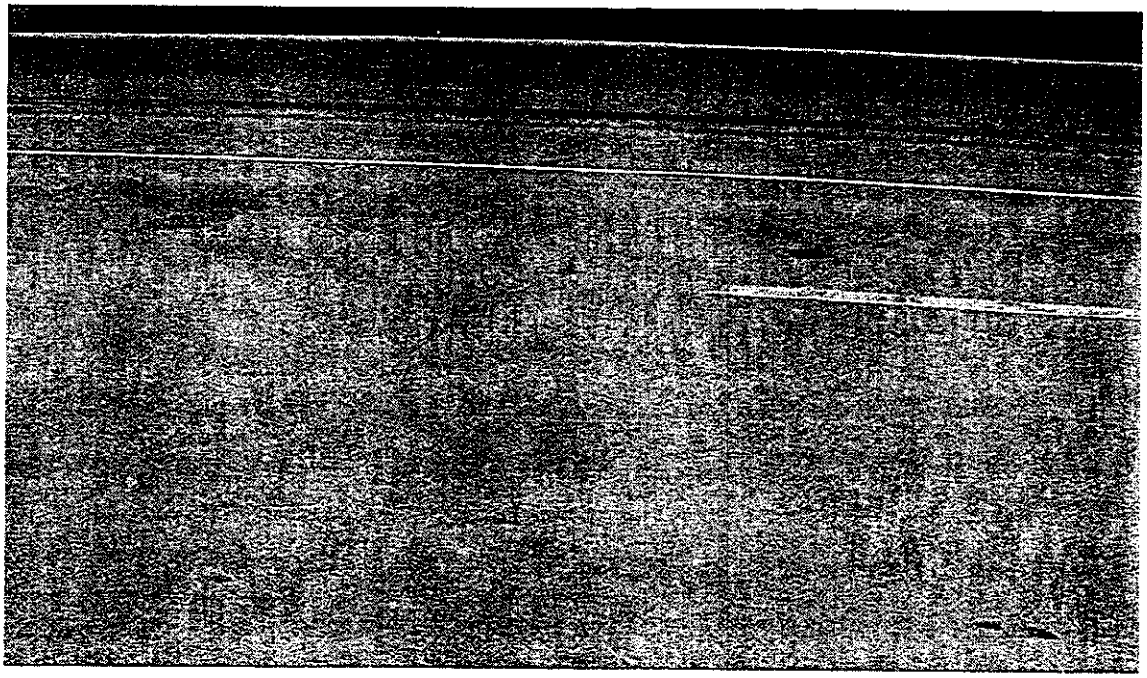


FIGURE 3.6 PROJECT 3F – NON CP-DECK: EVIDENCE OF SPALL AND TRAVERSE CRACKS

Pier Condition - District 5[5]

Pier no. 2 of the I-80 bridge over the Lehigh River, Conrail, and SR 1005 in Carbon and Luzerne Counties showed signs of deterioration of concrete and needed treatment. CP was chosen as the best method for prolonging the life of the pier.

Deck Condition - District 11[3]

The bridge in question exhibited moderate surface wear, poor riding quality, and a minimum reinforcing bar cover of one and a half inches. As a result of the poor condition of the existing deck surface, a new wearing surface was deemed to be necessary.
The chloride ion content ranged from 1.25 to 12.31 pounds per cubic yard of concrete. The half-cell potential readings demonstrated that active corrosion was undoubtedly widespread. In addition, the existence of deck delaminations confirmed the indications of active corrosion.

3.2 Construction Scope And Outline of Specifications[2]

District 3

The protective system that was required for these projects had to be compatible with the new rigid surface that was to be installed. Based on the Department's and industry experience, it was determined that anodes applied to the surface of the decks would adequately protect the top mat of deck reinforcement. Longitudinal anodes with 12-inch spacing were selected based on the electrical resistance of the deck concrete and the need to limit current density to maximize the life of the system. A constant current operational mode was specified because of the questionable long-term reliability of reference cells that are needed in potentially controlled systems.

Electrical interconnections were provided where possible to eliminate power supply problems that have occurred in the past with electrically separate and non-redundant zones. Since bridges 3A and 3B each had separate decks, a degree of redundancy was provided by two additional transverse carbon strand anodes placed near the ends of the deck in each span. However, the energizing of each deck was dependent upon the lead wire supplying power to at least a portion of the single transverse primary anode that was located near the centerline of each span. Greater redundancy was possible on bridges 3C and 3D because their decks are continuous without joints. The three transverse primary anodes at 35 to 40-foot spacing, and the transverse carbon strand anodes placed near the ends of the deck, provided multiple electrical interconnections.

As a result of high cost, the use of the field-tested and effective palatalized niobium copper core wire was limited to the primary transverse anodes on each bridge. These anodes distributed power to each of the continuous longitudinal secondary anodes. As a result of its economy and conductivity, carbon strands were used for the secondary anodes. The resistance of these anodes was designated to be not more than 1 ohm per linear foot. For further redundancy, carbon strands were also placed with copper wires in the primary anodes. The FHWA conductive polymer grout was used to help distribute the cathodic protection current, to minimize current densities at the deck and wearing surface/ concrete interfaces, and to bond and protect the anode wires and strands. The CP system specification requirements were detailed in the Contact Special Provision. The specifications were based on the department's past experience and on those developed by Strafull and Clear for the slotted system and were installed on another bridge in 1983. HARCO CORPORATION, Corrosion Engineering Division, Hatboro, Pennsylvania, provided the detailed CP system designs in accordance with the contract requirements.

The improvement project, including the bridge rehabilitation work, was carried out between September 1983 and February 1985. The bridgework included deck repairs, scarification and blast cleaning, overlay application (latex-modified concrete), installation of armored preformed neoprene compression dams, and the installation of the CP system. The repair of the substructure was also included.

Construction Details

The CP of bridges was carried out by closing only single lanes to minimize traffic interruption and by placing a precast concrete median barrier across the bridge on the outside lane beside the pavement centerline and in the shoulder area. Delaminated areas were outlined by the Department's construction inspector and contractor's representative. A 3/4-inch-deep saw cut was used to delineate removal boundaries and to provide a suitable edge for patching. Lightweight jackhammers (30-pound class and less) were used to remove the delaminated and unsound concrete and to slightly undercut the edges of the sound concrete. The existing concrete surfaces in the patch areas were blast cleaned and then coated with an epoxy-bonding compound just prior to placing the patching material. The Department's Class AAA Cement Concrete (f'c = 4,500 psi.) modified to use IB (AASHTO No. 8) coarse aggregate was used in all patches.

After the patches were cured, the decks were scarified to remove a minimum of 1/4 inch of the concrete surface. Pachometer was then used to verify that concrete cover still existed over all reinforcing bars and that there were no electrical shorts. In areas where the reinforcing bar cover was found to be less than 0.5 inches, a non-conductive epoxy was applied to electrically insulate the steel from the anode materials. At numerous locations, reinforcing bar tie wires were found to be at or very near the surface. A dab of epoxy material was used to insulate each of these potential problematic areas. The anode wires/strands were then covered with a continuous bead of conductive polymer grout (HARCO Anode-Crete). The beads were 1/2 inch high and had a cross sectional area of at least 0.6 square inches. Prior to curing, conductive carbon filler was sprinkled on the surface of the grout using a garden-type watering can. The prepared bridge decks were then blast cleaned and 1-1/4-inch-thick latex-modified concrete (LMC) wearing surface applied.

Monitoring Equipment

Several items were installed in the decks of each of the bridges for monitoring purposes. These devices included silver/silver chloride reference cells and Rohrback reinforcing bar corrosion probes. Lead wires from these devices and connecting wires to the rectifier-controller were terminated or spliced in the under-deck test stations. The splice between the primary anode wire and its insulated lead wire occurred in a flush deck-mounted test station.

Rectifier/ Controller

The CP systems were energized by three- and six-circuit automatic TIE rectifier/controllers, manufactured by the Goodall Electric Company. They are capable of operating in an IR-Drop free potential control, constant current, or constant voltage mode.

Bridges 3A and 3B were served by a single six-circuit unit with a maximum output rating of 40 volts, 48 amperes D.C. per circuit. Bridges 3C and 3D were serve by individual three-circuit units having a maximum output rating of 40 volts, 24 amperes D.C. per circuit. Each rectifier circuit was provided with a low voltage red warning lamp. The lamps were wired so that they would be "on" when the cathodic protection system was functional.

Stray Current Testing

The construction contract included a requirement that the contractor develop and implement a method for monitoring and measuring the stray currents in the prestressing strands within the spread box beams on bridges 3A and 3B. The methods developed and the results of the tests are detailed in HARCO's "Post Installation Test Report." There was an indication of very small corrosion activity on the outer strands, in part due to its electrical discontinuity with the inner strands and deck. Bonding of the outer to the inner strands was not recommended because the very minute amount of the applied CP current was being "picked up" on the reinforcing mats in the deck.

Monitoring Requirements

HARCO Corporation calibrated and set the systems and conducted initial testing. They documented this in their "Post Installation Test Report." The controllers were set to provide electrical current as shown in Table 3.3.

Table 3.3 Controller Output

Bridge	Circuit Number	Current Output (Amps)	Current Density on Deck Surface Area (mill-comps/S.F.)
3B	1	1.0	0.64
3B	2	1.2	0.77
3B	3	1.1	0.71
3A	4	1.5	0.96
3A	5	1.4	0.90
3A	6	1.0	0.64
3C	1	1.0	0.63
3C	2	1.3	0.81
3C	3	1.2	0.75
3D	1	2.5	1.66
3D	2	2.3*	1.53
3D	3	2.1	1.39

*Based on average results from circuits 1 and 3

It was the responsibility of the Department's District Engineering Office to monitor each of the systems and to ensure that they were functional and were provided with adequate protection. Monitoring activities were to be as follows:

1. Visually inspect decks and systems.
2. Measure and record.
3. For each circuit - electrical current output and voltage and the set and reference potentials.
4. Reinforcing bar probe potential.
5. Corrosion-meter probe reading.

In addition, the Department of Bureau of Bridge and Roadway Technology was consulted whenever the current output readings varied significantly from the set values listed above or when the probe readings changed. The rate of corrosion is determined by evaluating the changes in the corrosion-meter probe readings. The pre-stressed spread box beams were inspected at least once a year for cracking or for any stains, which could indicate corrosion activity.

District 5

This particular project included the installation of a CP system consisting of Titanium Anode Mesh, rectifier, and instrumentation including a remote monitoring system approved by the Department. The system designer and the engineer were responsible for the design and supervision of installation and testing. They were required to have at least 5 years of field and design experience in this type of work, be completely familiar with the system, and be pre-approved by the Department 30 days prior to the commencement of any work.

Initially, the surface was prepared and the reference cells were installed with the system negative connections made to the rebars. The titanium anode mesh system was then installed and was followed by shotcrete. The sequence of operations was jointly prepared by the contractor, supplier, and engineer and approved by the Department. It allowed for a smooth operation of the entire installation and as testing of the system. The Contractor and the consulting engineer were responsible for identifying the location of negative connections to the steel and for justifying plans for Department's approval. All spalled concrete and other loose material were removed using approved methods. Tests were conducted to ensure that there were no shorts in the system; any found were to be corrected. All equipment met the relevant specifications. Rectifiers and other equipment were protected from weather and vandalism and located on pier 6, Eastbound. They were to be placed 5 feet above ground so that they were accessible if necessary. A remote reading system was used so that the readings could be taken by telephone using an approved computer and software system.

Testing, Readings, and Equipment

All testing and readings were done using equipment and methods specified. Unless otherwise specified, equipment, testing, and data recording were done using methods indicated in FHWA Demo Project No. 84 (September 1991) titled "Corrosion Detection in Reinforced Concrete Bridge Structures." The readings were reported on approved forms. The Contractor/supplier had to submit a list of test equipment, readout equipment, remote data collection system, forms and method, and frequency of data collection for the Department's approval six weeks prior to construction.

District 11[3]

The construction project was carried out between October 1985 and January 1987. The work included deck repairs, scarification, cleaning, the application of a latex-modified concrete overlay, and installation of safety-shaped parapets, beam repairs, installation of new compression seal and installation of the CP system. The Prime Contractor for the overall project was Brodhead Construction Company of Pennsylvania. The installation of the CP system was subcontracted to Corrpro Companies of Ohio.

Delaminated deck areas were identified by the Department and confirmed by the contractor. Saw cuts of about 0.75 inches deep were used to mark out the boundaries of the delaminated areas. Light-rated jackhammers with 30 pounds of weight were used to remove unsound concrete, as well as to undercut a portion of sound concrete. Silica sand was used to blast clean the concrete surface prior to patching with Class AAA cement concrete ($f'c$ = 4,500 psi). The deck surface was then scarified to remove a minimum of 0.25 inches of concrete after curing. A pachometer was utilized to verify that the concrete cover was acceptable. In areas where the cover was found to be less than half an inch, a non-conductive epoxy was applied to form an insulation layer. It was noted that numerous bar ties were found to be near the surface. Epoxy was used to insulate each of the potentially damaging areas. Next, primary anode wires were installed in longitudinal directions. The system was connected with reference cells that previously had been installed in the decks. The overlay that was then placed included a layer of latex-modified concrete 1.25 inches thick to function as the main wearing surface.

3.3 Construction Problems and Service Life Performances

Construction Problems

For the installation of the Coke Breeze system with anode mounds, which was one of the first CP system used in District 3, it was reported that the contractor had a lack of control during the placement of the 'anode-crete' mounds[2,4]. This led to problems in the placement of the $1^1/_4$ inches thick latex-modified concrete overlay because the grout mound was found to be higher than the specified 1/2 inch in certain places. In certain cases, the coarse aggregate (AASHTO No. 8) would be dragged across the mounds by the screed-finisher and would result in 'opened surfaces.' Hand finishing would then be required to repair these areas. This lack in control was reported to be a result of the lack of fluidity of the grout and the inability to mechanically consolidate it with the supplied equipment. This resulted in the thickness exceeding the specified depth. In addition, small voids are suspected to have existed at the strand-deck interface. This also could have contributed to the indication of delamination.

The polymer grout bonded to the scarified deck very well, however, the bond between the grout and the latex overlay was not as good. Sounding of the latex surface revealed hollow areas in locations by some of the grout strips. Upon removal of the latex-wearing surface in these areas, its bond to the grout was found to be much weaker than that between the grout and deck.

The Department's construction inspectors also had difficulty monitoring the installation with confidence. They were unfamiliar with cathodic protection in general and were unsure of what was to be done and what quality of work was expected.

Several problems were experienced with the slotted system that was employed after the mound system in the District. Inadequate protection was found in one particular deck that utilized the slotted system with spacing of 24 inches.

CP System Performance[4]

Latex-Modified Concrete Overlay (LMC)

Approximately 20 months after the reconstruction, hairline cracking of the LMC overlay in Bridges 3A to 3D developed directly over the secondary anode mounds running in a longitudinal direction. The cracking existed at random locations on all four CP bridge decks and was most evident in the travel lanes. The cracks have not widened since their initial detection, but their number has increased.

Rectifier/Controller

The rectifiers were set up to operate in the constant current mode. Approximately 12 months after the systems on decks 3A to 3D were energized, it was found that the rectifiers automatically shut off the power to five zones. This problem was attributed to the reference potentials exceeding the -1.700-volt maximum set potential in each controller. This problem was overcome by placing a jumper across the reference cell and structure terminals in the affected circuits. As a result, the reference cell reading had to be taken with a portable voltmeter instead of the rectifier meter. Each rectifier circuit was provided with an externally mounted low voltage red warning lamp. These lamps were wired so that they were "on" when the cathodic protection systems were functioning and became a maintenance problem due to the short bulb life. The warning systems were turned off due to the maintenance problems and the isolated location of the rectifiers. Each rectifier has an RMS volt/amp meter with a digital readout display and selector switches for monitoring. Initially, this was convenient, but after prolonged use, accuracy became a problem, and a portable voltmeter had to be used for monitoring. Additionally, one of the meters failed and was replaced by the manufacturer at no cost to the Department.

Two of the rectifiers were located beneath the structures in areas not visible to the traveling public. Vandalism has been an ongoing problem because of the isolated locations.

Test Stations

Individual test stations were provided under the deck of each rebar probe and corrosion probe. Water seepage into one test station caused corrosion and ice build-up. This problem was immediately corrected by drilling drain holes in the PVC conduit and test station box. At one point, mice gained access into one test station conduit and damaged the corrosion probe lead wires, making it inoperable. All the test stations are located at the top of dirt sloped walls, which has proven to be inconvenient and time consuming when monitoring.

Data Interpretation

The lack of the required expertise, training, and the staffing needed to interpret data and solve problems appear be the District's foremost problem for properly maintaining the CP systems. Although many corrosion experts from the private sector have been helpful in solving problems when encountered, the District has had great difficulty in interpreting the compiled data that may indicate a problem.

Problems With Other CP Systems[4]

Subsequent to the earlier contract given out in 1983, District 3-0 installed CP systems on eight other structures located throughout the District. Various anode systems have been constructed on these bridges. Following each subsequent CP installation, the District reviewed and modified the contract's special provisions to minimize or eliminate problems encountered during construction. The following is a summary of those installations.

Raychem Ferex 100 Anode System

This system was used in 1986 on a three-span continuous steel I-beam bridge (41-0015-0400-0816) carrying S.R. 15 over Lycoming Creek in Hepburnville, Lycoming County. This bridge was built in 1967 and has a length of 169 feet. The deck CP system consisted of a Raychem Ferex 100 Anode System with a Latex-Modified Concrete overlay. The Ferex 100 Anode mesh is a flexible polymeric mesh of anode strands attached to the deck with pushpin fasteners. During construction, several problems were encountered. One involved initial brooming of the latex onto the prepared deck surface. This was done to achieve the glue-like adhesion typical of the latex material. Based on previous experience, it was anticipated that the coarse aggregates would be segregated from the mix and would congregate along the anode mesh of the C.P. mats. This would potentially cause weak bonding in such areas. To avoid this problem, the LMC was hand mixed and the coarse aggregates were omitted for the initial brooming operation; this created a minor problem with quality control.

From the time the system was energized in August 1986 to September 1989, it was fully operational approximately 25% of the time. Initially, this was due to delays in obtaining an operating manual for the system, but it was largely due to problems with the rectifier cards burning out. Lightning arresters were replaced in 1989, and several rectifier cards were refurbished.

Elgard 150 Anode Mesh System

This system was used on a three-span P/S I-beam bridge (59-0080-2070-0592) carrying Interstate 80E over S.R. 1010 (White Deer Pike) in White Deer Township, Union County, in 1992. The bridge was built in 1970 and has a structural length of 168 feet. The deck CP system consists of Elgard 150 Anode Mesh with a Latex-Modified Concrete overlay. The Elgard 150 Anode mesh used is a highly expanded titanium mesh that has a mesh surface area of 0.15 ft^2 mesh/f t^2 concrete area. The mesh was obtained on rolls 3.75 feet wide. This was used to cover the entire deck surface and was fastened to the deck with pushpin fasteners at a spacing recommended by the manufacturer. Although these were fastened down according to the manufacturer's specifications, the weight of the mesh caused major problems in the placement of the LMC overlay. The mesh expanded, moved, and buckled when brooming in the grout. During placement of the overlay, the mesh was caught by workers' feet, rakes, and in one case by the Bidwell finisher. The CP subcontractor was kept very busy repairing and refastening the mesh to the deck surface during the overlay process. In some instances, it was impractical to adequately repair the mesh. As a result, certain small areas were cut out so as not to delay the placement or finishing of the overlay. The contractor's unfamiliarity with latex placement, finishing problems with "flash setting" of the latex, and the hot weather combined with the cathodic mesh problems resulted in a very poor job.

Elgard-210 Anode Mesh System

This system was used on four Interstate I80 WB bridge rehabilitation projects between 1992 and 1994. The first was a three-span R.C. slab bridge (47-0080-2181-0093) over S.P. 3003 in Liberty Township, Montour County. Built in 1963, this bridge has a structure length of 110 feet. The second of these bridges is a three-span P/S I-beam bridge (59-0080-2071-0770) that spans over S.R. 1010 (White Deer Pike) in White Deer Township, Union County. This bridge is a twin to the Interstate 80 EB Bridge rehabilitated using Elgard 150 Anode Mesh. Built in 1970, this bridge also has a structure length of 168 feet. The third of the bridges is a three-span P/S spread box-beam bridge (59-0080-20712179) over white Deer Creek in White Deer Township, Union County. Built in 1970, this bridge has a structure length of 135 feet. The fourth of the bridges is a single-span R.C. slab bridge (59-0080-1991-1132) that spans over S.R. 4001 in White Deer Township, Union County. Built in 1970, this bridge has a structure length of 32 feet.

The deck cathodic protection system used on these bridges consisted of Elgard 210 Anode Mesh with a Latex-Modified Concrete overlay. Heavier than the 150 Anode mesh, the 210 Anode mesh has the same material properties and allows a higher current density. The Elgard 210 mesh was used to alleviate the construction problems incurred with the 150 mesh. The 210 Anode mesh comes in four-foot-wide rolls and was fastened to the deck with pushpin fasteners at spacing exceeding that recommended by the manufacturer.

During the placement of the overlay, the 210 Anode mesh showed some improvement over the 150 Anode mesh, but the same constructibility problems remained.

Corrpro Delam-X Strip Mesh System

This system was scheduled for installation on two interstate 80 bridges that were rehabilitated in 1995 and 1996. Both bridges are three-span continuous R.C. slab bridges (47-0080-2164-0028 and 47-0080-2165-0012) spanning over S.R. 3013 in Liberty Township, Montour County. Built in 1962, these bridges have structure lengths of 110 feet. The Delam-X Anode Strip Mesh used was a 24-inch-wide and 1/8-inch-thick and 40-foot-long titanium strip with a precious metal oxide coating. Concrete current densities were controlled by the center-to-center spacing of the mesh strips.

The eastbound bridge was completed during the 1995 construction season, while the westbound bridge was completed in 1996. The strip mesh was placed on the eastbound deck at a 12-inch spacing and was fastened to the deck at 2-feet intervals. Visually, the strip mesh appeared to be much heavier and more suited for resisting buckling than the roll mesh previously used; however, the strip mesh still moved during the grout brooming operation. In addition, it floated to the surface during the overlay placement. In at least one area, the anode mesh was seen on the surface of the finished overlay.

4. ECONOMIC CONSIDERATIONS

4.1 Introduction

It should be appreciated that the costs for the various systems are dependent on the various items required, such as distance to electric power, depth, width, and area of saw cut slots required. For example, in the mid-1980's, for a deck of 5,000 square feet, slots may have cost $2.00 per linear feet. This may have been significantly lower for a larger deck. In addition, the details of the specifications also played a major role. For instance, in 1984, the cost of the 0.064-inch-diameter palatalized niobium copper wire was approximately $7.00 per foot, compared to $1.20 to $1.50 per foot for the 0.031-inch-diameter wire. Use of carbon strand reduced the cost from 3 cents per foot to 10 cents per foot for the dacron-braided strand. Furthermore, the approach of placing the CP system below an overlay, such as dense low slump concrete or latex-modified concrete, has eliminated the need for the sawed slots and has also helped in the reduction of installation costs.[16] While it is appreciated that different systems will perform differently, the net effect of cost and function should always be considered.

The following exercise illustrates an evaluation of six bridges in District 3 that were rehabilitated and whose details form a part of this report. These bridges were selected based on their similarities. They are all located in the same area, carry similar volume of traffic, and most important, required rehabilitation works of approximately the same nature and scope at the same point in time.

It is not the purpose of this exercise to demonstrate with any level of high accuracy whether CP systems are more economically viable than conventional approaches used to rehabilitate Bridges 3E and 3F. However, the exercise gives an indication of whether the system is competitive in the District and worth the effort of further refinement.

4.2 Initial And Maintenance Costs

The following table includes cost estimates for the installation of CP systems and associated works for Bridges 3A to 3D in District 3. The total construction costs were obtained from the Initial Report prepared for the project and includes costs for the following works

1. Deck repairs
2. Scarification and Blast-Cleaning
3. Latex-Modified Overlay Installation
4. Modification of Existing Parapets
5. Installation of Compression Dams
6. Installation of CP System

In addition, because actual monitoring expenditures were not available, the costs used were obtained from evaluation estimates contained in the Work Plan for the project. In addition, evaluation costs were assumed to be comparable to the cost of providing proper monitoring of the CP systems during their service lives.

The bridges in question were all constructed in 1964, thus providing a service period of almost 20 years prior to the installation of the CP systems. It is reasonable to assume that an additional service period of 20 years should be expected after such an investment in the CP systems is made. In addition, outside of the evaluation and monitoring costs, it is assumed that total maintenance should be similar to that of a typical concrete bridge repaired without the use of CP, provided a simple and relatively low maintenance system is utilized. The total cost for providing CP to an existing bridge deck (i.e., construction cost and evaluating/monitoring costs), without the routine maintenance costs which would be required for both systems, can then be determined and compared to the construction cost for repairing a similar bridge by conventional means. The more economic system can then be identified based on the difference in costs and service life achieved.

4.3 Inflation

Based on the estimates provided, the average annual cost of evaluation in 1984 was $1,525.00. Assuming an average inflation rate of 5%, the cost in year 10 (1994), the mid-point of expected service life, would be as follows:

$FC = PC * (1+R)^n$

Where FC is the future cost, PC is the present cost, R is the inflation rate (5% assumed), and n is the time period.

Therefore, the average cost of evaluation/monitoring over a 20-year period would be

$$FC = \$1{,}525.00 * (1 + 0.05)^{10}$$

OR $$FC = \$2{,}484.00$$

4.4 Time Value

Assuming an average monitoring cost that accounts for inflation over 20 years of $2,484.00, which is the equivalent of spending that same figure per annum over the same duration, the average annual cost valued at the base year can be determined as follows:

$$P = F / (1+I)^n$$

Where P is the present worth (1984), F is the future worth (1994), and I is the interest rate (7% assumed)

$$P = \$2{,}484 / (1.07)^{10}$$

$$P = \$4{,}886.40 \text{ (average of four bridges)}$$

Assuming that the assumed additional service life expectancy of 20 years is realized, the total estimated cost of providing CP system to Bridges 3A to 3D are indicated below.

Table 4.1 – Cost of Various Repair and Protection Systems

Items	3A	3B	3C	3D	3E	3F
Total Initial Construction Costs (base Year 1984)	$41,164	$39,124	$43,787	$41,985	NA	NA
Construction Inspection (including expenses and overhead)	$4,754	$4,754	$4,943	$4,527	NA	NA
Evaluation Cost By District (Year 1)	$1,516	$1,516	$1,577	$1,491	NA	NA
Assumed Monitoring Cost (base year)*	$4,898.	$4,898.	$5,019.	4,728.	NA	NA
Total Evaluation and Monitoring Costs	$97,720.	$97,720	$100,380	$94,560.	NA	NA
Area of Bridge (SF)	4663	4663	4778	4501	4778	4507
Total Estimated Cost**	$138,884	$136,844	$144,167	$136,545	NA	NA
Total Estimated Cost per SF	$29.78	$29.35	$30.17	$30.33	NA	NA

* Values proportioned based on relative areas of decks.
** Excludes routine maintenance costs.
***Information not available.

The above analysis costs about $30.00 per square foot to install and monitor a typical CP system with associated works, excluding routine maintenance. The cost (per square foot) to execute conventional repairs cannot be checked because no data was available during these studies.

5. CONCLUSIONS AND RECOMMENDATIONS

5.1 General

Based on the National Association of Corrosion Engineers (NACE) Standards for evaluating the performance of cathodic protection in reinforced concrete decks, as of December 1995, the four systems installed in Bridges 3A to 3D were performing as expected.[4]

As of June 2000, Bridges 3A to 3F were found in a good to fair state. More corrosion-based damage is evident in the bridges without any cathodic protection (3E & 3F). A final evaluation of the additional benefit of the CP system can, therefore, only be made when extensive rehabilitation works are required on both systems. Regardless of the results of the final numbers, it appears that there is a potential for economic benefits associated with the use of cathodic protection systems for bridge structures; however, it is also obvious that the technology is not being employed as efficiently and effectively as it could be. As a result, serious emphasis should be placed on the fine tuning of the system.

While there appears to be some need for the manufacturers of these systems to focus more on the constructibility and maintenance issues, it is always imperative that any CP system be designed to address these same issues. The experience in installing these systems in the various districts has taught us several important points that we need to take into serious consideration before we embark on any such project.

Even though the technology has been around for some time now, installation and maintenance of CP systems to existing bridge decks is not a very common occurrence. As a result, the Department's engineers are unlikely to have acquired the experience required to properly supervise construction works and to monitor and maintain such systems. Provided that it is the District's opinion that CP systems will eventually be the method of choice, or even a viable alternative for the protection of bridge structures, then it seems that the District should concentrate on the factors over which they have most control, the first of these being training of their staff.

A simple approach in staff training would be to include external experts on each CP construction project. While this will obviously have cost implications, a net gain may indeed be typically realized because a series of problems based on lack of experience may be avoided. In addition, the experience gained by the district engineers will be invaluable, and they will eventually be able to supervise, monitor, and maintain these same project types without external help and with confidence.

While it is imperative that the consulting team be equipped with the expertise and experience required for the proper execution of a CP project, the same must be required of the contractors. It is, therefore, in the District's interest, recommended to provide incentives where possible so that regional contractors are trained to do such work. This, indeed, would be an economical and a political plus. The training program does not necessarily mean a formal training session. Instead, a simple modification of the typical CP bridge contract may be adequate. One format would be to invite experienced contractors to bid, with the requirement that certain portions of the work be subcontracted to a regional firm who meets certain criteria.

Another series of problems on which the District may have a reasonable amount of influence is constructibility. Based on past reports, it is obvious that significant fine tuning may be required with respect to the products that are being offered on the market. One approach is to institute a set of manufacturers' performance specifications within relevant Pennsylvania bridge contracts. These specifications should address the same problems that the Department and others has experienced as a result of the inadequacies of these products. This concept, however, should be an ongoing program that keeps changing with the culture and needs of the Department.

5.2 Specific Conclusions and Recommendations

1. A Cathodic Protection System can be successfully installed to the prepared surface of a bridge deck.

2. The FHWA conductive polymer grout bonds well to a prepared concrete surface. Its bond to a latex-modified concrete (LMC) overlay, however, is much less. It should be ensured that adequate bond between conductive polymers and LMC is achievable.

3. The conductive grout and anode wires typically are not damaged by relevant construction activities.

4. To help avoid future problems, the deck surface must be very carefully checked to ensure that all potential electrical shorts are found. This should be a contract requirement.

5. Delamination planes have been found to be able to adversely affect half-cell potential readings. Where possible, the extent of the effects should be determined.

6. Cathodic Protection can be considered as an integral component of bridge deck rehabilitation if ease of installation and constructability problems, as well as reliability and maintenance issues, can be resolved by the industry. Criteria to establish the applicability of CP on such projects should be established by the department.

7. If the Department decides to install additional systems, then a corrosion engineer should be hired to oversee all CP systems. This engineer should be a statewide expert for all CP activities. In addition, each district should have on staff a technically proficient individual and an electrical technician who should be responsible for design review, construction inspection, monitoring, and maintenance.

8. Literature and details on construction specifications, design guidelines, and installation and monitoring procedures should be readily available and regularly updated.

9. The rectifier/controllers and other system components should be standardized based on the following criteria:

a. All rectifiers should be manual constant voltage or manual constant current controlled. Electronic or potential controlled rectifiers should be avoided in order to reduce maintenance costs.

b. All rectifiers should be designed with at least one spare circuit. Spare parts should be adequately stocked and readily available.

c. To simplify field monitoring, all system monitoring devices should be terminated in the rectifier cabinet.

d. All terminals should be carefully labeled.

e. All high-voltage terminals within the rectifier should be labeled and shielded with clear 'Plexiglas' or similar material to prevent accidental electrical shock.

f. All rectifiers should be galvanized or of stainless steel and be made rodent-proof and waterproof.

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