CORROSION PROTECTION-CONCRETE BRIDGES

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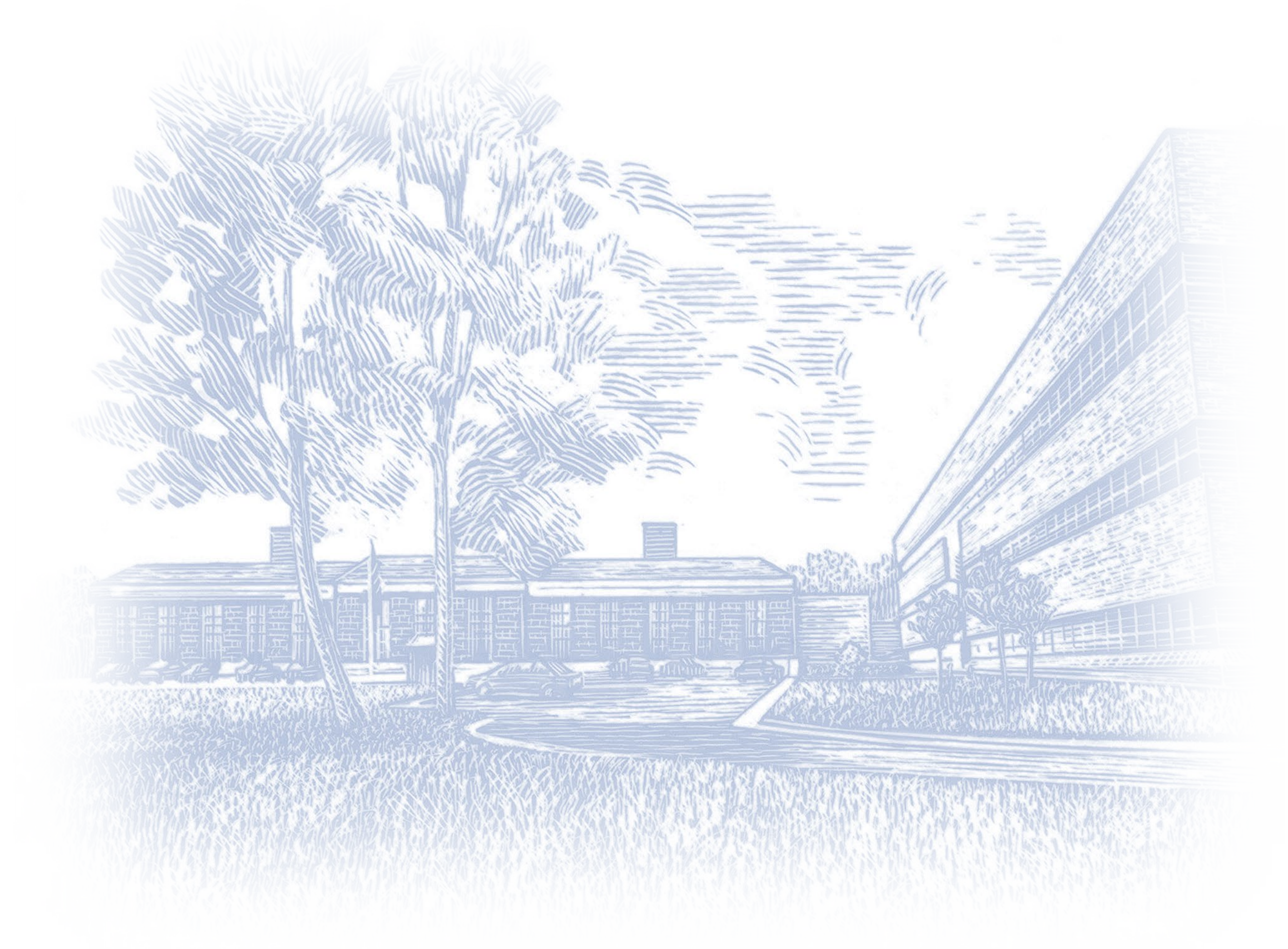

U.S. Department of Transportation
Federal Highway Administration

Federal Highway Administration
Turner-Fairbank Highway Research Center
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Foreword

The Federal Highway Administration recognized the problem of premature corrosion of reinforced concrete bridges in the early 1970's and established corrosion protection as one of its high-priority areas (HPA's) for research.

This report summarizes the research performed over the last 15 to 25 years of developing various cost-effective corrosion protection systems for both new structures and rehabilitating existing salt-contaminated, reinforced and prestressed concrete bridges.

Other forthcoming reports from Structures HPA's are: Geotechnical Engineering; Hydraulics/Scour; Seismic Protection; Corrosion Protection-Steel Bridges, and Timber Bridges.

This report will be of interest to owners, bridge engineers, consultants, and designers of reinforced and prestressed concrete bridges who are involved in the design, construction, maintenance, and rehabilitation of these structures.

Charles J. Nemmers
Director, Office of Engineering
Research and Development

NOTICE

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<tr><td colspan="3">16. Abstract

Premature corrosion of reinforcing steel has caused many concrete bridges in the United States to deteriorate before their design life was attained. Recognizing the burden that reinforcing steel corrosion imposes on natural resources, the Federal Highway Administration established Corrosion Protection for Concrete Bridges as one of the high-priority areas (HPAs) in its Structures Research Program. This HPA had two main objectives: (1) To develop effective and economical methodologies for arresting or reducing the extent of steel corrosion due to chloride-contamination of concrete bridges, thereby reducing maintenance costs of existing concrete bridges and minimizing interruption to traffic and, (2) To develop sound design and construction practices and materials for preventing corrosion of reinforcement in new structures, hence minimizing future deterioration. To meet these objectives, several research programs were developed and pursued in this HPA. This report summarizes the progress made through research efforts conducted under these programs.

Other reports will summarize progress from Structures High-Priority Programs for: Geotechnical Engineering; Hydraulics/Scour; Seismic Protection; Corrosion Protection - Steel Bridges; and Timber Bridges.</td></tr>
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SI* (Modern Metric) Conversion Factors

Approximate Conversions to SI Units

Symbol	When You Know	Multiply By	To Find	Symbol
Length				
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
Area				
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
Volume				
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
NOTE: volumes greater than 1000 L shall be shown in m^3				

Mass				
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
Temperature (exact degrees)				
°F	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	°C
Illumination				
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
Force and Pressure or Stress				
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

Approximate Conversions from SI Units

Symbol	When You Know	Multiply By	To Find	Symbol
Length				
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
Area				
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
Volume				
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3

m^3	cubic meters	1.307	cubic yards	yd^3
Mass				
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
Temperature (exact degrees)				
°C	Celsius	1.8C+32	Fahrenheit	°F
Illumination				
lx	lux	0.0929	foot-candles	fc
cd/m^2	$candela/m^2$	0.2919	foot-Lamberts	fl
Force and Pressure or Stress				
N	newtons	02.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

* SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380. (Revised March 2003)

Table of Contents

List of Tables

CHAPTER 1. INTRODUCTION

Corrosion of reinforcing steel has led to premature deterioration of many concrete bridges in the United States before their design life is attained. This has placed tremendous financial burden on many state and local transportation agencies in their attempts to halt ongoing reinforcing steel corrosion in the existing structures that are still functional (so that as much service life as possible can be salvaged from these) and to replace those structures that have already deteriorated to the point that it does not make any economic sense to keep on maintaining them. In addition, badly deteriorated bridges have considerable adverse effects on the nation's economic output and also place the safety of motorists at risk.

Recognizing the tremendous, adverse impacts that the problem of reinforcing steel corrosion poses, the Federal Highway Administration (FHWA) established Corrosion Protection for Concrete Bridges as one of the high-priority areas in its Structures Research Program. The other high-priority areas (HPAs) in this research program include:

- Geotechnical Engineering.
- Hydraulic Engineering.
- Corrosion Protection for Steel Bridges.
- Seismic Protection.
- Timber Bridges.
- Nondestructive Evaluation.

Through the HPA of Corrosion Protection for Concrete Bridges, FHWA has been developing solutions to this problem through various research programs that include investigations into different effective and economical ways to protect reinforcing steel in existing concrete bridges from further rapid deterioration and to prevent corrosion from occurring in new construction. It has also established ongoing cooperative efforts with the Office of the Technology Applications and some state transportation agencies to expediently disseminate beneficial findings resulting from this research program to state and local transportation agencies for implementation.

This research report summarizes the progress made in the research programs of this HPA of corrosion control of reinforcing steel in reinforced and prestressed concrete. This report deals with both new construction and rehabilitation of existing salt-contaminated concrete bridges.

CHAPTER 2. ACCOMPLISHMENTS

For New Bridges

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- The preferred primary corrosion-protection systems in many states has been fusion-bonded epoxy coated rebars (ECR), which have been used in approximately 20,000 reinforced concrete bridge decks. This rebar has performed very well in alleviating the problem of corrosion-induced deteriorations of concrete bridge decks. It is estimated that its use in the last 25 years has saved the taxpayers billions of dollars so far.

  With continuing improvements in the American Association of State Highway and Transportation Officials (AASHTO) and American Society for Testing and Materials (ASTM) specifications for ECR, this corrosion system will become even better. The improvements involved all possible aspects of the fabrication of ECR, including certification of coating plants, proper storage of coating powder at the plants, restriction on surface imperfections on the bars, removal of dust and salt from the surface of the bars prior to coating, and better quality control of thickness, continuity, flexibility, adhesion, etc. In addition, requirements related to job-site storage and handling of the coated bars have also been established. All of these will result in better ECR and more durable new concrete structures. Ongoing efforts to identify more effective organic coatings will also lead to more corrosion-resistant steel bars in the future.
-
- To provide even longer service life to the concrete decks; G years or longer— without any need to repair corrosion-induced concrete damage, a number of alloys and cladding have been developed for rebars. Most notable are solid stainless steel 316 rebars and stainless steel clad black bars, which have performed exceedingly well in accelerated screening corrosion tests. Both of these two new alternative reinforcing bars have the potential to provide an excellent corrosion protection system, albeit at a higher initial cost.
-
- The combined use of ECR and a corrosion inhibiting admixture, such as calcium nitrite, could serve as a very good corrosion protection system. However, the stability of this inhibitor is still under study. In addition, research efforts are under way to identify new inhibitors that are more effective than calcium nitrite.
-
- The combination of high temperature (38° C) and an intermediate level of humidity or moisture (75 percent) have been identified as environmental conditions that lead to high corrosion rates for steel in concrete. It was found that the use of a low water-cement ratio, incorporation of mineral admixture and proper selection of cement type and aggregates contribute significantly to producing low-permeability concretes.
-
- For the protection of high-strength, seven-wire strands encased in ducts, mix designs for corrosion-resistant grout for filling the ducts have been developed. In addition, an accelerated corrosion test method has been developed for evaluating new grout mixes. These developments have become the basis of a new specification to be published by the Post-Tensioning Institute (PTI) in 1998.
-
- Prompted by the recent sudden collapse of two post-tensioned bridges in the United Kingdom and one in Belgium, several different nondestructive inspection techniques were carefully

evaluated to identify those that may be suitable for detecting voids in post-tensioned ducts. From these, the impact-echo technique was selected for improvement; then, it was successfully evaluated in the field. The equipment for this technique is now commercially available. Further research is under way to develop a complementary magnetic-based, nondestructive technique for assessing section loss in the high strength steel strands in the ducts. It is anticipated that when used in combination, the impact-echo and the magnetic-based techniques will allow complete inspection of post-tensioned systems, reducing the likelihood of any sudden collapse of post-tensioned bridges in the United States.

For Rehabilitation of Existing Concrete Bridges

- Cooperative research with industry and states in the development of durable anodes, monitoring devices, installation techniques, etc. has led to application of impressed-current cathodic protection systems on bridge decks as a routine rehabilitation technique.
-
- For cathodic protection of substructure members, especially those in a marine environment, two very promising sacrificial anode systems have been developed. Initiatives in the industry and in some states, in cooperation with FHWA, have led to further developments and identification of anodes suitable for impressed-current cathodic protection of inland concrete substructures.
-
- Through extensive fundamental research and evaluation of cathodic protection systems that have been installed, significant advances have been made in the technology for cathodic protection of prestressed concrete components. Concerns about a loss of bond between the prestressing steel and concrete and possible hydrogen embrittlement (from overprotection of the prestressing steel) have been alleviated by the establishment of criteria for qualification of prestressed concrete bridge components for cathodic protection.

CHAPTER 3. HISTORY

Concrete is a construction material that is relatively easy to work with. However, concrete is very weak in tension in comparison to its compressive strength. Because of the low tensile strength of concrete, reinforcing steel bars are placed in regions of tension in a concrete member. This combination of concrete and steel provides a relatively inexpensive and durable material that has become widely used in construction of roadways and bridges. Reinforced concrete bridges have functioned reasonably well until the late 1960s, when premature concrete delamination and spalling, which used to be encountered only in coastal areas, became common in many of the reinforced concrete decks in the "snow belt" and concrete bridge decks were beginning to require maintenance after being in service for as little as 5 years. The emergence of this type of concrete deterioration, which was first observed in marine structures and chemical manufacturing plants, coincided with the increased application of deicing salts (sodium and calcium chlorides) to roads and bridges during winter months in those states where ice and snow are a problem to implement a "bare pavement policy".

It was recognized by the mid 1970s that this problem is caused by the corrosion of the reinforcing steel in the concrete which, in turns, is induced by the intrusion of even a small amount of chloride from the deicing salts into the concrete. It is difficult to estimate the cost of these corrosion-related damages to conventionally reinforced and prestressed concrete bridge components in the nation. According to a 1997 report, of the 581,862 bridges in and off the federal-aid system, about 101,518 bridges were rated as structurally deficient. Most of these bridges are not in danger of collapse, but they are likely to be load posted so that overweight trucks will be required to take a longer alternative route. The estimated cost to eliminate all backlog bridge deficiencies (including structurally and functionally) is approximately $78 billions (*1*), and it could increase to as much as $112 billions, depending on the number of years it takes to meet the objective. The average annual cost, through year 2011, for just maintaining the overall bridge conditions, i.e., the total number and the distribution of structurally and functionally deficient bridges, is estimated to be $5.2 billions. While corrosion of the reinforcing steel is not the sole cause of all structural deficiencies, it is a significant contributor and has therefore becomes a matter of major concern.

The magnitude of this corrosion problem in the transportation infrastructure has increased significantly in the last three decades and is likely to keep increasing. Even though the cost of maintaining bridge decks is becoming prohibitively expensive, the benefits provided by deicing salts are too great, however, that it's use is not likely to decrease in the future. In fact, the use of road deicing salts, which are extremely corrosive due to the disruptive effects of its chloride ions on protective films on metals, has actually increased in the first half of the 1990s—after a leveling off during the 1980s. Although an alternative effective and less corrosive deicing agent, calcium magnesium acetate (CMA), is available, its price is apparently not yet reasonable enough for winter maintenance engineers to use widely. Therefore, it can be expected that the road environment would likely remain corrosive, if not more, well into the future. In response to the tremendous economic burden that corrosion of reinforcing steel on concrete bridges placed on the national economy, the Structure Division of FHWA has placed emphasis on finding effective and economical solutions that can be easily implemented by various state and local transportation agencies.

Nature of Reinforcing Steel Corrosion in Concrete

In order to understand the various approaches by which this type of corrosion can be controlled, it is necessary to understand its nature. A few metals, notably gold, silver, and platinum, occur naturally. Engineering metals, including steel, must be derived from their ores by smelting. Ores are natural oxides, sulfides, and other reaction products of metals with the environment. During smelting, a metal absorbs the energy required to free it from the ore; and, this energy is retained within the metal after it is recovered. However, this metallic state is unstable, because the metal tends to rid itself of this extra energy by recombining with the environment to revert to its more stable and natural state as an ore. This reversion process is known as oxidation or, more specifically, corrosion.

A refined metal such as iron or steel has a natural tendency to corrode and thereby return to the stable state that it exists in nature, as iron ore (typically iron oxide, Fe_2O_3). The rate of steel corrosion depends on its composition, grain structure, and the presence of entrained stress from fabrication. It also depends on the nature of the surrounding environment, such as the availability of water, oxygen, and ionic species, pH and temperature.

In concrete, the presence of abundant amount of calcium hydroxide and relatively small amounts of alkali elements, such as sodium and potassium, gives concrete a very high alkalinity—with pH of 12 to 13. It is widely accepted that, at the early age of the concrete, this high alkalinity results in the transformation of a surface layer of the embedded steel to a tightly adhering film, that is comprised of an inner dense spinel phase (Fe_3O_4 / γ Fe_2O_3) in epitaxial orientation to the steel substrate and an outer layer of α-FeOOH (*2*). As long as this film is not disturbed, it will keep the steel passive and protected from corrosion. When a concrete structure is often exposed to deicing salts, salt splashes, salt spray, or seawater, chloride ions from these will slowly penetrate into the concrete, mostly through the pores in the hydrated cement paste. The chloride ions will eventually reach the steel and then accumulate to beyond a certain concentration level, at which the protective film is destroyed and the steel begins to corrode, when oxygen and moisture are present in the steel-concrete interface.

In 1962, it was reported that the required minimum concentration of chloride in the concrete immediately surrounding the steel to initiate corrosion, the chloride corrosion threshold, is 0.15 percent soluble chloride, by weight of cement (*3*). In typical bridge deck concrete with a cement factor of 7, this is equivalent to 0.025-percent soluble chloride, by weight of concrete, or 0.59 kg soluble chloride per cubic meter of concrete. Subsequent research at FHWA laboratories estimated the corrosion threshold to be 0.033-percent total chloride, by weight of concrete (*4*,*5*). (Although it is widely accepted that only water-soluble (ionized) chloride contributes to corrosion, it is more common in practice to determined the total (inorganic) chloride contents of concrete samples from bridges, because analytical methods available for soluble chloride (AASHTO T-260 and ASTM C-1218) are more laborious and not as precise as that for total acid-soluble inorganic chloride (AASHTO T-260 and ASTM C-1152). In addition, it is common practice to express chloride contents in terms of weight percentage of concrete, which would not required an additional elaborate analysis to determine the cement content in the hardened concrete sample.) There are indications that the chloride corrosion threshold can vary between concrete in different bridges, depending on the type of cement and mix design used, which can vary the concentrations of tricalcium aluminate (C_3A) and hydroxide ion (OH^-) in the concrete. In fact, it has been suggested that because of the role that hydroxide ions play in protecting steel from corrosion, it is more appropriate to express corrosion threshold in terms of the ratio of chloride content to hydroxide content, $[Cl^-]$ / $[OH^-]$, which was recently established tote between 2.5 to 6 (*6*,*7*).

Once corrosion sets in on the reinforcing steel bars, it proceeds in electrochemical cells formed on the surface of the metal and the electrolyte or solution surrounding the metal. Each cell is consists of a pair of electrodes (the anode and its counterpoint, the cathode) on the surface of the metal, a return circuit, and an electrolyte. Basically, on a relatively anodic spot on the metal, the metal undergoes oxidation (ionization), which is accompanied by production of electrons, and subsequent dissolution. These electrons move through a return circuit, which is a path in the metal itself to reach a relatively cathodic spot on the metal, where these electrons are consumed through reactions involving substances found in the electrolyte. In a reinforced concrete, the anode and the cathode are located on the steel bars, which also serve as the return circuits, with the surrounding concrete acting as the electrolyte.

When corrosion occurs on the reinforcing steel in concrete, the electrochemical reactions involved are dependent on the environments at the steel-concrete interface:

- When Oxygen Is Present:
-
    1.

$$Fe \rightarrow Fe^{+2} + 2\ e^- \tag{1}$$

2.
3. At the anode, iron is oxidized to the ferrous state, releasing electrons.
4. At the cathode, these electrons combine with oxygen and moisture to form hydroxide ions.

$$\tfrac{1}{2}\ O_2 + H_2O + 2\ e^- \rightarrow 2\ OH^- \tag{2}$$

5.
6. The ferrous ions combine with hydroxyl ions to produce ferrous hydroxide. Then the latter is further oxidized in the presence of moisture to form ferric oxide.

$$Fe^{+2} + 2\ OH^- \rightarrow Fe(OH)_2 \tag{3}$$

$$4\ Fe(OH)_2 + 2\ H_2O + O_2 \rightarrow 4\ Fe(OH)_3 \tag{4}$$

$$2\ Fe(OH)_3 \rightarrow Fe_2O_3 + 3\ H_2O \tag{5}$$

7.

- When Oxygen is Absent:
-

1.
2. At the anode, in the upper layer of steel in a bridge deck, the oxidized iron reacts with chloride ions to form an intermediate iron complex.

$$Fe^{+2} + 4\ Cl^- \rightarrow (FeCl_4)^{-2} + 2\ e^- \tag{6}$$

3.
4. This complex then reacts with moisture to form ferrous hydroxide.

$$(FeCl_4)^{-2} + 2\ H_2O \rightarrow Fe(OH)_2 + 2\ H^+ + 4\ Cl^- \tag{7}$$

5.
6. At the cathode, the hydrogen ions are reduced or combine with electrons to form hydrogen gas.

$$2\ H^+ + 2\ e^- \rightarrow H_2 \tag{8}$$

It has been suggested (*8*) that the chloride complex ions formed in reaction 6 may react with calcium hydroxide in the surrounding cement paste as follow:

$$(FeCl_4)^{-2} + Ca(OH)_2 \rightarrow Fe(OH)_2 + CaCl_2 + 2\ CL^- \tag{9}$$

It is apparent from the above reactions that, unfortunately, none of the chloride ions in the concrete are consumed and are, therefore, available again to contribute to corrosion. By forming hydrogen ions, or acid, in reaction 7, the pH at a local anodic site can reduce rapidly to values of 5 to 6. The resulting low-pH anode on a rebar is so different from that of other nearby rebars, which are surrounded by concrete of higher pH, that a powerful macro-cathode is created, which then feeds the original anodic spot. It is clearly obvious that moisture is required, not only to support the cathodic reactions but also to enhance the electrical conductivity of the concrete.

Corrosion can also occur even in the absence of chloride ions. For example, when the concrete comes into contact with carbonic acid resulting from carbon dioxide in the atmosphere, the ensuing carbonation of the calcium hydroxide in the hydrated cement paste leads to reduction of the alkalinity, to pH as low as 8.5, thereby permitting corrosion of the embedded steel:

$$CO_2 + H_2O \rightarrow H_2CO_3 \qquad (10)$$

$$H_2CO_3 + Ca(OH)_2 \rightarrow CaCO_3 + 2\ H_2O \qquad (11)$$

The rate of carbonation in concrete is directly dependent on the water/cement ratio (w/c) of the concrete, i.e., the higher the ratio the greater is the depth of carbonation in the concrete. In concrete of reasonable quality, that is properly consolidated and has no cracking, the expected rate of carbonation is very low. For example, in concrete with w/c of 0.45 and concrete cover 25 mm (1 in.), it will require more than 100 years for carbonation to reach the concrete immediately surrounding the steel (*9*). Carbonation of concrete or mortar is more of an issue Europe—thereby prompting the application of electrochemical realkalinization of concrete there—than in United States.

However, another possible source of problem to the durability of concrete that has not been widely recognized yet is the emissions of air pollutants such as sulfur dioxide (SO_2) and nitrous oxides (NO_x), resulting from burning of fossil fuels in the United States. When these gases come into contact with moisture in the atmosphere, they are converted to acids—in the form of acid rain and snow—that are considerably more corrosive than the carbonic acid resulting from carbon dioxide. These acids can cause acidification of cement paste that is more severe than the carbonization shown in reaction 11. Furthermore, there are indications that a combination of deicing salts and acid precipitation creates an even more corrosive environment than either of these substances alone does. In fact, there are data showing that corrosive areas in North America and Europe are, in general, areas where marine salts or deicing salts are present, while the most corrosive areas are those having a combination of these salts and acid precipitation (*1*). This synergistic effect of deicing salts and acids on metallic corrosion, which has been documented in laboratory studies, is attributed to the supply of cathodically reducible hydrogen ions for the corrosion reaction (*10*,*11*). It is conceivable that acid deposition from these gases can eventually have adverse effects on the concrete—increasing its permeability to intrusion by chloride ions and at the same time reducing the beneficially high alkalinity of the concrete around the reinforcing steel. The extent to which acid precipitation can become a threat to the durability of concrete deserves some investigations.

Each electrochemical corrosion cell exhibits a potential, or voltage, difference between the anode and the cathode, which drives the corrosion. In a reinforced concrete, the voltage difference may be created by a combination of the following:

- Differences in the surface of the steel bars. Steels are heterogeneous materials as they are alloyed with carbon and other elements. Their surfaces may be considered a patchwork of metal sites of slightly different electrochemical potentials. The presence of residual stress in the steel bars and even the presence of scratches on a portion of a bar and not on the other can create enough potential differences to drive corrosion.
-
- Differences in the electrolytes. These can be differences in the concentration of chloride, oxygen, moisture, hydroxide, etc., in the concrete surrounding the steel bars. Such differences can readily exist in concrete, since it is a very heterogeneous material, both chemically and physically and on the microscopic and the macroscopic scales. Microscopic differences give rise to microscopic electrochemical cells (micro-cells), wherein each pair of electrodes (the anode and the cathode) exist in the same steel bar; in a concrete bridge deck, these paired electrodes would be found mostly in the upper reinforcing mat. Macroscopic differences create macroscopic electrochemical cells (macro-cells), wherein an anode is located on a steel bar in the upper mat and its cathode is located in another bars in the same mat or in a lower mat. The latter macro-cell is also common in

bridge decks, where steel bars in the upper mat are anodic because of being exposed, in general, to higher amount of chloride ions and moisture than steel bars in the lower mat are.

The presence of cracks in the concrete can quickly give rise to these differences in different portions of the concrete surrounding the steel bars.

Lately it has been suggested that even heterogeneity in the distribution of porosity in the cement paste surrounding the reinforcing steel can affect surface corrosion of reinforcing steel—with corrosion reactions occurring preferentially on the surface of steel bars embedded in relatively denser hydrated cement paste (*12*). Measurements of surface impedance and phase angle, using a low-frequency impedance technique, revealed that depletion of oxygen at the steel/cement interface to form anodic areas may be the cause. It is no doubt that, in general, the greater these differences exist along the same steel bars or between neighboring steel bars, the faster is the resulting corrosion on the steel bars.

When corrosion on a steel bar progressed to the extent that the corrosion products (rust) occupy a greater volume than the steel and exert substantial stress on the surrounding concrete, the concrete begins to delaminate and then eventually spall. Estimates of the expansive force exerted on the concrete when steel converts to rust varied from 32 to 500 MPa (*13*,*14*). Similarly, different estimates have been made on the amount of corrosion product necessary to crack the concrete or mortar, and these have ranged from 0.1 to 20 mils (*15*,*16*). As corrosion takes place, the cross section of the steel is reduced, which leads to loss of bond between the steel and the concrete. This effect can be of serious concern in prestressed concrete bridge members, where the bonding between the high-strength tendons and the concrete is critical.

The rapid premature deterioration of many concrete bridge decks in the late 1960s had raised concern among the state highway agencies. As a result, the use of non-corroding fusion bonded epoxy-coated reinforcement as a corrosion protection system has become a standard practice since the late 1970s. For additional protection, low-permeability concrete (low water-cement ratio Portland cement concrete, latex-modified concrete, other specialty concrete, and improved mix design) and increased concrete cover over coated reinforcing steel began to be utilized. Use of waterproof membranes over concrete decks, in conjunction with an asphalt overlay, as a protection system has produced mixed results and is also used, although sparingly. Few states have used epoxy-coated rebars in conjunction with waterproof membrane and asphalt overlay as their preferred multiple corrosion protection system.

FHWA research and field data have indicated that the use of overlays, waterproof membranes, and sealers only serve to slow the corrosion rate but do not stop the ongoing corrosion process. These conventional rehabilitation methods marginally extend the life of a structure to a variable extent, depending upon the quality and the type of treatment employed. On the other hand, cathodic protection (CP) has proven to be successful in retarding and controlling chloride-induced corrosion in reinforced concrete (RIC) bridge components. In addition, the corrosion process can be stopped or slowed by the elimination of either chloride ions, moisture, or oxygen in the concrete. Electrochemical chloride extraction, which removes chloride ions from the contaminated concrete, is therefore another alternative rehabilitation technique that is being explored by a number of state highway agencies. More research is under way to bring this technology at par with cathodic protection.

Investigations are under way for development of protection systems that can be adopted in the construction of new reinforced concrete bridge substructures and prestressed concrete members (PS/C), such as piles, girders, box beams, etc., which are generally still being built with uncoated high-strength steel. Research in progress has shown promise for the use of epoxy-coated strands for PS/C structural concrete members. Another promising protective system consists of the use of a corrosion inhibitor as a concrete admixture during the mixing of concrete. Such admixture inhibits the oxidation of the reinforcing steel to the ferrous state and the subsequent formation of corrosion products. Corrosion inhibitors and low-permeability concrete have found extensive use in PS/C members in which black steel/strands are still generally used. However, as yet there is no generally accepted protection system for reinforced concrete substructure or prestressed concrete bridge members comparable to the acceptance of epoxy-coated reinforcement in concrete bridge decks.

The application of prestressed concrete technology in building bridges is relatively recent. Therefore, the existing prestressed concrete members in bridges are still relatively young, and the corrosion and the concrete deterioration problems associated with this type of concrete members have only became evident in the early 1980s. Although prestressed concrete members were generally manufactured with concrete of relatively higher strength, time has shown that they are subject to the same adverse effects of reinforcement corrosion as reinforced concrete members are. Documented cases of prestressed strands breaking as a result of corrosion make this a most pressing problem. Since PS/C members rely on the tensile strength of the strands to resist loads, loss of even a few strands per member could prove catastrophic. In addition, because of the high stress the strands are subjected to, corrosion effects are accelerated. Even small corrosion pits could cause fracture of a strand, as comp red to non-prestressed reinforcing steel that will literally rust away before breaking. Of the 581,862 bridges in the National Bridge Inventory, slightly more than 10 percent have prestressed concrete superstructures, all of which will eventually need some protective measures applied to them. In addition, many of the other structures not in the inventory have prestressed substructure members that will likewise need some degree of corrosion protection.

There is no doubt that corrosion of reinforcing steel in concrete bridges has become a costly problem for the nation's infrastructure. To address this, the Structures Division of the Federal Highway Administration has led the efforts to find solutions for preventing this problem in new constructions and for mitigating it in existing reinforced concrete bridges.

CHAPTER 4. OBJECTIVES

The HPA of Corrosion Protection for Concrete Bridges had two major objectives:

- To develop materials for preventing corrosion of embedded reinforcing steel in new construction and hence minimize premature corrosion-induced concrete deterioration in the future. To achieve this objective, promising approaches such as adoption of sound construction designs, development and use of alternative reinforcements, and use of corrosion inhibiting admixtures were investigated.
-
- To develop methodology for completely stopping or reducing the extent of ongoing corrosion of reinforcing steel in chloride-contaminated concrete bridges and hence reducing the costs in maintaining existing concrete bridges. To achieve this objective, besides improvements made on the more conventional approaches, durable anodes for the application of cathodic protection were developed and improvements in the electrochemical chloride extraction are being developed.

With the above objectives in mind, this research program has developed a number of new materials and methods to construct and rehabilitate the existing structures in a cost-effective manner that is consistent with the preservation of the environment.

CHAPTER 5. RESULTS

Since our total investment in the nation's highway infrastructure amounts in the billions of dollars, it is extremely important that all possible methods applicable to controlling corrosion in existing concrete bridges be developed so that these structures will not deteriorate prematurely. Equally important is developing methods to avoid this costly corrosion problem in all new concrete bridges to be constructed in the future. Accordingly, the research program undertaken in this HPA was divided into two major areas:

- Corrosion Control in New Concrete Constructions.
- Corrosion Control for Rehabilitation of Existing Concrete Structures.

To be effective, this research program involved not only the participation of FHWA, but other partners including another federal agency (National Institute of Standards and Technology), some state highway agencies, the academia, and the private industry. The following is a presentation of results or accomplishments achieved to date in these two areas.

A. Corrosion Control in New Concrete Constructions

Given the very harsh service environments that many bridges will be exposed to typically, it is extremely difficult - but not impossible - to build reinforced concrete bridges that would be free of steel reinforcement corrosion. The achievement of this goal would require the adoption of system approach, i.e., using a combination of different measures, such as adequate depth of concrete cover, quality concrete, corrosion inhibitors admixture, and corrosion-resistant reinforcement.

1. Adequate Concrete Cover

It is now widely accepted that, for a concrete structure to be durable in a corrosive environment, it is absolutely necessary to provide an adequate layer of concrete or depth of concrete cover over the first layer of reinforcing steel so that it would not be easy for chloride ions to reach the steel. This adequate depth of cover can be determined by application of Fick's second law of diffusion, which adequately models the intrusion or diffusion of chloride through a porous material such as concrete (*17*):

$$(\delta C_{x} / \delta t) = D(\delta^{2} C_{x} / \delta^{2} \chi) \qquad (12)$$

where, C_x is the concentration of chloride at depth c at time t, D is the diffusion coefficient of chloride. The solution to this equation for a semi-infinite slab is:

$$C(\chi, t) = Co \{ 1—\mathrm{erf} [\chi / (2 (Dt)^{½})] \} \qquad (13)$$

which is applicable to concrete structures where the chloride ions enter from one direction, such as concrete bridge decks and piers. Using this relationship, the minimum depth of concrete cover over the reinforcing steel that would be required so that the total amount of chloride ions that will accumulate at the depth of steel will not exceed the corrosion threshold—before a desired service life is reached—can be determined from the measured chloride diffusivity of the selected concrete mix design and the anticipated level of exposure of the structure to deicing salts. Finally, this minimum depth of cover must be incorporated with expected construction tolerances (that accounts for typical construction variance) to achieve a rational depth of cover specification (*18*). For example, if a minimum concrete cover of 50 mm is required and statistical surveys of the cover on recently built bridge decks show that the cover on a deck typically assumes normal distribution with a standard deviation of approximately 10 mm, then the specified cover must be either 67 mm or 73 mm for 95-percent or 99-percent compliance, respectively.

However, even with adequate concrete cover, corrosion of reinforcement can still occur because, invariably, concrete will crack. In addition, presence of variances in the concrete cover and in the density of the placed concrete across a structure will eventually create corrosion micro-cells (consisting of cathodes and anodes), which drive steel corrosion. Therefore, other supplementary protective measures also need to be adopted in a new construction.

2. Quality Concrete

Recently, there is also a heightened awareness that the quality of concrete is of utmost importance in determining the durability of reinforced concrete bridge members exposed to chlorides and subjected to intermittent wetting. Although concrete is outwardly a dense material, it contains pores; and many of these pores are interconnected to form a network of channels that allows water and oxygen, both important to steel corrosion, to permeate into the concrete. And, as a general rule, low water/cement ratio and good consolidation lead to either a lower number of pores or smaller pores in the concrete, both of which can lead to reduce penneability. In addition, reduced permeability also leads to reduction in the electrical conductivity of the concrete by reducing not only the amount of moisture intrusion but also the amount of chloride ions carried by the moisture into the concrete. A low water/cement ratio also offers higher strength to the concrete, which would extend the time before stresses resulting from steel corrosion cause the concrete to crack. Therefore, mixture proportions must be carefully selected to the keep the water/cement ratio to an absolute minimum.

In addition to making state transportation agencies become aware of the importance of using low water/cement ratio in concrete mixes, some recent FHWA research efforts were aimed at identifying concrete materials that can consistently provide superior performance when used in construction of bridges. These efforts included investigation into how concrete material and mix variables, such as water-cement ratio, air content, coarse aggregate type, fine aggregate type, mineral admixture, and cement type, affect the corrosion behavior of steel (*19*). The test matrix adopted in this investigation is shown in Table 1.

Table 1. Test Matrix

Variable	Number	Type
Cement type	6	Type I-low C_3A, Type I-high C_3A, Type I-low alkali, Type I-high alkali, calcium aluminate, magnesium phosphate
Mineral admixture	4	Silica flume, Class C fly ash, Class F fly ash, GGBF slag
Coarse aggregate	2	Limestone, quartz
Fine aggregate	2	Glacial sand, quartz sand
Water-cement ratio	3	0.3, 0.4, 0.5
Air content	3	2, 5, 8 percent
Exposure environment	2	Moderate, severe

Because only 30 concrete mix designs were used, which were insufficient to test all the possible combinations of the large number of variables, only weak correlation between corrosion rate or potential and these variables/types were obtained, for both moderate and severe environments. Nevertheless, it was possible to draw some general conclusions, the primary of which was that the corrosion rate of reinforcing steel varies significantly depending on the concrete mix components. Among the variables investigated, water/cement ratio, cement type, mineral admixture, and fine aggregate exhibited the most significant effects on the durability of the concrete. The other factors, such as coarse aggregate and air content, also affect behavior of steel in concrete, but to a lesser extent. The data indicated that the Type I cement (low C_3A), quartz fine and coarse aggregates, and silica fume appeared to be the best materials for improving resistance of concrete to deterioration (*20*).

The use of blended cements might, under certain circumstances, be detrimental because of a reduction in alkalinity in the concrete. However, this adverse effect may be more than offset by the beneficial effects that blended cements can offer, which include a substantial reduction in permeability and also a reduction in conductivity—especially where a reduction in the water-cement ratio is made possible.

In addition, the investigation also attempted to quantify the corrosive conditions fostering concrete bridge deterioration. The effects of environmental variables such as chloride concentration, temperature, and humidity, on the corrosion behavior of reinforcing steel in a typical concrete mix were examined (*21*). The adopted test matrix is shown in Table 2.

Table 2. Adopted Test Matrix

Chloride Level kg/m^3 (lb/yd^3)	Temperature °C (°F)	Rel. Humidity (percent)
0.59 (1)	4 (40)	43
1.77 (3)	21 (70)	75
5.93 (10)	38 (100)	98

From the data collected, a statistical regression model was developed to permit prediction of the corrosion rate as a function of these three variables. Even though the model has only a correlation coefficient of 0.50, it serves to demonstrate that, given any chloride concentration, the environmental factors of temperature and humidity also significantly affect the corrosion behavior of steel in concrete. This explains why the problem of reinforcing steel corrosion is considerably more severe in the marine environment of states like Florida than in some other states.

The following are the most important findings resulting from these investigations:

- Chloride concentrations from 0.59 to 5.9 kg/m^3 (1 to 10 lb/yd^3) can produce a wide range of corrosion behavior of steel in concrete.
-
- Environmental variables such as temperature and humidity are also significant in influencing the corrosion behavior of steel. Intermediate levels of humidity or moisture provide aggressive environments than either low or high levels.
-
- The interactions of chloride ion concentration, temperature, and relative humidity on the corrosion behavior of steel are complex.
-
- The corrosion rate of reinforcing steel can vary significantly based on the concrete mix components. The most significant beneficial effects were obtained from use of low water-cement ratio and proper selection of cement type, mineral admixture, and fine aggregate. Factors such as coarse aggregate, and air content had a lesser effect.

3. Alternative Reinforcements

Even if concrete can be made to be extremely impermeable (by the addition of pozzolans like microsilica, etc.) and not conducive to steel corrosion (by the addition of an effective inhibitor), it still would not completely solve the corrosion problem, since concrete has a tendency to crack eventually, especially the high-performance concrete if proper curing is not observed. The final line of defense against corrosion of reinforcing steel would still lie with the reinforcing steel itself. Unfortunately, the resistance of mild steel to corrosion can not be significantly improved by just modifying its composition, grade, or the level of stress (*22*). Therefore, to prevent corrosion of steel reinforcement in concrete located in corrosive environment, either the conventional mild steel reinforcement must be coated with an effective and economical barrier to prevent contact with chloride, moisture, and oxygen, or reinforcement made of corrosion-resistant materials must be used.

Among the above two options, application of a suitable coating on the mild steel reinforcement may be the most economical. The coated reinforcing steel must be resistant to damages during transport from a plant to a construction site, storage at construction site, and placement in the structure. It must also be durable in severe service environment and capable of maintaining its structural function throughout the service life of the structure, and be economical.

a. Steel Bars With Organic Coating

After roadway deicing salts was identified as the cause of the premature deterioration of many concrete bridges, solutions to control this problems have to be developed since banning of deicing salts would not be acceptable to the motoring public. A solution that was identified as potentially viable and economical is the application of a suitable, stable organic coating on the reinforcing steel to serve as a barrier for isolating the steel from moisture, chloride ions, and oxygen, thereby preventing corrosion. Therefore, in the early 1970s, the Federal Highway Administration sponsored a project at the National Institute of Standards and Technology, then the National Bureau of Standards, to search for organic coatings suitable for this application. In that study, 47 coatings—including 36 epoxies (in both liquid and powder forms)—were evaluated in the laboratory (*23*). Four fusion-bonded epoxy powders emerged as the most promising of the coatings studied.

The first bridge utilizing epoxy powder-coated reinforcing steel bars (hereafter called epoxy-coated rebars) was built in 1973 in West Conshohocken, Pennsylvania. Subsequent trial of some of these coatings in bridge decks, under the National Experimental and Evaluation Program, revealed some problems such as damage to the coating during transport and handling, and cracking of coating (in the bend areas) rising from bending of bars at construction sites. To eliminate or alleviate these problems, measures such as bending the bars before coating, increasing bar supports during shipping (to prevent abrasion between bars), and using padded bundling bands and nylon slings during loading and unloading, were adopted.

In a subsequent FHWA study using relatively large concrete slabs, the performance of some epoxy-coated rebars that failed specifications—by having excessive holidays and surface damage (in excess of 0.8 percent) and failing bend tests—was compared with that of black steel (*24*). It was found that even though the coating on those rebars failed specifications, it was effective in reducing steel corrosion in salted concrete. Based on the data, it was estimated that, relatively, if it required 1 year to consume a given amount of black steel, then 12 years would be required to consume the same amount of the coated rebars, when the epoxy-coated rebars were used only in the top mat bars; or, 46 years would be required, if the coated rebars were used in both top and bottom mats. (The shorter protection accrued when the coated rebars were used only in the top mat was due to the formation of macro-cells between the different types of rebars in the two mats, especially when electrical couplings exist.) It appeared that the presence of the insulating epoxy coating caused significant increase in the electrical resistance between the top- and the bottom-mat of steel bars, which typically behaved as macro-anode and the macro-cathode, thereby retarding steel corrosion.

These findings had lessened the concern that a small exposed area of steel would be susceptible to intense corrosion and, thereby, enabled the requirement for patching or repair of coating damages to be waived when the damage is less than 2 percent in straight bars, 5 percent in bent bars, and 3 percent after placing. This led to decrease in the cost of epoxy-coated steel bars and their wider use in bridges by many states. However, in 1992, a final report for the Canadian Strategic Highway Research Program, "Effectiveness of Epoxy-Coated Reinforcing Steel," raised some questions about the long-term corrosion and structural performances of this type of steel bars, based on observation that some of the epoxy films became disbonded, blistered, and cracked (*25*). Since then, both ASTM and AASHTO specifications for epoxy-coated rebars, on the presence of holidays and bare area and on coating thickness, were revised and tightened. The Canadian report also urged that the effect of coating adhesion loss on the structural bond and creep properties of concrete members reinforced with epoxy-coated bars be investigated.

To study this important issue, the FHWA conducted a series of short-term experiments to assess the possible effects of disbondment of the epoxy coating from the steel bars on critical bond stress and flexural strength (*26*). For comparison, a series of pull-out concrete specimens and flexural reinforced concrete slabs were fabricated with: (a) epoxy-coated bars with different levels (20 to 30 percent) of intentionally induced disbondment in coating, (b) epoxy-coated bars with good coating, and (c) black steel bars. Although the investigation was limited in the number of tests conducted and in some other aspects (including bar size, deformation pattern, grade of steel, and slab design used), it was adequately designed to detect any significant differences in the behaviors of concrete fabricated with coating-disbonded bars and normal epoxy-coated bars.

The positive moment tests showed that the mere presence of epoxy coating on the bars, with either well- or partially-bonded coating, appeared to reduce the average ultimate diagonal tension capacity by 14 percent. When the positive-moment flexural behavior of different slabs were compared, no significant difference was observed between bars with partially bonded coating and bars with well bonded epoxy coating. Furthermore, no significant differences were observed in the comparison of the negative-moment flexural behaviors of the concrete slabs cast with the three types of bars. Results from the pull-out tests showed that the critical bond strengths for the bars with partially disbonded coatings were measurably lower than those for good epoxy-coated bars. However, even with 20 to 30 percent coating disbonded, the coated bars still developed 80 percent of the mean critical bond strength for bare bars. Essentially, the study indicated that even with 20- to 30-percent disbondment in the coating, the structural capacity of concrete reinforced with epoxy-coated steel bars was not compromised.

In response to the concerns raised in some portions of the United States, especially in Florida, about potential problem with the long-term durability of present epoxy-coated rebars, a series of investigations were initiated by FHWA in 1993 with the following objectives:

- To reexamine the effectiveness of epoxy coatings on the steel rebars in substructural bridge members exposed to a simulated marine environment and to identify the cause of the problem, if any.
- To determine the up-to-date overall performance of epoxy-coated rebars in concrete bridge decks exposed to actual severe service conditions.
- To search for the most corrosion-resistant organic coating systems for steel bars.

The following are presentation of the various findings resulting from these investigations.

(1) Reexamination of the Performance of Epoxy-Coated Rebars in Concrete Structures Exposed to a Marine Environment.

Following some early success of epoxy-coated rebars in controlling corrosion in bridge decks, some state highway agencies started using them in substructure concrete members in the marine environments, despite some major differences between the exposure conditions existing in bridge deck and marine applications. Reinforced concrete in a marine environment is subjected to:

- Relatively higher temperature throughout the year.
- A continuous supply of moisture and chloride.
- More wetting and drying cycles to provide penetration of oxygen and chloride.

All these conditions contribute toward lowering the resistivity of the concrete and thereby facilitating the flow of corrosion current between anodes and cathodes in both micro and macro levels.

Therefore, it was not totally surprising when in the late 1980s, reports about the premature deterioration of a number of substructure concrete members in the splash zones of three bridges located in the Florida Keys began to circulate. Even though there were different opinions about the quality of materials and construction practices employed in those bridges, nevertheless it was not expected to encounter in such a short period the severity of corrosion observed. This report led to claims that: (a) when exposed to marine environments, epoxy-coated rebars are more susceptible to corrosion than bare rebars; (b) epoxy coatings are, therefore, not effective in providing long-term protection to rebars in salt-contaminated concrete, even in bridge decks; and (c) the technology of organic-coating of rebars, as practiced in North America, is (probably) flawed. Responding to this concern, a study was initiated to reexamine the effectiveness of epoxy coatings on the steel rebars in substructure bridge members exposed to a simulated marine environment and to identify the cause of the problem, if any (*27*).

In this study, two parallel sets of experiments were conducted to investigate: (a) anodic growth as a function of immersion time; and (b) cathodic disbondment plus wet-adhesion loss. Quantification of anodic growth was made using (nondestructive) infrared thermography; whereas, quantification of disbondment was made with a specially developed peel test apparatus. The variables included in each experiment were: (a) two commercial epoxy coatings — designated as Coating 1 and Coating 2; (b) two coating thicknesses—approximately 139 and 190 μ (5 to 8 mils); (c) two coating conditions—unscribed and scribed; (d) one immersion solution - saturated $Ca(OH)_2$ solution with 3.5 percent (by wt.) of NaCl; and (e) two immersion-solution temperatures—35°C and 50°C (95°F and 122°F). Approximately 200 steel panels coated with the two coatings were used in the experiments. The following observations were noted:

- Regardless of the coating used, none of the unscribed panels exhibited any visible sign of corrosion, even after 3,072 hours in the immersion solution at 35°C.

- As early as 2 to 5 hours after immersion at either 35 or 50°C, some of the scribed panels started to have anodic sites, within 2 to 4 mm from the edge of the scribes. Then, within 24 hours, all the scribed panels started to corrode and produced a black pasty material, that was thought to be magnetite (Fe_3O_4), underneath the coating. This confirmed that the corrosion rates of epoxy-coated steel panels depend largely on the presence or absence of defects through the thickness of the coating, since the diffusivity of chloride and sodium ions through intact coatings is known to be low.

- An increase in the temperature of the immersion solution from 35 to 50°C greatly accelerated the rate of corrosion of the scribed panels. The scribes acted like cathodes and remained free of corrosion, while corrosion take place around the scribes.

- Anodic sites have tendency to grow in size. When the number of anodic sites and the total area covered by the anodic sites were measured for panels immersed in the 50°C solution, it was found that after approximately 100 hours of immersion, the accumulative number of anodic sites did not change but that the total anodic area increased. After 500 hours of immersion at 35°C (or 100 to 500 h at 50°C), liquid- filled blisters started to form above each anodic site. Analysis of the blister fluid indicated that the concentration of Cl^- was 4 to 6 times greater than that of the bulk electrolyte, while the concentration of Na^+ was about the same as in the bulk solution. The pH of the fluid ranged from 5 to 5.5.

- This increased activity at the anodic sites was accompanied by an increase in cathodic reaction at the edge of the anodic sites, which produces OH^- and causes the coating to disbond from the steel. This process of cathodic disbondment contributed to a reduction in the strength of the bond between the coating and the steel. It was also found that the rate of cathodic disbondment was not affected by the type and the thickness of the coating, but was affected by the temperature of the immersion solution.

- At beyond the disbondment front, loss of bond strength occurred through wet-adhesion loss, whereby water molecules accumulated between the coating and the steel substrate. This process was influenced by coating type and immersion temperature, but not by coating thickness. However, unlike cathodic disbondment, wet-adhesion loss was found to be recoverable once the coating is dried.

- Eventually through the combined action of cathodic disbondment and wet-adhesion loss, the bond strength of the coating to the steel panels was lost so that the coating could be easily removed.

The basic implication of all these findings is that for epoxy-coated rebars to protect reinforcing steel from corrosion, it's extremely important that the coating is free of any significant damage before being embedded in concrete and special care be observed to avoid damaging the coating during concrete pouring and consolidation. Hence, strict compliance with specifications and use of vibrators equipped with rubber-lined heads were strongly recommended.

(2) Assessment of the Performance of Epoxy-Coated Rebars in Bridge Decks

Recently, 11 state highway agencies in the U. S. and the Canadian SHRP conducted investigations to evaluate the performance of epoxy-coated rebars on 92 bridge decks, 2 bridge baffler walls, and 1 noise barrier wall, which were exposed to harsh service conditions. To shed some light on the controversy regarding the performance of epoxy-coated rebars, FHWA assembled and analyzed the data available from these investigations (*28*). The bridge decks evaluated included some that were constructed with epoxy-coated rebars used in the top mat of reinforcement only and in both the top and the bottom mats. In addition, 7 different types of epoxy powders were involved in these structures, where the epoxy-coated rebars had been in service up to 20 years at the time of the investigations. In-situ and laboratory evaluations of the concrete and the epoxy-coated rebars were typically included in these investigations. The methodology used in the field or in-situ evaluations included some or all of the following:

- Visual examination of the concrete for cracking, spalling, and patches.
- Chain dragging to locate concrete delamination.
- Use of pachometer to determine concrete cover and to locate reinforcing steel.
- Drilling of concrete powder samples for determination of chloride content.
- Coring for determination of the quality and the chloride content of the concrete.
- Overall rating of deck condition.
- Survey of half cell potential.
- Survey of concrete resistivity.
- Determination of corrosion rate through measurement of linear polarization resistance.

The laboratory evaluations used some or all of the following procedures:

- Visual examination of the concrete in the extracted cores.

- Measurement of the concrete cover over the ECR in the extracted cores.
- Evaluation of the segments of ECR extracted.
- Measurement of the thickness of epoxy coating on the extracted ECR segments.
- Determination of the total or water-soluble chloride content in the concrete.
- Determination of the permeability of the concrete (in the extracted cores).
- Determination of the pH of the concrete adjacent to the ECR in the extracted cores.
- Determination of the compressive strength of the concrete.
- Determination of the unit weight of the concrete.

The results from the various investigations were summarized according to:

- <u>Concrete Cover and Chloride Content.</u> All the structures investigated appeared to have adequate concrete cover—with only 10 (5.4 percent) out of 184 extracted cores that were measured had concrete covers of less than 51mm (2 in). The concrete surrounding the rebars in many of the structures had accumulated sufficient chloride ions to exceed 0.6 kg/m^3(1.0 lb/yd^3), which is considered by many as the corrosion threshold level. In fact, of the 40 bridge decks, wherein the average total chloride contents in the concrete were measured, 33 (83 percent) decks had chloride content equal to or greater than this threshold level. In addition, in 11(28 percent) decks the chloride content was ≥ 3.0 kg/m^3(5.0 lb/yd^3), with the highest content being 6.8 kg/in^3(11.5 lb/yd^3). In another 16 bridge decks where the water-soluble chloride contents were determined, the chloride content was ≥0.6 kg/m^3 (1.0 lb/yd^3) in 5 (31 percent) of these decks.

- <u>Condition of Epoxy Coating on the Rebars.</u> Some of the segments of epoxy-coated rebars extracted from the cores were examined for holidays, thickness of the epoxy coating, and, on some selected bars, the blast profiles. Most, if not all, of the segments examined contained holidays or bared areas. The thickness of the coatings was generally within the limits specified at the time of construction. When the coating thickness did not meet specifications, it exceeded the upper limit, in most cases. (In present construction, the coating thickness is higher at the lower limit and is about the same at the upper limit. Overall, the distribution of coating thickness is much narrower now. Field data indicate that increasing the thickness at the lower end is more beneficial, as far as resistant to corrosion of the bars is concerned.)

- <u>Condition of the Steel.</u> The segments of coated rebars were also examined to determine the condition of the steel itself Of the approximately 202 rebar segments extracted from some of the bridge decks, 81 percent did not have any corrosion present. For some of the remaining rebar segments, it was suspected that the corrosion was already present at the time of construction, since the chloride contents in the surrounding concrete were below the corrosion threshold level. Only 4 (2 percent) rebar segments were reported to have significant corrosion. Similarly, 8 (80 percent) of the 10 rebar segments extracted from the barrier and noise walls did not have any corrosion present; while 1 rebar segment was reported as having significant corrosion. In general, the corroded segments came from locations in the structures where concrete cover was relatively shallow, the chloride contents were high and/or there were cracks in the concrete. Furthermore, the corrosion found on the rebar segments were typically located underneath visible holidays in the coating or where the coating was missing completely.

- <u>Adhesion of the Coating to the Steel.</u> With the exception of California, Indiana, and Michigan, none of the other states had reported any significant reduction in coating adhesion on the extracted rebar segments. In California, where a total of 32 ECR segments were extracted from bridge decks that were 7 to 10 years old, reduction in coating adhesion of various extents was found on 12 (38 percent) segments, on both non-corroded and corroded areas. On these 12 segments, the loss of coating adhesion varied from 3 to 100 percent of the rebar surface—with 6 segments having adhesion loss on more than 75 percent of rebar surface. And, visible holidays were found in 11 of these segments.

  In the Indiana investigations, which involved 6 bridge decks and slabs that were 8 to 17 years old, none of the extracted epoxy-coated rebar segments showed any signs of loss in coating adhesion. In fact, it was difficult to strip the coatings, even with a knife, that mechanical means was used for stripping the coatings from some steel rebars to allow examination of the coating underside.

Michigan evaluated 12 bridge decks that were 10 to 15 years and found that the epoxy coating on the rebar segments extracted from moist concrete in 3 experimental decks was easily removed by fingernail. However, on almost all of these rebar segments, steel corrosion was non-existent or very minor.

Preliminary results from an on-going investigation in Ontario, Canada, where rebar segments were extracted from some 12 bridges and examined, indicated that coating adhesion appears to decrease with time. On 73 percent of rebars segments from bridges built between 1979 and 1980, the coating can either be pried up in small pieces or peeled off from the steel. In contrast, only 40 percent and 12 percent of the rebar segments extracted from bridges built during in the period of 1982 to 1985 and 1990, respectively, had coatings in the same adhesion conditions. In addition, using the dry-knife adhesion test, the Canadian SHRP study found that the coating on 54 percent of the rebar segments, extracted from structures that were in service for 3 to 16 years, was very well bonded. The coating on the remaining 46 percent of the segments varied from somewhat easy to remove to totally disbonded.

- Condition of the Concrete. Examination of the structures indicated that the condition of the concrete was generally good. Some concrete cracks, ranging in severity from "little or none" to extensive, were found in some of the decks. Such cracking was generally transverse in nature, and was not attributed to corrosion of epoxy-coated rebars. Some concrete delamination was detected in only 10 of the bridge decks. Approximately half of the delaminations were about 0.1 m^2 (1 ft^2) in size. The rest varied from 0.3 m^2(3 ft^2) to approximately 2.8 m^2 (28 ft^2).

  Interestingly, a 19-year old bridge deck in West Virginia with a total deck area of 1,654 m^2(17,800 ft^2), had concrete delamination of approximately 3.7 m^2 (40 ft^2), or 0.25 percent of the total area. The largest of these delaminations was centered on a construction joint and was likely not corrosion related. According to transportation personnel in West Virginia, it is typical to expect 5-percent to 20-percent concrete delamination in a deck of the same age and design, using black steel bars. Even more interesting was an analysis conducted by Kansas of the most recent deck ratings for 757 bridges built from 1977 to 1994 using epoxy-coated rebars. It showed that only 6 decks (0.8 percent), built mostly between 1980 and 1984, had any concrete deterioration that may be associated with corrosion of the reinforcing steel. All of the older decks were in good condition with minor or no concrete deterioration.

  A comparison of the performances of epoxy-coated bars among these structures suggested that better performance was obtained when the coated bars were used in both top and bottom reinforcement mats than when used only in top mat. The overall results of these investigations indicated that, in general, the epoxy coating did not perform as well when the concrete was cracked as when the concrete was not cracked. In fact, no visible or neglible corrosion was found on rebar segments extracted from uncracked concrete, even when the concrete chloride contents were up to 7.6 kg/m^3 (12.8 lb/yd^3). This is not surprising, since cracks in the concrete provide moisture and chloride an easy and direct access to the epoxy-coated rebars as opposed to the normally slow diffusion process through sound concrete; perhaps more detrimental, the cracks probably allow the coating to remain wet longer than otherwise. The investigations also suggested that the use of quality concrete and adequate cover, proper finishing and curing of concrete, and proper manufacturing and handling of ECR are important in ensuring effective corrosion protection in concrete bridge decks.

In summary, the following conclusions can be made on the performance of epoxy-coated rebars in the bridge decks surveyed:

- ECR has provided effective corrosion protection of concrete bridge decks for up to 20 years. No maintenance had yet been performed on thousands of bridge decks constructed with ECR.
- A bridge deck in West Virginia had only 0.25-percent concrete delamination after 19 years of service life. The largest delamination was centered at a construction joint and was not attributed to rebar corrosion.
- No evidence of corrosion has been found on 81 percent of the ECR segments extracted from deck cores.
- Some of the corrosion was observed on ECR segments in concrete where the chloride concentrations were below the corrosion threshold level. This corrosion was attributed to superficial corrosion that was already present on the rebars at the time of construction.
-
- Most of the corrosion was observed on ECR extracted from cracked concrete, where chloride concentrations were high.
- In uncracked concrete, where moisture levels were typically nominal, ECR had tolerated higher concentrations of chloride. In fact, little or no corrosion was observed in uncracked concrete with chloride concentrations as high as 7.6 kg/m ^{3} (12.8 lb/yd ^{3}).
- The data from these field investigations, similar to those from previous and on-going research, have indicated that better corrosion performance is obtained when ECR was used in both mats of reinforcement than when it was used only in the top mat.
- Defects and holidays reduce the effectiveness of ECR in protecting steel bars from corrosion.
-

(3) Search for New Corrosion-Resistant Organic-Coated Bars

In 1993, following reports of contradictory performances of ECR in piles exposed to marine environment and in bridge decks exposed to deicing salts, FHWA initiated a 5-year research study to develop or identify corrosion resistant rebars that could provide a corrosion-free service life of 75 to 100 years (*29,30,31)*. This study investigated 60 different rebars, including rebars with organic, inorganic, ceramic, and metallic coatings, as well as solid metallic bars.

Among these 60 different rebars were steel bars with 33 different organic coatings, of which 22 were bendable and 11 were non-bendable coatings. The bars were obtained from 15 organizations in the United States, Canada, Japan, England, and Germany. Of these, 17 coatings were utilized in conjunction with new improved steel surface cleaning processes and/or chemical treatments. In addition, since reinforcing bars are typically subjected to various severe conditions during their transportation, storage, installation, and service, the testing was designed to simulate field conditions.

In the Phase I prescreening tests, these rebars were subjected to accelerated solution-immersion screening and cathodic disbonding (CD) tests, using both straight and bent (to 4 D) specimens, each with one 6-mm hole drilled into the coating to simulate severed damage in the coating (*29).* The adopted test conditions are presented in Table 3. For the immersion tests, four solutions, of different compositions and pHs, were selected to produce corrosion and disbondment that the researchers believed to represent what could be expected during storage at construction sites, installation operations, and in-service behavior within mature concrete (Table 3). The specimens were immersed in these solutions at 55°C for up to 28 days. After immersion for 1, 3, 7, and 28 days the coated bars were visually examined for blisters, cracks, corrosion, and adhesion loss. The adhesion of the coatings was evaluated on both the straight and the bent sections of the bars, using the knofe-peel adhesion tests described in ASTM G1, while the bars were wet and again after 1 and 7 days of air drying.

To assess coating quality, the coating industry has been utilizing cathodic disbondment tests, which are described in AASHTO M284, ASTM A775, ASTM D3963, ASTM G8, and ASTM G42. To make the CD test in this study more severe than similar tests conducted by others, this test was conducted on 4-D bent bars (instead of typically straight bars) to introduce bending stresses on the coatings. As Table 3 indicated, the tests were conducted at a potential of -1,000 mV (versus static potential) over a period of 28 days at 23°C in a solution of 0.3N KOH + 0.05N NaOH at pH 13.3. This solution was used because it had previously been shown to produce more disbondment of organic coatings than other solutions. After 1 hr, 7 days, and 28 days of CD testing, impedance measurements were performed on the specimens; and, coating adhesion evaluation of the specimens was conducted only after the 28-day period.

Table 3. Adopted Test Conditions

Test	**Solution Immersion**		**Cathodic Disbonding**
Specimen Type (Number)	**Straight bars (16)**	**4-D Bent bars (8)**	**4-D Bent bars (2)**
Solutions	1. deionized water (pH 7) 2. 3 % NaCl (pH 7) 3. 0.3N KOH+0.05N NaOH (pH 13.4) 4. 0.3N KOH + 0.05N NaOH + 3 % NaCl (pH 13.3) All at 55°C (13 1°F)	Same	0.3N KOH + 0.05N NaOH (pH 13.4) at 23°C (73°F)
Duration	1,3,7, and 28 d	Same	28 d (at 1000 mV)
Coatings Tested	22 bendable 11 nonbendable	20 bendable 6 nonbendable*	21 bendable 6 nonbendable

* applied on prebent bars

In Phase II, seven of the best performing coatings from Phase I and three other new coatings (the 3M 213 bendable epoxy coating and two non-bendable coatings—an epoxy and a vinyl) were vigorously screened (30). The adhesion of these 10 coating systems on straight, 4D, 6D, and SD bent bars were tested, after solution immersion tests and cathodic disbonding tests, which were conducted under the conditions listed in Table 4.

Table 4. Test Conditions

Test	**Solution Immersion**	**Cathodic Disbondment**
Specimen Type	1. Straight bars 2. 4-D bent bars 3. 6-D bent bars 4. 8-D bent bars	1. Straight bars 2. 4-D bent bars 3. 6-D bent bars 4. 8-D bent bars
Intentional Coating Damage		Two 6-mm drilled holes
Replicates	4 per solution	3
Test Solutions	1. Deionized water 2. 0.3N KOH+0.05N NaOH +9 % NaCl (Cl^-/OH^- 4.5)	1. 0.3N KOH+0.05N NaOH (pH 13.3)
Temperature	55°C (131°F)	23°C (73°F)
Potential Shift	-	-1,000 mV (CSE)
Test Duration	28 days	7 days

The following general observations were made:

- In the immersion test using deionized water (pH 7), any corrosion on the bars did not extend under the film, even if a coating was poorly bonded. Thus, loss of adhesion may not necessarily be a precursor to corrosion under the coating.

- Bending of coated bars induced significant strain on the coatings. This accounted for the observation of significantly better adhesion on straight bars than on bent bars, for both bendable and non-bendable coatings.

- In the immersion test, the non-bendable coatings exhibited significantly better adhesion than the bendable coatings. Furthermore, the adhesion of all coatings was significantly better when they were dry than when wet.

- In the cathodic disbondment test, the adhesion of all coatings appeared to be significantly better at locations away from the drilled hole (in the bar specimens) than at the hole. Overall, the non-bendable coatings exhibited excellent adhesion on more bars than the bendable coatings did—97 percent of the bars with the non-bendable coatings had marginal to excellent adhesion, while only 30 percent of the bars with bendable coatings had the same ratings. Furthermore, in contrast to what has been observed about coating adhesion in the immersion test, whenever adhesion is lost in the cathodic disbondment test, it was unlikely to be regained even when the coating was subsequently dried. This indicates that different adhesion deterioration mechanisms occur in each test.

The last phase of the study was aimed at evaluating the roles that various factors may contribute to the risk of long-term corrosion of epoxy-coated bars in uncracked and cracked concrete. The factors were: reduction in coating adhesion, percentage of coating damage allowed, hole size, bendable versus non-bendable coating technology, and use of steel surface pre-treatments (*31*).For this evaluation, 6 organic coatings had been selected:

- Two bendable epoxy coatings.
- Two non-bendable epoxy coatings.
- Scotchkote 213 epoxy coating (bendable).
- Post-baked epoxy coating (nonbendable).

These organic coatings—one from Phase I and five from Phase II—represent a very broad range of adhesion performances. The scotchkote 213 was also chosen since it is in most of the bridges currently in service. For control, a conventional black steel is also included. To more accurately represent real bridge structures, a new and even more severe test method that incorporates a new specimen configuration and cyclic salt ponding is being used. The following is a brief outline of this in-concrete testing of the rebars with the selected coatings:

- Rebar Parameters:
  -
  - Straight and bent.
  - Damage areas of 0.5 percent and 0.004 percent.
  - Exposure in uncracked and precracked slabs.

- Concrete Slab Parameters:
  - 300 x 300 x 175 mm—cast according to specifications for AASHTO Class A concrete, but with a w/c of 0.47 instead of 0.45.
  - Plastic dikes hold sodium chloride solutions on top of the concrete surfaces.
  - Two bars placed at 2.5-cm depth act as anodes and 4 bars placed at 15-cm depth act as cathodes.
  - Slabs are fabricated with and without 0.30 mm wide longitudinal cracks. The cracks are introduced to maximize exposure of the top rebars to chloride solutions.
  - Slabs are fabricated with 3 different configurations with 4 replicate for a total of 12 concrete slabs for each rebar type tested.
  - Wire connections to the rebars to permit constant measurement (by computer) of the voltage drops between the top and bottom bars for macrocell corrosion currents.

- Testing Schedule:
  - First 12 weeks: 3 days of drying at 38°C, followed by 4 days of ponding with a 15-percent NaCl solution between 16 to 27°C. (During drying, slabs are placed in hoods equipped with heat lamps that maintain a temperature of 38°C.)
  - Second 12 weeks: continuous ponding with the sodium chloride solution at 16 to 27°C.
  - 24-week cycles repeated 4 times.
  - Concrete slabs were autopsied at the end of 48, 72, and 96 weeks to reveal varying levels of rebar corrosion.

This in-concrete testing is complete. It is anticipated that analysis of the data and preparation of a report will be completed by the end of 1998 for 75 percent of the concrete slabs. The remaining slabs will be kept for long-term observation.
So far, the following are some of the major conclusions that have been drawn on the performance of ECR in simulated adverse environments in outdoor and indoor laboratory research:

- Defects and holidays in the coating would reduce the performance of ECR.
-
- ECR provided superior performance when it is utilized in both top and bottom mats of reinforcement in a bridge deck than when utilized only in the top mat.
- All coatings are permeable to water vapor transmission, which leads to some reduction in the adhesion between the coating and the steel, irrespective of presence or absence of chloride ions. However, adhesion is regained when the coating become dried again. Furthermore, despite having reduced adhesion, the epoxy-coating barrier systems are still very much functional, because they are still impermeable to chloride and oxygen.
- In addition, recent research has shown that the strongly adhered epoxies (where adhesion was enhanced by pre-treatinent of black steel prior to epoxy coating) performed relatively poorly in concrete at elevated chloride levels when compared to the weakly adhered epoxies. At this moment, it appears that adhesion does not correlate with corrosion performance.
- It also appeared that further research would be needed to understand the role of chemical pretreatment of the steel bars, coating adhesion, and ultimately the corrosion performance of coated rebars in salt-contaminated concrete for extended periods.

(4) Cost Impact and Potential Pay-Off from Use of Epoxy-Coated Rebars

(a) Cost Impact. For bridge engineers, cost and benefits are always important factors to consider when making materials selection. To illustrate the cost-benefit aspect of using ECR in bridge decks, consider the actual cost data for the construction of three bridge decks recently in Illinois using epoxy-coated rebars in lieu of black steel. The delivered on-site cost of uncoated black steel was $0.44/kg ($0.20/lb) and ECR was $0.62/kg ($0.28/lb). The cost associated with using the different rebars and the total costs of the decks are presented in Table 5. From this example, it is evident that the cost of decks constructed with epoxy-coated rebars averages about 1 percent more than if constructed with black steel.

Table 5. Costs of Bars and Decks

Deck	Cost of Bars ($)		Cost of Deck ($)	
	Black Steel	Epoxy-Coated	Black Steel	Epoxy-Coated
A	21,644	30,302	690,000	698,658
B	22,544	31,561	880,000	899,017
C	13,614	19,059	1,122,000	1,127,445

(b) Potential Pay-off The potential pay-off can be estimated conservatively on the basis that the ECR deck will provide at least twice the corrosion-free service life of decks built with black steel. On the federal-aid highway system, there are about 2.3 billion square feet of deck surface. So far, ECR has been used on only 100 million square feet of deck. Based upon the average replacement cost of $40/sq. ft. for bridge deck, the potential savings are: $40/sq.ft. x 100,000,000 sq.ft. = $4 billion in 50 years, or earlier. Considering that eventually the remaining decks built with black steel will need replacement, the estimated future savings on the decks will be: $40/sq.ft. x 2,200,000,000 sq.ft. = $88 billion in that same amount of time. Since the above estimated figures are for decks on the federal-aid highway system only, the potential saving is greater if the off-system bridges are also considered in the above scenario.

At present, epoxy-coated rebar is the most common protection system used by 48 state highway agencies. To date, fusion-bonded ECR has been used as the preferred protection system in about 20,000 bridge decks, which represent roughly 95 percent of the new deck construction since the early 1980s. And, the use of ECR has extended to other structures, such as continuously reinforced concrete pavements, parking garages, nuclear power plants, coal plants, aquariums, buildings exposed to the marine environments, and wastewater treatment tanks. At present, there are approximately 100,000 structures containing ECR. Due to the success and the confidence gained by using fusion-bonded ECR over the last 20 years, there are about 35 coating plants. In addition, there are a significant number of people employed in the manufacture of epoxy powders and fabrication of the epoxy-coated reinforcement.

b. Steel Bars with Metallic Coating and Alternative Solid Metal Bars

The success enjoyed by metallic coatings on protecting steel from corrosion in other environments has raised the prospect of similar success in concrete. Metallic coatings that can be applied on reinforcing steel to provide protection against chloride-induced corrosion in concrete can be classified into two categories: sacrificial or non-sacrificial (noble). Coatings made of metal such zinc, which has more negative potentials or less noble than iron, can provide sacrificial protection to steel. When this sacrificial coating on a steel is broken, a galvanic cell or couple is formed whereby the coating is slowly sacrificed. Noble metals such as copper and nickel can also be coated on steel; however, the protection exist only as long as the coating is unbroken, since any exposed steel is anodic to the coating.

Nickel and zinc claddings began to receive attention in the late 1960s and have been shown to be capable of delaying, and in some cases preventing, the corrosion of reinforcing steel in concrete (*32*,*33*,*34*,*35*). The nickel-clad bar is produced by applying a heavy layer of nickel to a billet before hot rolling to its final form. This results into a continuous surface barrier of wrought nickel, at least 0,025-mm (0.001-in) thick, with an underlying diffusion zone of alloyed nickel and iron, which provides additional corrosion protection in the event of a break in the wrought nickel. The corrosion resistance of nickel is high in alkaline chloride solution, and even if breaks occur in the nickel coating, corrosion of steel is not appreciably accelerated, even though steel is relatively less noble than nickel (*32*). The results of an 11-year period testing of nickel-coated bars in a marine environment showed that the coating was effective in delaying, or sometime completely preventing, corrosion of the rebars (*33*). However, steel bars cladded with adequate thickness of nickel are still expensive. In addition, additional research is required to ensure their effectiveness as a corrosion protection system.

(1) Galvanized Rebars

Zinc-coated, or galvanized, bars are produced by a hot-dip process, which consists of cleaning the steel by pickling it and then immersing it in molten zinc. Galvanized bars are typically dipped in a chromate bath to passivate the zinc surface and prevent it from reacting with the hydroxide in fresh concrete. Through this process, zinc is metallurgically bonded to the steel, providing a coating (usually not less than 0.086 mm or 3.4 mils) that is composed of an outer layer of pure zinc and a number of transitional zones of zinc-rich alloys. Since galvanized bars are commercially available, it was the subject of numerous laboratory (*32*,*36*,*37*,*38*) and field studies (*35*,*39*,*40*). The laboratory results of the performance of galvanized bars in concrete have been conflicting. (Perhaps the conflict pointed to possible influence of experimental techniques on the results and that there is a need to develop an evaluation method that can be standardized.) For example, it was found in one study that when concrete specimens were alternately exposed to 4-percent NaCl in the stressed state, those reinforced with galvanized bars cracked less and later than those reinforced with black steel (*38*). In another study, data obtained when lollipop concrete specimens were partially immersed in saturated NaCl solution showed corrosion began at roughly the same time for specimens made with galvanized and with black steel, suggesting that there was no benefit from galvanizing the steel bars (*36*).

A subsequent 10-year FHWA comparative testing of concrete slabs made with two different water-cement ratios and containing either conventional black steel or galvanized bars, wherein corrosion rate was estimated indirectly by measuring macrocell current between top- and bottom-mat bars, indicated that galvanized bars were subject to the same type of macroscopic corrosion as black steel bars (*41*). Furthermore, in concrete with 0.40 w/c, both the long-term exposure data and the rate-of corrosion data indicated that use of galvanized bars did not provide extra benefit over using black. However, in concrete with 0.50 w/c, when galvanized bars were used in both mats, the corrosion rate and the corresponding metal loss were about 30 percent and 22 percent, respectively, in comparison to black steel. (However, 86-percent reduction in corrosion rate and metal loss was achieved just by using black steel in concrete with 0.40 w/c.) And, in the same concrete, when galvanized bars were used only in the top mat, the corrosion rate was twice of that observed when only black steel was used in both mats. Essentially, these results suggested that the use of galvanized bars would not provide extra benefit over black steel bars, if water-cement ratio in the concrete is kept low. The results also pointed to the possibility of galvanic reaction between galvanized bars and black steel bars, when these bars are used in the same structure.

Field studies of the performance of galvanized bars in concrete structures exposed either to deicing salts or seawater have yielded conflicting results too. For example, accelerated field studies in Michigan over a period of six years have shown that galvanized bars will retard concrete delamination and spalling but will not prevent them, especially where cover over the reinforcement is shallow (*42*). Meanwhile, evaluation of salt-contaminated bridges that had been in service in Bermuda for up to 20 years noted no corrosion damage (*43*).

To resolve the discrepancy, a panel made a critical examination of literature on the performance of galvanized rebars from: (a) studies conducted in laboratory environment, (b) studies of model specimens exposed outdoor to either natural or artificial weathering, and (c) studies of reinforced concrete structures under service conditions (*44*). Briefly, the panel reported that these studies had found that:

- Laboratory studies with aqueous solutions had showed that zinc tolerates a higher chloride content than steel before corrosion begins, which at least partly explains why galvanizing delays the onset of corrosion in the outdoor specimens.

- However, there is indication from the outdoor exposure studies that once corrosion of galvanized steel bars begin, the rate at which corrosion-induced distress on the concrete occurs is more rapid than for black steel.
-
- Due to a lack of standardization in the methodology (such as specimen design, quality of concrete, exposure conditions, and criteria for distress) used in the various laboratory studies, it is difficult to extrapolate the laboratory results to field conditions for predicting service life.

- It was deemed impossible to predict their service life since most of the structures were not in service sufficiently long (at the time of the study) for chloride to accumulate in the concrete to a level at which corrosion of the reinforcement would be expected.

- In the case of some marine structures (where chloride levels are high), there is the possibility that high moisture contents cause insufficient oxygen to be available at the reinforcement to sustain significant corrosion activity.

- There is evidence that rapid corrosion will occur when galvanized and black steel are (used in the same structures and are electronically) interconnected in a salt-containing environment.

- When galvanizing is preceded by chromate treatment, the chemical reaction of zinc in the fresh concrete does not impair bonding of the steel to the concrete nor the mechanical properties of the reinforcing steel. Therefore, the same design criteria used for black steel bars can be used for galvanized steel bars.

Finally, based on the literature and the collective experience of its members, the panel estimated that, for a new concrete bridge deck with a 5.1-cm (2-in.) cover of 0.45 w/c concrete and assuming that normal construction practices are used, the use of galvanized steel bars may add 5 more years to the 10 to 15 years that is typically required for corrosion-induced distress to be manifested in unprotected bridge decks.

(2) Other Cladded Bars and Alternative Solid Metal Bars

With the ultimate goal of developing corrosion resistant reinforcement that will result in a 75- to 100-year design life for concrete structures, The FHWA sponsored a 5-year research project to evaluate corrosion-resistant bars. As part of that project, 24 different bars of various types were tested with newly developed accelerated tests that are considered to be more severe than most available test procedures and much more severe than typical field conditions (*45*). The 24 bars included 14 ceramic-, inorganic-, and metallic-clad bars:

(2) Ceramic-clad bars (using a micro-infiltrated macro-laminated coating),
(1) Inorganic zinc silicate-clad bar,
(1) Hot-dip galvanized bar,
(1) Zinc coated bar (using the Delot process),
(3) Zinc-rich cladded bars,
(1) Nickel-clad bar,
(1) Copper-clad bar,
(1) Copper alloy-clad bar,
(1) 304 stainless steel-clad bar,
(1) Galvalum (aluminum and zinc) clad bar,
(1) Reactive copper in an organic coating bar,

and 10 different types of solid metallic bars:

(1) Black bar,
(1) Titanium bar,
(1) Type 304 stainless steel bar,
(1) Type 316 stainless steel bar,
(1) Type 317 stainless steel bar,
(1) Type 304N stainless steel bar,
(1) Type XM-19 stainless steel bar,
(1) Nitronic 33 stainless steel bar,
(1) Corrosion-resistant steel alloy bar,
(1) Type C613000 aluminum bronze bar.

Initially, the 14 cladded bars, in both straight and bent forms, were tested for 28 days (in 112 cycles of 1.25 hours dipping in specified solutions and 4.75 hours drying in air) in two solutions (a 3-percent NaCl solution, which has a pH 7, and a solution of 0.3N KOH + 0.05N NaOH + 3-percent NaCl, which has a pH of 13). Both straight and bent bars were tested in three conditions: (1) as received, (2) with a 6-mm (0.25-in) hole intentionally drilled through the cladding, and (3) with cladding purposely abraded with black slag sprayed on the bars at a specified distance and then pressured for 5 minutes. Based on the polarization resistant data, along with visual examination of the condition of these 14 different cladded bars after 28 days of testing in the two solutions, the following five cladded bars were selected for additional testing:

- 304 Stainless steel cladded bar.
- Nickel cladded bar.
- Copper cladded bar.
- Ceramic cladded bar A.
- Ceramic cladded bar B.

These five cladded bars, along with the above-mentioned 10 different solid metallic bars, were further tested for longer periods and under more severe conditions. The very corrosive solutions and severe exposure conditions used sequentially in this test are tabulated in Table 6.

Table 6. Corrosive Solutions and Exposure Conditions

Test Order	Test Solutions	Test Duration
1	3 % NaCl (pH 7)	90 days (360 cycles)
2	3 % NaCl + 0.3N KOH + 0.05N NaOH (pH 13)	56 days (224 cycles)
3	9 % NaCl + 0.3N KOH + 0.05N NaOH (pH 13)	56 days (224 cycles)
4	15 % NaCl + 0.3N KOH + 0.05N NaOH (pH 13)	56 days (224 cycles)

The 3-percent NaCl solution was selected to provide the exposure to seawater that may be encountered by bars prior to installation into the concrete or at cracks after placement in concrete. So, after 90 days of exposure to this solution, the bars are subjected to a total of 168 days of additional exposure to the three more-severe solutions, thereby providing a total of 1,032 wet-and-dry cycles.

The measurements indicated that the polarization resistance of the black bars in the various NaCl solutions (at pH 13) averaged about 0.90, 0.51, and 0.26 ohm·m ^{2}, respectively. These correspond to corrosion current densities of approximately 29, 50, and 100 mAm ^{2} (2.7, 4.7, and 9.3 mA/ft ^{2}), respectively. Assuming that concrete cracking occurs after a metal loss of 0.0254 mm (0.001 in), the calculated metal losses for the current densities observed for the black bars at 56 to 168 day (d) accelerated tests indicated that the concrete could crack in 1 year or less.

Using the same assumption, the approximate time-to-cracking for concrete using the various reinforcements were predicted and shown in Table 7. And, based on review of the polarization resistance data and visual assessment for the 15 types of bars, the 10 best performing bars were determined to be: the 7 solid stainless steel bars, the stainless steel-clad bar, the aluminum bronze bar, and the titanium bar. However, after consideration of cost and other factors (including availability), only 4 bars were selected for additional testing, with 6 different epoxy-coated bars identified earlier. These 4 bars were the Type 304 stainless steel bars, the copper-clad bar, the galvanized bar, and a zinc alloy-clad bar (with a newer zinc alloy).

Table 7. Time-to-Cracking

Type of Bars	**Time-to-Cracking**
Zinc-clad	slightly better or worse than black bars
Zinc alloy-clad bars	slightly better or worse than black bars
Copper-clad bars	@ 5 years
Stainless steel-clad bars	23 to 50 years
Nitronic 33 bars	@ 100 years
Type 304 stainless steel bars	@ 100 years
I Type 316 sainless steel bars	@ 100 years
Titanium bars	@ 100 years

The 10 different bars and an ASTM A615 black bar (control), in both straight and bent shapes, were embedded in cracked and crack-free concrete test specimens of 9 varied test configurations, then subjected to in-concrete tests (*46*). As mentioned earlier, the final report for this ongoing research will be available by December 1998.

(3) Cost Comparison Between Black Steel, Epoxy-Coated, Stainless Steel, and Titanium Bars

As mentioned in an earlier section, selection of materials is usually based on performance and cost. To demonstrate that replacing black steel bars with the more expensive stainless steel bars may not be that significantly more costly, FHWA researchers used the actual 1994 costs associated with the delivered on-site costs of black steel and epoxy-coated steel bars at three bridge projects in Illinois, which were $0.44/kg ($0.20/lb) and $0.62/kg ($0.28/lb), respectively. And, according to industrial sources, the corresponding prices for stainless steel and titanium bars were approximately $2.60/kg ($1.20/lb) and $13.20/kg($6.00/lb), respectively. Based on these costs and the bid prices for the three bridges, the overall costs (including site work and finishing) of the three bridges for using stainless steel and titanium bars instead were estimated (Table 8). When these costs were compared with those bid prices for using black steel bars or epoxy-coated bars, it showed that use of stainless steel bars would have increased the initial total project costs by only 6 to 16 percent; while use of titanium bars would have increased the initial costs by 35 to 91 percent. Considering that use of stainless steel rebars in these bridges may increase the time-to-cracking of a concrete by 65 to 130 times, in comparison to black steel and based on data obtained from ongoing tests of these different bars in the pH 13 solution, the benefit-to-added cost ratios would have to be considered very favorable (*47*). Similar test data for the titanium bars indicated that its use would extend the time-to-cracking of concrete by approximately 130 times, which would not provide a benefit-to-added cost ratio as attractive as that for using stainless steel bars.

Table 8. Cost Comparison

	Cost of Rebar / Cost of Black Bar	Total Cost with Rebar / Total Cost with Black Bar		
		Bridge A	Bridge B	Bridge C
Black steel	1.0	1.00	1.00	1.00
Epoxy-coated	1.4	1.01	1.02	1.00
Stainless steel	6.0	1.16	1.13	1.06
Titanium	30.0	1.91	1.74	1.35

(4) Copper-Clad Steel Bars

Copper-clad reinforcing steel bar is another of the metallic coated bars that were recently selected for additional evaluation. This type of reinforcing bars was initially tested in concrete in 1980 and 1984. In 1980, FHWA fabricated an extensive series of large test concrete slabs that contained various types of reinforcement—including black steel bars (in concrete with and without admixed calcium nitrite), non specification epoxy-coated bars, and copper-clad bars—and had various amounts of chloride added to the top lift of concrete. Since then, these slabs have been inspected several times (*48*). The conditions of these slabs at different ages are shown in Table 9.

Table 9. Slab Conditions

Slab Type	Age of the Slabs (years)		
	2	7	"120"> 13
Black bars	Badly cracked and bars corroded	-----	-----
Black bars and calcium nitrite	No cracks, but minor rust spots	Fine to wide cracks and some spalls	-----
Nonspecification epoxy-coated bars*	No cracks	Cracked and bars corroded	-----
Copper-clad rods— in both mats	-----	-----	No cracking
Copper-clad rods— in top mat	-----	-----	No cracking
Black bars	-----	Severe cracking	Severe cracking

* Containing uncountable holidays and less than 0.80-percent damage in coating.

As the inspection results indicated, the copper-clad bars were far more resistant to corrosion than any of the other types of bars in the identical test concrete slabs, and providing protection even better than calcium nitrite inhibitor. The slabs containing black bars started to crack severely after 2 to 7 years, even when the concrete had calcium nitrite. The slab reinforced with an epoxy-coated bar that had uncountable holidays and less than 0.80-percent damage performed better, but finally showed symptoms of corrosion after 7 years. The slabs containing copper-clad bars in both mats and in top mat only, were still in good condition after more than 13 years of outdoor exposure. It must be noted that the average total chloride contents in the slabs containing the copper-clad bars were between 8.50 to 10.32 kg/m ^{3} (14.33 to 17.40 lb/yd ^{3}), which is at least 14 times of the corrosion threshold level. Examination of cores taken from these two slabs revealed that the cladded bars, with cladding thickness of about 0.5 mm (0.02 in.), had discolored the surrounding concrete to a gray-green color. Petrographic examinations indicated that there was significantly higher amount of unhydrated cement around the copper-clad bars—extending 0.25 to 0.5 mm (0.01 to 0.02 in.) into the concrete—than at other locations. It is well known that copper, lead, and zinc salts can be retard hydration of cement. Despite the presence of unhydrated cement, the cement paste surrounding the bar appeared to be sufficiently hard in the concrete.

These findings correlated well with the results from an earlier 1984 study, which involved 48-week cyclic wetting (in 15-percent sodium chloride solution) and drying tests of concrete specimens reinforced with copper-clad bars. The copper-clad bars showed similar good corrosion resistance—during the 48-week ponding test, none of the specimens that contained the copper-clad bars exhibited signs of chloride-induced corrosion activity. Similar to the 1980 study with the same type of bars, discoloration of the concrete and retardation of cement hydration in concrete surrounding the copper-clad bars were observed at the end of the testing. In addition, the copper cladding was generally blackish in color—probably the normal copper oxide film formed after the fabrication of the specimen.

Due to the exceptional corrosion resistance shown by copper-clad bars in these studies, along with successful performance of these bars in the recent 168-day screening tests that was described earlier, it was decided that these bars should be tested further in concrete. These tests, as mentioned earlier, are under way and the results will be available in December 1998. It is clear that this type of reinforcing bar has the potential of becoming a cost-effective option for corrosion protection system, since the cost of copper-clad bars could be under $1.20/kg ($0.54/lb). However, further study on the structural effect of the retardation of cement hydration is required, prior to using such bars in bridge structures.

4. Corrosion Inhibiting Admixtures

In the past decade, a promising new approach to controlling steel corrosion in concrete structures is the incorporation of a corrosion inhibitor in the concrete mixes. Corrosion inhibitors are materials, both inorganic and organic, that are added to water or other liquids or gases in small amounts to reduce or completely stop corrosion. Inhibitors have been classified in many ways, including by composition and mechanism of action. Substances that retard corrosion by forming protective precipitates or by removing an aggressive constituent from the environment are also considered as inhibitors. The mechanism of inhibition is often complex and varies according to the type of inhibitors. For simplicity, the major inhibitor classifications are:

- <u>Anodic Inhibitors.</u> These inhibitors passivate the metal by forming an insoluble protective film on anodic surfaces or by adsorption on the metal. Examples of this type of inhibitors are chromates, nitrites, molybdates, alkali phosphates, silicates, and carbonates. Certain anodic inhibitors, e.g., nitrites, can cause accelerated corrosion and pitting attack if used in insufficient concentrations.

- <u>Cathodic Inhibitors.</u> These inhibitors are generally less effective but safer than the anodic inhibitors, and function by forming an insoluble film or adsorbed on cathodic surfaces of a metal. Examples of cathodic inhibitors are zinc, and salts of antimony, magnesium, manganese, and nickel.

- <u>Organic Inhibitors.</u> These inhibitors function by blocking both anodic and cathodic reactions by adsorption on the entire surface of the metal. This type of inhibitors includes amines, esters, and sulfonates.

With the widely marketed calcium nitrite, inhibition occurs only at addition dosage that is sufficiently high to counteract the effect of chlorides. In addition, this inhibitor is water-soluble and is, therefore, subject to leaching from concrete and hence would become less effective after some time. Some of the inhibitors may cause adverse effects in concrete, such as low concrete strength, erratic setting times, efflorescence, and enhanced susceptibility to alkali aggregate reaction. Finally, the real long-term benefit provided by these materials in concrete - the commercial corrosion inhibitors in particular - remains uncertain. Despite the risks and uncertainty, the use of corrosion inhibitors, both inorganic and organic, as admixtures for the corrosion protection of steel in reinforced and prestressed concrete has increased in the last fifteen years. A recent survey (*49*) indicated that, at present, there are four commercial corrosion inhibiting admixtures. These are:

- Darex Corrosion Inhibitor (DCI and DCI-S) (W. R. Grace). The active ingredient in this admixture is calcium nitrite, in approximately 30 percent. The typical dosage is approximately 10 to 30 liters per cubic meter (2 to 6 gallons per cubic yard) of concrete, depending on the expected extent of chloride exposure. This inhibitor provides "active" protection of reinforcing steel by facilitating the formation of a passive oxide film on the steel surface, through the following reaction:

$$2\,Fe^{++} + 2\,0H^{-} + 2\,NO_2^{-} \rightarrow 2\,NO\uparrow + Fe_2O_3 + H_2O \qquad (14)$$

-
- Rheocrete 222 $^{+}$ (Master Builder). The active ingredient of this corrosion inhibiting admixture is a water based combination of amines and esters. According to its manufacturer, this organic mixture forms a protective film, by chelation process, on the reinforcing steel to serve as a barrier for actively inhibiting both the anodic and cathodic reactions. In addition, this mixture also lines the pores in the concrete with hydrophobic chemical compounds that provides a screen to "passively" reduce the ingress of chloride ions into the concrete (*50*). The recommended dosage is 5 l/m^3(1 gal/yd^3) of concrete, irrespective of the severity of the exposure environment.

- Ferrogard 901 (Sika). This is a water-based blend of surfactants and an amino alcohol (or alkanolamine), specifically dimethyl ethanolamine (DMEA). Using x-ray photoelectron spectroscopy (XPS) and secondary ion mass spectroscopy (SIMS), its manufacturer showed that the amino alcohol absorb on the steel bars by forming bond between the its amino functional group and the hydroxide group on the steel surface. This leads to the formation of insoluble iron oxide complexes, which stabilize the oxide surface and inhibit further corrosion. The recommended dosage is 10 l/m^3(2 gal/yd^3) of concrete, irrespective of the severity of the corrosive environment, irrespective of the severity of the exposure environment.

- Catexol 1000 CI (Axim Concrete Technologies). It appears that this corrosion inhibiting admixture combines both an organic film-forming inhibitor, specifically an amine derivative, and a nitrite-based inhibitor. (No specific data is available for this admixture.)

The bulk of the performance data for these corrosion inhibiting admixtures are obtained mostly from laboratory tests conducted by their manufacturers - using simulated environment and/or small concrete specimens; the rest come from investigations conducted by FHWA, some of the state transportation agencies, and the academia. Most of these data pertain to DCI, because this commercial inhibitor was introduced in 1978 while the others were introduced in the last five years. Due to the long time required to assess their performance and the lack of follow-up studies once a structure is built, field performance data on any of these admixtures are still very limited. Therefore, it is not yet possible to accurately define the actual benefits that can be expected from the use of a specific corrosion inhibiting admixture other than to state that delay of corrosion initiation is anticipated.

Investigations of the use of corrosion inhibitors in concrete started in the 1960s and included sodium nitrite, and the sodium and potassium salts of chromate and benzoate. It was found that the sodium and potassium salts of chromate and benzoate gave mixed results on inhibition and reduced concrete strength. In one early study, it was shown that nitrite and chromate were effective inhibitors (*51*). A subsequent study indicated that only sodium nitrite was effective but it exhibited deleterious effects on the strength of the concrete (*52*). In the late 1970s, calcium nitrite was introduced commercially as an alternate form of nitrite (*53*).

In 1980, the FHWA began an outdoor exposure study to assess the effectiveness of calcium nitrite as an admixture for inhibiting steel corrosion in concrete (*54*). Eighteen reinforced concrete slabs (0.6 m x 1.52 m x 0.15 m) were built to simulate bridge decks using bared black steel bars. Of these slabs, 13 slabs contained various amount of admixed chloride and 2.75-percent calcium nitrite (by weight of cement) in the top lift and 5 control slabs without admixed nitrite. These slabs were tested by periodic measurements of macro-cell corrosion current, half cell potential, driving voltage (i.e., potential difference between the top- and bottom mat reinforcing steel), and concrete electrical resistivity (i.e., resistance between the top and bottom mats), as well as visual inspection. Based on data at the time, it was concluded that:

- Calcium nitrite inhibitor appeared to function by not allowing a large electrical potential difference to develop between the bars, thereby preventing any voltage difference that may drive corrosion, and keeping the electrical potential of all steel bars at or near the passive range.

- Calcium nitrite inhibitor was effective in reducing the rate of corrosion of black steel bars, up to a chloride-to-nitrite ratio, $[Cl^-]$ /$[NO^-]$, of 1.79. At the ratio equal to or less than 1.25, 10 or more years would be required to consume the same of iron as that consumed in 1 year in a concrete with equal amount of chloride.

However, after testing of the same 18 concrete slabs were continued for 7 years, it was found that calcium nitrite was effective in reducing the rate of corrosion of black steel up to a $[Cl^-]$ /$[NO^-]$ of only 0.90, instead of the higher $[Cl^-]$ /$[NO^-]$ of 1.79 indicated earlier (*55*). In fact, the slabs fabricated with higher ratios, even up to only 1.11, showed some cracking and rust spots on their surfaces after 7 years, despite exhibiting a reduction in corrosion current by a factor of 10. Since the chloride and the nitrite were added at the same time when the test slabs were being fabricated, it is reasonable to assume that a $[Cl^-]$ / $[NO^-]$ higher than 1 may still be able to suppress steel corrosion, if the chloride ions penetrates instead naturally into the concrete as in a structure. However, the researcher still recommended that, if selected as the protection system in a new construction, sufficient quantity of calcium nitrite should be added to the fresh concrete mix so that the $[Cl^-]$ / $[NO^-]$ will not exceed 1.0 at the level of steel—throughout the expected design life of a structure.

Recently, several new inhibitors and the first three commercial inhibitor admixtures were examined in a preliminary study of inhibitors conducted in Virginia (*56*). Using electrochemical impedance spectroscopy and visual observation, the investigators measured and ranked the overall inhibition performance of these inhibitors in simulated pore solution. It was determined that the top three inhibitors were barium metaborate, disodium β-glycerophosphate, and one of the commercial inhibitors. A follow-up study is under way to allow: (a) additional testing of these inhibitors in concrete to determine if their use has any adverse effects on setting and other properties of concrete, (b) development of an unique method of inhibitor delivery for alleviating such adverse effects, if there is any, and (c) expansion of the search to other new inhibitors (*57*). Another study is under way in Florida to (a) evaluate the ability of commercial inhibitors to stay in the concrete, which would influence their effectiveness, (b) estimate the long-term effectiveness of inhibitors, (c) determine possible adverse side effects on the corrosion process and the concrete properties, (d) quantitatively assess extension of concrete durability, and (e) establish their suitability for applications in new construction and in rehabilitation (*58*).

Recently, the commercial inhibitors—particularly the first three products—have been promoted for use in repair of corrosion-damaged concrete in bridges, either by: (a) addition as admixture to patching concrete, (b) spray application onto affected areas, or (c) saturation treatment to affected areas. Unfortunately, data on the effectiveness of this approach of using inhibitors are relatively limited. Therefore, an on-going study in Virginia is attempting to develop some data from actual bridge rehabilitation projects and concrete slabs fabricated to simulate bridge situations for assessing the effectiveness of these commercial inhibitors when used in rehabilitation (*59*).

Due to lack of widely accepted standard evaluation test that can truly be applied to all types of inhibitors, regardless of their inhibition mechanisms, comparison of the effectiveness of different inhibiting admixtures present difficult challenges. It's likely that a system of different acceptable tests would have to be developed. Furthermore, the effectiveness of all inhibitors should be evaluated based on per unit weight of the active ingredient(s) - the commercial corrosion inhibiting admixtures in particular. Unfortunately, most of the manufacturers are reluctant to share information on the compositions, or even just the concentrations of the active ingredients, of their products with the users.

In summary, corrosion inhibitors are increasingly being employed as a part of multiple corrosion protection systems in conjunction with epoxy-coated rebars and low-permeability concrete. As yet, epoxy-coated seven-wire strands are not usually employed for prestressed concrete bridge members. In lieu of coated seven-wire strands, corrosion inhibitors have found their niche in the prestressed highway construction industry. In addition, corrosion inhibitors are finding use in cementitious grouts for filling the post-tensioning ducts of bridge members or the sheating of cable stays to protect the highly tensioned and uncoated black steel. Due to the premature failure of epoxy-coated rebars in the splash zone of piles on the Florida Keys bridges, some state highway agencies are relying on corrosion inhibitors as one of the alternative corrosion protection systems for marine application. For bridge deck exposed to excessively aggressive environment, the use of a corrosion inhibitor, in conjunction with ECR, as a redundant corrosion protection system, can enhance the service life of a structure with respect to corrosion. (Industry-sponsored research is under way on the combined use of ECR and corrosion inhibitors as a dual corrosion-protection system for bridge decks.) In addition, the corrosion inhibitors can play an important role in protecting uncoated high-strength steel in PS/C bridge members and cable-stayed bridges. Research is continuing to identify other corrosion inhibitors that will provide long lasting corrosion protection to bridge members.

5. Corrosion Protection of Prestressed Concrete Bridge Members

In the fall of 1992, the United Kingdom Ministry of Transportation imposed a temporary ban on the commissioning of grouted, bonded post-tensioned bridges. This resulted from the collapse of two footbridges in 1960, the collapse of a single-span, segmental post-tensioned bridge in Wales in 1985, and an examination of nine other segmental bridges. United Kingdom is not the only place with problems of voided ducts and insufficient coverage over prestressing steel in ducts, as evident by the collapse in 1992 of the post-tensioned Melle Bridge across the Schelde in Belgium, which was constructed in 1956. This last failure was traced to the corrosion of the ducted post-tensioning wires even though the bridge had been inspected, load tested, and rated satisfactorily. The UK moratorium was lifted in 1996 with the publication of advisory report "Post tensioned Concrete Bridges: Planning Organization and Methods for Carrying Out Special Inspections" by the construction industry and owners of this type of bridges.

The underlying difficulty is that there are no reliable, cost-effective, and rapid nondestructive methods for providing assurance to the owners that the built structures have met the construction specification. One of the major inspection concern is determining whether the ducts in the post-tensioned bridge members have been completely filled with the grouts and that there is uniform coverage over the prestressing steel. It has been found many times that, invariably, the ducts had big voided sections and were filled only partially. In addition, it is very difficult to assess the condition of anchorage areas.

Chloride-bearing water can find its way through anchorage into the ducts and eventually initiate corrosion at the anchorage and subsequently of the prestressing steel inside the ducts, especially if the ducts have voids and are partially filled. The salty water can also access the ducts through faulty and leaky joints. In time, the chloride ions also penetrate through concrete cover and corrode the prestressing steel in the ducts, either after the corrosion of the metallic ducts or through defective plastic ducts. Beside causing pitting of the prestressed strands, corrosion reactions lead to the evolution of atomic hydrogen, which is subsequently absorbed into the steel leading to hydrogen embrittlement of the steel. This embrittlement can cause the strands to fail much more readily under normal loads. Since prestressed concrete bridge members rely on the tensile strength of the strands to resist loads, loss of even few strands per member can prove catastrophic. In addition, due to the high stresses the strands are subjected to, corrosion effects are accelerated. Even though corrosion initiate on prestressed steel in the same manner as on mild reinforcing steel, there is one significant difference between the two: since prestressed wires have smaller cross sections than rebars, sections of prestressed wires are more readily broken or completely rusted away.

Corrosion protection methods adopted at present in the construction of prestressed concrete members included: (1) the use of highly impermeable concrete through the use of fly ash and silica fume and controlled curing at the fabrication site, and (2) the use of corrosion inhibitor admixtures. The use of epoxy-coated steel is not yet common in prestressed concrete members and research on this application of coated strands is needed.

The main thrust of FHWA field and laboratory investigations in this area of prestressed concrete consisted of the following:

- Determination of the condition of oldest prestressed concrete bridges located in adverse environments. This is to ascertain whether the same problems observed in Europe exist in the United States.
- Develop methodology for inspection of grout coverage over prestressing steel in the ducts of post-tensioned bridge members.
- Develop corrosion resistant grouts and methods to test them in the laboratory before their use in the field.
-

a. Field Investigations

Many problems associated with deterioration of prestressed concrete members can be traced to improper designs in a structure. To help identify some common examples of improper design, a total of 17 prestressed concrete bridges were surveyed between 1988 and 1992 and the collected data were analyzed (*60*,*61*). Built between 1954 to 1979, eleven of these bridges were pre-tensioned and six were post-tensioned—with some of the bridges containing both types of members. These bridges were located both in northern climates exposed to application of roadway deicing salts and in southern climates exposed to seawater spray. The procedures used in the examination included: visual examination, concrete delamination survey, chloride content survey, concrete cover survey, half-cell potential survey, rapid chloride permeability, and petrographic and metallurgical analysis of the prestressed steel.

The detailed examinations of these bridges revealed that, since most common sources of chlorides in concrete in the substructures are water running from bridge decks and spray from seawater, the corrosion of prestressed steel observed in these bridges was due commonly to one or more of the following causes:

- Water-permeable deck overlay.
- Leaking expansion joints.
- Poorly designed and malfunctioning drains.
- Inadequate concrete cover over the prestressed steel.
- Improper protection of the anchorage systems provided by mortar.
- Access of water and chloride into the sheathing through the anchorages.
- Poor grouting.
- Lack of grease in unbonded tendons.
- Exposure to direct action of seawater spray.
- High chloride permeability of the concrete, mortar, and grout.

Overall, the conditions of the bridges surveyed were judged to be adequate. And, to add to the durability of these types of concrete structures located in adverse environments, the following recommendations were made for:

- Pretensioned Box Beams.
    - Installation of protective membrane systems on the top flange of the beams.
    - Installation of a rigid wearing course or composite deck slab (provided the structure could accommodate the additional dead load).
    - Addition of stiffeners to prevent the movement of the beam faces.
    - Transverse post-tensioning of the beams to hold beam faces together.
    - Design of proper slope and drainage details to minimize the dwell time of deicer solutions on the deck.

- Pretensioned I-Beams.
    - Minimizing exposure of beam ends to deicing salts runoff from the deck. This can be done by running beams through diaphragms over piers at the center of the bridge (allowing beams to be made continuous) or by running beam ends into concrete abutments at the bridge ends.
    - Install and maintain a functional gutter and drainage system using large-capacity gutters and wide drainage pipes to carry water off the deck quickly.
    - Install rigid concrete overlay to prevent leakage of water through deck on to the superstructure.
    - Reduce or eliminate joints in the new construction and replace deteriorating deck joints.

- Post-tensioned Beams.
    - Use of corrosion-resistant grouts containing corrosion inhibitors, in addition to low w/c with silica fume and fly ash.
    - Use of plastic ducts with adequate thickness to eliminate puncturing by prestressing steel during pulling and grouting.

Finally, the researchers emphasized that routine inspection and proper maintenance are extremely important in eliminating major deck-to-steel water routes for prestressed substructure concrete elements.

b. Laboratory Investigation

(1) Corrosion-Resistant Grouts

Grout is the final line of defense against corrosion of steel tendons. Based on review of state highway agencies' specifications on grouts, it was determined that the design mixtures were inadequate and that there was no suitable method for evaluating the corrosion performance of grouts before their use in the field. Hence a study was initiated with the following objectives: (1) to develop new mixture designs, and (2) to develop and perform accelerated test methods on the new grouts. In this study, in addition to varying w/c, several modifiers and additives for grouts were examined, including high-range water reducers, fly ash, silica fume, latex-polymer modifier, expansive agents, anti-bleed additives, and corrosion inhibitors (*62*,*63*,*64*,*65*,*66*).

A large number of grout design mixtures were tested with an accelerated corrosion test method (ACTM). ACTM uses a test specimen that simulates a post-tensioning strand embedded in grout, but with a section of duct missing (representing a break in the duct). Other features of the test included minimal cover on the tendons, a 5-percent sodium chloride solution in direct contact with the grout, and forcing of chloride ions through the grout by an applied voltage. The study resulted in the following conclusions:

- Reduction in w/c had a marginal beneficial effect.
- Silica fume combined with low w/c had a significant beneficial effect. A 10-percent silica fume provided a two-fold increase in time-to-corrosion initiation.
- 33 percent (by weight of cement) flyash provided a three-fold increase in time-to-corrosion initiation.
- Latex polymer modifier and anti-bleeding admixtures reduced pressure-induced bleeding.

In summary, use of fly ash or silica fume, in conjunction with proper dosage of a corrosion inhibitor with a suitable anti-bleeding agent, can provide a grout which can protect the prestressing steel even if it is contained in defective ducts. Based on these recommendations, the Post-Tensioning Institute has written a grouting specification, which will be published in 1998.

(2) Inspection of Grout Cover Over Prestressing Steel and Voids in the Ducts

Prior to the discovery of problems with grouting of ducts in existing prestress sing concrete structures in UK and other parts of the world, little attention was given to this subject. Review of literature offered the following relevant techniques:

- Drilling and monitoring air flow at suspected locations. The disadvantages with this method are that it is a destructive method and there is always a slight chance of damaging or cutting the prestressing steel. In addition, the drilled hole, even after filling, becomes an avenue for future corrosion.
-
- Boroscope. This technique allows viewing of the inside of a duct by insertion of a fiber optic cable through a 12.5-mm (0.5-in) diameter hole drilled through the duct at the location of suspected void. A camera is typically used in conjunction with the optic fiber to record images of the ducts. This has the same disadvantages of the air-flow technique.
- Radiography. The French use a radiography system (Scorpion), which utilizes a portable linear accelerator to provide the ionizing rays, to inspect duct systems in post-tensioned concrete bridge members. With this system, they can inspect the grout cover and also find voids in the ducts containing prestressing steel.

Neither of these techniques are widely used in the United States, if at all, because the first two are destructive and the third technique, even though it is nondestructive, involves a bulky equipment and is expensive. Hence, FHWA initiated a research study to develop an equipment which is in the range of $15,000 to $25,000 and also not bulky. After completion of the first phase of this study, the impact-echo technique was selected for further evaluation and development, since it showed the most promise (*66*,*67*).

In this method, a short-duration stress pulse is sent through the duct location under inspection by striking the outer concrete surface. The reflected pulses are received by a transducer (which is in close proximity to the impactor) and is displayed on a computer screen as a frequency spectrum. The frequencies of the pulses reflected from a voided and/or partially filled duct are much lower than those reflected from a filled duct. This technique has been successfully used on a few bridges. The concrete cores taken from the suspected locations showing lower frequency had either a void in the duct or honeycombing around duct area. The correlation between the nondestructive impact-echo technique and the destructive examination of concrete cores was excellent.

B. Corrosion Control for Rehabilitation of Existing Concrete Structures

There are different remedial methods that can be applied, as part of rehabilitation of existing concrete structures, for controlling chloride-induced reinforcing steel corrosion. Each of the methods, which are used after all damaged concrete in a structure have been removed and patched, functions by one of the following principles:

- Providing a barrier on the surface of the existing concrete to prevent fixture ingress of chloride. The conventional rehabilitation methods belong in this category.
- Controlling the electron flow within the reinforced concrete environment to halt metal loss. Cathodic protection functions in this manner.
- Modifying the concrete environment to make it less corrosive. This is how electrochemical chloride extraction can extend the service life of chloride-contaminated concrete structures.

Conventional Rehabilitation Methods

> Methods that function by the first approach—such as application of an overlay of polymer concrete, low slump concrete, latex modified concrete, or high density concrete, on the existing concrete—provide a barrier that prevents continued intrusion of additional harmful chloride ions, moisture, and oxygen that are needed to sustain corrosion. In some cases, these methods have successfully extended the useful life of a structure. However, past experiences have also indicated that when such measures were used, without first decontaminating the existing concrete of the necessary corrosion ingredients, it was often that these substances become entrapped (in the old concrete) by the overlays in sufficient amounts to initiate new corrosion. Furthermore, the ensuing corrosion is often worse than before rehabilitation, due to arising electrochemical incompatibility between the new patching concrete and the surrounding old but physically sound concrete (*68*). The following is a brief summary of work performed on measures belonging to approach one.

FHWA undertook a long-term staff study in the 1980s to evaluate the effect of various conventional repair systems on the rate of corrosion of reinforcing steel (*69*). It had been debated whether the overlaying of a bridge deck with a relatively impermeable concrete or membrane would sufficiently arrest the corrosion to effect a permanent repair. The following overlays were placed on chloride-contaminated reinforced concrete slabs and tested:

- 50.8-mm (2-in) thick concrete overlay with 0.32 wlc.

- 50.8-mm (2-in) thick concrete overlay with 0.45 wlc.
- 31.7-mm (1.25-in) thick latex-modified (styrene-butadiene) concrete overlay, uncracked and badly cracked, with 0.40 wlc.
- 50.8-mm (2-in) thick internally-sealed concrete overlay with 0.50 w/c.
- 50.8-mm (2-in) AC overlay over preformed waterproof membrane.
- Silane surface treatment.
- Nominal 13-mm (0.5-in) thick built-up polymer concrete overlay (two with vinylesters, one with MMA, and one with polyester resins).
- Nominal 10-mm (0.4-in) thick acrylic (latex mortar with glass fabric overlay).
- Delamination patching only (as necessary; very little patching with salt-admixed concrete was actually done).

The concrete specimens were 1.2-m-wide x 1.5-m-long x 0.152-m-thick (4-ft. x 5-ft. x 0.5-ft.) slabs that were fabricated in 1971 using a top-mat reinforcing steel only. These slabs were subjected to daily salting for 8 years. All of the slabs in this phase of the study were fabricated using concrete with a 0.5 w/c and 25.4 mm (1.0 in) of clear concrete cover over the reinforcing steel, and exhibited some corrosion-induced concrete distress. Measured electrical potentials indicated the presence of corrosion on the rebars in all slabs. In order to make the slabs more closely simulate a bridge deck, a 63.5-mm (2.5-in) layer of chloride-free concrete containing a mat of reinforcing steel was placed on the bottom of each slab. Prior to concrete placement, the old bottom of each slab was sandblasted to exposed coarse aggregate. Care was exercised to insure the new bottom mat of steel was not electrically continuous to the top mat or bar chairs. Then lead wires were attached to the bottom-mat rebars and run outside the slabs. Other instrumentation at the bottom-mat level included thermocouples, and 152 mm x 50.8 mm (6 in x 2 in) copper mesh, which is used for measurements of concrete resistivity. The mix design and aggregate sources for the bottom-lift concrete were identical to that used for the original slabs. After concrete placement and 14 days of wet burlap curing, the sides were recoated with epoxy paint and the slabs were placed on 0.9-m (3-ft) posts. Six months of outdoor storage was used to re-establish a normal moisture gradient within the 0.216 m (8.5 in) simulated bridge deck sections.

Instrumentation was installed within each slab, at the top-mat rebar level, via slots cut into the surface. This included resistivity mesh positioned above that placed in the bottom lift, one or two rate-of-corrosion probes, thermocouples (at the top-mat rebar level and at the slab mid-depth), and new lead wire attachments to each top-mat rebar. Slot patch concrete had a 0.50 w/c and utilized aggregates from the same sources as the original concrete and contained chloride at a concentration of 11.9 kg/m ^{3}(20 lb/yd ^{3}). This chloride level was used because sampling indicated this to be the average chloride content which had penetrated to the top- mat rebar level during 8 years of daily salting. An instrumentation interface box was attached to the side of each slab and switch-wired in such a manner to permit the following measurements:

- Corrosion current flow between the top- and bottom-mat of rebars.
- Corrosion current flow between rate-of-corrosion probes and the rebar mats.
- Concrete temperature at eight points within the slab.

Following these measurements, the switch on each slab is deactivated and the following measurements are made as rapidly as possible:

- Electrical potential at three points on the top-rebar mat (taken through "wells" placed on the original top surface of each slab).
- Electrical potential at three points on the bottom-rebar mat (taken from the bottom of the slab).
- Electrical potential of each rate-of-corrosion probe (taken through one of the top surface "wells")
- Electrical resistance (1,000 cycles AC) between the rebar mats.
- Electrical resistance between each of the copper mesh (four measurements: R for mesh pairs 1 and 2 through the slab, R between the top mat mesh, and R between the two bottom mat mesh).
- Rate-of-corrosion probe readings (a resistance measurement made using a special meter).

After all these measurements, the couple switch on each slab is then reactivated and exposure of each specimen continues.

The major evaluation criteria in the testing are the macrocell corrosion current flow, between the top- and the bottom-mats of rebars in each slab, and the changes in current with time on each rehabilitated slab. The basis for comparison is the change on the slabs subjected to delamination repair only. By calculating a corrosion current ratio, defined as the 21 °C (70°F) corrosion current (at time x after rehabilitation) divided by the same corrosion current before rehabilitation of each slab, increases or decreases in corrosion rate can be monitored. Thus, a ratio of 1.0 indicates no change, while a value greater than 1 denotes an increase, and a value less than 1.0 denotes a decrease, in corrosion rate.

The corrosion current data on all rehabilitated slabs is summarized in Table 10 and can be used to define general trends. In the tabulation, are the 21°C (70°F) corrosion current ratio data for each test slab at approximately 0.4, 1.0, 1.4, 2.1, 3.1, and 4.1 years after rehabilitation, and averages for each variable.

Table 10. Corrosion Current Data on All Rehabilitated Slabs

Conventional Treatment	**21°C Corrosion Current Ratio, Years After Rehabilitation**					
	0.4y	**1.0y**	**1.4y**	**2.1y**	**3.1y**	**4.1y**
Patch delaminations only	0.96	1.17	0.85	1.04	1.08	0.79
Latex overlay (cracked)	0.88	0.81	0.59	0.77	0.93	0.98
Latex overlay (uncracked)	0.73	0.81	0.57	0.69	0.73	0.68
Concrete overlay (0.32 w/c)	0.91	0.84	0.67	0.73	0.79	0.91
Concrete overlay (0.45 w/c)	1.00	0.78	0.55	0.61	0.52	0.54
Internally Sealed Concrete	0.85	0.83	0.45	0.48	0.56	0.51
Preformed membrane and AC	0.92	0.94	0.81	0.79	0.73	0.57
Thin latex/fabric overlay	1.02	1.19	0.97	1.14	1.31	1.41
Polymer concrete (vinyl)	0.79	0.88	0.78	0.98	0.99	1.06

Polymer concrete(MMA)	1.26	1.67	1.61	2.02	2.26	1.89
Polymer concrete (polyester)	1.40	1.36	1.42	1.76	1.88	1.52
Silane treatment	0.88	1.29	1.09	1.29	1.23	1.15

The data for each time period after rehabilitation are the average of at least three separate measurements and up to 2 slabs were used for each treatment. These data indicated that none of the overlays, membranes, or sealers completely stopped corrosion of the rebars in the underlying salty concrete. Finally, rust stains, cracks, and delaminations appeared on almost all the slabs at only 4 years after rehabilitation, even though this testing was not accelerated. This indicates that such repair procedures are not permanent.

Traditionally, greater than 90 percent of the rehabilitation jobs used low-w/c concrete or latex overlay as the preferred method. A Strategic Highway Research Program report estimated the life of this rehabilitation method as about 30 years; however, state highway agencies believe it is somewhere around 15 years.

2. Cathodic Protection

Corrosion control measures such as cathodic protection, either by impressed-current or galvanic mode, function by the second approach. Their advantage is that the control of electron of electron flow overrides any tendency of the steel to corrode again because of electrochemical imbalances in the concrete. Removal of chloride ions from the concrete by electrochemical means belongs to the third approach.

The science of cathodic protection (CP) was born in 1824, when Sir Humphrey Davy made a presentation to the Royal Society of London: "The rapid decay of the copper sheeting on His Majesty's ships of war, and the uncertainty of the time of its duration, have long attracted the attention of those persons most concerned in the naval interest of the count. ... I entered into an experimental investigation upon copper. In pursuing this investigation, I have ascertained many facts ... to illustrate some obscure parts of electrochemical science... seem to offer important application." Davy succeeded in protecting copper against corrosion from seawater by the use of iron anodes. From that beginning, CP has grown to have many uses in marine and underground structures, water storage tanks, gas pipelines, oil platform supports, and many other facilities exposed to a corrosive environment. Recently, it is proving to be an effective method for protecting reinforcing steel from chloride-induced corrosion.

The Pourbaix diagram, a thermodynamic diagram of electrical potential versus pH, for iron pointed out that iron is immune to corrosion if its potential is more negative than in any naturally occurring condition, regardless of pH (*70*). The iron or steel can be made to be sufficiently electronegative by installing a suitable external electrode (anode) on the surface of or in the concrete, the electrolyte, and passing a current (of sufficient magnitude) from that electrode through the concrete to the embedded steel, which all become the cathode. There are two methods of supplying the required cathodic protection current: (a) by using an external DC source and connecting the anode to its positive terminal and the steel to the negative terminal, therefore called the impressed-current cathodic protection, and (b) by using an anode made of a metal or an alloy that is sufficiently more reactive or anodic than steel (such as magnesium, zinc, and aluminum) so that when this anode is electrically coupled to the steel, the sacrificial anode corrodes and gives away electrons, which flow to and protect the steel, therefore called galvanic cathodic protection.

Each method of applying cathodic protection has its own characteristics that make it more applicable to a particular situation and, therefore, more advantageous than the other. Table 11 provides a comparison of those characteristics.

Table 11. Comparison of Cathodic Protection Characteristics

Impressed-Current CP	Galvanic CP
External power required	Requires no external power
Driving voltage can be varied	Fixed driving voltage·
Current can be varied	Limited current
Can be designed for almost any current requirement	Usually used where current Requirements are small
Can be used in any level of resistivity	Usually used in low-resistivity Electrolytes

Protection of reinforcing steel from further corrosion in existing concrete bridge members can be achieved by either method, which one is more effective depends on the environment surrounding the structures.

Regardless of the method of CP used, it is critical to select the right anode, to determine the right amount of current or potential to apply to ensure that the steel is adequately protected, and to ensure that the protection current is applied uniformly across the structure being protected. Therefore, a considerable portion of FHWA research efforts was focused on the identification or development of the most suitable and durable anodes for use in the cathodic protection, either by the impressed-current or the galvanic mode, of each type of concrete bridge members and for the different types of environment that can surround that type of concrete members.

a. Impressed-Current CP of Concrete Bridge Decks

To install an impressed-current cathodic protection system, a few basic components are required, in addition to the reinforcing steel and the concrete in the structure to be protected:

- Controlled external DC power source (typically a rectifier).
- Current distribution system (anode system).
- Wiring.

- Evaluation or monitoring devices (reference cells, corrosion probes, etc.).

Among these components, the anode system is probably the most critical. For use on bridge decks, an anode system must have the following characteristics:

- Capability to withstand traffic loads and environmental influences.
- Sufficient durability to have design life equal to or greater than that of the wearing surface.
- Sufficient conductive surface area to minimize or completely prevent premature deterioration of surrounding concrete.
- Economical.

Since the first application of CP in 1973 in a bridge deck in California, which was an impressed-current system using an electrically conductive Coke-asphalt concrete overlay as an anode, other anode systems have become available. The following is a discussion of these various anode systems for bridge decks.

(1) Coke-Asphalt Anode System

In that first CP system for concrete bridge deck (*71*), a number of high-silicon cast iron disks were secured (with epoxy) on the deck to function as the primary anodes. Then, each primary anode was connected to a header cable back to a rectifier. To facilitate the distribution of the protection current across the deck and also extend the life of the cast-iron primary anodes, a 50-mm (2.0-in) thick layer of a secondary anode system—a conductive coke-asphalt mix—was placed over the cast-iron anodes and the entire deck, then covered by a nominal 50-mm (2.0-in) thick wearing course for a total thickness of about 100 mm (4 in). The conductive coke-asphalt mix, which resembled a typical asphalt mix, was developed from combination of asphalt and metallurgical coke breeze (in place of conventional aggregates). The coke breeze, which is high in carbon content, is a good conductor of electrical current but is weak structurally. That is why a strong surface wearing overlay made of conventional asphalt concrete was necessary. Unfortunately, this conductive coke asphalt anode system still exhibited some structural degradation after being in service.

To increase the stability and resistance to traffic loading of this conductive coke-asphalt mix, the Ontario Ministry of transportation and Communications modified it by addition of some conventional aggregate (*72*). This anode system is relatively inexpensive and had functioned effectively in several CP systems built in California and Canada. However, it has some disadvantages: (a) it adds dead load to a structure, (b) it requires the modification of expansion joints, drains, curbs, and approaches to accommodate the overlay system, (c) the uncertain durability of the entire overlay system, and (d) the risk of entrapping moisture under the overlay, which can lead to freeze and thaw deterioration of improperly air entrained concrete. These had briefly prevented wide acceptance of CP as an effective corrosion controlling method in rehabilitation of existing bridge decks.

(2) Non-Overlay Slotted Anode System

In 1980, a new method of distributing the protection current over a bridge deck was introduced. In this system, the old disk anodes were replaced by platinized niobium-copper wires laid in saw slots spaced regularly across the concrete surface and covered with grout (*73*). To provide an electrically conductive grout for grouting the saw slots, researchers in FHWA developed a conductive polymer concrete, made of vinyl-ester resin and carbon black (*74*). This material had a resistivity of less than 10 ohm-cm and a 4-hour compressive strength in excess of 4,000 psi. The development of this non-overlay slotted anode system contributed to significant advancement in CP of concrete bridge decks by eliminating the need for a thick overlay system of conductive asphalt concrete and asphalt wearing course, and thereby making it easier to install CP systems on bridge decks with minimal traffic interruption.

Following this development, one of the first impressed-current CP systems to utilize this non-overlay slotted anode system was built on a bridge deck in Buckingham County, Virginia, in 1983 (*75*). This CP system was designed in such a manner that the protection current was supplied by a rectifier to each of the three spans of the bridge deck by two primary anodes—0.78-mm (0.031-in) diameter Pt-Nb-Cu wires—that were laid transversely in the deck. Then the current was distributed longitudinally over each span by secondary anodes made of less expensive carbon strands that were spaced at 30.5 cm (1 ft) intervals across the width of a span. Both the primary and the secondary anodes were set in saw slots approximately 13 mm (0.5 in) wide and 19 mm (0.75 in) deep, and filled with the conductive polymer grout. This CP system functioned reasonably well at a constant current density of approximately 17 mA/m^2(1.6 mA/ft^2). Other CP systems of basically the same design were subsequently constructed in few other states, and those systems had also performed as expected. Unfortunately, the conductive grout was found to be susceptible to degradation arising from attack by acids that are produced by anodic reactions such as:

$$2\,H_2O + 2\,Cl^- \rightarrow 4\,H^+ + 2\,OCl^- + 4\,e^- \qquad (15)$$

$$4\,Cl^- \rightarrow Cl_2 + 4\,e^- \qquad (16)$$

$$2\,H_2O \rightarrow 4\,H^+ + O_2 + 4\,e^- \qquad (17)$$

Such acid attack was expected to be concentrated at the grout/concrete interface, because of the concentration of electrically charge or current at that interface. In fact, symptoms of this degradation—such as discoloration of the concrete at the end of the slots, at stress cracks, and at the boundary of concrete patches—became evident after 1 to 2 years of the operation of the first system in Buckingham County, Virginia, and the second system built on the deck of the Interstate Route 64 over 13th View Street in Norfolk, Virginia (*76*). This acid attack, when compounded by normal traffic impact and thermal stresses at that interface, can eventually cause some of the conductive grout to eventually become disbonded from the decks. Incidentally, this system is reasonably efficient, as indicated by the relatively low average circuit resistance of the latter installation, which was approximately 1.0 ohm (*76*). Other disadvantages with this non-overlay slotted anode system were the added cost associated with cutting 19-mm (0.75-in) deep slots into the concrete decks, which may exposed many steel bars in many decks that have shallow concrete covers.

(3) Conductive Polymer Mound Anode System

The excellent strength of the FHWA conductive polymer concrete allowed the development of another concept of anode system: the distributed anodes with non-conductive, rigid overlays. This system was also called "strip or mound system," because the Pt-Nb-Cu wires and the carbon strands are placed directly on the surface of the decks, instead of in saw slots, in the same grid system used in the slotted system. Each of the anodes is then covered or mounded with just enough amount of the conductive polymer concrete. The mound is typically 33 mm (1.25 in) wide and 12.5 cm (0.5 in) high. Before the polymer concrete has cured, calcined petroleum coke breeze is broadcast over the mounds to provide a good bonding surface and increase conductivity to the rigid concrete overlay (of either Portland cement concrete or latex-modified concrete) that is eventually placed on the entire deck.

This system provides slightly higher anode/surface area than the slotted anode system. It's installation is easier and compatible with existing concrete overlay field practice and requires only minor additional construction procedures. Unfortunately, significant stresses in the concrete overlay directly above the anode mounds caused the overlays to crack in several such installations (*77*).

(4) Conductive Polymer Anode Overlay

With the objective of developing another viable anode system that can be applied directly on a deck without requiring the deck to be scarified first, the FHWA investigated the possibility of developing a multi-layer, resin-filled overlay system in which the first two or three layers are made electrically conductive by using conductive fillers and aggregates, while the surface layer used conventional aggregate to impart a high degree of skid and abrasion resistance (*78*). Eighteen different resins and 16 different conductive fillers were examined to determine what combinations of resin and filler and ratios would produce composite systems with desirable electrical and physical characteristics (electrical resistivity and durability, thermal coefficient, compressive strength, shear bond strength (with concrete), flexural strength, freeze-thaw durability, and gas and water permeability). This preliminary screening identified 8 resins (3 ortho-phthalic polyester, 3 iso-phthalic polyester, and 2 vinyl ester resins) and 1 conductive filler (calcined coke breeze) for extensive evaluations.

In the subsequent phase of the study, the most promising premixed and built-up overlay systems made from the selected 8 resins and filler were further evaluated, with emphasis on the compatibility of the overlay systems with Portland cement concrete and their performance in a CP system. In addition, the weatherability characteristics of systems made from three (2 ortho-phthalic polyester and 1 vinyl ester) resins were evaluated outdoor. The results revealed that, for an overlay to be used in conjunction with a CP system for bridge decks, a premixed conductive polymer would be more promising than a built-up polymer system, because the inherent physical and mechanical weaknesses of built-up overlay systems lead to early deterioration (by about 31 to 40 percent) of the initial bond strength—after only 6 weeks to one year under CP. Furthermore, the premixed overlays have some advantages over a built-up overlay:

- Their initial bond strengths are 50-percent to 100-percent higher than built-up overlays made of the same resin.
- This resulted in better freeze-thaw durability.
- Their curing was more satisfactory than individual layers of built-up overlays.
- It was easier to make premixed overlays permeable to gases, when permeability is desired.

Although the premixed overlays exhibited slightly higher electrical resistivities than built-up overlays, they were still in the desirable range of 3 to 10 ohm-cm. Their average initial bond strenght was approximately 6.90 MPa (1000 psi).

A field study was then undertaken to test this developed premixed polymer concrete overlay on a concrete bridge deck. Through experimentation, it was identified that material formulations consisting of either Hetron Q6305, a modified vinyl ester resin (manufactured by Ashland Chemical Co.), or Polylite 32-044 (from Reichold Chemical Co.), a 50-50 (by weight) aggregate mixture of calcined coke-breeze and crushed basalt or silica sand, a promoter, an initiator, and some suitable coupling, and wetting agents, would yield overlay systems with good combination of bond strength, flexural strength, compressive strength, shrinkage, wear resistance, and electrical conductivity (*79*). These two formulations, in the proportion of 55.3 kg (122 lb) of aggregate blend per 9.5 l(2.5 gal) of the resin blend, were used separately to build sections of 12.7-mm (0.5-in) thick overlay and tested in conjunction with a CP system for a bridge deck in Virginia (Table 12).

Table 12. CP System for Virginia Bridge Deck

Span/Lane	Aggregate Blend (% wt)	Resin Blend (% wt)
1,2,3 / EBL	Calcined coke breeze 49.26 Silica sand 49.26 Wetting agent (S440) 1.07 Initiator (BZP-C50X) 0.41	Hetron Q6305 vinylester 98.62 Dimethylanaline, promoter 0.39 A-174, silane coupling agent 0.99
1,2,3 / WBL	Calcined coke breeze 49.26 Crushed basalt (1/8 in max.) 49.26 Wetting agent (S440) 1.07 Initiator (BZP-C50X) 0.41	Polylite 32-044 98.0 Cobalt naplithenate, promoter 0.5 A-174, silane coupling agent 1.0 5-440, wetting agent 0.5

The cost of the CP system (including the design, the overlays, etc.) was about \$180/m ^{2} (\$17.79/ft ^{2}). During the first several months of operation of the CP system, which covers 6 different zones (according to number of spans and lanes), the CP current density was set at 10 mA/m ^{2}(1 mA/ft ^{2}). In general, as indicated by the results of depolarization testing conducted on December of 1987 (Table 13), the CP system, with the electrically conductive polymer overlays serving also as the secondary anode, was providing sufficient protection to the steel bars.

Throughout the first 15 months of operation, the currents required in the six zones averaged 4.24 A and the corresponding driving voltages averaged 3.04 V - indicating that the conductive polymer overlay is a highly efficient secondary anode system. In terms of anode current density required in the six zones, it averaged 0.09 mA/m ^{2}(1.05 mA / ft ^{2}). Depolarization tests conducted on 05-03/89, i.e., after 15 month of operation, indicated that the system was providing good protection to the rebars (Table 14).

Table 13. Results of Depolarization Testing in December 1987

Zone (Span / Lane)	Depolarization (mV)		
	Min.	Max.	Mean
A/WBL	144	319	202
B/WBL	222	503	331
C/WBL	141	565	278

A/EBL	39	231	165
B/EBL	102	364	228
C/EBL	45	324	206

Table 14. Results of Depolarization Testing in May 1989

Zone	Depolarization (mV)	
	4-h	**24-h**
A/WBL	104	182
B/WBL	112	211
C/WBL	136	214
A/EBL	420	503
B/EBL	394	473
C/EBL	286	366

Within less than a year after construction, the conductive overlay on the east-bound lane (EBL) was completely replaced with the same resin formulation used in the west-bound lane (WBL). After approximately 19 months of service, it was found that the WBL and the EBL had 3 percent and 5 percent, respectively, of delaminations in the overlays. Skid tests conducted on the overlays indicated that after 2 years of service, the skid numbers obtained were good. Based on the field evaluations, it was concluded that the overlay system appeared to have good potential as a secondary anode for CP system. However, with only 18 months of service, the investigators were not able to extrapolate the service life of this anode system. It was believed that the overlay must be able to perform satisfactorily for 15 to 20 years to be a viable, cost-effective anode system (*80*). However, due to the subsequent availability of commercial anodes, some of which have proven to be more durable, further installations of this anode were not carried out.

(5) Conductive Polymer-Encased Copper Anode Grid

At the time, two commercial grid-type anodes became available. The first was Raychem Ferex 100 anode, which consisted of a stranded copper wire encased in a proprietary flexible and electronically conductive polymer, resulting in an anode cable with a nominal diameter of approximately 8 mm (0.31 in). This composite anode cable is then woven in the factory into a two-dimensional panel. Installation of this anode panel is very simple: the panels are spread over a scarified and repaired bridge deck, secured down with plastic cleats and pins (drilled into the concrete), then covered with a rigid overlays. There is no handling of potentially hazardous chemicals at the bridge site. Reported early performance of this anode on one half of a bridge deck in Cincinnati, Ohio, was good, with an average circuit resistance of 0.6 ohm (*76*,*77*).

The unique form of this anode allows for possible use on vertical surfaces of concrete piers, using shotcrete to cover the anode. Unfortunately, it is relatively difficult to completely cover the 8-mm high anode cables with shotcrete without any voids, especially at where the cables contact the concrete substrate. In fact, some disbondment of shotcrete with this anode installed on substructures has been reported. In addition, the conductive polymer casing around the copper wire also proved to be susceptible to attack by acids generated around the anode. Another disadvantage of this anode is that if the core copper wire is exposed, the exposed copper will corrode and causes an open circuit and local loss of cathodic protection current. (The marketing of this product was eventually discontinued by its manufacturer in the late 1980s or early 1990s.)

(6) Activated Titanium Mesh Anodes

The second commercial grid-type anode is a honeycomb-shaped titanium wire mesh that is coated with a proprietary mixed precious metal oxide catalyst, which serves as the current discharge surface. This mesh, which comes in .2-in (4-ft) wide panels, is first secured on a scarified concrete deck with plastic anchored bolts. The mesh panels are electrically connected by spot welding across all adjacent panels a titanium strip, which is then connected to a rectifier lead wire. Then the entire concrete deck is covered with either a conventional concrete or a latex-modified concrete overlay.

This type of anode mesh has many distinctive advantages over the other anodes developed so far. It offers a considerably higher anode/concrete area ratio, and, therefore, distributes the protection current much more uniformly across a structure. The coated titanium has been used in other applications that required extremely high-current outputs and, therefore, the mesh is projected to have long life as a cathodic protection anode for used in reinforced concrete, where the current requirements are relatively low. This anode was installed in the other half of the bridge deck in Cincinnati, Ohio, for testing with the Ferex 100 anode, and functioned reasonably well—yielding an average circuit resistance of approximately 0.5 ohm with a latex-modified concrete overlay (*76*,*77*). Similar installation, with an acrylic-cement based coating as an overlay, on the sidewalks of that same bridge deck yielded an average circuit resistance of 2.0 ohm. Some cracking of the overlays on the deck and the sidewalks were observed, however.

This mesh anode may also be installed on substructures, and covered with an overlay of shotcrete. However, it faces the same risks of some portions of the mesh not being completely covered by the shotcrete and disbondment of shotcrete that the Raychem Ferex 100 anode cables faced when installed on substructures, but probably less because the mesh is considerably thinner. In fact, some ends of the mesh were exposed when it was field tested on the two concrete piers of the bridge in Cincinnati, Ohio (*76*,*77*). A possible indication of the difficulty of achieving good shotcrete bonding was the relatively high average circuit resistance of that installation, which was approximately 57.0 ohm.

The titanium mesh anode, when used in conjunction with a conventional rigid overlay, has so far performed better than all the other anodes tested in reinforced concrete bridge decks. Consequently, it is now the anode preferred by many state highway agencies and parking garage owners for rehabilitating horizontal concrete surfaces.

b. Impressed-Current Cathodic Protection of Concrete Bridge Substructural Members

To date, there is no significant amount of corrosion reported on continuously immersed reinforced concrete members (piles) and hence very few structures are using cathodic protection technology on this type of concrete members. However, there is a significant problem with corrosion of reinforced concrete piles exposed to the marine environment, particularly in the splash zone (between high and low tides). Also, there is a definite problem with corrosion in the other reinforced concrete components exposed to marine environment.

Many concrete piers in the snow belt are rapidly deteriorating as a results of reinforcing steel corrosion, induced by deicing salts dripping through faulty deck joints that are often located directly above the piers or splashes from nearby traffic. In addition, the concrete beams—typically the end portions, which are often situated underneath deck joints—are also susceptible to corrosion problem. These concrete components are not exposed to traffic impact and have more vertical surfaces than horizontal surfaces. Therefore, cathodic protection anodes intended for use on concrete piers do not have to be resistant to traffic load and wear. However, the anodes must be relatively easy to install or apply on the vertical concrete surfaces, and, if they are liquids, must be of the proper consistency so that they would not run. As discussed earlier, the conductive polymer encased copper anode grid (Ferex 100) and the activated titanium mesh had been marketed also for used on concrete piers by their manufacturers, although there is considerable doubt as to their suitability for this use.

(1) Sprayable Conductive Polymer Coating Anode

To provide state and local transportation agencies in the snow belt with some economical anodes for effective distribution of cathodic protection current across concrete piers, the FHWA sponsored the development of an electrically conductive polymer coating that can be applied on vertical and overhanging concrete surfaces, by mechanical means such as spraying or shotcreting (*81*). The study resulted in the development of a mixture of calcined coke breeze, a vinylester resin (Dow 8084), thixotropic agent (made from a bentonite clay), and a catalyst, methyl ethyl ketone peroxide (MEKP), that will stay in suspension for several days and can be sprayed on vertical and overhanging concrete surfaces without dripping and sagging. Before spraying, the resin, coke filler, and additives were premixed in a 19.3 1(5 gal) bucket and then pumped through a hose to a spray nozzle. Meanwhile, the catalyst was pumped from a separate pressurized container to the spray nozzle through a second hose. The sprayer uses an external impingement mixing technique that allows the catalyst to be added to the coke-filled resin mixture approximately 5 cm (2 in) beyond the end of the nozzle. The spray is regulated with air pressure controlled by valves.

When sprayed to a thickness of 10 to 15 mil, the mixture provides a coating that has a resistivity of 5 ohm-cm and is 50 percent as permeable as the Portland cement concrete (substrate). After being charged for up to 4 months at current levels several times that normally used in CP systems, the coatings did not show any decay in bond strength or weatherability. It was estimated that it would cost approximately \$10.76/m ^{2}(\$1.00/ft ^{2}) to spray the conductive polymer coating on concrete piers. This cost does not include the installation of the primary anode, cost of the air compressor and spray equipment, scaffolding, etc.

This sprayable polymer coating was utilized as an anode for CP of one of the two piers of the bridge in Norfolk, Virginia, where it functioned effectively in distributing current over the pier—yielding an average circuit resistance of 5.3 ohm. However, during the first 1.5 to 2 years of operation, at least up to a dozen of 2.5-cm (1-in.) diameter blisters were found in the coating. Although still unconfirmed, this may indicate that the coating may not be completely permeable—in at least some locations over the pier—to gases formed by some of the anodic reactions.

(2) Metallized Zinc Coating

In 1983, the application of metallized zinc coating extended from structural steel components to CP of concrete piers when California Department of Transportation field tested this material on a pier of the Richmond/San Rafael Bridge in San Francisco Bay (*82*). Zinc can be applied on the surface of a reinforced concrete using either the flame spray or the arc spray technique, whereby zinc wire is continuously fed through a hand-held gun where it is melted with an oxygen-acetylene flame or a high-voltage arc and spray on the concrete with a jet of compressed air. Electrical connection of the metallized zinc to the rectifier is facilitated by small stainless steel or copper plates secured (with epoxy) on the concrete surface prior to spraying. Fortunately, because the zinc coating is metal, it is very conductive and, therefore, only a limited number of these contact plates are needed.

Since the metallization process is similar to spray painting, the zinc coating can be applied easily on vertical surfaces, corners, and undersides of a structure. And, unlike the carbon-based conductive polymer coating, the zinc coating is very comparable in color to concrete and requires no decorative overcoat. As with other anode systems that completely covered the concrete, metallized zinc coating provides efficient distribution of protection current. However, as was observed during field trial on a concrete pier, it exhibited a higher tendency than the other conductive coatings to create electrical shorts, with chairs and discarded tie wires that are often found at the bottom of pier caps. As had been observed on a concrete pier of the Route 64 Bridge in Norfolk, Virginia, these metal parts will either cause shorts during the metallization process or, will corrode while the CP system is in service and then cause small pieces of concrete to pop off (*76*).

Another mode of failure of metallized zinc, which is common among all coating anodes (including the conductive polymer coating discussed earlier and the conductive paint to be discussed in a later section), is fracture and separation in the surface layer of concrete adjacent to the coating, after being subjected to varied extents of current passage. This resulted from the formation of acid, by anodic reaction, at the coating-to-concrete interface, which ultimately decreased the adhesion of coating. For metal coatings, corrosion products formed on the coating at low pH were also significant cause of coating failure (*83*,*84*).

(3) Conductive Paints

Paints, using water or organic solvents as the carrier, can be made electrically conductive by adding finely dispersed carbon particles. The resulting materials can be applied on an entire concrete member as a continuous distributive or secondary anode, so that protection current brought in from a rectifier by primary anodes can be uniformly distributed. The primary anodes can be platinum wires secured on the surface of the concrete members. Since these materials can be applied by using common tools such as sprayers, rollers, or brushes, they are very suitable for use on concrete piers. In addition, these materials are relatively inexpensive.

Earlier field testing on some concrete piles surrounded by water in Florida showed some localized disbondment of the conductive paint at the tidal level of the pile but none at above that level. Recently, a water-based conductive paint was tested in two CP systems that were built, one 9 years ago and the other 7 years ago, to protect the concrete piers of four inland bridges in Virginia (*85*). Measurements made at various times of circuit current, voltage, steel potential, and 4-hour polarization indicated that the conductive paint was performing adequately and the CP systems were providing sufficient protection to the reinforcing steel in the piers. However, the circuit resistance for the various circuits at the energization of these CP systems were relatively high—varying from 12 to 61 ohms for the 9 years old system and from 5.6 to 31.0 ohms for the 7 years old system.

Some deterioration or degradation of the conductive paints was observed, mostly at unsheltered portions of the piers (specifically the ends of pier caps). This deterioration, which ranged from 0 to 0.37 percent in the older system and 0 to 0.14 percent in the newer system, occurred at the coating-cement interface. In a recent study on cathodic protection of prestressed concrete components, similar degradation of carbon-filled conductive coatings was observed and it was suggested by the researchers that this may be related to poor wetting of the carbon particles by the water, which may be improved by the use of adhesion promoter (*84*).

Based on the performance of the water-based conductive paint in the first 7 to 9 years of service of the two CP systems in Virginia, it was projected that the service life of this conductive paint can be at least 15 years, especially when any small deterioration is touched up as early as possible.

(4) Conclusions on the Performance of Various Anodes on Concrete Piers.

Based on all the laboratory and field data available so far on the various anodes that have been tested for impressed-current CP of concrete substructure bridge components, the following conclusions can be made:

- None of the anodes that need to be anchored to the (vertical) surface of concrete pier caps and columns and then overlaid or encased in shotcrete had performed satisfactorily, due to the debonding of the shotcrete after a short time. In addition, since their installation was difficult and time consuming, this type of anodes was abandoned in favor of electrically-conductive coatings.
- Conductive polymer slurry, which was one of the conductive coatings tested, had performed satisfactorily. However, it was not used much in the field since the trial in Virginia, due to unavailability of satisfactory equipment for the mixing and spraying of the activated resin mixture and coke breeze and the subsequent development of conductive paints and metallized zinc.
- The field performance of conductive paints have been mixed. This is probably because many of the applications were inappropriate, particularly on concrete components that were either surrounded by water and therefore remained wet most of the time or received considerable splashes from nearby traffic. The recent applications of conductive paint in CP systems for inland concrete piers in Virginia have yielded very satisfactorily results.
- Flame or arc sprayed zinc or metallized zinc has performed maybe more satisfactorily than the others, but still has many deficiencies.
-

(5) Present Status of Impressed-Current Cathodic Protection

Titanium mesh anode, used in conjunction with a concrete overlay to distribute protective current, is filling the need for a durable anode for use in impressed-current CP of reinforced concrete bridge decks and is, in fact, now widely accepted by state and local transportation agencies.

On the other hand, the progress in the anode technology for impressed-current CP of other concrete bridge components has lagged behind and no single anode has yet been identified to be very satisfactory in terms of being able to provide long-term durability. One of the reasons for this relatively slow development is that recognition of corrosion-induced concrete deterioration as a problem in these components came much later than for bridge decks. The titanium mesh anode would have had a place in CP of concrete piers, because it is expected to be able to perform equally well on any concrete components. However, it is still not possible to use it as such because the mesh cannot be installed on vertical concrete surfaces and covered with shotcrete without invariably creating some problems.

The use of conductive carbon-based coatings or polymers has some possible drawbacks, including: (a) debonding of the coating that arises when the materials are used in environments wherein the concrete is constantly wet or when the materials are applied before the concrete is sufficiently dry; (b) degradation of carbon or binder after extended current passage; and (c) increase in the electrical resistance between the anode and the steel due to insufficient moisture or accumulation of insulating byproducts at the anode/concrete interface. The flame or arc-sprayed zinc is free of the debonding problem; however, it suffers from some of these same problems. Recently, the use of flame or arc-sprayed titanium metal—over concrete coated with a conductive oxide solution—as a new anode has shown some promise when it was tried partially on a bridge in Oregon.

c. Galvanic Cathodic Protection of Concrete Bridge Decks

Galvanic CP was first tested on a bridge deck in Illinois in 1977, as part of a National Cooperative Highway Research Program study. In that field test, two different sacrificial anodes were installed in the deck:

- Perforated sheets of high-purity zinc fastened on the deck with a bed of mortar then covered with a free-draining concrete overlay.
- Conventional zinc alloy ribbons embedded in grooves cut into the concrete surface.

Except for the early failure of the concrete overlay used on the perforated zinc sheets, which necessitated the use of a free-draining asphalt overlay over both anode systems, these had performed satisfactorily for nearly 14 years. In 1991, these sacrificial anode systems had to be removed due to the failure of the asphalt overlay and the need to widen the structure. Examination of the removed anode materials revealed that considerable mass of each anode was still left and that the systems had the potential for many more years of service if not limited by the lifetime of the overlay. A 1989 revisit of the systems indicated that the system current output was 16.8 mA/m ^{2}(1.5 mA/ft ^{2}); this compared very well to the outputs of 10.8 to 32.2 mA/m^2 (1.0 to 3.0 mA/ft ^{2}) during the earlier days of the systems.

To continue that effort, a laboratory screening program was undertaken to identify most promising sacrificial anodes (*86*). The selection of potentially useful sacrificial anodes for the laboratory screening was limited to scrap metals and to materials that are already commonly used in the CP industry, which are aluminum, magnesium, and zinc alloys.

- Scrap zinc (from penny stamping).
- Special high-grade zinc (meeting ASTM B6).
- Magnesium alloy (meeting ASTM B90 alloy AZ31B).
- Galvalum I alloy (Galv I).
- Galvalum III alloy (Galv III).
- Structural-grade expanded aluminum mesh (Al-5005).

This laboratory screening involved coupling 50 x 150 mm (2 x 6 in.) strips of these anodes with reinforcing steel bars in sand conditioned to various resistivities (23, 100, and 260 ohm-m) and treated with a solution to simulate concrete pore solution, for a period of 18 weeks. During that period, measurements of current flow, potentials, circuit resistance, and 4-hour depolarization of the steel, were taken. Afterward, the anode strips were removed from the sand and their weight losses were determined. Results from this screening indicated that:

- As expected, current output from each anode to the steel decreases with increased resistivity in the surrounding medium or environment. When the resistivity reached the levels typical of those found in atmospherically exposed concrete structures, current flows from these anodes were less than 10 mA/m ^{2} (1 mA/ft ^{2}).

- At the two largest resistivities investigated, the aluminum-alloy anodes provided the largest current outputs; while at the lowest resistivity studied, the magnesium anodes provided the most. Consistent with these observations, the driving voltages were greatest for the series of aluminum-alloy anodes and much smaller for the zinc and magnesium anodes. This indicates that the latter two may not be capable of protecting steel in the high-resistivity environment of dry concrete.

- For most of the anodes tested in all three environments, the observed 4-hour depolarization throughout the 18 weeks testing exceeded the 100-mV criterion. However, the investigators believed that these results may not be realistic because the steel bars did not exhibit any significant corrosion, even when not coupled to the anodes.

- Weight loss in an anode may be due more to self corrosion of the anode than to its galvanic reaction. This was especially true for the aluminum-alloy anodes.

Three anodes were selected for further evaluation in reinforced concrete slabs. These were the special high-grade zinc (SHG-Zn), the Galvalum III alloy (Galv III), and the structural-grade expanded aluminum mesh (Al-5005). The test concrete slabs were constructed with the following four different types of overlay concrete: (a) normal-weight concrete, (b) lightweight concrete, (c) free-draining Portland cement concrete, and (d) free-draining latex-modified concrete. Each of the slabs was cast with two lifts of concrete, with the top lift containing 5.9 kg/in ^{3}(10.0 lb/yd ^{3}) of chloride.

During the 24-weeks testing, the slabs were subjected to periodic wetting with a saline solution; and, at weekly intervals, measurements of current flow and circuit resistance. In addition, at 4 and 12 weeks, depolarization measurements were made on the steel for up to 72 hours, instead of the usual 4 hours. Based on these screening tests and detailed testing in concrete slabs, the structural-grade expanded aluminum mesh (Al-5005) was tested on a bridge deck located in a northern area, with a normal-weight concrete overlay. The performance of this system was, however, not satisfactory.

d. Galvanic Cathodic Protection of Concrete Substructures

Most anodes applied to atmospherically exposed concrete structures utilize impressed current supplied by an auxiliary power source. For all situations, this mode of cathodic protection provides adequate power to the structures and permits complete flexibility for control of the CP system. However, the use of impressed current may overprotect the structures and reduce anode life and cause hydrogen embrittlement of high- strength prestressed steel. In addition, the need for additional electrical components compromises the reliability of the impressed-current CP systems and increases the monitoring and maintenance requirements.

Certain metals that are anodic in comparison to steel, such as aluminum, magnesium, and zinc, each possess an inherent characteristic electrical potential that allows it to galvanically supply cathodic protection current—without the need for an external power source. However, the driving voltage and, therefore, the amount of protective current that an individual galvanic anode can supply is limited by the magnitude of the potential that characterizes that anode. Thermally sprayed zinc appeared to be able to function as a sacrificial anode in galvanic CP systems installed for protecting concrete in the seawater splash and tidal zone. However, the anode surface must be periodically wetted in order to supply adequate current and, in addition, the protective current decreases with time. Recent data indicated that the metallized coating eventually became passive, due to reduction in pH at the anode-to-concrete interface (*87*). In addition, it is also recognized that the system has a relatively short useful life.

Due to the inherent advantages that galvanic CP has over the impressed-current CP, particularly the almost complete freedom from need for regular maintenance, the FHWA initiated multiple research studies to develop galvanic anode systems for counteracting the corrosion currents in both dry and wet environments (*87*,*88*). This section describes two such promising galvanic anode systems for substructure concrete bridge members.

(1) Zinc-Hydrogel Anode System

The ability of sacrificial anodes (such as aluminum, aluminum alloys, and zinc) to deliver galvanic current, when in contact with different commercial conductive hydrogel adhesives, were investigated (*88*). Zinc was included in the test because it has good current efficiency and relatively low cost. The aluminum anodes were included for their potential to be 30 percent thinner, 74 percent lighter, and 65 percent less expensive than zinc. However, testing indicated that neither aluminum nor its alloys were suitable for use in conjunction with the hydrogel adhesives tested, because they either exhibited unstable behavior or provided inadequate working potential. In addition, the aluminum anodes were voluminous and difficult to accommodate. Zinc was identified as the best anode candidate, with a stable working potential of 1.1 volt (SCE) at the typical range of current densities used in cathodic protection of concrete, and showing no tendency to passivate with time.

The conductive hydrogel adhesives tested were hydrophilic coagulated colloids of acrylic-sulfonamide copolymers. These adhesives, which are used for medical purposes, have satisfactory resistivity, adhesive tack, and drying resistance. However, accelerated testing indicated that they could not tolerate a charge transfer greater than that equivalent to 6 years of operating life. A special hydrogel formulation, the 3M-3, that is capable of tolerating a total charge equivalent to about 11 years of service life was then developed and is being combined with zinc anode for field testing.

This best combination of sacrificial anode and conductive adhesive, the zinc/3M-3 hydrogel system, is being field tested in at least three installations. In the following installations described below, a 10-mm- to 20-mm-thick zinc sheet is affixed to the concrete with the ionically conductive-hydrogel adhesive.

- Long Key Bridge. Marathon. Florida. In February, 1996, the zinc/hydrogel anode was installed on five V shaped piers of the Long Key Bridge. The steel density in this area is high and corrosion was severe. Recorded start-up current and steel polarization are shown in Table 15.

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Table 15. Recorded Start-Up Current and Steel Polarization

Time- on- Line	Current Flow, mA/m^2 (of Steel)			Average Polarization
(days)	Pier 16	Pier 32	Pier 43	-
0	9.5	11.1	9.5	----
98	----	----	----	261 mV
132	----	----	----	I98 mV

The zinc/hydrogel galvanic anode at this installation has performed satisfactorily during the evaluation period. Measurements showed that the anode was providing sufficient polarization of the steel bars, exceeding the 100-mV CP criterion. Visual inspection indicated that the system is properly controlling corrosion activity in the piers.

- Cape Perpetua Bridge, Yachats, Oregon. An installation of about 225 m ^{2} of zinc/hydrogel anode was installed recently on this bridge in Oregon. This installation is intended to prevent chloride-induced corrosion of steel in the beams and the underside of the bridge deck. No data is available yet.

- South Bridge Fishing Pier, Ft. Pierce, Florida. The zinc/hydrogel anode was installed on about 300 m ^{2} of concrete surface of this fishing pier in Ft. Pierce, Florida, during May and September of 1995. The protected members included 4 piles, 4 pile caps, and 2 double-tee beams. The edges of the anodes were protected by polyurethane caulk to prevent the conductive adhesive from coming in direct contact with the seawater; and the system was painted with a gray paint for aesthetic reasons. Table 16 suimnarizes the performance of the system to date.

Table 16. Performance of the System to Date

Time-on-Line (Months)	Current Flow, mA/m^2 (steel)			Average Polarization
	Piles	Caps	Beams	-
0	37.5	17.0	12.5	------
2	28.2	16.9	13.6	------
4	18.5	14.4	13.6	------
6	15.7	4.1	4.0	------
9	14.4	2.9	2.5	120 mV
13	13.4	5.8	2.1	114 mV
20	14.1	4.1	1.1	108 mV

These data show a slow decrease in the amount of current delivered, apparently related to relative distance of the concrete member from the water. Despite this decrease in current, steel polarization is sufficiently adequate to meet CP criterion of 100 mV, during the first 20 months of operation. Past experience has shown that the current required for meeting this criterion typically decreases with increased length of operation of a CP system.

The following are interim findings resulting from the effort to develop a combined sacrificial metal/hydrogel anode system:

- The aluminum and its alloys were found to be unsuitable for use as sacrificial anodes in conjunction with the hydrogel adhesives, because they exhibited either unstable (passive) behavior or inadequate working potentials. In addition, they are voluminous and difficult to accommodate.
-
- Zinc was identified as the best anode, with a stable working potential of—1.1 V (SCE) at the typical range of current densities used in CP of concrete.
- The galvanic and accelerated impressed-current tests of the different hydrogel adhesives indicated that the proprietary 3M-3 hydrogel was the best adhesive for this particular type of CP applications. It tolerated a total charge of 1140 A-hr/m ^{2} (105.8 A-hr/ft ^{2}) or equivalent to 12 years of operation under typical CP current densities.
- To ensure maximum service life, the adhesive must be prevented from direct contact with water or seawater, by caulking the edges of the zinc/hydrogel anode system,
- The system is relatively simple to install and required no construction contractor. After two years, the system adhesion and appearance remained good in the field installations described.

In addition to these three installations, independent testing by Virginia Department of Transportation is also under way on some inland concrete piers and marine prestressed concrete piles in Virginia.

(2) Sprayed Alloy Anode System

In another FHWA developmental effort, a different approach is taken to develop good sacrificial anodes: finding new sacrificial metals or alloys that can be applied by the conventional metallization (flame- or arc-spraying) process to form a metallized coating on the concrete members (*87*). The goal of this research was to develop a new cost-effective, sacrificial anode material that is free of the problem associated with metallized zinc coating and is capable of providing sufficient current density to polarize the steel so that the steel will not corrode at all or corrode at an acceptable rate. Fifteen different commercially available anodic materials were tested for: (1) anode capacity; (2) anode efficiency; (3) cyclic polarization; (4) atmospheric corrosion characteristics; (5) susceptibility to effects of environmental (temperature and humidity); and (6) effect of pH on anode performance. The sprayable anodic materials tested included pure zinc, pure aluminum, binary Zn-Al alloys, binary Al-Zn alloys, binary Al-Mg alloys, a tertiary Al-Zn-Mg alloy, and a tertiary Al-Zn-Sn alloy. The study found that:

- In general, these anodes produced more current at higher temperatures and higher relative humidity. At conditions of high temperature (≥ 32°C) and high humidity (≥ 90 percent), all the sacrificial anodes tested produced sufficient current.

- At low temperature and low relative humidity, the zinc alloys and most of the aluminum alloys did not produced sufficient current to protect the embedded steel. The zinc-based anodes generally exhibited passivation after a time.

- The current output and potential of all anodes showed decreased as the pH of the environment decreased, especially at pH below 12. The decreasing pH at the anode-to-concrete interface accounted for the decrease, with time, in the current outputs of all anodes. However, this effect was the least on the Al(55)- Zn alloy, which also remained relatively more active than the other anodes at low temperature and low humidity.

Since none of the evaluated 15 anodes demonstrated sufficient utility for use as sacrificial anode for reinforced concrete, especially in low-pH environment, the last portion of this study involved developing new alloy for this application. This effort centered on improving the Al-Zn alloy by adding indium, titanium, or zirconium, which in effect are tertiary alloys, and finding the optimum concentration for each of these three components. Among the numerous combinations of the tertiary alloys tested, the best performing alloy was found to be Al(20)-Zn-In(0.2), because it exhibited the lowest anode polarization and maintained a relatively active static potentials in all environments. Field testing of this new alloy is under way in Florida, Pennsylvania, and Virginia. The following is a description of the progress made in the Florida installation on the Bryant Patton Bridge.

This structure has considerable seawater-induced corrosion damage on the prestressed concrete piles. In July, 1995, two piles in one bent was sprayed with the Al-Zn anode; and, for comparison, conventional pure zinc was sprayed on two other piles. Both arc and flame spray equipment were used for the thermal spray application of the metals to the concrete piles. The concrete surface on each pile where an anode was applied is approximately 3.7 m ^{2}**,** and the corresponding embedded steel surface was approximately 2.1 m ^{2}. The anodes are not normally exposed to rainwater because the treated piles are located beneath a wider pile cap.

Both materials are operating as galvanic anodes. The four piles are fully instrumented with rebar probes, silver-silver chloride reference cells and current measuring circuits. To facilitate depolarization testing, provision was made so that the anode-to-steel circuit can be opened (disconnected) remotely. A data logger and a modem were installed at the site so that all of these parameters can be monitored remotely by using a cellular phone.

The prospect of extending the service life of the Al-Zn anode with a suitable top coat is also being tested. For this testing, one pile in a second bent was treated with the Al/Zn anode and then covered with an aliphatic urethane coating, while a second pile was treated with the anode and then covered with a water-based acrylic paint.

The system was kept in operation for two years, during which anode-to-steel current, rebar probe current, steel potential, and depolarization data were obtained monthly. At the end of this period, additional data (such as anode depolarization, cathode depolarization (using both the embedded reference and surface electrodes), anode-to-concrete adhesion, anode-to-concrete interface pH, anode-to-structure driving voltage, anode-to- structure resistance, and visual condition) were collected. In addition, core samples were also taken to determine the composition and extend of corrosion products at the anode-to-concrete interface, to allow for estimation of anode depletion rate.

The following are findings made from this testing:

- The current output of galvanic anode of Al-Zn-In alloy was higher current than that of the zinc anode; in fact, it might be more than necessary for the warm climate of Florida.
- The higher level of current from the Al-Zn-In anode resulted in higher cathode depolarization than the Zn anode. The depolarization achieved by the former exceeded 100 mV during the two-year test period. The Zn anode did not provide enough current to provide depolarization levels sufficient to meet the CP criterion of 100 mV.
- The current outputs of both anodes fluctuated with changes in relative humidity, temperature, and wetness of the concrete.
- The Al-Zn-In anode continued to adhere well to the piles during the test period—exhibiting adhesion values comparable to those of the Zn anode.
- Both Al-Zn-In and Zn anodes can be expected to provide a life expectancy of above 15 years, at thickness of 12 mils (305 μm).
- Topcoating of the Al-Zn-In anode is not necessary. In fact, the coating might disbond from the anode as a result of accumulation of corrosion products on the outside face of the anode.

e. Cathodic Protection of Prestressed Concrete Bridge Members

In the last 25 years, FHWA administrative contracts, in-house research, State Planning and Research studies, National Cooperative Highway Research Program studies, and private industry research have made significant advances in the development of improved CP systems for reinforced concrete bridge decks. This development has resulted in the installation of various CP systems on approximately 500 bridge decks by the state highway agencies. However, the development of CP technology for prestressed concrete has lagged behind. In 1988, FHWA initiated a major laboratory study to identify limitations and potential problems, which may exclude the application of CP to PS/C bridge members. Based on this study on PS/C, it became apparent that the susceptibility to hydrogen embrittlement of embedded prestressed steel can be avoided. This can be accomplished by careful monitoring and control of the amount of steel polarization through the use of a current- and voltage-limiting rectifier and embedded reference cells. Based on this knowledge, FHWA initiated a study in 1991 with the objectives of: (1) performing fundamental research that is aimed at resolving some concerns related to PS/C and, (2) installing and evaluating a field CP system for some PS/C components, to gain knowledge about the practical application of CP to prestressed concrete bridge members.

In addition to the hydrogen embrittlement susceptibility issue, there was also another concern. It is well known that under CP, there is a migration of cations (e.g., potassium, sodium, etc.) toward the embedded strands resulting in possible softening of the mortar paste around the reinforcement. This phenomenon may, in turn, cause some reduction of bond between the prestressed strands and concrete, over the life of a structure, and hence the overall structural integrity of the bridge members and their ability to perform their designed function. This possible partial loss of bond is of no real consequence in reinforced concrete structures, but in prestressed concrete members, it could be a major problem after CP is applied for an extended period.

There was also a concern about the behavior of corrosion pitted embedded prestressing strands when under cathodic protection, even at below the hydrogen evolution potentials. Earlier limited research had indicated that the probability of notched prestressing strands undergoing hydrogen embrittlement and then failing under service loads is much greater than for smooth and uniformly corroded specimens. This raised the question: If the embedded strands in concrete bridge members behave more like notched specimens, what would be the most appropriate CP criteria to use or should CP even be considered for prestressed concrete members?

To address these concerns and raise CP technology for prestressed concrete to a par with that for reinforced concrete, a number of laboratory and field studies were initiated.

(1) Laboratory Investigations

Laboratory investigations (*89*) were conducted to address the following issues that are particularly relevant to PS/C bridge components:

- Determination of the electrochemical potentials at which hydrogen is generated on steel embedded in concrete.
- Determination of whether hydrogen generation has a detrimental effects on the ductility of high-strength steel embedded in concrete.
- Assessment of the effect of notch severity.
- Study recovery of mechanical property after hydrogen-generating potentials have been applied and released.
- Determination of any relationship between steel tendon properties (composition and microstructure) and the tendency for occurrence of CP-induced embrittlement.
- Assessment of the significance of reduction in bonding between tendon and concrete.
- Determination of how prestressed concrete members may be qualified for CP.
- Identification of the effective field techniques and operating parameters for CP of prestressing steel in concrete.

To facilitate resolution of these issues, the study was focused in four major areas:

- <u>Hydronen Embrittlement.</u> Experiments included constant extension rate testing (CERT) of smooth, notched, and pre-corroded (pitted) tendon wire specimens in a simulated concrete pore solution at two cathodic potentials of - 0.90 V and - 1.30 V (SCE), which represent adequate and excessive protection, respectively. It was intended for these tests to replicate actual service conditions, wherein existing corrosion resulted in cross-section loss in the strands and excessive cathodic polarization may promote reduce strength and ductility in the strands. (It is believed that this procedure is particularly revealing with regard to hydrogen embrittlement susceptibility and any tendency of CP to induce brittle fracture.)

  A second set of experiments involved a series of pre-tensioned concrete beams for which the embedded tendon was first corroded to various degrees by anodic polarization and then cathodically protected excessively to promote embrittlement. Subsequently, these beam specimens were autopsied. Situations that were found to have caused tendon fracture and those that did not were characterized.

The following findings were made:

  - Hydrogen is generated at the surface of prestressed steel embedded in concrete at potential levels consistent with thermodynamic theory. Therefore, -0.90 V (SCE) was confirmed as the safe cathodic protection limit.

  - The ultimate tensile strength (UTS) of pre-corroded Grade 270 prestressing steel tendon decreased in direct proportion to the magnitude of loss in its cross-sectional area and was independent of corrosion acuity (depth-to-length ratio of the corrosion attack).

  - Either a constant load, a constant stress condition, or a combination of both, arises for corroding tendons, depending on the corrosion conditions and the extent to which the tendons debond from the concrete. The fact that relatively few wire fractures were detected for the pre-tensioned concrete beam specimens subsequent to CP and, instead, numerous wires appeared to have simply corroded in two during the corrosion phase of the tests (prior to application of CP) is consistent with a state of constant stress having been maintained as the wires corroded.

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- Bond Loss. In response to concerns that CP of pre-tensioned concrete could cause a loss in bond, pull-out tests were performed where the bond strength of tendon and individual wires, as a function of total charge transferred, was determined. By assuming that bond strength depends solely on the magnitude of total charge transferred and is independent of current density, bond strength corresponding to long term CP at realistic levels of current density was projected from the results observed in experiments wherein the specimens were subjected to short term CP of very high current densities.

  Analysis of the data from these tests indicated that the tendon-to-concrete bond of the pull-out specimens was either retained or experienced only a modest decrease during the course of charge transfer that was equivalent to as much as 160 years of cathodic protection. Hence, it was concluded that loss of tendon bond is not a problem for cathodically protected pre-tensioned concrete members.

- Qualifying Pre-tensioned Concrete members for Cathodic Protection. Based on results of the above experiments, a protocol for qualifying pre-tensioned members for CP, termed Dimensional Analysis, was developed. This method requires simply that: (1) the pre-tension stress either be measured or an appropriately conservative value be assumed; and (2) the remaining tendon cross section at the most corroded location be measured. The following procedures and criteria were proposed for qualifying a particular prestressed concrete member as being suitable for CP:

  - If no corrosion-induced concrete cracking and spalling are evident, then the member is automatically qualified

  - If these symptoms are evident, visually inspect already exposed tendon(s), by removing cracked and spalled concrete from areas damaged by corrosion—making sure that the concrete being removed will not significantly compromise the load-bearing capacity. Identify locations on the exposed steel tendon where corrosion is uniform and/or where it is localized. Qualify the structural member for CP based upon the following criteria:

    - At locations of uniform corrosion, the remaining wire cross section is at least 85 percent of the original cross section, or
    - At locations of localized attack, the remaining cross section is at least 90 percent of the original, or
    - At locations where both uniform and localized corrosion occurred, the remaining cross is at least 90 percent of the original.

-
- Remote Monitoring and Criteria. Of particular importance in using cathodic protection on pre-tensioned concrete members and structures is the need to ensure that the steel potential is maintained within prescribed safe limits. Remote monitoring systems provide a means for economically accomplishing this. For this reason, a remote monitoring hardware and software system was evaluated by monitoring, controlling, and testing a CP system that was installed on some pre-tensioned beams in Florida but remotely controlled by researchers from Northern Virginia. This included control according to: (1) constant rectifier voltage, (2) constant rectifier current with limitation on voltage, and (3) control on potential. Also, the degree to which the 100-mV protection criterion was achieved was evaluated by depolarization testing.

Based on evaluation of the computerized rectifier system and the control methods investigated, it was concluded that CP systems for prestressed concrete structures can be effectively monitored and controlled by existing remotely operated hardware and software.

Field Installations

Selected anode systems were tested in the cathodic protection of some prestressed box-beams (located in a snow belt state) and prestressed concrete beams and piles (located in a marine environment). The aims were to: (1) develop methodology for CP installation, (2) evaluate the effectiveness of several CP systems with potential application in combating corrosion in prestressed concrete components, and (3) identify all system limitations and potential risks to the structures. These CP systems are briefly described in Table 17 (*90*,*91*).

Table 17. CP Systems Selected for Testing

Bridge	Component	Anode System	CP Mode
H. Frankland Br. Tampa, El.	Prestressed beams	Metallized zinc coating	Galvanic
	Prestressed beams	Metallized zinc coating	Impressed current
	Prestressed piles	Titanium mesh/cement grout jacket	Impressed current
	Prestressed piles	Titanium mesh/conductive rubber	Impressed current
Abbey Rd. Ohio	Soffit of prestressed box-beams	Metallized zinc coating	Galvanic (then Impressed current)
West 130th St. Ohio	Soffit of prestressed box-beams	Conductive paint (Permarock)	Impressed current

After nearly three years of operation and monitoring, these CP systems and bridge components were evaluated. Analysis of the data lead to the following conclusions:

- In CP of prestressed concrete members where the electrical resistivity of the concrete is heterogeneous, it may be possible to achieve sufficient protection at locations where resistivity is high while at the same time precluding hydrogen generation at locations where resistivity is low. (Diverse resistivity in a structure is often caused by a difference in moisture content, but may also be caused by factors such as depth of concrete cover, chloride content, anode contact resistance, and distance from anode power feed.

- The operation of CP systems for prestressed concrete members is best accomplished in constant voltage mode—with the operating voltage selected such that potentials required to generate hydrogen at the surface of the prestressed steel will never be reached. However, the same can be achieved by operating in constant current mode while limiting the compliance voltage, provided that the same strategy is used in selection of the compliance voltage.

- Examination of prestressed steel and concrete specimens extracted from the tested concrete bridge members in Florida and Ohio, after 32 and 37 months of cathodic protection, respectively, revealed no adverse effect on the structural integrity of the concrete adjacent to the steel and the bond between the steel and the concrete. In addition, the relatively high ductility exhibited by the steel specimens, including those extracted from locations where steel potentials were frequently very negative, i.e., <-900 mV (SCE), indicated that little or no evidence that hydrogen embrittlement had occurred (*91*).

In summary, CP of prestressed concrete bridge members can be accomplished safely and reliably; however, concrete members which are very heterogeneous in moisture content are not considered good candidates.

3. Electrochemical Chloride Extraction

When a direct current is conducted through concrete, the relatively mobile ions (such as chloride, hydroxide, sodium, potassium, calcium, etc.) in the concrete would migrate, with each ion moving toward the electrode with the charge opposite to it. The feasibility of removing the undesirable chloride ions from a contaminated concrete by such electrochemical means, instead of excavation of the contaminated concrete from a structure, was studied in the mid-1970s by Kansas Department of Transportation, which used a sacrificial copper anode to show that chloride ions can be expelled from concrete by passing a direct current between the steel bars and the anode, as in cathodic protection except at considerably higher level of current (*92*). This feasibility of electrochemical removal of chloride ions from concrete was confirmed in another study, a total of 20 m ^{2} (200 ft ^{2}) of concrete on a bridge deck was treated with an average current density between 23 to 28 A/in ^{2} (2.3 to 2.8 A/ft ^{2}), under a constant voltage of 100 V for 12 to 24 hours (*93*).

However, the unnecessarily high levels of direct current used in these early investigations had some adverse effects (such as decreasing the concrete-to-steel bond and cracking in the concrete) on the concrete. The concern about these adverse effects on treated concrete has delayed the use of electrochemical chloride extraction as a remedial method for the permanent rehabilitation of concrete bridges. To address this concern, one of the SHRP studies conducted investigations on laboratory slabs and on portions of some concrete bridge components, and thereby found that if the level of current applied is kept below 5 A/m ^{2}(0.5 A/ft ^{2}), the chloride removal treatment is unlikely to have any adverse effect on the concrete (*94*,*95*). It was also found that the treatment removed 20 to 50 percent of the chloride ions admixtured into the concrete slabs and redistribute the remaining chloride well away from the reinforcing steel. In an independent study in Canada, portions of a concrete pier column were treated and the results confirmed the removal of some of the chloride in the concrete and the corresponding passivation of the steel bars (*96*).

These studies showed that electrochemical chloride extraction carried out with considerably less electrical current than that used in the earlier studies used, thereby avoided any adverse effects on the concrete. Since this approach has a very important advantage over cathodic protection as there are no electrical components or anode materials to be maintained after the treatment is completed, these findings had rekindled interest in this method. Since these studies involved only concrete slabs and very small sections of several bridges, pilot electrochemical removal of chloride treatments were conducted recently on some full-sized concrete bridge deck spans and piers in Virginia (*97*,*98*) and some full-sized concrete piers in South Dakota (*99*). These pilot treatments have demonstrated that it is feasible and simple to conduct the treatment on fill-sized reinforced concrete bridge members, although it is comparatively more difficult to conduct the treatment on concrete piers.

A difficulty encountered in planning for these full-sized treatments was predicting the necessary length of treatment required to reduce the chloride concentration at the steel level to below the corrosion threshold or some concentration equilibrium. The SHRP studies suggested that a total charge of 600 to 1,500 A-hr/m ^{2}would be sufficient in most cases, and for most concrete, this would mean a total treatment time of 10 to 50 days. Unfortunately, this guideline is not sufficiently definitive to allow for efficient logistic scheduling of the different rehabilitation operations that would typically be involved before and after a treatment and of necessary traffic control. Another issue relates to the uncertainty on how long the various beneficial effects of a treatment would last on a concrete structure, which is critical in estimating the life cycle cost of a treatment. To address these two issues, an ongoing study co-sponsored by FHWA and Virginia Department of Transportation is attempting to develop procedures for estimating optimum treatment time for any candidate concrete structure (based on factors such as initial and target final chloride concentrations, concrete resistivity, etc.) and a model for estimating the beneficial life of a treatment (*100*).

CHAPTER 6. SUMMARY

Salt-induced reinforcing steel corrosion in concrete bridges has undoubtedly become a considerable economic burden to many state and local transportation agencies. Since the iron in the steel has a natural tendency to revert eventually to its most stable oxide state, this problem will, unfortunately, still be with us for the next few decades—although likely in a lesser extent because of some of the corrosion protection measures that came into practice in the last two decades in building new concrete bridges. There is no doubt that adoption of corrosion protection measures, such as use of sensible construction designs, adequate concrete cover depth, low-permeability concrete, corrosion-inhibiting admixtures, and coated reinforcing steel, in new construction will help in significantly prolonging the occurrence of reinforcing steel corrosion in the new bridges.

The use of good construction design and procedures, adequate concrete cover depth, corrosion-inhibiting admixture, and low-permeability concrete alone will not abate the problem, because concrete has a tendency to crack inordinately. In fact, it has been observed lately that the new low permeability concrete or high-performance concrete (made from partial substitution of Portland cement with silica fume or fly ash) has an even more pronounced tendency than conventional concrete to crack—potentially trading a normally slow intrusion of chloride ions into the concrete (by the diffusion process) for a potentially faster gravity-assisted flow of salt-laden water. Even corrosion-inhibiting admixture for concrete would likely not be of use when the concrete cracked. This situation essentially leaves the reinforcing steel itself as the last line of defense against corrosion. For this very reason, the use of a barrier system on the reinforcing steel, such as epoxy coating or other organic or even other possible metallic coatings, is even more critical in abating this costly corrosion problem.

It is likely that there may never be any organic coating that can hold up to the extreme combination of constant wetting and high temperature and high humidity that reinforcing steel is exposed to in the marine environment in Florida, especially in the splash zone, and that either steel bars coated with a sufficiently-stable metallic coating or a type of corrosion-resistant solid metal bars would have to be used in conjunction with the use of sound construction designs and concrete. However, as discussed in an earlier section, there were very convincing reports of good corrosion resistance performance shown by epoxy-coated steel bars in concrete bridge decks, where unlike in coastal bridges in Florida, the concrete does not remain constantly wet and the other exposure condition is not as severe. And, just recently, good performance by epoxy-coated bars has been observed in bridge decks surveyed in Pennsylvania and New York by researchers from a corrosion engineering firm in Virginia (*101*) and in cracked and uncracked concrete by researchers from the University of New Brunswick (*102*). It must also be mentioned that unfavorable performance by epoxy-coated bars has recently been claimed, albeit unconvincingly (*103*,*104*). In one latter case, the poor performing epoxy-coated rebars were located mostly in noise walls, where the quality of the concrete was known to be poor, and in concrete expansion dams (beside expansion joints), where it is suspected that the coated rebars may have been cleaned by abrasive blasting before pouring of the concrete.

The many successful performance of embedded epoxy-coated steel bars in places outside of Florida and possibly other similar locations, indicates that when used in exposure conditions that do not keep the concrete constantly wet, the epoxy coating will provide a certain degree of protection to the steel bars and, thereby, delay the initiation of corrosion. The recent claims of poor performance of epoxy-coated rebars serve, at most, to indicate that the corrosion protection provided by ECR (more accurately, the old generation of ECR) is not permanent and also to raise the question: For how long does the use of ECR, in a particular exposure condition, delays the initiation of steel corrosion in the concrete? And, for a prospective user, the next question is: Is the savings in maintenance and traffic control costs resulting from this extra time worth the initial extra cost of using ECR instead of black steel bars? Unfortunately, accurate determination of the actual field performance of ECR in a particular state or region or exposure condition is extremely difficult, if not impossible, since many contributing factors are involved and have to be accounted for (*98*). Needless to say, the recent improvement of specifications for ECR by the industry and the tightening of requirements on proper storage and handling of ECR at construction sites will ensure good corrosion protection.

The ongoing research study on steel bars coated with new organic and metallic coatings and alternative solid metal bars should result into identification of more corrosion-resistant and, hopefully, cost-effective alternative reinforcement for future use in concrete bridges.

For construction of new prestressed concrete bridge members, the use of a corrosion-inhibiting admixture in the concrete or the grout, in conjunction with use of good construction designs and practices, would provide some corrosion protection. However, the long-term effectiveness of the commercial inhibitor admixtures has not been verified yet.

For existing chloride-contaminated concrete bridge decks, impressed-current cathodic protection—using titanium mesh anodes—provides the ultimate and permanent solution to stopping reinforcing steel corrosion in the structures, as long as associated rectifiers and electrical wiring are properly maintained. Electrochemical chloride extraction provides an alternative rehabilitation method for stopping steel corrosion in contaminated concrete, albeit less permanently. This alternative has the advantage of having no rectifier or wiring to maintain after the treatment.

These same corrosion protection methods are applicable for existing inland concrete substructures. However, in cathodic protection of such concrete members, the appropriate anodes to use include the arc- or flame-sprayed zinc coating and the water-based conductive paints. Although electrochemical chloride extraction is applicable to concrete sub structures, it is more difficult to conduct this treatment efficiently on these bridge members than on bridge decks, because it is relatively difficult to set up the necessary treatment system on vertical concrete surfaces to keep them wet during the entire treatment.

For various prestressed concrete bridge members, either impressed-current or galvanic cathodic protection or both modes can be applied, depending on the types of the bridge members and their surrounding environments. However, before CP is applied to any of these concrete members, it should be qualified first following the proposed guidelines discussed earlier. And, to ensure that no prestressed concrete member is overprotected, which may lead to hydrogen embrittlement of the high-strength prestressed steel strands, the use of electronic remote monitoring systems should be incorporated with any installed CP system.

For prestressed concrete bridge members, such as beams, box beams, etc., impressed-current cathodic protection—using arc-sprayed zinc or titanium as the anode—is the most suitable mode. However, when applying CP, measures must be taken to ensure that the drainage system on a bridge is always properly maintained, so that there would not be any uneven electrical resistivity across the concrete members that would lead to uneven distribution of protection current and subsequent overprotection of the wet areas. Otherwise, use of CP on such members has to be excluded.

For prestressed concrete piles in marine environment, either mode of CP can be applied—with the impressed-current mode allowing for control of the current density being applied on the prestressed steel strands. The use of the zinc-hydrogel and the arc-sprayed Al-Zn-In coating as a galvanic anode in this application appeared to be promising. However, the long-term durability of these galvanic anodes is still being evaluated.

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