# EVALUATION OF CORROSION PROTECTION METHODS FOR REINFORCED CONCRETE HIGHWAY STRUCTURES

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## ABSTRACT

Since the 1970s, research projects and field studies have been conducted on different methods for protecting reinforced concrete bridges from corrosion damage. The methods include alternative reinforcement and slab design, barrier methods, electrochemical methods, and corrosion inhibitors. Each method and its underlying principles are described, performance results of laboratory and/or field trials are reviewed, and systems are evaluated based on the results of the trials. Using performance results from the studies and costs obtained from transportation agencies, an economic analysis is used to estimate the cost of each system over a 75 year economic life using discount rates of 2, 4, and 6%.

Epoxy-coated reinforcing steel is the most common corrosion protection method used in the United States today. Although controversial in many areas, epoxy-coated reinforcement has performed well in many states, including Kansas, since it was introduced in the early 1970s and is a low-cost backup to many other corrosion protection options. Research on stainless steel reinforcement indicates that it may remain free of corrosion in chloride contaminated concrete for more than 75 years. At a low discount rate (2%), solid stainless steel reinforcement is a cost-effective option compared to other options, but at higher discount rates (4%+), the present value cost of a deck with solid stainless steel is significantly higher than that of an unprotected deck. Stainless steel clad reinforcement is much less expensive than solid stainless steel reinforcement. The performance of stainless steel-clad reinforcement will be similar to that of solid stainless steel bars if the stainless steel coating is continuous and if the black steel core, exposed at the bar ends, is protected so that it does not come into contact with concrete pore solution. The present value of the cost of a bridge deck built with stainless steel-clad reinforcement is significantly lower than the present value for the cost of any other corrosion protection system. This method should be considered for experimental use. Solid stainless steel should be considered, as well, if a low discount rate (around 2%) is used. Hot rubberized asphalt membranes are the least expensive option, other than stainless steel-clad reinforcement. Hot rubberized asphalt and spray-applied liquid membranes should be considered for use on future projects. In laboratory tests, corrosion inhibitors have been shown to provide protection to steel in chloride contaminated concrete, but information on their performance in the field is limited. Both calcium nitrite and organic corrosion inhibitors have the potential to be cost-effective, if they perform as well in the field as they have in the laboratory, and should be considered for experimental use.

**Key Words:** bridge decks, calcium nitrite, cathodic protection, corrosion protection, electrochemical chloride extraction, epoxy-coated reinforcement, galvanized reinforcement, organic corrosion inhibitor, overlays, sealers, silica fume, stainless steel-clad reinforcement, stainless steel reinforcement, waterproof membranes

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# INTRODUCTION

## 1.1 GENERAL

Corrosion of reinforcing steel in bridges is a significant economic and safety problem for many transportation agencies, preventing many bridges from attaining their design life. A 1997 report (The Status 1997) estimated that of over 580,000 bridges in the United States, both in and out of the federal aid system, over 100,000 were structurally deficient. The cost to eliminate all of these bridge deficiencies was estimated to be from $78 billion to $112 billion, depending on how long it took to make the repairs. While the corrosion of reinforcing steel is not the only cause of structural deficiencies in bridges in the United States, it is a significant contributor, and therefore a matter of major concern (Virmani and Clemena 1998).

## 1.2 BACKGROUND

Until the late 1960s, reinforced concrete bridges performed reasonably well, with problems such as delaminations and spalling limited to structures in coastal areas. Deterioration of bridge deck concrete in snow belt states coincided with the increase in the application of deicing salts in the winter months as states began to initiate a bare pavement policy. By the late 1960s, many snow belt states began to experience maintenance problems with bridge decks after as little as 5 years of service. By the mid 1970s, it was recognized that the problem was caused by the corrosion of reinforcing steel, caused by the ingress of chloride ions from deicing salts. The benefits of deicing salt are too great for its use to be discontinued, or even decreased, even though the cost of maintaining bridge decks is very high (Virmani and Clemena 1998); so methods of effectively and economically protecting concrete from corrosion damage have received special attention since the early 1970s.

## 1.3 CORROSION OF REINFORCING STEEL IN CONCRETE

Some metals, such as gold, silver, and platinum, occur naturally in their pure form. Many other metals, including iron, are found in their natural state as ores, natural oxides, sulfides, and other reaction products. These metals must be derived from their ores by smelting, from which the metal absorbs and retains the energy needed to free it from the ore. This metallic state is unstable, however, because the metal tends to recombine with elements in the environment and return to its natural state, losing the extra energy in the process. The process of a metal reverting to its natural state is called oxidation, or corrosion (Virmani and Clemena 1998).

Steel has a natural tendency to corrode and to return to its natural state as iron ore, typically ferric oxide, $Fe_2O_3$. The rate of steel corrosion depends on the availability of water, oxygen, and aggressive ions, as well as the pH and temperature of the surrounding environment, and on the internal properties of the steel, such as composition, grain structure, and entrained fabrication stresses (Virmani and Clemena 1998).

The high pH (about 13) of concrete pore water solution results in the formation and maintenance of a passive film (oxide layer) on the surface of the reinforcing steel (Bentur et al. 1997).

As long as the passive film on the reinforcing steel remains intact, the rate of corrosion is very low. However, if the oxide layer is broken, oxygen will be able to react with the steel, resulting in corrosion (*Cathodic* 1998). Significant corrosion does not occur for steel in concrete that is either very dry or continuously saturated because both air and water are necessary for corrosion to be initiated. Steel will remain corrosion resistant in concrete if the concrete cover prevents air and water from reaching the embedded reinforcement (Jones 1996).

Dissolved chloride ions are a big contributor to corrosion in concrete because they impair the passivity of the reinforcement and increase the active corrosion rate of steel. Oxidation is enhanced through the formation of an iron chloride complex, which is subsequently converted to iron oxide and chloride ions, which are then available to again combine with iron in the reinforcement. When corrosion products are deposited, they induce tensile stresses on the surrounding concrete, which cause cracking to occur (Jones 1996).

For corrosion to occur, an electrochemical cell must be present. A cell consists of an anode and a cathode, separated by an electrolyte, and connected by a metallic conductor (*Cathodic* 1998). The anode is the area in which oxidation occurs, or where electrons are released. The cathode is the area where reduction occurs, where electrons are consumed (Jones 1996). The electrolyte is generally an aqueous solution that can carry ions, such as salt water, or, in concrete, alkaline pore solution (*Cathodic* 1998). According to Bentur et al. (1997), "Variations in chemical activity from place to place are associated with corresponding differences in electrical potential; it is these electrical potential differences which are the actual driving forces for the corrosion reactions." Steel in reinforced concrete acts as a cathode when it is in the passive state. When the passive layer is lost, either uniformly by a reduction in the pH of the concrete due to carbonation, or by local breakdown due to chloride ions, parts of the steel act as an anode and start to corrode. Ferrous ions, $Fe^{2+}$, are lost into the solution, which frees up electrons in the steel and makes the potential more negative. Potential differences between cathodic and anodic sites within a structure cause current to flow in the pore solution of the concrete, and through the metal reinforcement (Polder 1998).

When oxygen is present, as is usually the case, the oxidation and reduction reactions at the steel-concrete interface are, respectively,

$$2\ Fe \rightarrow 2\ Fe^{2+} + 4\ e^{-} \qquad (1\text{-}1)$$

$$O_2 + 2\ H_2O + 4\ e^{-} \rightarrow 4\ OH^{-} \qquad (1\text{-}2)$$

Eq. (1-1) is the anodic reaction. Iron is oxidized, releasing electrons and ferrous ions, which dissolve in the solution surrounding the steel. The electrons are deposited on the steel surface, lowering its potential.

Eq. (1-2) is the cathodic, or reduction, reaction. Electrons released by Eq. (1-1), at the anode, flow towards higher potential (cathodic) sites, where they combine with water and oxygen molecules to form hydroxyl ions. The corrosion reaction will only continue if there is a cathodic reaction to accept released electrons, so these corrosion reactions can be stopped if oxygen and water are not available at the cathodic sites on the steel (Bentur et al. 1997).

During the corrosion process, current flows in a closed loop. In addition to electrons flowing through the steel, an external current is carried through the pore solution of the concrete by the movement of charged ions to complete a closed loop. The external current consists of negatively charged hydroxyl ions moving from the cathode to the anode, and positively charged ferrous ions moving from the anode to the cathode. Because this current flow is required to complete the loop, the corrosion reaction will be slowed considerably if the pores in the concrete are dry or are not interconnected very well (Bentur et al. 1997).

Although it is the cause of most corrosion damage in concrete, rust is only a byproduct of the corrosion process, and does not necessarily accumulate where the corrosion occurs (Bentur et al. 1997). Rust is formed according to the following reactions:

$$Fe^{2+} + 2\,OH^- \rightarrow Fe(OH)_2 \qquad (1\text{-}3)$$

$$4\,Fe(OH)_2 + 2\,H_2O + O_2 \rightarrow 4\,Fe(OH)_3 \qquad (1\text{-}4)$$

$$2\,Fe(OH)_3 \rightarrow Fe_2O_3 + 3\,H_2O \qquad (1\text{-}5)$$

When the moving ferrous ions and hydroxide ions meet, they react to form ferrous hydroxide $Fe(OH)_2$ [Eq. (1-3)]. If moisture and oxygen are present, the ferrous hydroxide is then further oxidized to form ferric oxide, or rust [Eq. (1-4) and (1-5)] (Virmani and Clemena 1998).

In any electrochemical cell, corrosion is driven by a potential, or voltage, difference between the anode and the cathode. Voltage differences in reinforced concrete may be created either by differences in the surface of the steel bars or by differences in the electrolyte. Steel is a heterogeneous material, with a patchwork of sites of slightly different potentials on the surface. Potential differences great enough to drive corrosion can be found in areas of residual stress, and even in places where there are scratches on a bar. The electrolyte, or concrete pore solution, surrounding the steel reinforcement may have different concentrations of chloride ions, oxygen, moisture, and hydroxyl ions. These differences can set up microscopic electrochemical cells (microcells), where both the anode and the cathode exist on the same bar. Microcells occur most often in the upper mat of reinforcing, while macroscopic electrochemical cells (macrocells) can be set up by potential differences between two bars in the same mat or between the top and bottom mat of reinforcement in a bridge deck. Macrocells between the reinforcement mats are common in reinforced concrete bridge decks because the upper mat of reinforcement is generally exposed to a significantly higher chloride and moisture content than the bottom mat (Virmani and Clemena 1998).

Galvanic corrosion can occur in concrete when two dissimilar alloys are electrically connected in the presence of an electrolyte. One of the metals will corrode, while the other, the one with the more positive potential, will be protected (Jones 1996). This can be either good or bad. Zinc, aluminum, and magnesium have more negative potentials than steel, meaning that if they were to come into contact with steel in an aggressive environment, they would corrode and the steel would be protected galvanically. Other materials, such as copper and stainless steel, have more positive potentials than steel, so if they were to come into contact with exposed steel in an aggressive environment, they would be protected, while the steel corroded.

## 1.4 CORROSION POTENTIAL

Electrochemical techniques are useful for evaluating the behavior of steel in concrete because many of the methods are non-destructive and can be used to monitor corrosion over time. Measuring the corrosion potential is the easiest electrochemical technique for monitoring corrosion in a structure (Bentur et al. 1997).

Corrosion potential depends on activity at the anode and the cathode, and the electrochemical reaction can be described by curves of potential versus the log of current density (E versus log I). Fig. 1.1 shows the anodic and cathodic polarization curves for steel in an alkaline environment (Bentur et al. 1997).

On the anodic polarization curve, when the potential is very negative (active potential), the corrosion rate increases rapidly with an increase in potential (point 1 to 2), and then drops quickly when the potential gets high enough for the formation of ferric oxide, which passivates the steel surface (point 2 to 3). This is known as the primary passivation potential. After the passivation potential is reached, corrosion remains negligible over a wide range of potentials (points 3 to 4). This is the passive region, above which, water is broken down to produce oxygen, passivity is lost, and severe corrosion begins to take place (point 5). The cathodic polarization curve shows that the rate or the cathodic reaction (producing $OH^-$) decreases with an increase in potential. The corrosion potential ($E_{corr}$) is the potential at which the anodic and cathodic reactions are in balance (the currents for the reactions are equal). The current at $E_{corr}$ is defined as the corrosion current ($I_{corr}$). In non-carbonated, chloride free concrete, the intersection of the two curves is in the passive zone. However, chlorides in the concrete will cause pitting to be initiated and corrosion will occur at lower potentials (Fiq. 1.2) (Bentur et al. 1997).

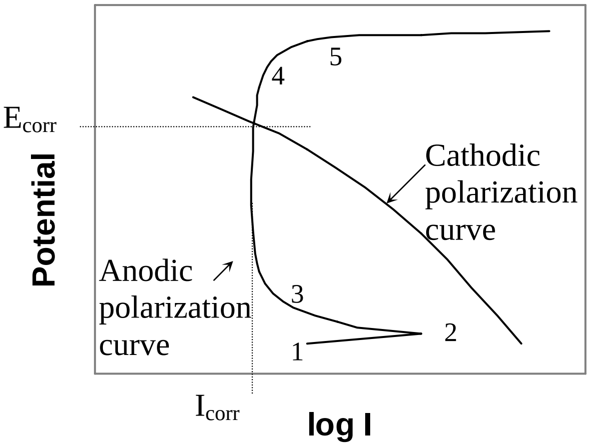


**Figure 1.1 Polarization Curves for Steel in Concrete**
(Bentur et al. 1997)

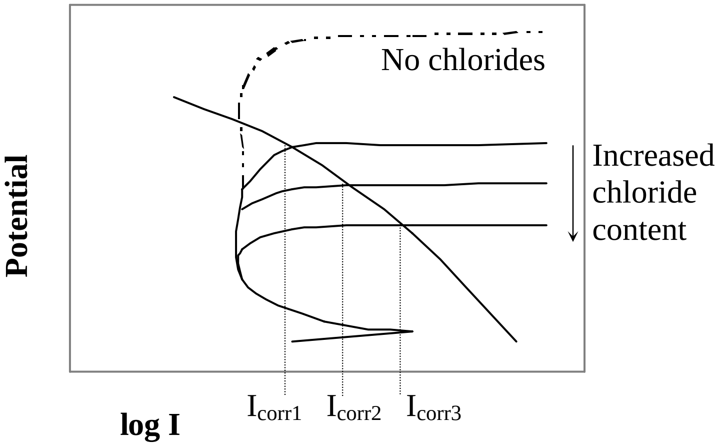


**Figure 1.2 Polarization Curves in the Presence of Chlorides**
(Bentur et al. 1997)

The most common method to determine the corrosion potential of steel in reinforced concrete without having to destroy the structure or the specimen is to take half-cell measurements. Half-cell measurements compare the potential of the reinforcement with that of a reference electrode exposed to the same environment. Reference electrodes are made out of materials with behavior that is basically independent of the surrounding environment (Bentur et al. 1997). In the field, half-cell measurements are usually taken using a copper-copper sulfate (CSE) electrode, and are fairly accurate when proper prewetting of the concrete is used (ACI 222 1996). Other common reference electrodes for use on steel in concrete are the saturated calomel electrode (SCE) and the silver/silver chloride electrode (Ag/AgCl) (Bentur et al. 1997). Potentials measured with the SCE and Ag/AgCl electrodes are 74 mV and 94 mV less negative, respectively, than potentials measured with a CSE.

At a measured half-cell potential more negative than -0.350 volts (CSE) there is a very high probability that active corrosion is present. At measured half-cell potentials more positive than -0.200 V (CSE), there is a very high probability that corrosion is not occurring. In the range of -0.200 to -0.350 V (CSE), the degree of corrosion is uncertain (ACI 222 1996).

In the "uncertain" range, potential differences and other corrosion detection methods must be used. Although potential measurements are not uniquely related to corrosion, they can provide a good indication as to whether corrosion is active in a particular area. Typically, on bridge decks, potential readings are taken in a grid with a spacing of about 1 m. These readings are used to create a potential contour map. Regions that have more negative potentials than adjacent regions and regions where the contour lines are the densest are areas that should be investigated further (Bentur et al. 1997). In Federal Highway Administration studies, potential differences within structures where corrosion was not active have rarely exceeded 100 mV, but potential differences were often more than 200 mV when significant corrosion was occurring (Clear 1981).

## 1.5 CHLORIDE CONCENTRATIONS

It has been difficult to determine an exact "chloride corrosion threshold", the chloride content below which the risk of corrosion is negligible for all concrete mix ingredients and all exposure conditions, but it has been generally accepted that this threshold does exist. According to Bentur et al. (1997), the integrity of the passive oxide film on reinforcement is controlled by two competing processes: the repair, or stabilization of the film by $OH^-$ ions, and the disruption of the film by $Cl^-$ ions.

Therefore, the concentration of chloride ions at the corrosion threshold is dependent on the ratio of $OH^-$ ions to $Cl^-$ ions. However, the threshold is not generally presented as a ratio of $Cl^-/OH^-$ in specifications, but is presented as a total weight of chloride ions in the concrete. ACI 318 allows a maximum water-soluble chloride content of 0.15% by mass of cement (ACI 222 1996), while some studies have indicated that the threshold level may be as high as 0.40% chloride by mass of cement (Locke and Siman 1980, Browne 1980). In a typical bridge deck with a cement content of 390 $kg/m^3$ (658 lb or 7 sacks/$yd^3$), 0.15% soluble chlorides by weight of cement is equivalent to about 0.025% soluble chlorides by weight of concrete, or 0.59 kg soluble chloride per $m^3$ (1 lb/$yd^3$) of concrete. The accepted corrosion threshold for reinforcing steel in concrete is generally 0.6 to 0.9 $kg/m^3$ (1.0 to 1.5 lb/$yd^3$).

There are many different ways in which chloride ions can be introduced to a reinforced concrete structure. Chlorides can be introduced to the concrete mix in an admixture or in contaminated aggregates or mix water, or they can diffuse into hardened concrete from external sources such as salt water or deicing salts. When chloride ions are included in a concrete mix, a substantial portion of the chloride ions are combined in solid cement hydration products and do not remain in the pore solution. Because only the chloride ions that remain in solution are free to directly disrupt the passive layer on the reinforcement, the corrosion threshold will be higher for concrete in which chloride ions were introduced in the mix than for concrete that was exposed to chloride ions after hardening (Bentur et al. 1997).

The transport of chloride ions through hardened concrete is controlled by absorption, diffusion, and capillary action or wicking. Fick’s Second Law of Diffusion is often used to model the diffusion of chloride ions through concrete.

$$C(x,t,C_o,Deff) = C_o\{1 - erf\,[x/(2(tD_{eff})^{1/2}]\} \qquad (1\text{-}6)$$

where

$x$ = depth

$t$ = time
$C_o$ = surface chloride concentration
$D_{eff}$ = effective diffusion coefficient
$erf$ = error function

This equation is a special case of a generalized diffusion model in which three assumptions are true, all of which are violated in the case of chloride ions migrating through concrete. First, the material through which the diffusion takes place must be permeable and homogeneous. Concrete is permeable, but it is not homogeneous. Second, the diffusion properties of the material cannot change with either time or diffusant (ion to be transported) concentration. In most cases, the diffusion properties of concrete are effected as hydration proceeds and as chloride ion concentrations change. Third, no binding or chemical reactions take place between the diffusant and the material. Chloride ions are bound by hydration products of the aluminates in the cement paste and by supplementary cementing materials (Detwiler et al. 1999).

Another limitation of this equation is that it only considers diffusion and does not consider other methods through which chloride ions enter concrete. Near the surface, when concrete has been allowed to dry for a period, capillary action plays a significant role (Detwiler et al. 1999). According to Pettersson (1994), the effect of capillary action can be up to 20 times as great as that of diffusion in a 0.40 water/cement ratio concrete dried at room temperature at 60% relative humidity. However, according to Whiting and Mitchell (1992) "Except for the near-surface region of concrete, where capillary forces may be active under drying conditions, the predominant mechanism for transport of chloride ions in crack-free concrete is by ionic diffusion through the water-filled pore system."

Finally, Fick's Second Law assumes that the surface concentration of the diffusant is constant over time. In the case of chloride ions on concrete, this is most often not the case. Chlorides are applied to bridge decks as deicing salts during the winter months, and can be washed away by rain during the other months of the year

(Whiting and Detwiler 1998). In marine areas, chloride concentrations on bridge substructure members change as they are exposed to wet and dry cycles of salt water in tidal and splash zones.

Despite its limitations, Fick's Second Law provides a useful and realistic way to compare data for concrete from tests using the same set of exposure conditions. Some studies use the term apparent diffusion coefficient to emphasize the fact that this method does not calculate an exact diffusion coefficient (Detwiler et al. 1999).

## 1.6 PREVENTION METHODS

Since the late 1960s, when corrosion of steel in reinforced concrete structures was first recognized as a problem in non-coastal areas, many methods have been developed with the intent of preventing corrosion from occurring. These methods can be divided into four different categories, based on how they provide protection:

(1) *Alternative reinforcement* and *slab design* includes materials that electrically isolate the steel from the concrete and create a barrier for chloride ions, materials that protect steel galvanically, and materials that have significantly higher corrosion thresholds than conventional reinforcing steel. Concrete slabs have been designed without any internal reinforcement.

(2) *Barrier methods* protect reinforced concrete from corrosion damage by preventing water, oxygen, and chloride ions from reaching the reinforcement and initiating corrosion.

(3) *Electrochemical methods* use current and an external anode to protect the reinforcement, even when the chloride ion concentration is above the corrosion threshold.

(4) *Corrosion inhibitors* offer protection by raising the threshold chloride concentration level, by reducing the permeability of the concrete, or by doing both.

## 1.7 OBJECT AND SCOPE

Since the 1970s, there have been many research projects and field studies conducted on different methods for protecting reinforced concrete bridge structures from corrosion damage. Good quality concrete is generally the primary protection system for any structure, but in many situations, quality concrete must be combined with other corrosion protection methods to ensure adequate durability. There is no one corrosion protection method that is entirely effective in all situations. Most transportation agencies have selected a few standard methods for corrosion protection, but these standard methods vary between agencies, and many of these agencies are also experimenting with new methods. Few of the methods have been directly compared in a laboratory or field study.

This report reviews research and field trials of many different methods used for the protection of reinforcing steel in concrete highway structures. The methods discussed include: (1) alternative reinforcement and slab design, consisting of epoxy-coated steel, galvanized steel, solid stainless steel, stainless steel-clad, nickel-clad, titanium, copper-clad, and fiber reinforced plastic reinforcement, and steel-free slabs; (2) barrier methods, including the use of low permeability concrete, low water/cement ratio mix designs, the use of mineral admixtures such as silica fume, fly ash, and blast furnace slag, overlays, waterproof membranes, sealers, and deep polymer impregnation, (3) electrochemical methods, including cathodic protection and electrochemical chloride extraction, and (4) corrosion inhibitors, including products commercially available in the United States.

The object of this report is to:

(1) Describe each method and its underlying principles,

(2) Look at performance results in laboratory and/or field trials,

(3) Evaluate the systems based on the results of the trials, and

(4) Determine the cost effectiveness of each system.

# CHAPTER 2

# ALTERNATIVE REINFORCEMENT AND SLAB DESIGN

## 2.1 GENERAL

The first line of defense against the corrosion of reinforcing steel is high quality concrete. However, high quality concrete can crack and, over time, chlorides can penetrate even the best low permeability concrete. Therefore, the final line of defense against corrosion in reinforced concrete is the reinforcement itself. Conventional mild steel is not very resistant to corrosion, so to improve its performance in contaminated concrete, it must either be coated with an effective and economical barrier, or replaced with a more corrosion-resistant material (Virmani and Clemena 1998).

Barriers tend to be more economical than solid corrosion-resistant bars, and can be separated into two basic categories: organic and metallic. The most common coating for reinforcing steel in the United States is epoxy, which is organic. Epoxy coating electrically isolates the steel from the concrete and creates a barrier to chloride ions. Metallic claddings for mild steel reinforcement include materials that can be either noble or sacrificial compared to the mild steel core. In bars with sacrificial coatings, the reinforcement will be protected at breaks in the coating; however, in bars with noble coatings, the integrity of the coating is important because the steel core will corrode at any breaks in the coating.

A good alternative reinforcement must be resistant to damage during shipping, storage at the construction site, installation, and concrete placement. The reinforcement must also be able to maintain its structural function for the service life of the structure, even in severe service environments. Finally, the alternative reinforcement must be economical when compared to conventional reinforcement (Virmani and Clemena 1998).

In an attempt to completely eliminate the problem of steel corrosion in reinforced concrete, bridge deck slabs have been developed that account for internal arching action and do not require any internal reinforcement. Currently, there are five of these steel-free bridge deck slabs on bridges in Canada.

## 2.2 EPOXY-COATED REINFORCING STEEL

Currently, the primary corrosion protection system for bridge decks in the United States is epoxy-coated reinforcing steel. Epoxy-coated reinforcing steel has been used in approximately 20,000 reinforced concrete bridge decks since the early 1970s, and is estimated to have saved taxpayers billions of dollars in rehabilitation costs (Virmani and Clemena 1998).

### 2.2.1 Background

Epoxy-coated reinforcement was developed in the early 1970s, in response to the need for better corrosion protection on reinforced concrete bridge decks. The method of coating reinforcing steel with epoxy was adapted from the method used by utility companies for coating pipes in the petroleum industry. The bar is cleaned by blasting with grit to a near-white finish to remove millscale, rust and contaminates. The bar is then heated to the temperature required for the application of the epoxy powder, typically 230°C, and passed through an electrostatic spray that applies charged, dry epoxy powder to the steel. The epoxy melts, flows and cures on the bars, which then are quenched, usually with a water spray bath (Manning 1996).

Epoxy resins are thermosetting plastics that have good long-term durability in concrete and are resistant to solvents, chemicals and water (Pike 1973). Tests have shown that the diffusion rates of oxygen and chloride ions through a quality coating of adequate thickness (177 μm, 7 mils) are extremely low, even in severe exposure conditions (Clifton et al. 1974, Pike 1973). However, epoxy based coatings are not impermeable to water (Lee and Neville 1967). Epoxy coatings function in two ways, first by acting as a barrier, keeping oxygen and chloride ions from reaching the surface of the steel, and second by increasing the electrical resistance between adjacent steel locations. Epoxy coatings reduce the magnitude of macrocell currents, which are responsible for extensive deterioration when they develop in bridge decks (Clear and Virmani 1983). Because the protective ability of epoxy coatings depends on their ability to act as both a physical and electrical barrier, effective quality control measures must be taken during coating of the bars and subsequent handling, shipping and storage of the bars (Clear et al. 1995). ASTM A 775, A 934, and D 3963, and AASHTO M 284 provide guidelines to be followed during these processes.

The first use of epoxy-coated steel on a bridge deck was in 1973 on a bridge over the Schuykill River near Philadelphia (Kilereski 1977). By 1976, 19 states were participating in a national experimental program to evaluate epoxy-coated reinforcement (Coated 1976). By 1977, 17 states had adopted epoxy-coated bars as standard and 9 more were using them on an experimental basis (NCHRP 1979). While there were 10 different types of prequalified coatings; the dominant coating, because of its relatively quick production capabilities and flexibility, was Scotchkote 213, which was manufactured by the 3M Company (Manning 1996). As the use of epoxy-coated reinforcement increased, some problems were revealed, such as cracking of the coating during bending and damage to the coating during shipping and handling on the job site. New methods, such as bending the bars before coating, increasing the number of supports during shipping, padding the bundles, and using nylon slings for loading and unloading, were developed in an attempt to overcome these problems (Virmani and Clemena 1998). Specifications also started to require plastic or epoxy-coated chairs and tie wires when they were in contact with the epoxy-coated reinforcement to minimize stray currents and to avoid the creation electrical couples within the structure (Manning 1996).

Epoxy-coated steel, along with higher quality concrete and deeper cover, has provided effective protection against corrosion distress in bridges in the Unites States and Canada for more than a decade (Clear et al. 1995). Many investigators are of the opinion that epoxy-coated steel is a viable option for long-term protection of reinforced concrete structures (Edgell and Riemenschneider 1992). In their opinion, reports of problems with epoxy-coated reinforcement are isolated, and each problem is caused by some shortcoming in the specific materials or construction in the particular structure (Clear 1991). However, recently, several investigators have been led to question the use of epoxy-coated reinforcement as a realistic strategy for preventing corrosion damage to concrete structures (Clear et al. 1995). These analysts believe that the failures of epoxy-coated steel in structures are indicative of generic shortcomings in the technology, and that additional problems will develop as structures continue to age (Clear 1991).

In the 1970's, epoxy-coated bars were only used in the top mat of reinforcement in bridge decks. However, even at this time, it was known that macrocells could develop between the top and bottom layer of reinforcement. It was also understood that the rate of corrosion was controlled by the cathodic reaction. A large bottom mat acting cathodically could cause the development of highly anodic areas at defects in the coating on the top deck steel (Manning 1996). A 1980 study by the Federal Highway Administration (FHWA) comparing test slabs with epoxy-coated reinforcement in the top mat only to slabs with epoxy-coated reinforcement in both slabs indicated that corrosion was reduced by 11.5 times with coated steel in the top mat only, and by 41 times when both mats of steel were coated (Virmani et al. 1983). In the 1980's, many states began using epoxy-coated reinforcement in both the top and bottom mats in bridge decks.

In 1974, a report was published by the FHWA entitled *Nonmetallic Coatings for Concrete Reinforcing Bars* (Clifton et al. 1974). The report covers immersion studies of epoxy-coated bars in water, saltwater, and other liquids, chloride permeability studies of epoxy films, and electrical resistance studies on coated bars and coated bars embedded in concrete partially immersed in 3.5% sodium chloride solution. The tests showed that both powder and liquid epoxy coatings with a thickness of 25 to 100 μm (1 to 4 mils) exhibited excessive holidays. The National Bureau of Standards performed two studies on coated bars in 3.5% sodium chloride solution. The results from the studies indicate that high resistance ratios correlated with good corrosion performance, since the resistance test served as a sensitive holiday detector (Pfeifer et al. 1992).

Many of the epoxy films in the FHWA study appeared to be fairly impervious, even at a thickness of only 75 µm (3 mils). The three films tested that had a thickness between 178 and 254 µm (7 and 10 mils) had no measurable chloride ion permeability after 37 weeks in a permeability cell (Clifton et al. 1974). It is still believed that epoxies are relatively impermeable to chloride ions, but not water (Pfeifer et al. 1992). The concept that the adhesion of epoxy-based coatings to steel is significantly reduced by exposure to water is not new, and was presented in the *Handbook of Epoxy Resins*, published in 1967 (Lee and Neville 1967). Results from a number of studies have supported this idea, indicating that the adhesion between the epoxy coating and the reinforcement is reduced as the moisture content in the concrete increases (Smith and Virmani 1996, Weyers et al. 1997, Schiessl 1992). Some studies have suggested that once the coating is allowed to dry, some of the original adhesion to the bar is salvaged (McDonald et al. 1998). The adhesive strength necessary for epoxy-coatings to protect the underlying steel has never been established. Some studies have indicated that the mere loss of adhesion does not indicate that the coating is unable to protect the steel, but that in the presence of defects, performance is directly related to the adhesion of the coating (*Adhesion* 1995). Other studies have identified the loss of adhesion of the epoxy coating as an important failure mechanism for epoxy-coated reinforcing steel (Clear 1992a).

### 2.2.2 Studies

Field experience with epoxy-coated reinforcement in the United States has generally been good, with a few significant exceptions. This section discusses the results of various laboratory and field studies on epoxy-coated reinforcement.

**Florida's Experience —** In 1979, the Florida Department of Transportation began construction of a series of bridges in the Florida Keys using epoxy-coated steel.

Signs of corrosion started to appear in the substructures in 1986, only six years after construction. Over the next seven years, each of the five major bridges began to show signs of corrosion damage in the splash zone of piers. Upon examination, it was found that the coatings were disbonded and in many cases, liquid with a pH as low as 5 was found beneath the coating (Manning 1996). In response to the failures of the Florida Key Bridges, laboratory and field studies were started by the Florida Department of Transportation with the University of South Florida to investigate possible causes for the failures (Sagues et al. 1994). The studies determined that extensive disbondment of epoxy coating can be initiated by a number of different factors that include exposure to salt water, mild levels of cathodic polarization, and anodic conditions while corrosion is underway. It was also determined that disbondment could occur in chloride-free concrete and that sodium ions, and possibly potassium ions, were instrumental in the process. It was assumed that handling and storage in salty air before installation of the bars on the structures made normal imperfections in the bars deteriorate further. It was also assumed that some of the bars were already disbonded before they were cast in the structure.

**Adhesion Studies —** According to a study on adhesion loss conducted by the University of Western Ontario for the Ontario Ministry of Transportation in 1993, the loss of adhesion on epoxy-coated reinforcing steel does not change short-term performance in the absence of defects; but if defects are present, performance is directly related to the adhesion of the coating. The study also concluded that adhesion was decreased in the presence of contaminants, and improved by an increase in surface roughness, and that water penetrating the coating and displacing the epoxy from the steel surface seemed to be the main mechanism of adhesion loss (*Adhesion* 1995).

The condition of epoxy-coated reinforcement taken in 1996 from 18 bridge decks, constructed between 1977 and 1995 in Virginia, was evaluated by Pyc et al. (2000). With the exception of too many holidays in bars from one deck, all of the epoxy-coated reinforcement samples evaluated met the specifications for coating thickness and holidays that were in place at the time of construction. The adhesion of the epoxy-coatings was evaluated using MTO – Draft 93 10 27 “Hot Water Test for Epoxy-Coated Reinforcing Bars.” In this test, an “x” cut is made in the epoxy coating between deformations, and an area of the bar is exposed by inserting the blade of an Xacto knife under the coating. The adhesion of the coating is given a value of 1 to 5. A newly coated bar would have an adhesion value of 1, meaning that the blade tip was unable to be inserted under the coating. If less than 2 $mm^2$ of steel is exposed, the adhesion rating is 2. Exposure of 2 to 4 $mm^2$ of steel gives an adhesion rating of 3. If more than 4 $mm^2$ of steel is exposed, the adhesion rating is 4. Finally, if the blade slides easily under the coating and levering action removes the entire section of coating, the adhesion rating of the coating is 5.

No reduction in adhesion was detected in bars from three decks, one built in 1983 and two built in 1995. The bars from the other decks exhibited a decrease in adhesion, with adhesion ratings ranging from 2 to 5. The study concluded that, although the reinforcement appeared to be in good shape, the loss of adhesion was a matter of concern for the long-term performance of epoxy-coated reinforcement in concrete. Ninety-four percent of the sample size showed evidence of adhesion reduction, including some complete coating disbondment. This coating disbondment was not caused by excessive coating damage or by the presence of chloride ions on the steel surface, but rather by water penetrating the coating.

According to the authors, these results indicate that epoxy coating will not maintain its bond to reinforcing steel in moist concrete environments. If, when chloride ions arrive at the surface of the reinforcement, the coating has already disbonded, corrosion will take place under the coating. This corrosion will occur at a rate similar to that of a bare reinforcing bar in an acidic environment, which is faster than the corrosion of bare steel in concrete.

Other studies have found that reinforcement in concrete with high moisture contents generally suffers reduced adhesion of the coating (Smith and Virmani 1996, Weyers et al. 1997, Schiessl 1992). Unfortunately, epoxy-coated reinforcement is never entirely free of defects. Florida stopped specifying epoxy-coated steel for reinforcement in bridge substructures in 1988, and stopped using epoxy-coated reinforcement in all construction in 1992 (Manning 1996). It is quite possible that organic coatings will never be able to protect reinforcing steel in a hot, humid, salt contaminated environment such as the Florida Keys where the concrete stays wet continuously; but research has been more favorable towards the use of epoxy-coated reinforcement in bridges where the concrete does not stay wet on a continuous basis (Virmani and Clemena 1998).

**Michigan Studies —** In 1973, Michigan proposed a study to look at three of the most promising non-metallic protective coatings, and compare their performance to that of uncoated and galvanized reinforcement, as well as comparing epoxy coatings with different amounts of surface preparation (McCrum and Arnold 1993). The study was performed in cooperation with the Federal Highway Administration. Specimens were cast using galvanized steel and epoxy-coated steel from various companies. The results for the specimens containing epoxy-coated steel were mixed; some performed well, while one performed even worse than plain, black steel. This reinforces the idea that, “an epoxy coating is only as good as its ability to act as a barrier.” It was found that it is important to clean the bars well before coating them, and that it is also important to get a good bond between the bar and the coating.

Michigan began a second study a few years later (McCrum et al. 1995). In 1976, three bridge decks were constructed using epoxy-coated, galvanized, and black steel in adjacent spans. (In 1972, five bridge decks had been constructed using galvanized and uncoated reinforcement in adjacent spans.) In 1991, these decks were evaluated along with nine newer decks that had been constructed using epoxy-coated steel between 1977 and 1982. Because the decks with epoxy-coated steel were at most only 15 years old, it was not surprising that the sections had not exhibited any signs of deterioration. However, cores indicated that the epoxy-coated steel was in the best shape out of the three bar types, especially in cores taken near cracks in the concrete (McCrum et all, 1995). When the cores from the bridges were evaluated, it was observed that the epoxy coating did not strongly adhere to the bars and could be removed easily with a fingernail while the bars were still wet. Concern was expressed that once the steel was exposed to deicing salts at holidays, corrosion could progress along the reinforcement under the coating and create a situation worse than that of an uncoated bar. The authors were unable to compare the effect of the three different types of reinforcement at the deck level because none of the spans showed any signs of distress.

According to McCrum et al. (1995), the extra cost of using epoxy-coated steel in both the top and bottom mat of bridge decks is justified by an expected increase in service life. The steel costs (material alone, 1995) for an average, 10000 $ft^2$ (929 $m^2$) bridge deck containing 72000 lbs (32,700 kg) of reinforcing steel were estimated as $41,760 for uncoated steel and $46,000-$50,400 for epoxy-coated steel.

**Concrete Reinforcing Steel Institute Studies —** A number of studies of the ability of epoxy-coated steel to prevent corrosion damage have been funded by the Concrete Reinforcing Steel Institute (CRSI), starting in 1985. CRSI is a member-supported trade organization headquartered in Schaumburg, Illinois (Manning 1996). The first study started in 1982 and was a long-term exposure study performed by Kenneth C. Clear, Inc. of Sterling, Virginia. For this study, concrete slabs were poured using a w/c ratio of 0.42, with either both mats black steel, both mats epoxy-coated steel, or top mat epoxy-coated steel and bottom mat black steel. The epoxy-coated bars were certified as meeting the applicable specifications for 1982. The slabs were ponded with 3% sodium chloride solution for three days a week for a period of three years until there was a high concentration of chloride ions at the level of the bars. After the ponding cycles were discontinued in 1985, the specimens were left outdoors to weather naturally. The measured chloride concentration at the level of the reinforcement at this time was about 6 $kg/m^3$ (11 $lb/yd^3$) (Pfeifer et al. 1992). All of the slabs containing black steel in both mats cracked within the first 1.5 years as the result of corrosion of the reinforcement. After 6.5 years, cores were taken from the slabs containing coated bars. No evidence of corrosion was found, but some softening of the coating had taken place (Scannell and Clear 1990). After 8.5 years, no cracks had appeared in the slabs, but macrocell corrosion currents and half-cell potential readings indicated that active corrosion was occurring in the specimens (Kenneth C. Clear Inc. 1992). All but one of the specimens, which had been removed for another study after 8.8 years, showed signs of corrosion induced cracking after 9 years of exposure (Clear 1992a).

In a report for a related study for the Canadian Strategic Highway Research Program (C-SHRP), (Clear 1992a), Clear performed an autopsy on half of one of the slabs that contained hairline cracks. The slab contained epoxy-coated steel in the top mat only, and significant corrosion was found on the bar beneath the crack even though macrocell measurements were low. The other bar was in uncracked concrete and had not corroded. When the corroded bar was evaluated, the pH of the solution under the coating was found to be between 4.0 and 5.0. This lead Clear to suggest that corrosion of quality epoxy-coated steel occurs in two phases. First, the coating disbonds from the steel in the absence of significant macrocell action. Corrosion begins underneath the coating and eventually splits it open, exposing the bar and allowing macrocells to be set up between the steel mats.

In 1988, another study was initiated to investigate the effectiveness of epoxy-coated reinforcement in corrosion protection (KCC 1992). The study was performed by KCC and funded by CRSI until 1991. This new study involved laboratory and outdoor exposure testing of bars from eight different suppliers in the United States. Twenty different variables were accounted for in the study including straight versus bent bars, bend diameter, coating thickness, application of the coating before or after bending bar, rate of bending, temperature of steel during bending, patching of damaged areas, and control (black steel). Sixty slabs were cast, 3 for each variable. Each slab had one bent and two straight epoxy-coated bars in the top mat, and black steel in the bottom mat. After the specimens had cured, two slabs for each variable were exposed to 70 weekly "southern exposure" cycles, each consisting of 4 days continuous ponding with 15% sodium chloride solution followed by 3 days air drying and exposure to ultraviolet light at a temperature of 100° F. The third specimen for each variable was exposed in an outdoor facility in northern Virginia and was to serve as a control.

After the 70 weekly southern exposure cycles, the test slabs that had been exposed to the cycles were subjected to sustained tapwater ponding for periods of 3 to 10 ½ months and then stored outdoors, exposed to natural Virginia weather for about 15 months.

The four black bar control specimens were experiencing heavy corrosion by the end of the 70 weeks of southern exposure cycling. Cracks and rust stains were clearly visible on the specimens and corrosion current densities averaged about 2.15 $\mu A/cm^2$ (2 $mA/ft^2$), which corresponds to a corrosion rate of about 25 μm/yr (1 mil/yr). No cracks or rust stains were found on any of the specimens containing epoxy-coated bars in the top mat at the end of the southern exposure cycles. However, some of the specimens started to crack due to corrosion stresses during the subsequent tapwater ponding, and mat-to-mat resistance decreased significantly in some specimens. KCC reported to CRSI that the reasons that some epoxy-coated bars performed so well, while others performed poorly, had not been discovered. The only variable that seemed to have a significant effect on the corrosion protection of epoxy-coated reinforcement was the source of the bar.

The third CRSI-funded study to be undertaken by KCC was an evaluation of 13 bridge decks that were built using epoxy-coated steel as reinforcement. The bridges ranged from 9 to 13 years old at the time of the study and were located in Virginia, Wisconsin, Pennsylvania, New York and Ohio. All of the bridges had been exposed to freeze/thaw cycles and deicing salt. The average cover over the top mat of steel was 2.5 inches, and the chloride ion concentration at the level of the reinforcement had reached the accepted threshold level for corrosion of 0.6 $kg/m^3$ (1 lb/yd$^3$) in about half of the structures. KCC reported that 87% of the top-mat epoxy-coated reinforcement was found to be basically free of corrosion, and that all of the bars exhibiting significant corrosion came from cores that had cracks that extended all the way down to the level of the reinforcement. KCC concluded that epoxy-coated reinforcement had performed well up to the date of the study on the bridge decks investigated, but that predictions of future performance were not possible.

In August of 1991, CRSI asked Wiss, Janney, Elstner Associates, Inc. (WJE) to review the draft report and research on the three studies that KCC had performed for CRSI (Pfeifer et al. 1992). Twenty-two slabs, 11 southern exposure and 11 control (outdoor exposure) specimens, as well as a number of left over bars from the eight-source study were obtained from KCC for the WJE review, along with four test slabs from the long-term exposure study. The 11 corrosion-tested slabs from the eight-source study received by WJE exhibited a wide range of corrosion performance and were the main focus of the study.

The first conclusion that WJE reached about the eight-source study was that it was an unusually severe test. The use of 15% sodium chloride solution for 70 weeks of southern exposure cycling resulted in high chloride contents at the bar level. The extended tapwater ponding also increased the severity of the test and may have created osmotic pressures. According to WJE, studies by the oil and pipeline industry have found that epoxy films allow the infiltration of aggressive elements under high temperature and pressure. WJE argued that the tapwater ponding on the specimens could have induced adequate pressure to allow these aggressive elements to penetrate the epoxy coating on the steel. The corrosion currents in the control specimens of this test greatly exceeded the corrosion currents of control specimens in previous studies (Pfeifer et al. 1992).

It should be noted that in a later study for the Federal Highway Administration (McDonald et al. 1998), WJE used a similar ponding procedure for its test specimens. This procedure consisted of three days of drying at 38° C (100° F) and 60 to 80% relative humidity followed by four days of ponding with a 15% sodium chloride solution at 16 to 27° C (60 to 80° F) for a period of 12 weeks. This cycle was followed by 12 weeks of continuous ponding with a 15% sodium chloride solution (rather than tapwater as KCC had done) at 16 to 27° C (60 to 80° F) and 60 to 80% relative humidity. The entire 24-week cycle was repeated four times for a total of 96 weeks.

In review of the same test in a NCHRP report, Clear speculates that the "southern exposure" cycles used by KCC may actually have given "misleadingly optimistic" results. Clear suspected that the samples had dried out somewhat during the cycles, so that even though the chloride level in the concrete was high, there was not enough moisture to initiate corrosion on the epoxy-coated bars. Once the tapwater ponding began, the moisture content increased and accelerated the corrosion process (Clear et al. 1995).

WJE concluded that the corrosion performance of epoxy-coated reinforcement is directly related to the number of holidays in the coating. To protect a bar, the coating needs to maintain a high electrical resistance, which depends on coating thickness and quality (not too many holidays). The bars from the eight-source study that contained less than one holiday per foot consistently exhibited good corrosion protection throughout the study, maintaining corrosion current densities of less than 1 $mA/ft^2$. Eighty percent of the bars containing up to 30 holidays per foot performed well, exhibiting corrosion current densities of less than 1 $mA/ft^2$ during the 70-week southern exposure cycling. However, many of these bars exhibited moderate to high levels of corrosion after the tapwater ponding. Bars containing more than 30 holidays per foot performed poorly during both parts of the study (Pfeifer et al. 1992).

WJE also concluded that "electrical resistance is a good measure of holidays and the potential corrosion protection qualities of epoxy-coated rebar." According to WJE, electrical resistance tests confirm that KCC used bars with a wide range of holidays, which resulted in a wide range of corrosion protection performance (Pfeifer et al. 1992).

The epoxy-coated bars that were obtained from KCC from the eight-source study were subjected to the following tests: A scanning electron microscope and energy dispersive X-ray (EDS) detector were used to map chloride ions and other materials on the backside of the epoxy coating after its removal from the bar. The backside of the coating was also checked for mill scale, rust, salts, and other evidence of contamination. The bars were cleaned and surface roughness was measured. Water absorption of the epoxy coating was tested, and coating thickness and holidays were measured. According to WJE, even the basic observations necessary to predict the performance of epoxy-coated bars such as holidays, film thickness, steel surface condition, and backside contamination of the epoxy coating were not properly reported in the KCC report. According to coating thickness measurements recorded by KCC, 7 of the 22 slabs sent to WJE had bars with more than 10 percent of their coating thickness measurements less than 125 μm (5 mils) (Pfeifer et al. 1992), which is considered cause for rejection according to ASTM A 775.

KCC contended that the bar measurements used by WJE to draw their conclusions were taken from retained bars that had been sitting in the lab for 3 years rather than from the measurements taken by KCC on the actual test bars before they were cast in the specimens. According to Clear, WJE’s measurements detected more holidays in the coatings because of: deterioration during storage and damage during previous testing, shipping and handling; measurements taken near the bar ends, which KCC did not include in its data because the ends were not embedded in concrete; differing definitions of the word holidays; and the fact that the bars with the lowest holiday count were picked out by KCC and used in the test specimens (Clear 1992a).

Four of the long-term exposure slabs from the KCC study were shipped to WJE for additional testing for CRSI along with the specimens for the eight-source study. Two of the slabs had never been exposed to salt solutions, and the other two had been exposed to salt solutions and contained hairline cracks above some of the bars (Manning 1996). The slabs were cut in half and one half was ponded with salt solution while the other was ponded with tap water. One slab that was cracked and was ponded with salt solution showed notable secretions of rust and was autopsied after 175 days. The coating beneath the crack could be removed from the bar easily, and about half of it was blistered and cracked. About one third of the coating on a nearby bar was well bonded to the steel, while the rest of the bar was actively corroding where adhesion had been lost. About 10% of the other two bars was corroding or the epoxy coating had lost its adhesion to the steel (Pfeifer et al. 1993).

In January 1992, Clear made a presentation of the studies that he had performed over the past years for the corrosion committee at the annual meeting of the Transportation Research Board. Clear reported that his findings had led him to a different conclusion about the effectiveness of epoxy-coated steel than had previously been reported and that bars coated to meet current specifications would not ensure long-term durability in structures subject to severe chloride exposure (Manning 1996). Clear's presentation was documented in the form of a memo, the contents of which were published in the May 1992 edition of *Concrete International* along with a response from CRSI (Clear 1992b).

CRSI filed a civil action against Kenneth C. Clear and Kenneth C. Clear, Inc. in April 1992. The suit was not specifically about the effectiveness of epoxy-coated reinforcement. The actual charges were racketeering, fraud, breach of contract and malpractice in connection with the work that KCC had performed for CRSI over the past 6 years. The racketeering charges were dismissed at the preliminary hearing in May of 1992 (Top 1992), and the remaining charges were dismissed in October of the same year following CRSI's presentation of evidence, before the defense had even presented it's case. The judge commented that, "…there is an underlying current here, which I think the evidence supports, and that is that this case is less about money than it is about an attempted discrediting of the defendant" (*CRSI vs. KCC* 1992). No costs were awarded. The judgement was not appealed, but as a result of legal fees, Clear was forced to close the research laboratory (Rebar 1992).

**Canadian Strategic Highway Research Program Study —** In 1990, the Canadian Strategic Highway Research Program (C-SHRP) began a study to investigate the effectiveness of epoxy-coated reinforcing steel (Clear 1992a). The project was awarded to Kenneth C. Clear, Inc. The study consisted of testing epoxy-coated reinforcing bars obtained from 12 different coaters, seven jobsites, and 19 field structures built between 1974 and 1988 in Canada and the United States. It was found that the number of holidays per meter of the bars from the field structures was much higher than expected, averaging more than 20 per meter. Ninety-four percent of the bars had more than 10 holidays per meter. Underfilm contamination, a measure of the cleanliness of the bar when it was coated ranged from 10-70%, with an average of 25%, which is often accepted as a maximum contamination level by the pipeline industry.

In the report, Clear questioned the validity of the traditional method for evaluating long-term corrosion tests. He argued that most tests, including “southern exposure” ponding tests, simply speed up the rate of chloride contamination of the concrete, but do not accelerate the corrosion of the specimen once the corrosion process has begun. This means that results should be evaluated accordingly. For example, if the time to reach the chloride contamination level in an outdoor exposure specimen is 1 year, then a 5 year test is only equivalent to a field structure in the first 4 years after it reaches chloride contamination. The approach used to estimate service life extensions in the report was to subtract the time-to cracking of the black steel from the time-to cracking of the epoxy-coated steel rather than dividing it, as had been done in previous studies. The time to cracking in this study was 1.5 years for slabs with uncoated reinforcement, and 8 years for slabs with epoxy-coated reinforcement. Therefore, if the time to contamination in a structure is 10 years, the service life for epoxy-coated steel would be determined to be 10 years + 6.5 years, or 16.5 years, rather than 8/1.5 X 10 years, or 53 years, as would have been determined using the traditional method.

The study concluded that in the northern US and Canada, the time before corrosion damage to a structure will occur is only improved by three to six years by the use of epoxy-coated steel. The study identified:

> An unexpected failure mechanism involving progressive loss of adhesion and underfilm corrosion in the highway concrete environment . . . active in northern and southern field structures and in ‘perfect’ coated rebars (i.e., coated bars which have zero holidays, zero bare areas and comply with all aspects of all North American specifications including those recently proposed for adoption by NACE, AASHTO and ASTM). (Clear 1992a).

The study’s conclusions bring up several important and controversial points. Clear concluded that simply producing epoxy-coated reinforcing bars that have fewer holidays and bare areas, both after fabrication and after outdoor exposure on a construction site, will not greatly improve the long-term performance of the bars. He concluded that, in addition to fewer breaks in the coating, better adhesion must also be obtained to improve the long-term performance of epoxy-coated reinforcement in concrete, and that; “There is presently no evidence that such can be routinely achieved using present day powders and coating technology.”

Because of the controversy surrounding the topic of epoxy-coated reinforcement when the final draft report was received in 1992, and because of ongoing litigation between KCC and CRSI, the final report, (Clear 1992a), was subjected to a detailed review. The review by Schiessl (1992), supported Clear’s major conclusions. He presented a theory in which corrosion begins at defects in the epoxy coating. These bare areas become anodic and induce adjacent areas beneath the coating to become cathodic. While corrosion continues, cathodic areas become anodic as water and chlorides penetrate beneath the film, increasing the anodic area and the rate of disbonding. Moisture and oxygen pass through the coating causing cathodic disbondment. Disbondment is also caused by hydroxyl ions that undercut the coating at defects. According to Schiessl, even in the absence of defects, the penetration of water, and possibly to a lesser extent, oxygen and chloride ions, through the film could cause small amounts of corrosion, causing adhesion loss and eventually blistering. Results of both laboratory and field investigations have supported the observation that the rate of deterioration of the coating begins slowly and speeds up over time. The results also indicate that deterioration increases with an increase in the moisture content of the concrete (Schiessl 1992).

Schiessl also noted that the corrosion reaction could accelerate through macrocell action in structures with an uncoated bottom mat of reinforcement that is electrically connected to an epoxy-coated top mat. High macrocell current densities could be set up between the mats once corrosion has begun because of the relatively small surface area of defects on the coated bars in the top mat as compared to the uncoated bars in the bottom mat.

**Federal Highway Administration Studies —** In 1993, the Federal Highway Administration started a research project on corrosion-resistant reinforcement for concrete. The purpose of the study was to develop cost effective new types of organic, inorganic, ceramic, and metallic coatings as well as metallic alloys that could be used as reinforcement in concrete. These new types of coatings and reinforcement were to provide a corrosion-free design life of 75 to 100 years. The research was performed by Wiss, Janney, Elstner and Associates, Inc. (McDonald et al. 1998).

Prescreening tests were performed on 22 different bendable (ASTM A 775) and 11 different nonbendable (ASTM A 934) organic-coated bars. Bars were tested for adhesion in solutions and by cathodic disbondment tests. Seven of the coatings that showed the best performance were selected for screening tests, four bendable and three nonbendable. 3M Scotchkote 213 was also selected because it had been the most commonly used epoxy coating for reinforcement for many years. (Scotchkote 213 was no longer manufactured after March of 1993 for environmental reasons.) The adhesion of the nonbendable coatings was significantly better than that of the bendable coatings, especially in bent bars. Nonbendable coatings were not damaged by the bending process because the coating was applied after the bars were bent. The coatings exhibited better adhesion when the bars were dry than when they were tested wet, but once adhesion was lost in cathodic disbondment tests, it was unlikely to be recovered after drying. In most of the samples, adhesion loss was not a problem in straight areas away from the holes that were drilled in the coating.

The two best bendable and nonbendable coatings were selected for in-concrete tests, along with Scotchkote 213. Test procedures (described earlier in the discussion of WJE's work for CRSI), consisted of four cycles of 12 weeks of alternate wetting and drying followed by 12 weeks of continuous saltwater ponding.

The results of the study support the continued use of epoxy-coated steel as reinforcement for bridges under the following conditions. Epoxy-coated steel should be used in both the top and bottom mats. The use of epoxy-coated steel in both mats significantly reduced the corrosion rates in all cases, "suggesting that the corrosion mechanism of epoxy-coated bars may be the inhibition of the cathodic reaction that requires electrons, oxygen, and hydroxide to be present at the cathode at the bar surface." Damage to the bar coatings should be minimized during shipment and placement. Coating damage and any cracks in the concrete should be repaired on site.

A clear relationship was found between the mat-to-mat resistance values and the corrosion resistance of the epoxy-coated bars. Systems that maintained high electrical resistivities provided better corrosion protection. (High resistivities were directly related to low amounts of damage to the coating.) This is consistent with previous findings by WJE. It was also determined that solution immersion and cathodic debonding tests are poor predictors of the actual performance of epoxy-coated steel in concrete, although cathodic debonding tests are a good measure of the consistency of coating during the application process. The chloride concentration threshold for the initiation of corrosion for damaged epoxy-coated bars was determined to be the same as that of black bars.

In 1996, the Federal Highway Administration published a report entitled *Performance of Epoxy-Coated Rebars in Bridge Decks* (Smith and Virmani 1996). The report summarizes investigations performed by highway agencies and universities in the United States and Canada and C-SHRP. The performance of epoxy-coated reinforcement was evaluated on a total of 92 bridges and three barrier walls in 3 provinces and 11 states, including Kansas.

"The overall condition of the bridge decks was considered to be good. Even though deck cracking was prevalent, it did not appear to be corrosion related." There were very few delaminations and/or spalls reported, and little or no maintenance work had been required on most of the bridges studied. Eighty-one percent of the epoxy-coated reinforcement samples that were extracted from the structures exhibited no signs of corrosion, although the chloride concentration at the level of the bars was above the threshold level in most of the decks. Corrosion was worse in locations with cracking, shallow cover, high concrete permeability, and/or high chloride concentrations. Performance also appeared to be affected by the number of defects in the coating and the amount of disbondment that the coating has experienced. Disbondment often occurred as a result of prolonged exposure to moisture. The study also resolved that the use of epoxy-coated bars in recent construction has been complimented by the use of adequate cover with quality concrete, adequate inspection, and proper finishing and curing of the concrete, all of which improve the corrosion performance of the structure.

### 2.2.3 Epoxy-Coated Prestressing Strands

Since 1982, the FHWA has required the use of corrosion protected reinforcing steel and tendons in bridge decks. Epoxy-coated prestressing strands were developed as a way to provide extra corrosion resistance in prestressed concrete (PCI 1993).

Generally, epoxy-coated strand consists of a low-relaxation seven-wire prestressing strand coated around the outside, to a thickness of 0.64 to 1.14 mm (25 to 45 mils). The epoxy coating can be either smooth, or can have grit particles embedded in it to improve the bond between the strand and the concrete. The smooth strands are used in unbonded post-tensioned concrete systems, external post-tensioned systems, and cable stays, while the bond controlled (grit impregnated) strands are used in bonded pretensioned and post-tensioned systems. Epoxy-coated prestressing strands are also available with epoxy filled in between the wires, and are generally used when there is a risk of contaminants or moisture entering the ends and migrating through the strands (PCI 1993).

The corrosion protection of epoxy-coated prestressing strands is considered to be similar to that of epoxy-coated reinforcing steel (Dorsten et al. 1984), but because the epoxy coating on the prestressing strands is much thicker and is not allowed to have any holidays (pinholes), it may perform better.

### 2.2.4 Summary

Reoccurring conclusions from studies of epoxy-coated reinforcement include:

- The performance of epoxy-coated reinforcing steel is enhanced by quality concrete and adequate cover.
- Epoxy coatings lose their adhesion to steel reinforcement when exposed to moisture. Whether this adhesion loss is a direct cause of corrosion damage is under debate.
- Performance of epoxy-coated reinforcement is related to the number of defects (holidays) in the coating. These defects directly effect the electrical resistivity of the reinforcement.
- Most problems that have been reported with epoxy-coated reinforcement have occurred in environments where the concrete is continuously wet, yet oxygen is still available (splash zones on piers, or areas of high humidity). Often these environments have high average temperatures.
- There is little doubt that the time to corrosion-induced cracking is increased in many concrete structures containing epoxy-coated reinforcement over the time to corrosion-induced cracking in bridge decks with no protective measures.

However, the question remains open as to whether epoxy-coated steel provides adequate long-term protection to reinforced concrete highway structures that are exposed for moisture and chlorides.

## 2.3 METALLIC COATINGS AND SOLID METALLIC REINFORCING BARS

For years, metallic coatings have been used successfully to prevent corrosion of steel in environments other than concrete, leading to the anticipation that they could also protect steel in concrete. Metallic coatings for reinforcing steel can be divided into two categories, sacrificial or noble. Sacrificial coatings are made up of metals that have a more negative corrosion potential than steel, such as zinc. If the sacrificial coating is broken, a galvanic cell is set up and the coating corrodes while protecting the steel. Noble coatings such as copper and nickel do not corrode in concrete, but if the coating is broken, any exposed steel becomes anodic and will corrode (Virmani and Clemena 1998).

Solid metallic bars made of corrosion-resistant materials such as stainless steel have also been used in reinforced concrete structures. These materials must be able to serve as reinforcement as well as resist aggressive environments.

### 2.3.1 Galvanized Reinforcement

Evidence began to surface in the early 1960s that hot-dipped zinc-coated steel reinforcement may provide superior performance to that of uncoated steel (McCrum and Arnold 1993). Zinc-coated, or galvanized, bars are produced by a hot-dip process. This consists of cleaning the steel bars by pickling, and then immersing them in molten zinc. Typically, in accordance with ASTM A 767, galvanized bars are dipped in a chromate bath after coating to passivate the zinc surface and to prevent it from reacting with the hydroxide in fresh cement paste (Virmani and Clemena 1998).

Like steel, zinc has corrosion products that occupy more volume than the original metal, and, as a result can cause concrete to crack. An advantage of galvanized reinforcement is that, when zinc corrodes sacrificially, a hydrated oxide [$Zn(OH)_2$] is formed on the surface that acts as an electrical insulator. This insulator is thought to form a barrier layer at active corrosion sites that will prevent further corrosion from occurring (McCrum and Arnold 1993).

In the early 1960s, the American Hot Dip Galvanizers Association began to circulate a letter, dated August 14, 1961, from the Director of Public Works in Bermuda. The letter discussed a deteriorating bridge over salt water that was demolished in 1935. The bridge was believed to have been built in the early 1900s and contained both uncoated and galvanized steel reinforcement in the deck. When the deck was demolished, it was found that the uncoated reinforcement had corroded enough to cause cracking in the concrete while the galvanized reinforcement had not (McCrum and Arnold 1993).

**Studies —** In 1968, Michigan proposed a study in cooperation with the FHWA to compare galvanized reinforcement to uncoated reinforcing steel and also look at the effects of concrete cover and water/cement ratio on corrosion performance (McCrum and Arnold 1993). In 1973, the study was expanded to compare epoxy-coated reinforcement to galvanized and uncoated reinforcement. Specimens were constructed using galvanized, epoxy-coated, and plain “black” steel. The specimens were then placed outdoors in a testing facility. Dikes were built around the edges of the specimens to retain water and a saltwater solution was applied to the surfaces on a regular basis throughout the winter months.

After six years of outdoor exposure, the specimens were examined. There was roughly the same amount of cracking over the galvanized and uncoated steel for covers of 25.4 mm (1 in.) or more, but for the specimens with only 12.7 mm (½ in.) of cover, cracks were present over all of the uncoated bars and only sporadically over the galvanized bars. The specimens used for the first portion of the study contained uncoated transverse reinforcement in one half and galvanized transverse reinforcement in the other. The bars were electrically connected by longitudinal galvanized reinforcing bars, which acted galvanically to slow the corrosion of the uncoated steel.  Because of this linkage, it is significant that any difference in performance of galvanized and uncoated reinforcement appeared in this study.

Upon visual inspection, the specimens containing galvanized steel seemed to have performed better than either the uncoated steel or the epoxy-coated steel. However, the researchers pointed out two specific problems with the test procedures in the second part of the study that gave the galvanized steel samples an advantage over the epoxy-coated samples.  First, the ends of the bars were exposed outside of the specimens and were not coated.  This was a significant disadvantage for the epoxy-coated bars because it was an easy place for corrosion to begin.  The galvanized bars did not exhibit corrosion at these ends because the zinc protects the bars sacrificially.  The second advantage of the galvanized reinforcement was that the top and bottom mats in the specimens were not electrically connected.  In actual structures, these two mats are usually connected, even if attempts are made to keep them both isolated.  This connection allows for macrocell corrosion between the top and bottom mats of steel, as the chloride concentration is generally quite different in the two mats.  Since the mats were not connected, microcell corrosion between areas of different concentrations on the same mat was the dominant reaction.  It was concluded that this was an advantage to the galvanized steel because epoxy coating isolates the steel from the outside environment in the concrete and therefore also reduces the effects of macrocell corrosion.  Had the mats been connected, the increase in corrosion on the galvanized specimens would have been, in all likelihood, much greater than the increase in corrosion of the epoxy-coated specimens.  Because of this, the conclusion of the study was that epoxy-coated specimens would have performed better than the galvanized ones.

The authors concluded that the benefits of providing galvanized steel for bridge decks in Michigan would outweigh the cost increase, assuming that the cost of galvanized reinforcement to be 1.65 times that of black steel, but that epoxy-coated steel is a better option for long-term corrosion protection.

In 1972, Michigan began a study to compare the performance of galvanized and uncoated steel in bridges (McCrum et al. 1995); five bridge decks were constructed using galvanized and uncoated reinforcement in adjacent spans. Three more bridge decks were constructed in 1976 using epoxy-coated, galvanized, and black steel in adjacent spans. In 1991, these decks were evaluated. Cores taken from the first five bridges indicated that the galvanized reinforcement provided better corrosion protection than black steel under similar conditions, but no significant differences were found in the condition of the concrete in the decks. For the second part of the study, cores indicated superior performance for the epoxy-coated, followed by the galvanized and then uncoated reinforcement. Performance was measured using half-cell potentials and visual inspection and ranking of the reinforcement that was extracted from the cores. No spalling or delamination occurred in any of the decks built in 1976.

The study concluded that, when chloride ions are diffusing slowly through the concrete cover, galvanized reinforcement could provide a few extra years of protection before the chloride concentrations reach the higher corrosion threshold level of the galvanized reinforcement. However, galvanized reinforcement provides no advantage once the concrete cracks and chloride ion concentrations increase to levels well above the threshold of either black or galvanized steel.

According to this report, the extra cost of using epoxy-coated steel in both the top and bottom mat of bridge decks is justified by an expected increase in service life; while the extra cost of using galvanized steel in bridge decks is not justified by the questionable extension in service life that it provides.

Galvanized steel was tested as part of a study of the effectiveness of corrosion inhibiting admixtures published by the Virginia Transportation Research Council in 1999 (Zemajatis et al. 1999). Three types of commercially available corrosion inhibiting admixtures and one type of galvanized steel were evaluated in concrete specimens subjected to ponding and drying with a salt-water solution over a 3½ year period. Galvanized steel was observed to be providing good protection in the higher quality concrete. The specimen reinforced with galvanized steel seemed to be performing well for concrete with a w/c ratio of 0.45, with no cracks from corrosion visible at the time of inspection. The other galvanized steel specimen had a w/c ratio of 0.71, and exhibited excessive cracking.

**Conclusions —** Results of both laboratory and field studies on galvanized reinforcement have been conflicting (Virmani and Clemena 1998). A panel was formed in 1982 to resolve these discrepancies, and came to the following conclusions (Manning et al. 1982): Laboratory studies with aqueous solutions have shown that zinc has a higher chloride concentration level than steel at the onset of corrosion. This higher threshold helps to explain why galvanizing can delay the onset of corrosion in outdoor specimens. There is, however, an indication from these outdoor specimens that once corrosion begins on galvanized bars, corrosion-induced distress in the concrete occurs faster than for specimens containing black bars. It is difficult to use laboratory performance to predict service life in the field because of a lack of standardization in laboratory tests. It is also not possible to predict the service life of most structures containing galvanized steel because they have not been in service long enough to accumulate a concentration of chloride ions above the threshold for corrosion at the reinforcement level. In marine structures where the chloride concentration in the concrete is high, sufficient oxygen is often not available to support corrosion. Rapid corrosion has been found in chloride-contaminated structures containing interconnected black and galvanized reinforcement. When galvanizing is followed by chromate treatment, the chemical reaction of the zinc with the fresh concrete does not reduce the bond of the steel to the concrete. And finally the panel concluded that, assuming that normal construction practices are used, for a new concrete bridge deck with 51 mm (2 in.) of cover and a w/c ratio of 0.45, the use of galvanized steel may add 5 years to the typical 10-15 years before corrosion damage occurs on a typical unprotected bridge deck.

### 2.3.2 Stainless Steel Reinforcement

The term stainless steel refers to a group of corrosion resistant steels that contain a minimum of 12% chromium. The chromium creates an invisible surface film that helps to make the stainless steel corrosion resistant by resisting oxidation. Other metals can be added to increase the corrosion resistance. Stainless steels are divided into four types, ferritic, ferritic-austenitic, austenitic, and martensitic. Ferritic steels are low carbon steels with less than 17% chromium. Austenitic steels are low carbon steels with around 18% chromium and 8% nickel. Ferritic-austenitic steels (duplex steels) typically contain 22-28% chromium and 4-8% nickel. Martensitic stainless steels have a chromium content of 12-18% and a carbon content as high as 1.2%. Austenitic and ferritic-austenitic steels can be produced as ribbed bars within the normal range of strength and deformability. However, the strength of these bars "as rolled" is not sufficient. Either special heat treatment or cold and warm working (the temperature of the steel is raised sufficiently so that the steel is easier to form, but not so high as to change any of the metallurgical properties) is generally used to increase the strength of the bar (*Stainless* 1996).

There are four types of corrosion in steel: general corrosion, intergranular corrosion, pitting and crevice corrosion, and stress corrosion. Pitting corrosion is the most common type of corrosion of stainless steel (as it is for black steel) in concrete, especially in the presence of chloride ions. Welded bars experience greater corrosion than unwelded bars; shot blasting the weld to remove scale and temper colors can help (*Stainless* 1996).

There are various grades of stainless steels that have been developed for use as reinforcement in concrete and to resist chloride environments. Stainless steel offers many advantages: Stainless steel has inherently good corrosion resistance and does not require the aid of other corrosion protection methods, such as cathodic protection or corrosion inhibitors. Stainless steel has good strength and ductility, and many of the commonly used grades also exhibit good weldability. The advantages of using solid stainless steel as reinforcement, rather than stainless steel clad carbon steel, include the fact that solid stainless steel bars can withstand shipping, handling and bending without the danger of damage to the coating. Exposed ends are also not a problem in solid stainless steel bars, and do not have to be repaired as they do in stainless steel clad bars (Smith and Tullmin 1999). The reason that solid stainless steel is used rather than stainless steel clad carbon steel in Europe is that the process of fusing the two types of metal together is not considered to be cost effective (*Stainless* 1996).

Stainless steel is often used in areas where sufficient cover cannot be obtained or at construction joints and critical gaps between columns and decks. Because of the cost of stainless steel, many engineers do not expect that it will become a standard for all reinforcement (*Stainless* 1996). The types of stainless steel reinforcement that have been most commonly used are types 304, 316, and 316LN. All three types are austenitic stainless steel (Smith and Tullmin 1999).

The FHWA study, *Corrosion Evaluation of Epoxy-Coated, Metallic-Clad, and Solid Metallic Reinforcing Bars in Concrete* (McDonald et al. 1998), looked at two different types of solid stainless steel in concrete exposure specimens, ASTM A 955 Types 304 and 316. Specimens were concrete slabs that contained either stainless steel in both the top and bottom mats of reinforcement, or stainless steel in the top mat and black steel in the bottom mat. Some of the specimens were precracked above the reinforcement, and others contained bent bars in the top mat of reinforcement. All of the test bars were subjected to damage over 0.5% of their surface before being cast into the slabs. The specimens were subjected to four 24-week cycles consisting of 12 weeks of wetting for four days with a 15% sodium chloride solution and drying for three days, followed by 12 weeks of continuous saltwater ponding.

The lowest corrosion rates for the Type 304 bars were obtained when the stainless steel was used in both mats. Cracks in the concrete did not appear to affect the performance of the bars. Half of the bars from specimens that contained black steel in the bottom mat exhibited moderate to high corrosion currents and had red rust on them. When the stainless steel bars were used in both mats, the specimens did not exhibit any signs of chloride-induced corrosion, even when the slabs were precracked. Both conditions (cracked and uncracked concrete) had about 1,500 times lower corrosion than the black steel specimens. Corrosion currents on bars with black bottom mats were 20 and 700 times lower than for black steel specimens, for bent and straight bars, respectively.

All of the specimens containing Type 316 solid stainless steel showed good corrosion performance. There was no distinguishable difference between precracked and uncracked slabs or between slabs with a black steel cathode or a stainless steel cathode. Measured corrosion for all conditions was about 800 times lower than that of the black steel specimens. During visual inspection of the slabs, only one of the bars exhibited any corrosion, and it was considered to be minor.

Both types of stainless steel were able to tolerate chloride levels much higher than the threshold level of black steel prior to the initiation of corrosion, especially when both mats were stainless steel. The threshold for Type 304 stainless steel with a stainless steel cathode was 7 to 18 kg/m$^3$ (12 to 30 lb/yd$^3$). For Type 316 stainless steel in both mats, the chloride concentration threshold ranged from 12 to 20 kg/m$^3$ (20 to 33 lb/yd$^3$). Even when the stainless steel bars were coupled to black steel cathodes, the chloride concentration for the initiation of corrosion was still about twice that of black steel for Type 304 and 15 times the threshold of black steel for Type 316. The results from the study indicate that Type 316 stainless steel bars may be better than Type 304 bars for use in concrete because they are less susceptible to galvanic effects if they are coupled to carbon steel bars.

Stainless steel and stainless steel-clad reinforcement have been used in a number of structures in the past 15 years, but none of these structures is old enough that corrosion damage would be expected, even if no protection measures had been used. So far, stainless steel reinforcement is performing satisfactorily. For example, in 1984, Type 304 stainless steel reinforcing bars were installed in part of a bridge deck north of Detroit. The rest of the bridge was built using epoxy-coated steel. The deck was inspected and cores were removed in 1993 by Michigan DOT officials. No delaminations or corrosion-induced cracks were present on the deck. Two of the cores had longitudinal cracks (from temperature and shrinkage) that intersected the reinforcing steel, but no evidence of corrosion was found, except for minor staining on one bar at the crack location. The chloride ion concentrations had approached the corrosion threshold for black steel, but had not exceeded it significantly (McDonald et al. 1995b).

Table 2.1 gives the ratios of the actual costs for three bridge projects that were completed in Illinois in 1994. The actual cost of each bridge, constructed with black and/or epoxy-coated steel is compared to what the costs would have been, according to industry experts, had stainless steel or titanium reinforcement been used. The use of epoxy-coated reinforcement had very little effect on the overall price of the projects, while stainless steel would have increased the total cost by 6 to 16%. Titanium would have increased the total bridge cost by 35 to 91%.

**Table 2.1 Reinforcement Cost Ratios** (Virmani and Clemena 1998)

| | Cost of reinforcement/ cost of black steel | Total cost / total cost with black steel | | |
|---|---|---|---|---|
| | | Bridge A | Bridge B | Bridge C |
| 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 |

Results of both field and laboratory studies of stainless steel as corrosion resistant reinforcement have been promising. Stainless steel exhibits excellent corrosion resistance in severe corrosion environments. No corrosion-induced damage has occurred in any of the studies reviewed for this report. A 1994 report indicated that stainless steel should extend the time-to-cracking in reinforced concrete structures by 65 to 130 times, compared to black steel. In situations where long-term durability is important, the extra cost of using stainless steel appears to be justified by the expected service life extension provided (McDonald et al. 1995).

### 2.3.3 Stainless Steel Clad Reinforcing Bars

Studies have shown that solid stainless steel reinforcement can provide significantly longer service lives than conventional reinforcement in reinforced concrete structures that are exposed to corrosive environments (McDonald et al. 1995b), but its use is limited due to its relatively high cost. Stainless steel clad bars are being studied as a lower cost corrosion resistant reinforcement option. Concerns with the use of stainless steel clad reinforcement include difficulty in bonding the cladding to the bars (*Stainless* 1996), and, if areas of mild steel are exposed in the concrete, those areas will corrode (Darwin et al. 1999).

A study performed by Darwin et al. (1999) at the University of Kansas compared the performance of stainless steel clad reinforcing bars to that of uncoated black steel bars. Corrosion rates and corrosion potentials were monitored for both stainless steel clad and conventional black steel reinforcing bars to determine the corrosion resistance of the stainless steel bars.

The stainless steel clad reinforcement was produced by spraying 304 stainless steel on the surface of a portion of a billet and then rolling the steel in the normal fashion. The stainless steel remained on the outside of the steel, giving the No. 19 (No. 6) bars a cladding than ranged from 0.196 to 0.894 mm (7.7 to 35 mils) in thickness, with an average thickness of 0.467 mm (18 mils).

Rapid corrosion potential and time-to-corrosion tests were performed for both plain bars and bars embedded in mortar, and a scanning electron microscope was used to determine the thickness and continuity of the cladding. For the corrosion potential and time-to-corrosion tests, the bars were placed in simulated concrete pore solution with a 1.6 molal ion concentration of sodium chloride.

The study concluded that the stainless steel clad bars corroded at a rate about two orders of magnitude lower than that of black steel bars, except in cases where the mild steel core at the ends of the bars was exposed to the test solution, in which case the stainless steel clad reinforcement behaved similarly to black steel.

### 2.3.4 Nickel-Clad Reinforcing Bars

Nickel claddings for steel reinforcing bars were first used in the late 1960s. The corrosion resistance of nickel in alkaline chloride solutions is high, and although steel is less noble than nickel, the corrosion of the underlying steel in not substantially accelerated if breaks occur in the barrier (Tripler et al. 1966). Nickel clad bars are produced by applying a heavy layer of nickel to a billet before it is hot rolled. The result is a continuous coating of wrought nickel on the surface of the bar with a diffusion zone of alloyed nickel and iron underneath. The alloyed zone provides additional protection should there be a break in the wrought nickel barrier. Although research has been promising so far, steel bars with a nickel cladding of ample thickness to prevent corrosion damage are relatively expensive (Virmani and Clemena 1998).

### 2.3.5 Titanium Reinforcing Bars

A 1994 study, (McDonald et al. 1995), found the corrosion rate of titanium bars to be 14,000 times less than that of black steel in a pH 7 salt solution, and 135 times less than that of black steel in a high pH salt solution. The study estimated that titanium bars would extend the time-to-cracking of concrete by about 130 times, which would not be as cost effective as using stainless steel considering that the cost of titanium bars, $13.20/kg ($6.00/lb) is about six times that of stainless steel, $2.60/kg ($1.20/lb).

### 2.3.6 Copper-Clad Reinforcement

Copper-clad reinforcing steel bars were first tested in concrete in 1980 in connection with an FHWA study (Virmani et al. 1983). Copper-clad reinforcing bars were compared to black steel in concrete with and without calcium nitrite corrosion inhibitor and to nonspecification epoxy-coated bars. The nonspecification bars had been coated for an earlier study and stored outdoors for over two years; the bars used in the study all contained more that 25 holidays per foot and failed the ASTM bend test. Visible damage of the epoxy-coatings was estimated at less than 0.05% of the surface area. The copper-clad bars were not discussed in the FHWA report, but the results were published by McDonald et al. in 1996. The results are summarized in Table 2.2. These results indicate that the copper-clad bars with a coating thickness of about 0.5 mm (0.02 in.) exhibited much better corrosion resistance than the other types of reinforcement in the study, even the black steel with calcium nitrite corrosion inhibitor. Slabs containing copper-clad bars in the top mat only and in both mats were still in good condition after 13 years of outdoor exposure with no visible cracks. The average total chloride contents in the slabs at this time were 8.50 to 10.32 kg/m$^3$ (14.33 to 17.40 lb/yd$^3$), which is well above the corrosion threshold level of steel.

**Table 2.2 Slab conditions after exposure** (McDonald et al. 1996)

| **Reinforcement Type** | **Age of the Slabs (years)** | | |
|---|---|---|---|
| | **2** | **7** | **13** |
| Black Bars | Badly cracked and bars corroded | ----- | ----- |
| Black Bars | ----- | Severe cracking | Severe Cracking |
| Black Bars with 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 Bars in Both Mats | ----- | ----- | No cracking |
| Copper-Clad Bars in Top Mat Only | ----- | ----- | No cracking |

Copper, lead and zinc salts have been known to retard the hydration of cement. Upon examination of cores taken from the slabs, it was found that the copper-clad bars had discolored the surrounding concrete, turning it a gray-green color. Petrographic observation indicated that there was a significantly higher amount of unhydrated cement around the copper-clad bars than elsewhere in the slab. This change extended 0.25 to 0.5 mm (0.01 to 0.02 in.) from the copper-clad bars into the surrounding concrete. “The paste surrounding the copper-clad rod is still considered to be hard, even though relatively unhydrated.” (McDonald et al. 1996).

In a similar study performed in 1984 (Virmani and Clemena 1998), copper-clad reinforcement again showed good corrosion resistance. Slabs containing copper-clad bars were subjected to 48 weeks of cyclic wetting and drying with a 15 % sodium chloride solution, followed by 48 weeks of ponding. None of the specimens containing copper-clad bars exhibited any signs of corrosion activity, but discoloration of the concrete and retardation of the cement surrounding the copper clad bars was observed. The copper cladding had also turned black, which was assumed to be normal copper oxide film, formed after fabrication of the specimen.

Further tests were performed for an FHWA project in 1995. Specimens consisted of slabs, some of which contained copper-clad bars in both mats, and others that contained black steel in the bottom mat and copper-clad steel in the top mat. Some of the specimens were also precracked above the reinforcement. Test procedures consisted of four cycles of 12 weeks of wetting for four days and drying for three days followed by 12 weeks of continuous saltwater ponding. The results indicated that corrosion rates were lowest when copper-clad bars were only used in the top mat and when the concrete was not cracked. However, corrosion rates for copper-clad bars were significantly lower (23 to 92 times lower) than for black bars under all conditions. No cracking or staining was observed on any of the specimens after 96 weeks of testing (McDonald et al. 1998).

Tests up to this time have shown excellent corrosion resistance for copper-clad reinforcing bars in concrete. Copper-clad bars could prove to be cost effective, as their price in 1996 was estimated to be as low as \$1.20/kg (\$0.54/lb) versus \$0.44/kg (\$0.20/lb) for black steel and \$0.62/kg (\$0.28/lb) for epoxy-coated steel (delivered, on site). However, before copper-clad reinforcement can be put to use in any structures, more research needs to be performed on the structural effect of the retardation of cement hydration that is caused by these bars (McDonald et al. 1996, Virmani and Clemena 1998).

## 2.4 FIBER REINFORCED PLASTIC REINFORCEMENT

Recently, a number of studies have looked at the possibility of using fiber reinforced plastic as a noncorroding reinforcement option for concrete. By 1996, more than 15 highway and pedestrian bridges had been built in Europe, Japan, and Canada using fiber-reinforced plastic bars (Erki and Riskalla 1993).

There are three elements that make up a fiber-reinforced plastic bar. The first is fibrous material, which reinforces the laminate and gives it strength. Most commercial fibers are less than 20 μm in diameter and have a length more than 100 times their diameter (>2 mm). Most fibers used today are carbon, glass or aramid. Fibers cannot act alone because, although they are strong in tension, they cannot support compression, torsion or bending loads. To overcome this problem, the fibers are embedded in a resinous synthetic polymer, which is the second element. Resins are much lower in strength and stiffness than fibers, but act as adhesives and can transfer loads to the fibers through shear. The third element is a finish, or coupling agent, that enhances the adhesion between the other two components (Phong 1997).

A study by Liao et al. (1997) looked at the long-term durability of glass fiber reinforced plastic composites for use in infrastructure applications. Preliminary results suggest that water and salt solutions reduce the life of the material when it is exposed to long-term fatigue loading. Exposure to freeze-thaw cycles in 2% sodium chloride solution resulted in a decrease of the dynamic modulus, flexural modulus, flexural strength and toughness of the composite.

The first bridge to be built in the United States using fiber-reinforced plastic in the concrete deck is a bridge across Buffalo Creek in McKinleyville, West Virginia. The bridge was completed in 1996 and consists of a glass-fiber-reinforced plastic bar reinforced concrete deck on steel stringers. On the construction site, the fiber reinforced plastic bars were handled similar to steel reinforcement. Manpower requirements were reduced because composite bars are relatively lightweight and easy to handle. The use of leather gloves was recommended while handling the bars, and some workers were disturbed by the presence of razor sharp edges on the bars. The fiber reinforced plastic bars were more flexible under foot traffic than conventional steel bars, so chairs had to be spaced closer together to limit deflections under construction loads. The chairs were also used to tie down the mesh because there was concern that the composite reinforcement would float when the wet concrete was vibrated (Thippeswamy et al. 1998).

The bridge is currently being monitored and tested for its response to loads, aging and temperature changes. Initial (short-term) load test results indicate that superstructure deflections are within allowable limits, but enough time has not passed to allow for determining the long-term response of the bridge (Thippeswamy et al. 1998).

Fiber reinforced plastics are also being studied for use as prestressing tendons. The first highway bridge to be prestressed with composite tendons is a two span continuous concrete bridge, built in Germany, which was post-tensioned with fiberglass tendons (Phong 1997). A new system, using a double-T girder cross-section bridge, reinforced with glass fiber reinforced plastic bars and prestressed with carbon fiber reinforced plastic strands, was developed and evaluated at Lawrence Technological University in Southfield, Michigan. The initial response of the system during construction was promising, and the authors hoped that it would continue to perform well under testing (Grace and Abdel-Sayed 1996).

Up to this time, fiber reinforced plastic reinforcement is performing well when used experimentally as corrosion resistant reinforcement in concrete highway structures. There are, however, some limitations to the use of fiber reinforced plastic that need to be taken into consideration. Carbon fiber reinforced plastic strands exhibit superior fatigue behavior, high tensile elastic modulus and low relaxation, but they also have a low tensile strain at failure (Agony et al. 1992). Carbon composites can have a modulus of elasticity that is higher than steel, which would lead to lower deflections. A significant problem is that composites are generally elastic and tend to undergo little strain up to the point of failure. Because glass and aramid composites have a significantly lower tensile modulus and shear stiffness than steel, deflection rather than strength usually governs design (Phong 1997). Another problem with glass composites is that they are made of silica, which can combine with calcium hydroxide in concrete, resulting in a loss of reinforcement.

## 2.5 STEEL-FREE SLABS

A system in which all of the internal reinforcing steel is removed from concrete deck slabs on steel girder bridges has been developed. For a typical bridge, the slab is assumed to be a transverse bending member supported by a series of girders and, thus, subject to flexural failure. This leads to the need for steel reinforcement in the slab. Research (Bakht and Markovic 1986), however, has revealed that the actual mode of failure in bridge decks is not flexural since failure occurs under concentrated loads that are several times higher than those predicted by flexural theory. Rather, compressive membrane forces, referred to as internal arching forces, are developed in the slabs. Because of these forces, bridge decks have a substantial load capacity beyond that based on flexural design. The amount of reinforcing steel can be reduced significantly if the internal arching action is considered. For many situations, calculations using an empirical method indicate that a reinforcement ratio of just 0.3% reinforcing steel in each direction, in top and bottom layers, is needed. The Ontario Highway Bridge Design Code, developed in 1979, includes design provisions for this empirical method (Newhook et al. 1997).

More recently, it has been determined that it is possible to remove all internal steel reinforcement from a concrete slab on a girder bridge deck, as long as adequate transverse and lateral restraint is provided to develop sufficient membrane compressive forces in the concrete (Mufti 1993). This concept was used in the construction of the first steel-free concrete bridge deck over Salmon River in Nova Scotia. The necessary lateral restraint was provided by transverse steel straps underneath the slab, which tie adjacent girders together (Newhook and Mufti 1996).

In December 1995, the Salmon River Bridge was opened to traffic. The bridge is being tested under conditions of severe northern winter conditions in Nova Scotia. The bridge is frequently subjected to deicing salts and undergoes many severe freeze-thaw cycles each winter. The bridge is divided into two simply supported spans of 31.20 m (102.4 ft) each. The first span was constructed using conventional steel reinforcement, and the second span has the reinforcing steel-free deck (Newhook and Mufti 1996).

The design details for the two parts of the bridge deck are essentially the same. Both use a girder spacing of 2.7 m (8.9 ft), a slab thickness of 200 mm (8 in.) and a concrete strength of 35 MPa (5000 psi). The steel reinforcement in the conventional bridge is 1.9% of the slab cross-sectional area, while the steel straps in the steel-free slab equal 0.5% of the slab cross-section (Newhook and Mufti 1996). Low modulus polypropylene fibers were added to help control cracks and to provide some ductility to the concrete after cracking. These fibers are considered secondary reinforcement, since they do not improve concrete strength (Newhook et al. 1997).

The concrete deck acts compositely with the steel girders through the use of shear studs that were placed in three rows, spaced at 300 mm (12 in.) in the midspan sections and at 250 mm (10 in.) in the end sections (Newhook and Mufti 1996). When a concentrated load is applied to the deck directly between two girders, the top flange of each girder has a tendency to deflect away from the load. This deflection is prevented by the steel straps that are welded to the top flange of each girder. Each strap is 100 x 14 mm (4 x 0.55 in.) and made up of uncoated weathering steel.

Because the straps are uncoated, a design thickness of 12 mm was used to account for 2 mm of loss in section due to weathering. The straps develop tension forces and restrain the deck, causing compressive membrane forces to develop in the concrete slab. After the concrete has cracked due to flexure, these compressive membrane forces allow the slab to support the load through arching action, even though the slab no longer has any flexural capacity. The controlling failure mechanism of the deck is punching shear. Depending on the degree of lateral restraint, the ultimate load can be much higher than the flexural failure load of a reinforced concrete deck (Newhook et al. 1997).

The Salmon River Bridge is currently being monitored for field performance. The total cost, in Canadian funds, of the conventional reinforced concrete deck span was \$63,000, or \$143/m$^2$ (\$13.3/ft$^2$), while the total cost of the steel free deck was \$66,850, or \$152/m$^2$ (\$14.1/ft$^2$). However, based on the experience from the Salmon River Bridge, the authors felt that a 175 mm (7 in.) thick deck slab could be used in the future, along with a lower fiber content in the concrete. These changes would reduce the cost of the steel free deck to \$134/m$^2$ (\$12.4/ft$^2$) (Newhook and Mufti 1996).

Bakht and Mufti (1998) discuss five different bridges, built with steel-free deck slabs in Canada. All of these bridges were performing well under unrestricted traffic, and laboratory models had further confirmed static and fatigue strength of the slabs. The first bridge discussed was the Salmon River Bridge. On the second, the Catham Bridge, the cost of the steel-free slab ended up being significantly higher than that of a conventional deck, because of expensive carbon fiber reinforced plastic grids used in negative moment regions resulting from the use of cantilever overhangs. However, the third bridge, the Cowchild Trail Bridge, used glass fiber reinforced plastic bars in the negative moment regions, and the steel-free deck was actually chosen as the cheaper alternative when compared to a conventional deck. The fourth bridge, the Waterloo Creek Bridge, did not have any cantilevers, so like the Salmon Creek Bridge, it did not require tensile reinforcement. The fifth bridge discussed in the paper is a single-lane, single-span forestry bridge with a precast arch steel-free panel. These panels are significantly cheaper than conventional precast reinforced concrete panels.

Both the cast-in-place and precast steel-free deck slab concepts are protected by patents (Bakht and Mufti 1998).

# CHAPTER 3
# BARRIERS

## 3.1 GENERAL

There are many different corrosion protection methods for reinforcing steel in concrete that can be classified as barriers. Some of these methods work by preventing chloride ions from penetrating the concrete and depassivating the reinforcement. Others work by reducing the movement of air and moisture into the concrete and therefore slowing the corrosion process. Procedures such as using good quality, low water/cement (w/c) ratio concrete and adequate cover have become standard in bridge construction. Many states place low-permeability concrete overlays or waterproof membranes with asphalt overlays on their bridge decks. Sealers are used on bridge decks and substructures in some states in an attempt to keep water and chloride ions from penetrating the concrete.

Barrier methods are often used in conjunction with other corrosion protection methods, such as epoxy-coated steel or other alternative reinforcement.

## 3.2 LOW PERMEABILITY CONCRETE

Probably the most important requirement for any reinforced concrete structure is the use of good quality concrete. Concrete contains many interconnected pores, which form a network of channels that allow water and oxygen to infiltrate the material. Generally, a low w/c ratio and good consolidation results in fewer and smaller pores, as well as fewer voids, and a subsequent reduction in permeability. In addition to slowing the ingress of water, oxygen, and chloride ions, reduced permeability leads to a reduction in electrical conductivity since fewer chloride ions are carried into the concrete by moisture (Virmani and Clemena 1998).

### 3.2.1 Cover

The amount of concrete cover over reinforcement should be as large as possible, within the limits of good structural design, the service environment, and cost. The ingress of chloride ions into cement paste is accompanied by both physical adsorption and chemical binding, so the effect of cover depth on chloride concentration is not linear. The effectiveness of the concrete cover depends on the lack of cracks, the porosity, and diffusion rate of the concrete. Diffusion rates can be reduced by using low w/c ratios, or by adding pozzolans, such as silica fume to the concrete. ACI 318 specifies a minimum cover of 50 mm (2 in.) for No. 6 bars and larger and 38 mm (1½ in.) for No. 5 bars and smaller in concrete exposed to weather. Some studies have recommended that, for severe exposure environments, such as bridge decks and parking structures, the minimum cover should be higher (ACI 222 1996).

The 1998 AASHTO LRFD Bridge Specifications require a minimum concrete cover of 100 mm (4 in.) for concrete that is in direct exposure to sea water, 75 mm (3 in.) for coastal concrete, and 65 mm (2½ in.) for concrete that is exposed to deicing salts or on deck surfaces that are subject to tire stud or chain wear. The specification allows the minimum concrete cover over epoxy-coated reinforcement to be reduced to 40 mm (1½ in.).

### 3.2.2 Low Water/Cement Ratio

A low w/c ratio will reduce the permeability and increase the strength of concrete. This will extend service life, both by limiting the amount of water, oxygen, and chloride ions that are able to penetrate into the concrete, and by extending the time until the stresses caused by corrosion of the steel reinforcement cause the concrete to crack (Virmani and Clemena 1998).

Studies have shown that essentially impermeable concrete could be produced using conventional mixes with w/c ratios between 0.30 and 0.32 (Sherman et al. 1996). ACI 318 limits the w/c ratio to a maximum of 0.40 for corrosion protection of reinforced concrete structures exposed to deicing chemicals. Even with a low w/c ratio, proper consolidation is important to ensure low permeability concrete (ACI 222 1996).

In 1992, a 35 year-old bridge in Windsor, Connecticut was demolished to make way for a wider bridge. As longitudinally sawed sections of the bridge slab were observed during demolition, it became evident that the performance of the slab in terms of corrosion resistance had been excellent. No reinforcement corrosion was found in any of the demolition debris or in any of the cores taken from the concrete. The good performance of the bridge was attributed to a low w/c ratio, high cement content, well graded fine aggregate, a high dosage of water reducer/retarder, good consolidation, minimum cracking, and a bituminous wearing course that shielded the concrete from some chloride ions, although problems have been reported with such wearing courses because asphalt is porous, and allows water to seep through and cause deterioration of the concrete underneath (see section 3.4.1). The concrete cover over the reinforcement on the bridge ranged from 32 to 50 mm (1¼ to 2 in.), with most areas having very close to 38 mm (1½ in.) of cover. The concrete mix used on the bridge contained 446 kg/m$^3$ (752 lb/yd$^3$) of Type III portland cement and had a w/c ratio of 0.35 (Schupack and Stark 1998).

The Kansas Department of Transportation uses a w/cm ratio of 0.40 to 0.44 on monolithic decks and subdecks and a w/cm ratio of 0.36 to 0.40 in bridge deck overlays overlays (Miller and Darwin 2000).

### 3.2.2 Mineral Admixtures

A pozzolan is a material that contains silica that is able to react with calcium hydroxide. Advantages to using pozzolans in concrete include improved workability of harsh mixes, lower heat of hydration, and improved permeability and durability of hardened concrete. When pozzolans are used in concrete, the rate of early strength gain is often reduced, but over time, the strength of the concrete is improved. However, if the water requirement of a concrete mix is increased by the addition of a pozzolan, an increase in drying shrinkage and creep should be expected (Mindess and Young 1981).

There are many different materials that can function as pozzolans. Some occur naturally, such as volcanic ashes, pumicite, opaline cherts, clays, and shales. Natural pozzolans often need to be ground and clays and shales also need to be calcined for their pozzolanic properties to be maximized. Other pozzolans are synthetic, often industrial waste products, such as silica fume, fly ash, and quenched boiler slag (Mindess and Young 1981).

**Silica Fume —** Silica fume is a manufacturing by-product of silicon alloy production in a submerged arc electric furnace (Berke et al. 1988). Most silica fumes consist of at least 85% amorphous silicon dioxide (Luther 1988). Silica fume is available as a powder, densified powder, or liquid slurry (Sherman et al. 1993). Silica fume is a highly effective pozzolan because of its high silica content, fine particle size, about 0.1 μm (3.9 x $10^{-6}$ in.), and large surface area. When silica fume is mixed into concrete, it reacts with excess calcium hydroxide to form calcium silicate hydrate binder, which causes the material to be stronger and less permeable. The small size of silica fume particles allows them to fill in voids in the cement paste and between the paste and the aggregate. As a result, concrete made with silica fume has a less permeable microstructure with fewer gaps or large calcium hydroxide crystals (Berke et al. 1988).

Silica fume was first used in bridge decks in Scandinavia, beginning in the 1970s. The first use of silica fume concrete on a bridge deck in the United States was as an overlay in Ohio in 1984 (Luther 1988). By the early 1990s, the use of silica fume in the construction of concrete bridge decks had been accepted as a way to improve durability (Luther 1993). The first silica fume concrete in bridge decks in the United States contained about 15% silica fume by mass of cement. This relatively high percentage of silica fume made the concrete expensive and difficult to mix and finish. Later studies have determined that the optimal percentage of silica fume for bridge decks is between 6 and 8% by mass of cement (Detwiler et al. 1999).

According to Luther (1988), there are four reasons to use silica fume concrete: for protection of steel reinforcement from ingress of chloride ions, for high ultimate strength, for abrasion resistance, and for improved bond strength. When combined with a low water/cementitious material (w/cm) ratio (0.32 to 0.37), however, placement of silica fume concrete requires extra care because the concrete is sticky and suffers from loss of slump, air, and bleed water. Silica fume concretes are susceptible to cracking prior to setting due to plastic shrinkage if they are not cured properly (Gjørv 1995). Proper curing will prevent the concrete surface from drying out too quickly. The Kansas Standard Specifications for silica fume overlays require the application of a liquid membrane forming curing compound immediately behind the tining float. Fogging is generally required throughout placing and finishing until wet burlap can be applied to the surface. After the burlap is placed, soaker hoses or occasional spraying is required to keep the burlap wet for the entire 7 day curing period (Special Provisions 90M-158-R8). Practical concrete mixes containing silica fume for cast-in-place field construction generally use w/cm ratios of about 0.40 to 0.45 (Sherman et al. 1996). Silica fume concrete mixes for bridge deck overlays in Kansas have w/cm ratios of 0.38 to 0.40 (Miller and Darwin 2000).

In 1995, the Precast/Prestressed Concrete Institute funded a study (Sherman et al. 1996) with the main purpose of determining the rate of chloride ingress for a range of concretes both with and without silica fume. Tests were conducted in accordance with AASHTO T 259, except for increasing the 90-day salt water ponding period for the specimens to 365 days. AASHTO T 277 / ASTM C 1202 “rapid chloride permeability,” or “coulomb” tests, which have been used exclusively by many studies, were also evaluated.

Concrete was mixed containing 0%, 5%, and 7.5% silica fume, by weight of cement, for w/cm ratios of 0.32, 0.37, and 0.46. Three 300 x 300 x 125 mm (12 x 12 x 5 in.) slabs were cast for each batch of concrete. The slabs were cured by one of three different methods: heat curing, tank curing in lime-saturated water, or burlap-curing. All of the silica fume samples were burlap cured.

AASHTO T 277 / ASTM C 1202 tests were performed 42 days after the specimens were cast. Two 100 mm (4 in.) diameter cores were taken from the center section of an unponded test slab from each category. The top 50 mm (2 in.) of the cores were used for the "coulomb" testing, leaving the bottom 75 mm (3 in.) of the 125 mm (5 in.) cores to be used to determine the absorption and permeable void values for the specimens. The average coulomb values for the specimens ranged from 637 to 3410 coulombs passed during the 6-hour test. At a given w/cm ratio, the concrete containing silica fume exhibited the lowest coulomb values, while the heat cured concrete without silica fume exhibited the highest coulomb values. Most specifications that use the AASHTO T 277 test require a coulomb value of less than 1000 for a mix to be accepted.

The lowest values of absorption and permeable voids were measured in the heat cured concretes at all three w/cm levels. The permeable voids for the 5 and 7.5% silica fume concretes were 100 and 50% greater, respectively, than those of the heat cured specimens at each w/cm ratio. In fact, the silica fume concretes had the highest volume of permeable voids for each w/cm ratio. This was attributed to the higher susceptibility to cracking of silica fume concrete.

After 365 days of continuous ponding with a 3% sodium chloride solution, two 75 mm (3 in.) diameter cores were taken from the two remaining slabs for each concrete mixture. These cores were sliced into depth increments of 13 to 22 mm ($^1/_2$ to $^7/_8$ in.), 25 to 35 mm (1 to 1$^3/_8$ in.), and 38 to 48 mm (1$^1/_2$ to 1$^7/_8$ in.) and tested for chloride content. A 100 mm (4 in.) diameter core was taken to determine the chloride content in the top 6 mm ($^1/_4$ in.).

No measurable chloride was found 38 to 48 mm (1½ to 1$^7/_8$ in.) below the surface for any of the samples, and no measurable chloride was found 25 to 35 mm (1 to 1$^3/_8$ in.) below the surface for the samples taken from the silica fume concrete. The chloride contents of the silica fume concretes were only about 20% as high as those of the conventional concrete mixes at the 13 to 22 mm (½ to $^7/_8$ in.) depth interval, but at the 0 to 6 mm (0 to ¼ in.) depth, the chloride contents in the silica fume mix with the highest w/cm ratio was slightly higher that that of the conventional mixes. This higher chloride content near the surface of the silica fume concrete with a w/cm ratio of 0.46 was attributed to higher absorption and the higher volume of permeable voids in that concrete. A chloride diffusion coefficient was calculated for each mix using a least-squares curve fitting technique.

The time-to-corrosion was calculated for each concrete mix using Fick's law [Eq. (1-6)], surface chloride concentrations, and diffusion coefficients. Assuming burlap curing, a 50 mm (2 in.) cover, and a w/cm ratio of 0.37, the time to corrosion for concrete containing 0, 5, and 7.5% silica fume by mass of cement would be 6, 11, and 11 years, respectively, based on a corrosion threshold of 0.022% acid-soluble chloride ion by concrete mass [about 0.59 kg/m$^3$ (1 lb/yd$^3$)]. Table 3.1 shows the estimated time-to-corrosion for specimens with different w/cm ratios and silica fume contents tested under continuous ponding. These times are considered to be conservative for field applications in which chloride exposure is intermittent.

When the results of the chloride permeability, AASHTO T 259 and coulomb, AASHTO T 277, tests were compared, it was clear that, despite a 1200 coulomb difference in the rapid chloride permeability test, the heat-cured conventional concrete with a w/c ratio of 0.37 and the burlap-cured silica fume concrete with a w/cm ratio of 0.46, (2800 and 1600 coulombs, respectively), had essentially the same diffusion coefficient and time-to-corrosion. In fact, only the mixes with silica fume and w/cm ratios of 0.32 or 0.37 would have passed the 1000 coulomb maximum criteria that is used for acceptance in some project specifications.

**Table 3.1  Estimated Time-to-Corrosion for Continuously Ponded Specimens** (Adapted from Sherman et al. 1996**)**

| Cure Type — Percent Silica Fume | Water/Cementitious Material Ratio | Time to Corrosion (years) | |
|---|---|---|---|
| | | 50 mm (2 in.) cover | 75 mm (3 in.) cover |
| Heat — 0 | 0.46 | 2 | 4 |
| Tank — 0 | 0.46 | 1 | 2 |
| Burlap — 0 | 0.46 | 1 | 3 |
| Burlap — 5 | 0.46 | 5 | 12 |
| Burlap — 7.5 | 0.46 | 6 | 13 |
| Heat — 0 | 0.37 | 5 | 11 |
| Tank — 0 | 0.37 | 4 | 9 |
| Burlap — 0 | 0.37 | 6 | 14 |
| Burlap — 5 | 0.37 | 11 | 25 |
| Burlap — 7.5 | 0.37 | 11 | 25 |
| Heat — 0 | 0.32 | 9 | 20 |
| Tank — 0 | 0.32 | 11 | 26 |
| Burlap — 0 | 0.32 | 9 | 20 |
| Burlap — 5 | 0.32 | 12 | 28 |
| Burlap — 7.5 | 0.32 | 17 | 38 |

The study concluded that an incorrect assumption; that the electric charge, in coulombs, passed through a concrete specimen is directly related to the long term diffusion properties of that concrete; is often made when specifying the AASHTO T 277 / ASTM C 1202 test.  According to the authors, the "coulomb" test was never intended to determine the amount of chloride that would penetrate into a given concrete unless proper correlations were made with long-term ponding tests for that specific concrete mix.   Review of the AASHTO T 277 test results determined that the coulomb value of conventional concrete could decrease by 5 to 10 times when 7%

silica fume was added, but the reduction in chloride ingress after the 90-day ponding test was only about 2 times.

The study also concluded that the dominant factor in decreasing concrete permeability is a low w/cm ratio. All of the concrete mixes with a 0.32 w/cm ratio had very low chloride concentrations at all depths, except for the surface. The first part of the study determined that, despite having coulomb values of 1000 to 5000, essentially impermeable concrete could be produced using conventional mixes with w/c ratios between 0.30 and 0.32.

A study by Detwiler et al. (1999) took a statistical approach to establish the ingress of chloride ions into silica fume concrete. Silica fume concrete mixtures were prepared with 0 to 12% silica fume by mass of cement, and w/cm ratios ranging from 0.30 to 0.45. Two types of cement, Type I/II portland cement, and Type K shrinkage-compensating cement were used. Three slabs, measuring 305 x 305 x 75 mm (12 x 12 x 3 in.) were cast for each mixture. After curing, slabs were ponded with a 3% sodium chloride solution in accordance with AASHTO T 259, except that the 90-day ponding period was extended to 180 days. After the ponding, a 100 mm (4 in.) diameter core was taken from the center of each slab. The cores were milled in approximately 1 mm (0.04 in.) increments, and the chloride content of each section was measured. The chloride concentration versus distance from the surface was plotted and a least-squares fitting method was used to determine an apparent diffusion coefficient according to Fick’s Second Law.

The results showed that the concrete containing silica fume was much more resistant to the ingress of chloride ions than the concrete without silica fume, but that there was not much difference in the chloride ion concentration between concretes containing 6% and 12% silica fume. The results also showed a decrease in chloride ion concentration with a decrease in w/cm ratio, although the decrease was not as significant as the decrease resulting from a silica fume addition. The study concluded that the greatest benefit came with adding between 6 and 8% silica fume by mass of the cement to the concrete. In this range, the apparent diffusion coefficient of the silica fume concrete was reduced by three times compared to that of conventional concrete. The decrease in the apparent diffusion coefficient that occurs with a reduction in w/cm ratio is almost linear, but is less than the decrease as the silica fume content is increased. For the mixtures with 6% silica fume, the apparent diffusion coefficients were about three times lower at a w/cm ratio of 0.30 than they were at a w/cm ratio of 0.40.

The study found no difference in the apparent diffusion coefficients of concrete using silica fume slurry versus concrete using the same amount of silica fume in dry densified form. The concrete in the study was well mixed so that the silica fume was uniformly distributed, but this may not always be the case in the field. There was also no significant difference in the apparent diffusion coefficients when Type K cement was used.

Field applications have consistently indicated that silica fume concretes can be effectively placed on bridge decks as long as sufficient care is taken throughout the process. To be effective, the silica fume must be evenly distributed throughout the concrete mix. When mixed in the truck, the volume of mixed concrete should not exceed 63% of the volume of the drum, as per ASTM C 94. For high dosages of silica fume, it is usually necessary to add a high-range water-reducer (HRWR) at the batch plant to ensure that the concrete is mixed adequately. If the HRWR is added at the batch plant, the slump of the concrete will generally have decreased by the time the concrete is at the job site, and sometimes requires the admixture to be added again (Holland 1988).

The biggest difference between silica fume concrete and conventional concrete becomes apparent during finishing. No significant differences appear for silica fume contents up to about 5%, but as the silica fume content increases, bleed water decreases, and eventually vanishes (Holland 1988). Because of the lack of bleeding, water loss from the surface cannot be readily replaced. Before the concrete has hardened, when it is in a plastic state, rapid surface drying can lead to plastic shrinkage cracks, the most common problem with silica fume concrete, because the tensile strength of the concrete is low (Whiting and Detwiler 1998). Methods to reduce plastic shrinkage cracking include minimizing delays, beginning curing immediately after finishing (not allowing the surface to dry out), minimizing the evaporation rate (evening or cool weather placements and fog spraying during placement), and using polypropylene fibers (Luther 1988). Seven days of continuous wet burlap curing has been effective at preventing shrinkage cracking in many situations (Luther 1988, Whiting and Detwiler 1998). Air-entrainment is also necessary to ensure adequate freeze-thaw durability (Luther 1988).

Laboratory studies have shown that a suitable reduction in the permeability of chloride ions can be achieved with a substitution of as little as 5% silica fume by mass of cement. This reduction in permeability can be further enhanced by either lowering the w/cm ratio or raising the percentage of silica fume in the mix (Sherman et al. 1996, Detwiler et al. 1999). In field applications, the percentage of silica fume may need to be more than 5%, depending on the surrounding environment and the amount of deicing chemicals that are used (Ozyildrim 1992).

A study by Miller and Darwin (2000) at the University of Kansas compared the performance of concrete bridge decks to that of concrete bridge decks with silica fume overlays in Kansas. Field surveys were conducted to determine crack density and to take cores for chloride content analysis and rapid chloride permeability testing. The study found that, although the decks with silica fume overlays had much lower coulomb values than the conventional concrete decks in the rapid chloride permeability test, the effective diffusion coefficients for the two different types of decks were similar at ages between 500 and 1500 days. The effective diffusion coefficient of the silica fume concrete increased slightly as the air content increased. Information on crack densities is given in Section 3.3.3.

**Blast-Furnace Slag —** Blast-furnace slag is a by-product of the iron and steel industry. It is composed of lime, silica, and alumina, with smaller amounts of magnesia, alkali oxides, and iron oxides. The composition of slag is dependent upon the industrial process and the raw materials used, but must be at least 40% lime to be used as cement. To be effective as a cement, slag must be cooled rapidly because its physical structure depends on the rate of cooling. Slag is usually quenched, which forms a reactive glass. Slag will not hydrate unless it is activated by the addition of other compounds, generally calcium hydroxide or calcium sulfate, which is most easily supplied by the hydration of portland cement. ASTM C 595 for Type IS cement allows 25 to 65% slag to be blended with portland cement (Mindess and Young 1981).

Studies have shown that blast furnace slag can reduce the permeability of concrete (see next section), but no information was reviewed on the corrosion effects of blast furnace slag in concrete.

**Fly Ash —** Fly ash is a byproduct of powdered coal burning. It is an inorganic residue that is trapped by electrostatic precipitators, mostly in coal-fueled power stations. Fly ash is a good pozzolan because it is already finely divided and, unlike most natural pozzolans, it is commonly in the form of tiny spheres, meaning that it can increase the workability of concrete without increasing the amount of water needed. Another reason that fly ash is popular is that it is readily available, especially near population centers, where a significant percentage concrete construction takes place (Mindess and Young 1981).

The results of studies on mixes containing fly ash have been mixed. In some studies, where chlorides were admixed into the concrete, there was an increase in corrosion in the fly ash concrete specimens compared to conventional concrete specimens (Haque and Kawamura 1992). Studies that applied the chlorides externally have found fly ash specimens to perform better than control specimens (Corrosion 1994, Swamy and Laiw 1995).

A study comparing the chloride resistance of concretes containing fly ash, silica fume, and ground granulated blastfurnace slag was performed by Swamy and Laiw (1995). Mixes were designed with 65% slag, 30% fly ash, or 10% silica fume cement replacement, by mass. All mixes had w/cm ratios of 0.60. Large slabs, measuring 1000 x 500 x 150 mm (40 x 20 x 6 in.), and reinforced with high tensile strength steel, were made using each mix. This w/cm ratio is higher than the recommended value for use in chloride exposure concretes, but was used to highlight the benefits of pozzolans, even at a high w/cm ratios, and also to simulate worst case field applications.

Both the fly ash and the ground granulated blast-furnace slag increased the workability and slump of the mixtures, while the silica fume reduced the slump by about 70% compared to the portland cement concrete. The slabs were ponded with 4% sodium chloride solution in cycles that consisted of 7 days of ponding, followed by 3 days of drying. Chloride concentrations were measured at depths of 5-25, 25-45, 45-65, 65-85, and 85-105 mm (0.2-1, 1-1.8, 1.8-2.6, 2.6-3.3, 3.3-4.1 in.) after 10, 20, and 50 exposure cycles. Table 3.2 gives the measured chloride concentrations as a percent of the cement weight. The threshold for corrosion is about 0.20% chlorides by weight of cement (about 0.75 kg of chlorides per $m^3$ of concrete). The mix containing silica fume had the lowest chloride ion concentrations at the depths tested, followed by the slag mix, the fly ash mix, and then the control.

**Table 3.2 Chloride Concentrations (% of cement weight) in Slabs After 50 Exposure Cycles** (Adapted from Swamy and Laiw 1995)

| | Chloride concentration Depth from concrete surface (mm) | | | | |
|---|---|---|---|---|---|
| Admixture | 5-25 | 25-45 | 45-65 | 65-85 | 85-105 |
| Control | 4.52 | 2.32 | 1.39 | 0.87 | 0.47 |
| Slag (65%) | 2.58 | 1.05 | 0.59 | 0.21 | 0.08 |
| Fly Ash (30%) | 5.02 | 1.41 | 0.81 | 0.32 | 0.17 |
| Silica Fume (10%) | 2.05 | 0.20 | 0.10 | 0.00 | 0.00 |

### 3.2.4 Summary

The studies described in this section have identified a number of ways to increase the quality of concrete for bridge structures. These are some of the reoccurring conclusions:

- A minimum of 25 mm (1 ½ in.) concrete cover over the reinforcement should be used on all bridge structures.
- The w/cm ratio of concrete to be used in reinforced concrete bridges should be kept below 0.40. The w/cm ratio in Kansas bridges ranges from 0.40 to 0.44 on subdecks and from 0.36 to 0.40 in overlays.
- Fly ash and blast furnace slag can reduce the permeability of concrete to chloride ions, but do not do so as effectively as silica fume.
- Silica fume can reduce the permeability of a concrete mix to chloride ions; however, the workability of the mix will be reduced, requiring a high-range water-reducer, and the use of proper construction and curing techniques is important to minimize plastic shrinkage cracking.
- The optimal amount of silica fume in silica fume concrete for bridge decks is 6 to 8% by weight of cement.

## 3.3 OVERLAYS

The purpose of concrete overlays is to create a low permeability protective layer over the conventional concrete on bridge decks. An overlay serves as a barrier to chloride ions, therefore increasing the time required for the concentration of chloride ions at the level of the reinforcement to reach the threshold for corrosion. Low permeability overlays also decrease water penetration into a structure, allowing it to dry out, which reduces chloride ion mobility. Overlays can be applied to new decks, or as a rehabilitation method on existing decks. However, overlays are not as effective when applied to existing decks because if chloride ions are already present in the deck when the overlay is placed, then the only protection that the overlay can offer is a decrease in moisture infiltration (Sherman et al. 1993).

The most common type of overlay is a low-slump dense concrete overlay, which has been effective in extending the service lives of damaged bridge decks in many states. Silica-fume concrete and latex-modified concrete overlays have also been successful in extending the service lives of contaminated structures, but are more expensive than the low-slump overlays. Silica-fume and low-slump overlays are used in new construction. Polymer concrete overlays are generally used as a temporary repair method on damaged bridge decks.

### 3.3.1 Latex-Modified Concrete Overlays

Latex-modified concrete overlays consist of a conventional portland cement concrete supplemented by a polymeric latex emulsion, usually styrene-butadine latex. The water in the emulsion hydrates the cement, while the polymer provides additional binding properties. This produces a low w/cm ratio concrete with good durability and bonding characteristics and a low permeability to chloride ions. Because of high material costs and good performance, latex-modified concrete overlays are typically either 40 or 55 mm (1.5 or 2 in.) thick, thinner than most overlays (ACI 222 1996).

Although they are not waterproof, the permeability of latex-modified concrete overlays has been found to be only about 12% that of conventional concrete. Latex-modified concrete overlays consistently outperformed conventional and low slump dense concrete overlays in 16-year tests in Virginia. Although they did not completely stop corrosion, latex-modified overlays extended the service lives of the bridge decks in the study (Sprinkle 1992a). Latex-modified overlays can be expected to last up to 25 years, after which they usually need to be replaced due to rutting and general wear (Babaei and Hawkins 1990). Latex-modified overlays have had problems with plastic shrinkage cracks that deepen with age (Sprinkle 1992a) and with scaling in continuously saturated areas (Babaei and Hawkins 1990).

### 3.3.2 Low-Slump Dense Concrete Overlays

The concept of dense concrete overlays is basically that the natural capillary void system of a concrete can be reduced by increasing the cement content of a mix to over 470 kg/m$^3$ (800 lb/yd$^3$) and decreasing the w/c ratio to about 0.30 to 0.32. This leads to a decrease in permeability and an increase in strength. Dense concrete overlays work by preventing chloride ions and moisture from penetrating into the underlying bridge deck (Sherman et al. 1993).

The main problems with low slump dense concrete overlays is that they are difficult to place, expensive, and have trouble with surface cracking. Also, the skid resistance of low slump dense concrete overlays is less than that of latex-modified concrete overlays. Some of the construction problems can be overcome with the use of a high-range water-reducing admixture (superplasticizer), but overworking or the addition of water in the field will reduce the performance of the overlay. Proper curing is important to prevent cracking (Babaei and Hawkins 1990).

Low-slump dense concrete overlays were first placed in the early 1960s in Iowa and Kansas, and are often referred to as “Iowa low-slump overlays”. When bridge decks in Iowa need rehabilitation, the standard has been to apply a 50 mm (2 in.) overlay of dense, low-slump concrete on top. The goal is 50 mm (2 in.) total of cover above the steel. In places where the concrete above the reinforcement is sound, it is left in place, but in areas where the steel is exposed, the concrete is removed and replaced below the bars. For bond, Iowa requires a water cement grout (sometimes it also contains sand) that is sprayed onto the concrete immediately before the concrete hits the deck. The grout must be applied to a dry deck (this is thought to help it to be absorbed into the dry concrete) and must still be wet when the concrete is placed. This method has worked well, and most of the overlaid decks are still in service. The first low-slump overlay in Iowa lasted for 23 years before it had to be replaced. Later overlays are still in place, after more than 25 years. During 1999, over 37,300 m$^2$ (44,600 yd$^2$) of dense concrete overlays were placed on bridge decks in Iowa. Silica fume overlays have also been used in Iowa, but the traditional low-slump overlays have been performing well and are cheaper, so silica fume was not determined to be cost effective for use in Iowa (Grove 1999).

In 1985, surface defects, delaminations, half-cell potentials, and chloride concentrations were examined on each of 50 randomly selected bridges in New York State on which low-slump concrete overlays had been placed as a rehabilitation method during the period of 1979 to 1981 (Chamberlin 1988). The process of installing the overlays was basically the standard at the time. Spalled and delaminated concrete was removed deep enough to expose sound concrete, and to at least 25 mm (1 in.) below any exposed reinforcement. Concrete was also removed to at least 25 mm (1 in.) below the reinforcement in any areas where the measured half-cell potential was more negative than –350 mV versus a copper-copper sulfate electrode. All other concrete surfaces were scarified to a depth of 7 to 13 mm (¼ to ½ in.), and all exposed steel reinforcement and concrete was sandblasted to remove rust and to eliminate microfractures. The excavated areas were filled to the level of the scarified deck with either conventional concrete or low-slump concrete and cured. Finally, the entire deck was overlaid with a minimum of 50 mm (2 in.) low-slump concrete bonded with portland cement mortar grout, compacted with an oscillating screed, and cured with wet burlap for at least 72 hours.

At the time of inspection, about 5 years after the overlays were placed, physical damage that could potentially have been caused by corrosion, delaminations, spalls, and patches, was found on 30 (60%) of the spans and effected 0.84% of the total deck area in the study. All of the spalling, and almost half of the damage was around joints, which did not reflect the integrity of the overlays according to the author. Damage of concern, that which occurred on the interiors of the slabs, was found on 10 (20%) of the study spans and affected 0.39% of the total study area. All of this damage was in the form of delaminations, and all of it occurred in areas of the decks that had been scarified. When chloride concentrations in the overlays were examined, they were found to be the same as would be expected in conventional concrete. The service lives of these overlays was estimated to be 25 years when about half of the surface of the deck was excavated to beneath the reinforcement before the overlay was placed.

In 1991, 305 bridge decks, 152 with latex-modified concrete overlays and 153 with low-slump dense concrete overlays, were evaluated for a Strategic Highway Research Project (Weyers et al. 1991). The service lives of the overlays were estimated by looking at historical data and evaluations of the condition of the overlays, which ranged in age from 1 to 20 years at the time of evaluation. Performance of the overlays was measured according to the percent of the deck area that was damaged by delamination or spalling.

The results of the study indicated that the performance of the overlays was less dependant on the type of overlay than on the methods used to prepare the deck. Both overlay types performed best when concrete was removed from areas that had half-cell potential measurements more negative than –350 mV versus a copper-copper sulfate electrode (rather than only from damaged areas), when concrete was removed to below the reinforcement, and when the exposed surface was sandblasted to remove microcracks. Chamberlin and Weyers (1994) estimated that when these procedures are followed, both latex-modified and low-slump dense concrete overlays have service life potentials of 30 to 50 years, assuming that the end of service life occurs when 40% of the total deck area is damaged.

### 3.3.3 Silica Fume Concrete Overlays

Studies have indicated that silica fume can reduce the ingress of chloride ions into concrete that is exposed to salt-water (Swamy and Laiw 1995, Sherman et al. 1996, Detwiler et al. 1999). While silica fume concrete is sometimes used for an entire bridge deck, silica fume concrete overlays are often used as corrosion protection measures on bridge decks. Specific information about silica fume concrete is given in the earlier section on silica fume.

Three silica fume concrete overlays were placed on bridge decks in Virginia as part of a study to evaluate the characteristics of silica fume concrete as thin overlays (thickness not reported) for corrosion protection (Ozyildrim 1992). In this study, the Virginia Department of Transportation was attempting to determine the minimum amount of silica fume needed to reduce the permeability of the decks to chloride ions to a low or very low level.

The first overlay for the study was placed on a four-span bridge with moderate traffic volume in 1987 (Ozyildrim 1992). Concrete containing 7% and 10% cement replacement by silica fume, added in slurry form at the batch plant, was used. HRWR was added at the plant in all seven truckloads, and again at the job site to two of the trucks. The concrete deck was sandblasted, wetted, and covered to keep it in the saturated condition on the day before the overlay was to be placed. During placement, the deck was kept saturated, but clear of free standing water. The silica fume concrete was deposited on the deck and leveled with a vibratory roller screed. When the deck was inspected two weeks after the bridge had reopened; one visible crack and a number of delaminations were found. Over the next four years, the cracks increased both in number and in length.

The installations of the next two overlays were similar to the first, except the concrete was misted after screeding and covered with wet burlap and plastic for four days followed by the application of a curing compound. These decks did not have any plastic shrinkage cracks after 2 and 6 months of service. The study concluded that silica fume concrete could be used effectively in thin overlays for bridge decks, but that plastic shrinkage was a concern and proper placing and curing procedures needed to be followed.

A study by Miller and Darwin (2000) at the University of Kansas compared the performance of silica fume overlays with that of conventional concrete overlays in Kansas. While the age of the silica fume overlays was generally less than that of the conventional overlays, the few decks that were in the same age range had similar crack densities and chloride contents. If this small sample was an accurate representation of all of the bridge decks in Kansas, then the results indicate that the use of silica fume in bridge deck overlays provides no advantage over conventional concrete overlays. Therefore, the extra cost and construction requirements of silica fume overlays may not be justified.

The chloride contents of samples taken from decks with silica fume overlays and conventional concrete overlays were similar for decks in the same age rage. Chloride contents of samples taken from crack locations at depths just above and just below the reinforcement [66.7 mm (2.625 in.) and 85.7 mm (3.375 in.), respectively], were above the corrosion threshold in as little as 1000 days (2.7 years) for all bridge types.

In general, crack densities for conventional and silica fume overlays decks were similar. The highest crack densities in decks with silica fume overlays occurred when the slump was greater than 90 mm (3.5 in.), while the highest crack density in the decks with conventional overlays occurred when concrete with zero slump was used. For decks with silica fume overlays, the crack density decreased as the relative humidity decreased and with the use of fogging and precure materials during and after finishing, and the crack density for both types of overlays increased as the air temperature range for the date of placement increased.

### 3.3.4 Polymer Concrete Overlays

Polymer concrete is concrete in which portland cement is replaced by a polymer. Because they are nearly or completely waterproof when they are uncracked, polymer concrete overlays are used to protect bridges from the ingress of water and chloride ions. The use of polymer concrete overlays is generally limited to the rehabilitation of corroding bridge decks because they are relatively expensive and difficult to place. However, they are occasionally used as a preventative measure on sound decks in which corrosion is anticipated. These overlays are generally less than about 12 mm (0.5 in.) thick and are seeded with fine aggregate or sand to improve friction properties. Polymer concrete with larger aggregate is often used for repairs because of its high early strength and short set time (Sherman et al. 1993).

The performance of a polymer concrete overlay is dependent on the strength of the bond between the overlay and the concrete underneath, which is dependent on surface preparation, cleanliness, and field application techniques. Most failures of polymer concrete overlays are attributed to workmanship or improper handling of materials (Sherman et al. 1993).

The Missouri Department of Transportation began using epoxy-polymer overlays to rehabilitate bridge decks in 1990. These overlays are cost-effective, and they protect the deck for 10-15 years by keeping water out. The overlays consist of a thin two-part epoxy with aggregate filler placed at a minimum thickness of 6 mm (¼ in.) (Wenzlick 1999).

A number of different materials for polymer concrete overlays were investigated in the late 1970s and early 1980s, but most have since been abandoned (Locke 2000).

## 3.4 WATERPROOF MEMBRANES

Waterproof membranes are installed on bridge decks for two reasons. The first is to protect the concrete deck slab from deterioration induced by freeze-thaw cycles, and the second is to protect the reinforcement from corrosion. Many types of waterproof membranes have been used on bridge decks since the problem of corrosion of reinforcement in concrete bridges was first recognized as a major problem in the 1960s. These membranes can be divided into three general categories, built-up membranes, preformed sheet membranes, and liquid membranes. Some agencies have had good experience with membranes as corrosion protection systems, while others have not and no longer use them (Manning 1995). For example, all of the bridges in Ontario are waterproofed using a hot rubberized asphalt (liquid) membrane. These membranes have been used in Ontario since the early 1970s and have worked extremely well. Most of the membranes are still in place (Manning 1999). Vermont has used sheet membrane systems as the standard corrosion protection method on its bridge decks since the early 1970s. In the late 1990s, Vermont began experimental use of other forms of waterproofing membranes, including torch applied sheet membranes and spray applied liquid membranes (Graham 2000). There are several bridges in Kansas that have membranes with asphalt overlays. Some have performed well and some have not (Meggers 1999). When used in Iowa, however, membranes did not perform well, and Iowa now has a policy of “nothing that’s not concrete” on bridge decks (Grove 1999).

### 3.4.1 Use of Membranes on Bridge Decks

The use of waterproof membranes started in the early 1960s to protect concrete that was overlaid with an asphalt-wearing surface. Some states use bituminous surfaces on bridge decks because they have higher skid resistance and provide a better riding quality surface than concrete; however, because asphalt is porous, water seeps through the wearing surface and causes deterioration of the concrete underneath. States that are opposed to the placement of bituminous overlays argue that the overlays do not perform well and that they hide deterioration in the concrete. Many attempts to waterproof decks using linseed oil or silane as sealers in the 1960s had been unsuccessful, leading to the development of built-up membranes (Manning 1995).

The most common of the early built-up systems consisted of two or more layers of glass fabric mopped with coal-tar pitch emulsion (NCHRP 1970). These built-up membranes are not used very much today, although they are still used in Illinois and permitted in Connecticut. The systems are generally too labor intensive and slow to construct to be cost effective. Oregon allows the use of a different kind of built-up membrane, which consists of a polypropylene fabric rolled into a hot-rubberized-asphalt membrane (Manning 1995).

Several states began using waterproof membranes in 1972, when the federal government introduced a policy that required bridge decks on the federal-aid system that were likely to be subjected to deicing salts to be protected against corrosion. However, data from surveys conducted between 1974 and 1994 show a constant decline in the number of state agencies that use waterproofing membranes in new construction. Table 3.3 summarizes the results from four different surveys conducted during this period.

**Table 3.3 Percentage of State Agencies Using Waterproof Membranes**
(Manning 1995)

| | For New Construction | | | | For Rehabilitation | | |
|---|---|---|---|---|---|---|---|
| Year of survey | 1974[1] | 1977[2] | 1986[3] | 1994 | 1977[1] | 1989[4] | 1994 |
| Membrane use | 74% | 69% | 53% | 25% | 58% | 51% | 46% |
| Standard use | — | 40% | 38% | — | 46% | 47% | — |
| Experimental use | — | 29% | 15% | — | 12% | 4% | — |
| No. of responses | 42 | 48 | 45 | 48 | 48 | 47 | 48 |

[1] From Van Til et al. (1976)
[2] From NCHRP (1979)
[3] From Babaei and Hawkins (1987)
[4] From Chamberlin, W.P. (1989).“Summary of the Field Survey Questionnaire,” unpublished report, June.

One reason for the decline in the number of agencies using waterproof membranes in new construction is that a number of alternative systems, such as epoxy-coated steel, are now available. The number of states using membranes in rehabilitation has remained relatively constant, at about 50%. This higher number reflects the fact that there are fewer options for protecting existing decks from corrosion damage than there are in new construction (Manning 1995).

A 1994 NCHRP survey of all 50 states in the United States and all 10 provinces in Canada, asking about membrane use, produced replies from 48 states and 6 provinces. All of the provinces responding to the survey report using membranes in both new construction and rehabilitation. The use of membranes within the United States is scattered. Membranes have been used in the Northeast for decades, but are not used in most Midwestern states for reasons that are not clearly explained by climate. Figures 3.1 and 3.2 show the states that reported using membranes on new decks and for rehabilitation, respectively (Manning 1995).

According to the results of the 1994 survey, the area of membrane that was installed in the United States during 1992 on new bridge decks and on existing decks for rehabilitation was 58,000 m$^2$ (620,000 ft$^2$) and 309,000 m$^2$ (3,300,000 ft$^2$), respectively (Manning 1995).

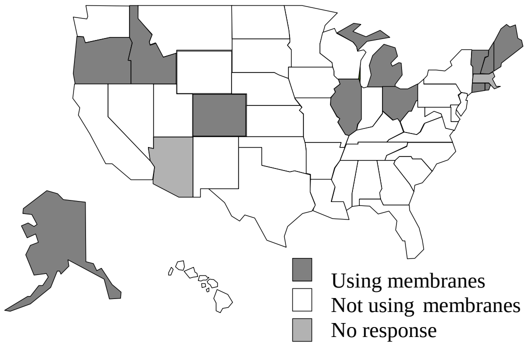


Figure 3.1 States using waterproofing membranes in new construction (Manning 1995)

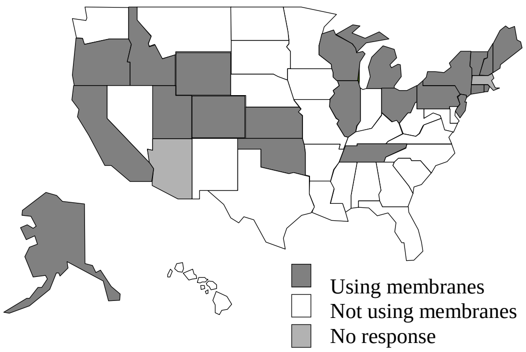


Figure 3.2 States using waterproofing membranes in rehabilitation (Manning 1995)

### 3.4.2 Membrane Types

Currently, there are three types of waterproofing membrane in use in North America: preformed sheets, liquid membranes, and built-up systems. Preformed sheets are used most often in the United States, while hot-applied rubberized asphalt, which is a liquid membrane, is used almost exclusively in Canada and is the most common liquid membrane used in North America. Responses to the 1994 survey indicated that 14 states were using preformed membranes, and 5 states were using both preformed and liquid membranes. Of the other 3 states, 1 was using built-up membranes exclusively, 1 allowed built-up membranes to be used as an alternative to preformed membranes, and the other allowed the use of any of the three systems (Manning 1995).

Each system has its own advantages and disadvantages. Preformed systems tend to perform better than liquid systems in laboratory evaluations. The quality and thickness of preformed sheets can be controlled at the manufacturing plant, while the consistent quality and thickness of liquid systems is more difficult to ensure during field installation. However, preformed systems can be labor intensive to install. Preformed systems require laps at each edge, and are difficult to install on curved or rough decks, while liquid systems can be applied in one application by spray or squeegee, without laps, and their application does not depend on the geometry of the deck. Preformed systems are more vulnerable to poor workmanship at critical locations, such as curves, expansion joints, and drains, and blisters have to be repaired by puncturing and patching the membrane. Liquid systems are less vulnerable to poor workmanship, and blisters and pinholes are easy to repair in self-sealing materials, but not in thermosetting materials. Liquid systems are generally less expensive than preformed systems (Kilareski 1977). Built-up systems are not often used because they are generally labor intensive and expensive (Manning 1995).

A more detailed classification system was developed as part of an investigation of waterproof membranes in the United Kingdom (Price 1989). In the system, preformed sheets are divided into four categories: asphalt-impregnated fabric sheets, polymeric sheets, elastomer sheets, and asphalt-laminated boards. Asphalt-impregnated sheets consist of a central core of absorbent material, either polyester fleece, glass cloth, or woven polypropylene, impregnated and coated with asphalt cement. Polymeric sheets are blends of various polymers, binders, plasticizers, and inert fillers. Elastomer sheets are made up of vulcanized butyl or polyisoprene rubber. Asphalt-laminated boards consist of a core of finely crushed aggregate saturated with asphalt-cement between layers on asphalt-saturated felt. These boards are also used as protection boards for some systems.

Liquid membranes are divided into two categories: bituminous and resinous, each consisting of one or two component solutions that are either moisture or chemically cured. Bituminous systems are either solutions or mastics, which require heat to be converted to a liquid for application. Resinous systems are urethanes, epoxies, or acrylic resin-based systems (Price 1989).

In 1970, NCHRP began a study of waterproof membranes (Van Til et al. 1976). This study was initiated to compare the effectiveness of 147 of the membrane systems available at the time. Five of the most promising systems were selected for detailed evaluation following laboratory testing. The systems consisted of preformed sheets, all of which required the application of an adhesive to attach the membrane to the deck. Three of the five sheets were vulcanized rubber. Although the preformed systems performed better than liquid membranes in the laboratory tests, Phase II of the study established that these systems were difficult and labor intensive to install. Blisters were common, the large sheets were difficult to handle in windy conditions, and the performance of the membranes was susceptible to the quality of workmanship (Van Til 1978). Today, preformed self-adhesive reinforced polymer sheets are used in an attempt to overcome the installation problems and high cost of the earlier preformed systems (Manning 1995).

### 3.4.3 Other Components of Waterproofing Systems

Whether a waterproofing membrane is a preformed sheet, a liquid, or part of a built-up system, it is not used by itself, but rather is part of a system consisting of several components that have to work together to be successful. The membrane needs to be bonded to the deck, usually by means of a primer, which is used to penetrate the deck and improve adhesion of the membrane to the concrete. Adhesives, which help to bond the membrane to the deck, are required with some sheet membrane systems (Manning 1995).

In Europe, ventilating layers, made up of either a thin [13 mm (0.5 in.)] lift of sand asphalt or a perforated sheet made of felt or other non-woven fabric, are sometimes used in an effort to dissipate vapor pressures and reduce blistering. Most agencies in North America do not recommend ventilating layers because they reduce the bond of the membrane to the deck (Manning 1995).

Protection board, placed above the membrane and consisting of either roofing felt or a sandwich of asphalt and mineral filler between layers of asphalt-impregnated felt, is used regularly in both Canada and Europe to prevent damage to the membrane, especially during construction. Protection board also helps to prevent the membrane from being punctured by aggregate particles under traffic loads. Protection board is often used in liquid membrane systems, but is usually not used with preformed membranes (Manning 1995).

Tack coats may be used to improve the bond between the membrane or protection board and the bituminous surface, which is the final protective layer in the system. Some agencies use only one layer of bituminous concrete over membranes, but most use a bottom layer as a base course on top of the membrane, and a top layer as a wearing course. It has been determined that when the right materials are used and installed correctly, the service life of membranes is often determined by the life of the bituminous surface, which is generally 15-20 years. Studies have shown that the thickness of the asphalt surface over a membrane has a significant impact on the performance of waterproofing systems, and that a minimum thickness of 75 mm (3 in.) should be specified (Manning 1995).

### 3.4.4 Requirements for Waterproofing Systems

There are a few simple requirements that an ideal waterproofing membrane system must meet. The system must be watertight after installation and remain watertight throughout the service life of the system, and the system must be economical. A more detailed list of requirements would include the following: during installation, the system must be tolerant of a rough or unclean surface and changes in temperature and humidity. The system must be easy to install, regardless of the geometry of the deck, and bond well to the deck, especially at the edges. The system must resist damage from loose particles, fuel spills, foot traffic, and dropped objects before the asphalt overlay is applied, and must be able to withstand paving equipment and asphalt application temperatures of up to 180° C (365° F) without damage. The membrane must also bond well to the asphalt overlay. In service, a waterproofing system must not be affected by changes in service temperature, ranging from -40 to 60° C (-40 to 140° F). The system must remain watertight and bonded to both the deck and the asphalt overlay throughout its service life, and must resist shear and puncture by aggregates under traffic loading. The system must bridge cracks in the deck slab and be unaffected by salt and water. It also should be possible to replace the asphalt overlay without having to replace the membrane (Manning 1995).

Many agencies that use membranes only use them for specific applications, such as on secondary route bridges. This implies that membranes have not performed well under heavy traffic loading. Membranes have also had problems on structures with steep grades, and at locations where heavy vehicles brake or turn at low speed (Manning 1995). Membranes do not work well on badly corroding decks, because they do not stop corrosion, and extensive patching is necessary. If the deck is rough, a leveling course of sand asphalt must be applied before the membrane can be applied (Manning and Bye 1984). Another reason that membranes are not used in some situations is that the asphalt overlay is not part of the structure in terms of strength, and adds a considerable dead load to the system. In other cases, waterproofing membranes with asphaltic overlays are used because they are easier to tie into the approach pavement than concrete overlays. Kansas and Michigan install membranes with asphalt overlays to restore a riding surface until a permanent concrete overlay can be placed (Manning 1995).

### 3.4.5 Service Lives of Membranes

Although tests have indicated that membranes may be able to provide up to 50 years of service before corrosion of reinforcement becomes a problem (Frascoia 1984), states responding to the 1994 NCHRP survey anticipated the service lives of their membrane systems to be between 10 and 30 years. Most of the longest anticipated service lives came from states in New England, where membranes have been used for the longest time and contractors have many years of experience at installing them (Manning 1995). Most agencies surveyed for the 1994 NCHRP Synthesis (Manning 1995) indicated the same anticipated service life of membranes installed on new decks as membranes installed to rehabilitate an existing deck. The fact that most of the estimates were the same indicates that the service life of a membrane is determined by the asphalt overlay rather than the membrane itself. Bituminous pavements are usually resurfaced every 15 to 20 years, and bridge repaving is often included in the contract. This is especially true on smaller bridges and in rural areas where there is often no alternative and paving equipment will not be in the area again for 15 to 20 years. Unless the waterproofing system can be expected to perform properly until the next repaving contract, another 15 to 20 years down the line, most agencies will make any necessary repairs to the bridge deck, including replacing the membrane, while the contractor and equipment are on site (Manning 1995). Ontario’s experience has generally been that the standard hot rubberized asphalt membranes do not need to be replaced until the second repaving, 30-40 years after their application. At the first repaving, the top 40 mm of asphalt is removed and replaced (Manning 2000).

### 3.4.6 Problems with Membranes

Some highway departments have had trouble with debonding of membranes and stripping of asphalt overlays, requiring the removal and replacement of the membrane in ten years or less, depending on both the traffic and the environment. Other membranes deteriorate after about 15 years of service due to traffic stresses and age embrittlement. One of the major causes of debonding and stripping of the asphalt overlay is water that is trapped on top of the membrane. Freezing and thawing, along with pressure from traffic weaken the bottom part of the asphalt overlay and the bond between the overlay and the membrane (Khossrow and Hawkins 1988). To prevent this, proper drainage should be provided so that water can drain quickly from the deck, and seepage drains should be provided at low points to prevent water from sitting on top of the membrane (Manning 1995). Blisters in the membrane can also be responsible for these problems. Some departments have also experienced problems with poor wear resistance of asphalt concrete overlays (Khossrow and Hawkins 1988).

The properties of the asphalt overlay are important to prevent deterioration and stripping. A high-density overlay with quality aggregate is important, as well as proper seams, compaction, bonding techniques and adequate drainage (Khossrow and Hawkins 1988). The dead load from an overlay of 50 to 75 mm (2 to 3 in.) may be effective in preventing blisters (NCHRP 1979). Most agencies in North America apply asphalt overlays that are 65 to 80 mm (2.5 to 3.5 in) thick. These overlays are applied in two lifts (Manning 1995).

Techniques that have been used to prevent blistering include applying the membrane when the deck temperature is higher than the ambient air temperature, such as in the evening or at night, sealing the concrete deck with silane prior to applying the membrane, and minimizing the time between membrane and asphalt placement. In most cases, using a minimum 50 mm (2 in.) asphalt overlay on top of the membrane has been sufficient to keep blisters from forming after the overlay has been placed. Studies have also shown that protection board is effective in reducing damage to membranes during placement and compaction of the asphalt surface and under traffic loads. Only a few states in the United States require the use of protection board, but it is widely used in Europe and in Canada. It is also recommended that liquid membranes be more than 2 mm (0.08 in.) thick, and preformed membranes be more than 2.5 mm (0.10 in.) thick (Manning 1995).

### 3.4.7 Field Studies

Field experience with waterproof membranes in the United States has ranged from satisfactory performance to dramatic failure. Most studies have included at least one deck on which the waterproofing system did not perform well (Manning 1995). This section discusses the results of various field studies.

**Vermont —** Beginning in 1971, 33 different waterproofing systems were placed on 69 new bridge decks in Vermont and were evaluated in 1983 after 5 to 12 years in service. The systems tested included 15 preformed sheet systems, 7 epoxies, 4 thermoplastics, 4 polyurethanes, and 2 tar emulsion systems. The preformed systems included the 5 systems selected for the field study described in the NCHRP report by Van Til et al. (1976). The performance of the membranes was evaluated based on the chloride content of the concrete from cores taken 0.3, 1.5, and 4.5 m (1, 5, and 15 ft) from the curb line. The concrete was considered to be contaminated when the chloride ion content was more than 50 ppm [0.12 kg/m$^3$ (0.2 lb/yd$^3$)] above the chloride content measured immediately after construction. This does not mean that corrosion will be initiated, the corrosion threshold is between 0.6 and 0.9 kg/m$^3$ (1 and 1.3 lb/yd$^3$), but it does mean that the membrane is not entirely effective at preventing chloride ions from entering the concrete. The decks were subjected to 75 to 115 freeze thaw cycles per year, 1.8-3.5 m (70-140 in) of snowfall, and a maximum salt application of 15 kg/m$^2$ (3 lb/ft$^2$) per year (Frascoia 1984).

Direct comparison of the different methods is difficult because they had all been in service for different amounts of time; however, some of the best performances came from systems that had been in place for the longest time. After 11 years of exposure, all of the samples taken from two bridges that had been waterproofed with rubberized asphalt membranes were still free of chloride ion contamination at the 25-50 mm depth. Only 9% of the samples were contaminated at the 0-25 mm depth, with an average chloride concentration of the contaminated samples of 73 ppm [0.17 kg/m$^3$ (0.29 lb/yd$^3$)].

After 8 years of exposure, all samples taken from 1 deck with a 1.75 mm (69 mils), 100% solids polyurethane system and 1 deck with a 2.3 mm (90 mils), 100% solids epoxy system were free of chloride contamination at both the 0-25 mm and the 25-50 mm levels. The other 7 decks with epoxy systems had high levels of chloride contamination at both levels, as did a tar modified polyurethane system.

As a group, the standard preformed sheet membranes provided the best overall performance, with an average chloride ion concentration in the 0 to 25 mm (0 to 1 in.) zone of 0.20 kg/m$^3$ (0.33 lb/yd$^3$), and only 7% of the cores were contaminated in the

25 to 50 mm (1 to 2 in.) zone. Polyurethane systems had the lowest chloride ion concentrations in the 0 to 25 mm (0 to 1 in.) zone, 0.076 kg/m$^3$ (0.13 lb/yd$^3$) above the base level [0.11 kg/m$^3$ (0.18 lb/yd$^3$)], and only 8% of the cores were contaminated in the 25 to 50 mm (1 to 2 in.) zone. The tar emulsion systems exhibited the worst overall performance, with average chloride concentrations of 0.25 kg/m$^3$ (0.42 lb/yd$^3$) above the base level in the 0 to 25 mm (0 to 1 in.) zone, and 35% of the cores showing contamination in the 25 to 50 mm (1 to 2 in.) zone. The five NCHRP preformed systems performed well, but were difficult to place; raising questions as to whether they could be placed properly in typical field conditions.

Of the more than 1,600 field samples taken from decks with experimental membranes (all but the tar emulsion and control decks), less than 2% had chloride ion concentrations above the corrosion threshold level of 325 ppm [0.11 kg/m$^3$ (0.18 lb/yd$^3$)]. All but three of these samples came from within the top 25 mm (1 in.) of the decks. The overall conclusion of the study was that membranes had been performing well in Vermont, and that the better systems could offer 50 years or longer of service before corrosion became a serious problem.

Table 3.4 presents a summary of the performance of each general category of membranes evaluated at the time of the study and again 3 years later, and is followed by more detailed descriptions of the performance of the membranes in each category. The average base level of chloride ions in the decks was 45 ppm [0.11 kg/m$^3$ (0.18 lb/yd$^3$)]. Corrosion of the reinforcing steel is initiated at a chloride content of about 325 ppm [0.77 kg/m$^3$ (1.3 lb/yd$^3$)].

**Table 3.4  Summary of Membrane Performance by Type**

(Frascoia 1984, 1987)

| System (number of decks) | Average years in service | % of samples contaminated (50 ppm (0.12 kg/m$^3$) above base level) | | Avg. Cl$^-$ in contaminated samples ppm (kg/m$^3$) | |
|---|---|---|---|---|---|
| | | 0-25 mm (0-1 in.) | 25-50 mm (1-2 in.) | 0-25 mm (0-1 in.) | 25-50 mm (1-2 in.) |
| Standard Preformed (21) | 7<br>10.2 | 19<br>25 | 7<br>13 | 125 (0.30)<br>153 (0.36) | 86 (0.20)<br>95 (0.22) |
| Thermoplastic (7) | 9<br>11.3 | 17<br>23 | 8<br>12 | 209 (0.49)<br>197 (0.47) | 130 (0.31)<br>140 (0.33) |
| Polyurethane (5) | 9<br>11.6 | 26<br>27 | 17<br>14 | 83 (0.20)<br>82 (0.20) | 70 (0.17)<br>83 (0.20) |
| NCHRP preformed (5) | 8<br>11 | 28<br>31 | 15<br>17 | 120 (0.28)<br>147 (0.35) | 109 (0.26)<br>95 (0.22) |
| Miscellaneous preformed (10) | 5<br>7.9 | 30<br>27 | 12<br>13 | 125 (0.30)<br>125 (0.30) | 91 (0.22)<br>88 (0.21) |
| Epoxy (8) | 9<br>11.9 | 50<br>51 | 22<br>19 | 116 (0.27)<br>114 (0.27) | 68 (0.16)<br>84 (0.20) |
| Tar emulsion (7) | 10<br>13.1 | 60<br>65 | 35<br>34 | 163 (0.39)<br>162 (0.39) | 122 (0.29)<br>123 (0.29) |
| No treatment (3) | 7 | 95 | 65 | 1,559 (3.69) | 791 (1.87) |
| Linseed oil (3) | 12 | 100 | 98 | 1,855 (4.39) | 945 (2.24) |

After an average of 10 years of exposure, all 6 control decks in the study were contaminated within the top 25 mm (1 in.). Three of the decks had no protection, and the other three had a mixture of linseed oil and mineral spirits applied to them. Ninety-eight percent of the samples from the control decks were contaminated at the 25 to 50 mm (1 to 2 in.) level with an average chloride ion concentration of 887 ppm [2.0 kg/m$^3$ (3.5 lb/yd$^3$)]. At the time of evaluation, the control decks appeared to be in satisfactory condition, but copper-copper sulfate half-cell potential measurements indicated that corrosion was active in an average of 30% of the deck areas. Delamination was also detected on one deck.

The standard preformed sheet membranes [1.7 mm (65 mils) reinforced rubberized asphalt, 1.8 mm (70 mils) reinforced tar and synthetic resin modified, and 1.9 mm (75 mils) reinforced bituminous] provided the best overall performance. After an average of 7 years exposure for 3 different systems on 21 bridges, only 7% of the samples had a chloride content more than 50 ppm [0.12 kg/m$^3$ ((0.20 lb/yd$^3$)] higher than the base level at a depth of 25 to 50 mm (1 to 2 in.). These membranes provided a consistent thickness, good cold weather flexibility, and were relatively easy to apply. There were, however, some problems getting a seal at the curb line and with blisters. The problem with the curb line seal appeared to be alleviated by the application of a compatible liquid polyurethane sealant along the edge of the membrane along the vertical curb face. Beginning in 1973, contractors in Vermont were allowed to use any of the three standard preformed systems, and since then, nearly all of the nonexperimental bridges in the state have used one of these.

After an average of 5 years of exposure, the results were mixed on the 7 miscellaneous preformed systems installed on 10 bridges. Two of the systems remained free of detectable chloride contamination for the 4 years of their evaluation. Other systems did not perform as well, such as two sheets that had holes to prevent blistering. The holes apparently did not reseal as well as they were supposed to, and although they did prevent blistering, they did not prevent chlorides from penetrating the membranes, showing 27-40% contamination at the 0-25 mm (0-1 in.) level.

After 8 years of exposure, the 5 preformed sheets from the NCHRP study had performed satisfactorily, except along the curb lines, where 52% of the samples were contaminated. Although they were well designed and had excellent physical characteristics, the sheets were expensive and difficult to place. These membranes were not recommended for further use.

After an average of 9 years of exposure, the 4 thermoplastic systems, installed on 7 bridges also performed well; only 8% of the samples were contaminated. The best performance for this class of systems was provided by a hot rubberized asphalt system, although for future applications, the use of protection boards was recommended, as was limiting the grade to 3% because of potential instability under traffic.

After an average of 9 years of exposure, the four Polyurethane systems, applied on five bridges, had the lowest chloride ion concentrations in the 25 to 50 mm (1 to 2 in) zone. Decks with two of these systems were still free of chloride after 8 years of service, while the other two had problems because of leakage. The polyurethane systems had an excellent bond at the curb line, satisfactory cold temperature flexibility, and were relatively easy to install. To prevent pinholes in the membrane, it was suggested that the polyurethane be applied in the afternoon or evening when the temperature was falling and that it be applied in multiple coats. Protection board was recommended to help establish a satisfactory bond with the asphalt overlay.

After an average of 9 years of exposure, the 7 epoxy systems installed on 10 bridges performed relatively poorly, except for one system. They exhibited poor cold temperature flexibility and had a tendency to form pinholes during application. However, the epoxy systems were relatively easy to apply and generally bonded well to the concrete.

After an average of 10 years of exposure, the 7 decks with one of the two tar emulsion systems exhibited the worst overall performance; 60% showed contamination. However, the average chloride concentration of the samples taken within the top 25mm (1 in.) of the decks with tar emulsion systems [0.25 kg/m$^3$ (0.43 lb/yd$^3$)] is still significantly lower than the average chloride concentration of cores taken from the exposed concrete decks [4.12 kg/m$^3$ (6.97 lb/yd$^3$)]. The results suggest that, even though they performed poorly relative to the other systems in the study, tar emulsion systems, which had been the standard treatment for bridge decks in Vermont during the 1960's, still offered a substantial level of protection to the concrete.

Currently, Vermont and New Hampshire are experimenting with two new types of waterproofing membranes, torch applied sheet membranes and spray applied liquid membranes (Graham 2000). Although these systems are more expensive than standard sheet membrane systems, testing has indicated that both of these membranes have a stronger bond with the concrete deck than the traditional "peel and stick" sheet membranes (Roberts 2000). The torch applied systems consist of a primer and a 4.5 mm (177 mils) thick sheet membrane with granular material to help increase bond with the deck (Graham 2000). The sheets are laid on the deck and then heated with torches, either by hand or by machine, until the underside is melted, causing the hot liquid bitumen to flow and form a bond to the concrete deck. This method has only been used since 1997, but the expected service life of this system is more than 20 years (Roberts 2000). The spray-applied membrane system consists of a primer, two layers of methylmethacrylate resin membrane, approximately 1.5 mm (60 mils) thick, and a tack coat to promote a good bond with the asphalt overlay (Graham 2000). Like the torch-applied membranes, spray-applied membranes have only been used since 1997, but it is expected that the service life of this system will be up to 50 years (Roberts 2000).

**Oklahoma —** Oklahoma installed 9 different membrane systems on 35 bridge decks between of 1973 and 1975 (Ward 1978). Five of the systems were preformed sheets, three were liquid membranes, and one was a built-up system. The thickness of the asphalt surfacing was 40 mm (1.5 in.), except for a few cases where an additional 25 mm (1 in.) of sand asphalt was added to help protect the membrane. Many problems were encountered during installation, including wrinkles, blisters, fish mouths along the lap joints, water beneath membranes, membrane and protection board being picked up by paving equipment, and unbounded sand asphalt. Three of the membranes had to be removed within a few days of installation because of water under the membrane or damage during the paving operation. A fourth membrane lasted only 18 months before it broke up under traffic. Poor workmanship at the time of installation was reported, as well as aggregate under the membrane.

**Minnesota —** The Minnesota Department of Transportation began a study to look at corrosion protection methods on rehabilitated bridge decks in 1972 (Hagen 1982). After 7 years, it was concluded that membrane systems were effectively preventing chloride ingress into new decks, but that the durability of the overlays was poor. After only three years, cracking and debonding began to appear at the interface between the membrane and the overlay on several of the decks. Bridge decks with high traffic volumes had serious problems with debonding and the traffic breaking up the overlays.

**Kansas —** Kansas has a long history of waterproof membrane use on its bridges. In 1926, a layer of asphalt-cotton fabric was placed over a new bridge deck, covered with cement-sand mortar, and topped with paving bricks. Fifty-six years later, the chloride content of the concrete under the membrane was less than 0.45 kg/m$^3$ (0.75 lb/yd$^3$), which is about 75% of the corrosion threshold (Bukovatz and Crumpton 1984).

In the period between 1967 and 1974, nearly 10,000 m$^2$ (12,000 yd$^2$) of interlayer membranes were installed on salt-contaminated bridge decks in Kansas. These membranes have performed well, with little maintenance. Asphalt riding surfaces have ranged from satisfactory, with some cracking, to excellent. The authors pointed out that the evaporation rate is higher than the precipitation rate in Kansas, which may be a factor in the good performance of these membranes.

In 1967, one quarter of a bridge deck was covered with a polypropylene membrane and an asphalt overlay. When the bridge was inspected 16 years later, this section was in much better shape than the other sections of the bridge. 91% of the deck in the area without the membrane contained patches, spalls and hollows, versus only 6.5% of the covered area (and all of these problems were around the edges of the membrane). The chloride content in the uncovered area ranged from 3.5 to 9.5 kg/m$^3$ (6 to 16 lb/yd$^3$), while the chloride content under the membrane was only 1 to 2 kg/m$^3$ (1.5 to 3 lb/yd$^3$). The chloride level on the bridge at the time the membrane was placed was not known, but the bridge had been in service for six years before the membrane was placed on the deck. Total traffic over the membrane had been about 3 million vehicles at the time of evaluation.

The only partial failure of the 9 membrane systems that were evaluated for the report was a 9-year old membrane that had been placed on a bridge with a steep downhill slope and a stop light at the bottom. After more than 76 million vehicles had passed over the bridge, the part of the membrane that was on a flatter slope was still in place.

Currently, there are several bridges in Kansas that have membranes with asphalt overlays; some have performed well and some have not. According to David Meggers (1999) of the Kansas Department of Transportation, various products have been used for the membranes–basically whatever was on the market at the time that they were installed. Some were sprayed on; others consist of boards impregnated with asphalt.

**Ontario —** According to David Manning (1999) of the Ontario Ministry of Transportation, agencies are fairly polarized on the issue of membranes, but the ones that have had success with membranes have generally used a thicker layer of asphalt over the membranes. All bridges in Ontario are waterproofed using a hot rubberized asphalt membrane. Because all bridge decks in Ontario are overlaid with 80 mm of asphalt to ensure a smooth riding surface, waterproof membranes are used to protect the concrete. The membrane comes in 22.7 kg (50 lb) cakes that are melted and spread onto the deck with a squeegee. Protection board is placed between the membrane and the asphalt overlay. These membranes have been used in Ontario since 1972 and have been working extremely well.

### 3.4.8 Summary

Field studies have produced varying results on the effectiveness of waterproofing membranes. Many of the studies that produced poor results took place in the early 1970's. In more recent studies (Frascoia 1984, 1989), waterproofing membranes have generally performed properly, especially those implemented by agencies that have had years of experience with their installation. To perform well, a membrane must be well bonded and undamaged, characteristics that depend on the quality of workmanship during installation. Waterproof membranes cannot stop corrosion that is already underway in existing decks, but along with an asphalt overlay, they can extend the service life of a deck by providing a smooth riding surface, preventing the development of potholes, and possibly slowing the rate of corrosion in the deck by limiting additional chloride contamination (Frascoia 1989, Manning 1995). The average service life of waterproof membranes applied to new decks is 15 to 20 years, while the service life of rehabilitated decks is 5 to 10 years, if minimal repairs are made, or 15 to 20 years if extensive repairs are made and the overlay thickness is at least 75 mm (3 in.). In many cases, the service life is determined by the asphalt overlay (Manning 1995).

According to *NCHRP Synthesis of Highway Practice 220: Waterproofing Membranes for Concrete Bridge Decks* (Manning 1995), waterproofing membranes can be a cost effective method for protecting reinforcing steel from corrosion in concrete bridge decks in the United States, but the following points need to be considered:

- Proper drainage should be provided so that water can drain quickly from the deck. Seepage drains should be provided at low points to prevent water from sitting on top of the membrane.
- Protection board or a layer of sand asphalt should be placed over membranes that are susceptible to damage during installation and overlaying.

- A minimum asphalt overlay thickness of 75 mm (3 in.) should be specified. Studies have shown that the thickness of the asphalt overlay influences the performance of the system.
- Softer, thicker membranes are vulnerable to rutting and shoving when used on grades exceeding 4 % or where heavy trucks brake or turn. Different membranes or another protection methods should be used in these situations.
- Liquid membranes should be more than 2 mm (0.08 in.) thick, and preformed membranes should be more than 2.5 mm (0.10 in.) thick.
- Ventilating layers are not recommended because they reduce the bond between the membrane and the deck.

## 3.5 CONCRETE SEALERS

When it comes to corrosion protection for concrete highway structures, sealers have an advantage in the fact that they can be used to protect all of the exposed concrete surfaces of the structure, including bridge decks, substructure members, and deck undersides (Zemajtis and Weyers 1996).

Currently, there are many sealers on the market. The purpose of a sealer is to reduce corrosion of reinforcement in concrete by preventing capillary action at the surface, therefore preventing water and chloride ions from penetrating the concrete. Sealers can either be pore blockers, forming a microscopically thin (up to 2 mm) impermeable layer on the concrete surface, or they can penetrate into the concrete slightly (1.5 to 3 mm) and act as hydrophobic agents (Zemajtis and Weyers 1996). Most pore blockers are not appropriate for use on bridge decks because they do not offer good skid resistance and do not hold up under traffic wear (Sherman et al. 1993).

An important property of a sealer is its vapor transmission characteristics. Moisture within the concrete needs to be able to pass through the sealer and escape to prevent high vapor pressures from building up in the concrete during drying periods which could cause the sealer to blister and peal (Sherman et al. 1993).

### 3.5.1 Linseed Oil

The first sealer to be used on concrete bridge decks in the United States was linseed oil. Linseed oil was first used as a method to reduce scaling of the deck surface as a result of deicer applications, but its use was expanded to corrosion protection when it was recognized that deicing salts were causing corrosion problems on bridge decks (Sherman et al. 1993). Although its use has been discontinued in most states because of environmental reasons or because many engineers feel that it does not work well and is not cost effective (Meggers 1999), linseed oil is still used in some states. Missouri, for instance, applies a mixture of linseed oil and mineral spirits to all new decks before they are opened to traffic, and then again one year later. The linseed oil combination is applied at a rate of 0.23 $l/m^2$ (0.05 $gal/yd^2$). According to J. D. Wenzlick of the Missouri Department of Transportation, other sealers have been tried, but none have worked as well as the linseed oil/mineral oil mixture (Wenzlick 1999). Texas also uses a linseed oil/mineral spirits mixture as a standard surface treatment on many of its bridge decks. Results of research on the performance of the mixture have varied, but it has generally performed well in regions that are not exposed to frequent deicing salt applications (Cox 2000). Kansas has not used linseed oil on its decks for at least the past 10 years (Meggers 1999).

Linseed oil works by preventing water and chloride ions from penetrating the concrete, while allowing water vapor to escape. Arguments for the use of linseed oil are that it is a well-known product, that most contractors have had experience with its application, and that it is one of the least expensive corrosion protection strategies available. However, linseed oil does need to be reapplied every 2 to 5 years to maintain its performance (Sherman et al. 1993), and some engineers question whether it actually provides any protection against corrosion. Tests indicate that linseed oil requires exposure to ultraviolet light to produce a durable coating (Pfeifer and Scali 1981).

### 3.5.2 Epoxy

Epoxies have been used as both penetrating sealers and as coatings. Epoxies were chosen for use as sealers because they have good adhesion to concrete, aggregate, and steel, are resistive to chemicals and weather, and cure quickly without requiring high temperatures or pressures (McCaskil et al. 1970). Problems experienced with epoxy coatings include pinholing, blistering, and debonding from the deck. Epoxy coatings were applied to a number of Kansas bridge decks in a 1970 study (McCaskill et al.). When the coatings were evaluated, they were found to be full of pinholes and to have not sealed the decks.

### 3.5.3 Silane and Siloxane

Silane and siloxane sealers are silica-based materials that function as hydrophobic agents. Silane and siloxane sealers do not block the pores of the concrete like most oil based sealers, but react chemically with the concrete surface to form a hydrophobic layer under the surface that repels water and chloride ions while allowing water vapor to pass through (Pfeifer and Scali 1981). Different types of concrete need different coverage rates and different sealers, depending on porosity and capillary size. Siloxanes are simply silanes that have been allowed to polymerize slightly, which makes them larger in size (Sherman et al. 1993).

Proper surface preparation is important when silanes or siloxanes are applied. The deck must be cleaned and be free of all oil, curing compounds, and general road grime to make sure that the treatment material can actually reach the surface (Sherman et al. 1993). Prescreening tests for an NCHRP study found that concrete coated with silane had a water absorption value that was 30% of the absorption of untreated concrete when the saline was applied to clean concrete. Absorption increased to 47% of the value for untreated concrete when the silane was applied on top of linseed oil (Pfeifer and Scali 1981). The deck must also be dry to allow the silane or siloxane to penetrate into the concrete and bond with it chemically. Prior applications of silane or siloxane do not need to be removed before reapplication, which is recommended every 5 years (Sherman et al. 1993).

The advantage of silane and siloxane sealers is that they are easy to apply, can be applied to any part of a structure, and can be applied at any time, during or after construction. Disadvantages include surface preparation requirements, and the fact that the materials are difficult to screen and are expensive when purchased in small amounts. Agencies could overcome the last two problems by choosing a standard product and testing and using it on a district-wide basis rather than selecting a different product for each job (Sherman et al. 1993).

### 3.5.4 Methacrylate

Another sealer that has been used extensively in California, Virginia and Alberta is high molecular weight methacrylate. Methacrylates can function as crack sealers or as fine overlays, using sand and other fine aggregate for skid resistance. Methacrylates are generally applied as a three component system consisting of a monomer, a promoter, and an activator, mixed together before application (Sprinkel 1992b). These sealers are usually low-viscosity materials, and are applied as a spray or with a broom or squeegee (Sherman et al. 1993).

There are some significant problems associated with the use of high molecular weight methacrylates. Over time, cracks tend to reopen through the polymerized material, decreasing the effectiveness of the methacrylate. It should also be noted that treatment will not entirely refill cracks, and will not restore concrete strength that is lost with cracking. Field application problems are encountered with this sealer because most field crews are not familiar with application procedures of methacrylates, are bothered by the smell, and because the set and hardening of the material are highly sensitive to the environment (Sherman et al. 1993).

### 3.5.5 Previous Studies

A 1981 report for NCHRP evaluated the effectiveness of five different generic sealers (Pfeifer and Scali). Three were found to exhibit consistently good performance throughout the tests. These materials consisted of an epoxy, a methyl methacrylate and a silane. All three materials performed best when applied after the concrete had been allowed to air dry for at least a 5 days. The silane and methyl methacrylate appeared to function as chloride screening materials, since the water that was absorbed through these sealers had a lower chloride content than that of the original solution. These materials also provided protection to the reinforcement in cracked concrete slabs, suggesting that the materials can penetrate into existing cracks to provide corrosion protection in the presence of cracks up to the 0.25 mm (0.01 in.) in width.

The epoxy sealer demonstrated a low capability for water vapor transmission, so it should not be applied to fresh concrete until the concrete has had a reasonable period of air-drying. The epoxy sealer also resulted in a glossy surface on the concrete, which would be dangerous for driving or walking, limiting its application to areas not exposed to these uses. The silane, on the other hand, did not appear to change the surface texture of the concrete and was the only sealer tested that did not change the color of the concrete. Although the saline had the highest material cost of the three sealers, it was also the only sealer that required just one coat, which could reduce the overall cost of its use (Pfeifer and Scali 1981).

The 1981 NCHRP study also included linseed oil with mineral spirits because it was the most widely used sealer at the time. Linseed oil performed relatively poorly in the Series I screening tests, in which the material was applied to concrete blocks that were then soaked in a 15% sodium chloride solution to evaluate water absorption characteristics. Vapor transmission characteristics were then evaluated during an air-drying period. The concrete that had been treated with boiled linseed oil had a water absorption value that was 80% of the value of untreated concrete, as compared with values of 10 to 30% for the other three sealers in the study. A group of specimens was pretreated with linseed oil before they received the test sealer to simulate an older deck that had been previously treated with linseed oil. All of the pretreated specimens had lower water absorption values than the regular test specimens, except for the one that was sealed with saline. In the Series IV tests, cube specimens were exposed to different environmental conditions, including ultraviolet light. Specimens treated only with boiled linseed oil and those pretreated with boiled linseed oil prior to coating with another sealer had low chloride contents at the end of the tests for all exposure conditions. The better performance of the boiled linseed oil in the Series IV tests was attributed to the exposure of the fresh linseed oil to ultraviolet radiation immediately after application (Pfeifer and Scali 1981)

Three bridge decks in Vermont that had been treated with a mixture of linseed oil and mineral spirits were evaluated as part of a study on waterproof membranes (Frascoia 1984). Performance was evaluated based on the chloride content of the concrete from cores taken from the decks. The concrete was considered to be contaminated when the chloride ion content was more than 50 ppm (0.12 kg/m$^3$, 0.2 lb/yd$^3$) above the chloride content measured immediately after construction [the average base level of chloride ions in the decks was 45 ppm (0.10 kg/m$^3$, 0.18 lb/yd$^3$)]. The decks were subjected to 75 to 115 freeze thaw cycles per year, 1.8-3.5 m (70-140 in) of snowfall, and a maximum salt application of 15 kg/m$^2$ (3 lb/ft$^2$) per year.

After an average of 11.7 winters of exposure, all of the samples taken from the 3 decks treated with linseed oil and mineral spirits were contaminated within the top 25 mm (1 in.). After an average of 7.3 winters of exposure, 95% of the samples taken from untreated decks were contaminated within the top 25 mm. Ninety-eight percent of the decks treated with linseed oil were contaminated at the 25 to 50 mm (1 to 2 in.) level, versus 65% of the samples taken from untreated decks. The average chloride ion concentration at the 25 to 50 mm depth in the contaminated samples was 791 ppm (1.9 kg/m$^3$, 3.2 lb/yd$^3$) for the untreated decks and 945 ppm (2.2 kg/m$^3$, 3.8 lb/yd$^3$) for the decks treated with the linseed oil mineral spirits mixture. Both of these chloride ion concentrations are well above the accepted threshold for corrosion (about 0.6 kg/m$^3$, 1 lb/yd$^3$). At the time of evaluation, all of the decks appeared to be in satisfactory condition, but copper-copper sulfate half-cell potential measurements indicated that corrosion was active in an average of 30% of the deck areas.

It is difficult to compare the effectiveness of the linseed oil and mineral spirit mixture to the untreated decks because the decks had not been exposed for the same amount of time, but the fact that the decks treated with linseed oil had average chloride ion concentrations well above the corrosion threshold after only 11 years indicates that it did not effectively seal the concrete.

The results of a year long study to evaluate the effectiveness of epoxy, methyl methacrylate, and polyurethane sealers, was published by researchers at Virginia Polytechnic Institute and State University in 1995 (Zemajtis and Weyers). All three sealers are surface sealers, and therefore would not be applied in areas that would be exposed to traffic wear. Two specimens were fabricated with a horizontal surface and two vertical surfaces, or legs. Each coating was used on half of a specimen, leaving half of one specimen as a control. The coatings were applied according to the manufacturers' specifications. The dry thickness of the polyurethane sealer was 625 μm (25 mils), the epoxy was applied in two 200 μm (8 mils) thick coats, and the methyl methacrylate consisted of three layers, each approximately 200 μm (8 mils) in thickness. The specimens were exposed to accelerated wet and dry cycles with 3% sodium chloride. The bottom part [280 mm (11 in.)] of the legs was continuously submerged.

The effectiveness of the coatings was evaluated by measuring the chloride concentrations in the concrete before the exposure cycles were started and then after specific numbers of exposure cycles had been completed. The average background chloride content was 0.26 kg/m$^3$ (0.44 lb/yd$^3$), and the corrosion threshold was considered to be 0.71 kg/m$^3$ for this study. Table 3.5 shows the average increase (kg/m$^3$) in chloride ion concentration above the background concentration in the slabs at a depth of 13 mm below the horizontal surface after 14, 21, and 30 weekly exposure cycles. Relative performances were similar for the other exposure conditions. Chloride gains were generally lower in the specimens that were coated with polyurethane and methyl methacrylate than in specimens that were not coated or were coated with epoxy sealers (Zemajtis and Weyers 1995).

**Table 3.5 Average Gain in Chloride Concentration Above Background Value After Weekly Cycles (kg/m$^3$)** (Adapted from Zemajtis and Weyers 1995)

| Coating | Chloride gain above background value (kg/m$^3$) | | |
|---|---|---|---|
| | 14 cycles | 21 cycles | 30 cycles |
| Control | 9.10 | 8.99 | 9.34 |
| Epoxy | 0.29 | 0.37 | 0.30 |
| Methyl methacrylate | 0.03 | 0.09 | 0.12 |
| Polyurethane | 0.04 | 0.15 | 0.14 |

A study to evaluate the effectiveness of four different sealers was published by researchers at Virginia Polytechnic Institute and State University in 1996 (Zemajtis and Weyers). The sealers that were evaluated included water-based epoxy, solvent-based epoxy, silane, and siloxane. Concrete slabs and a wall surface were fabricated, cured, sealed, and then exposed to direct sunlight and weekly cycles consisting of 3 days of continuous ponding with 3% sodium chloride solution followed by 4 days of air drying. Two bridge decks, with average annual daily traffic volumes of 12,430 and 24,270, were also sealed as part of the study.

The four sealers were evaluated based on visual observations and the level of chloride in the concrete. Chloride concentration levels were measured at depths of 13, 25, and 38 mm (0.50, 1, and 1.5 in.). The measured wear rate on the bridge deck with the higher traffic volume was 0.17 mm (7 mils) per year. At this rate, a sealer that penetrated 1.5 mm (60 mils) would be worn off in less than 9 years. This is consistent with the findings of Weyers (1994) that indicate a service life for hydrophobic sealers of about 7 years for bridge decks and 10 years for members not subjected to traffic wear. The service life, or reapplication period, of pore blockers (water-based epoxy and solvent-based epoxy) on surfaces not exposed to abrasion is considerably shorter, generally about 3 years. Visual observations on the bridge decks determined that the pore blockers had worn off in less than 1 year on both structures (Zemajtis and Weyers 1996).

Table 3.6 shows the average increase in chloride ion concentration, over the background concentration, in the slabs at the 13 mm depth after 10, 20, and 30 weekly exposure cycles. While all of the sealers had smaller concentration increases than the control specimens, the hydrophobic sealers (silane and siloxane) performed significantly better than the pore blockers (epoxies) in terms of reducing chloride diffusion at the surface.

**Table 3.6 Average Gain in Chloride Concentration Above Background Value After Weekly Cycles (kg/m$^3$)** (Adapted from Zemajtis and Weyers 1996)

| | Chloride gain above background value (kg/m$^3$) | | |
|---|---|---|---|
| Sealer | 10 cycles | 20 cycles | 30 cycles |
| Control | 1.22 | 2.27 | 4.18 |
| Water-based epoxy | 0.80 | 1.74 | 3.86 |
| Solvent-based epoxy | 1.10 | 2.18 | 4.14 |
| Silane | 0.17 | 0.13 | 0.13 |
| Siloxane | 0.14 | 0.14 | 0.11 |

The study concluded that the service lives of concrete sealers are affected by exposure conditions, traffic wear, ultraviolet light exposure, surface orientation (horizontal or vertical), and traffic wear. The study also concluded that the use of high quality concrete is still important when sealers are being used (Zemajtis and Weyers 1996).

### 3.5.6 Summary

The studies described in this section have identified a number of sealers that can reduce or prevent the corrosion of reinforcement in concrete structures. Important considerations when selecting a sealer include:

- The performance of some sealers appears to be affected by application rates, while the performance of others is not. The materials that are not dependent on application rate are preferable because field application will always have some level of variability (Pfeifer and Scali 1981).
- Some sealers do not penetrate the concrete, and therefore are not suitable for use on deck surfaces because they have poor skid resistance and are quickly worn off by traffic (Zemajtis and Weyers 1995).
- Even sealers that do penetrate the concrete surface must be reapplied on a regular basis (Weyers 1994).
- Many sealers are solvent-based chemicals that require the user to be aware of safety precautions that need to be taken during application (Pfeifer and Scali 1981).

## 3.6 DEEP POLYMER IMPREGNATION

Polymer impregnation protects reinforcement from corrosion in concrete by limiting the ingress of water, oxygen and chloride ions into the concrete by sealing the surface pore system of the concrete. Polymer impregnation also immobilizes chloride ions that may already be present in the deck. Because it is applied externally after the structure is constructed, polymer impregnation can be used both on new construction and as a repair method. Deep polymer impregnation is achieved by filling the concrete pores to a depth below the top layer of reinforcement with polymer. This is done by first applying heat to dry out the deck to the desired depth of impregnation. Next, a monomer is applied, that infiltrates the concrete and is polymerized in place by reheating the concrete (Cady et al. 1987).

Polymer impregnation can be applied for the protection of all areas on bridge structures, but its use is limited mainly to decks because of its associated high material, labor and equipment costs. Studies have shown that deep polymer impregnation can prevent deterioration of concrete bridge decks and also improve wear resistance if technically and economically feasible means for application can be developed (Cady et al. 1987). However, polymer impregnation is not commonly used because it is difficult to apply. The biggest problem with polymer impregnation is achieving sufficient depth of polymer penetration. It is important to have low moisture in the deck, and grooves may need to be sawed into the deck to help attain deep impregnation. Drying the deck for the monomer application can be difficult and heating and cooling the concrete can lead to excessive thermal stresses (Sherman et al. 1993).

### 3.6.1 Field Study

A joint research project, carried out by Lehigh University and Pennsylvania State University beginning in 1972, investigated deep polymer impregnation as a possible method to prevent reinforcing steel corrosion in sound concrete decks (Cady et al. 1987). The research ended with a field trial on a bridge in Pennsylvania. The field trial was carried out in 1975 on a bridge in Bethlehem, Pennsylvania. At the time of polymer impregnation, there were no spalls or patched areas on the deck. A 1 x 3.5 m (3.5 x 11.5 ft) area along the right wheel path in the area of the deck with the highest measured chloride content was selected as the test area. Mean chloride contents measured in the deck in 1975 before impregnation were 1.43 kg/m$^3$ (2.41 lb/yd$^3$) for the deck not including the test area, and 2.14 kg/m$^3$ (3.60 lb/yd$^3$) in the area that was to be impregnated. It is not known at what depth these measurements were taken, but it was determined that the differences between the chloride content in the concrete within the test area and the rest of the deck at the initiation of the study were not statistically significant.

Upon visual examination of the deck in 1983, there were obvious differences in the performance of the section of the deck with the polymer impregnation and the rest of the deck. Several areas of the deck had been patched, but there were no patches or spalls in the test area. In 1984, cores were removed from the deck and a field investigation was performed. Delaminations were detected by sounding with a hammer in areas surrounding the test area, but none were found within the polymer-impregnated area. Chloride contents at the level of the reinforcement [25-50 mm (1 to 2 in.)] ranged from 0.83 to 1.02 kg/m$^3$ (1.39 to 1.71 lb/yd$^3$) in the impregnated area, compared to 1.60 to 2.20 kg/m$^3$ (2.70 to 3.71 lb/yd$^3$) in the other areas. This difference was determined to be statistically significant. Wheel-path wear measurements in the test area and adjacent areas indicated that polymer impregnation had reduced the wear of the deck by about 65% over the 9-year period (Cady et al. 1987).

Using microscopic analysis, the depth of impregnation in the cores was found to range from 76 to 83 mm (3 to 3.25 in.). Air void analyses were performed on polished sections of the cores to evaluate whether the impregnated concrete contained an adequate air void system for freeze-thaw protection. The tests indicated that one of the polymer-impregnated cores had an air void system that could be susceptible to freeze-thaw damage, but there was no evidence of frost damage in the core (Cady et al. 1987).

### 3.6.2 Summary

Polymer impregnation has been successful in protecting bridge decks from corrosion damage in limited field studies. However, polymer impregnation is not commonly used because it is difficult to apply and expensive. The biggest problem with polymer impregnation is achieving sufficient depth of polymer penetration.

# CHAPTER 4

# ELECTROCHEMICAL METHODS

## 4.1 GENERAL

Electrochemical methods include cathodic protection and electrochemical chloride extraction. Both of these methods have the ability to stop corrosion in chloride contaminated concrete and are most often used as rehabilitation methods, although cathodic protection is also be used for structures.

## 4.2 CATHODIC PROTECTION

The science of cathodic protection began in 1824 when Sir Humphrey Davy used iron anodes to protect the copper sheeting on the bottom of the British Navy's sailing ships. Since its development, cathodic protection has been used in several areas including marine and underground structures, storage tanks, and pipelines (Virmani and Clemena 1998). However, cathodic protection was not applied to reinforced concrete structures until it was used to protect a reinforced concrete bridge deck in 1973 (Stratfall 1974). At first, cathodic protection was used mainly to prevent further corrosion after repair of damaged structures, but recently, cathodic protection has been incorporated in new construction in an effort to prevent corrosion from starting (Polder 1998).

According to Jackson (1982), "The only rehabilitation technique that has proven the stop corrosion in salt-contaminated bridge decks regardless of the chloride content of the concrete is cathodic protection." An advantage of using cathodic protection as a repair method for reinforced concrete bridges is that only spalls and detached concrete need to be repaired. Chloride contaminated concrete that is still sound can remain in place because the cathodic protection system will prevent further corrosion, and, in fact, reduce the concentration of chloride ions adjacent to the protected reinforcing bars. This can significantly reduce repair costs (Polder 1998).

Currently, the state of Missouri has about 110 bridge decks that are being cathodically protected with impressed current systems. Overall, they have had mixed results with cathodic protection. Two types of anodes are currently in use in Missouri: platinum anodes and expanded titanium mesh anodes. The biggest problem with cathodic protection systems in Missouri has been that, over time, the anode material deteriorates and needs to be replaced. The Missouri Department of Transportation expects that out of the 145 cathodic protection systems that have been installed on bridge decks since 1977, about half of these systems will still be providing protection to the decks 20 years after their initial installation. The first deck to have cathodic protection applied to it in Missouri in 1977 is still being supplied with current, and is still standing. Whether or not the cathodic protection system is actually working in the deck is not known (Wenzlick 1999).

Since the late 1980s, cathodic protection has been applied to over 20 structures in The Netherlands. It has been successful in stopping reinforcement corrosion in each case, even when other repair methods have failed.

For cathodic protection to be successful, repair materials must have similar electrical resistivity to the concrete in the structure, the current must be uniformly distributed throughout the structure, and the system must be regularly monitored and inspected to ensure that polarization is in the desired range (Polder 1998).

### 4.2.1 Components of a Cathodic Protection System

A cathodic protection system for reinforced concrete consists of a number of basic components, including the reinforcement to be protected, an anode, a power source, concrete surrounding the steel, a monitoring system, and cabling to carry the system power and monitoring signals (*Cathodic* 1998). Each cathodic protection system contains two types of anodes. The primary anode, or “anode conductor”, acts as a contact point and a power supply line for the secondary anode. The secondary anode, usually referred to as the “anode”, is the material that distributes the current over the surface of the structure. Generally, for existing structures, the deteriorated surface of the concrete is repaired, and the anode is placed on top of the new surface (*Cathodic* 1993). The positive terminal of the power source is connected to the anode, and the negative terminal is connected to the reinforcement, which becomes the cathode. A small amount of direct current (DC) is then applied, causing current to flow through the electrolyte from the anode to the reinforcement, making the reinforcement become cathodic in relation to the anode. The electrolyte is generally the pore water in the concrete, which contains alkalis that allow the transfer of current from the anode to the reinforcement. Figure 4.1 shows the basic setup for a typical cathodic protection system. Oxygen and moisture must be present in the concrete for the cathodic protection system to work. The anode should be designed to resist deterioration while acting to protect the reinforcement (*Cathodic* 1998).

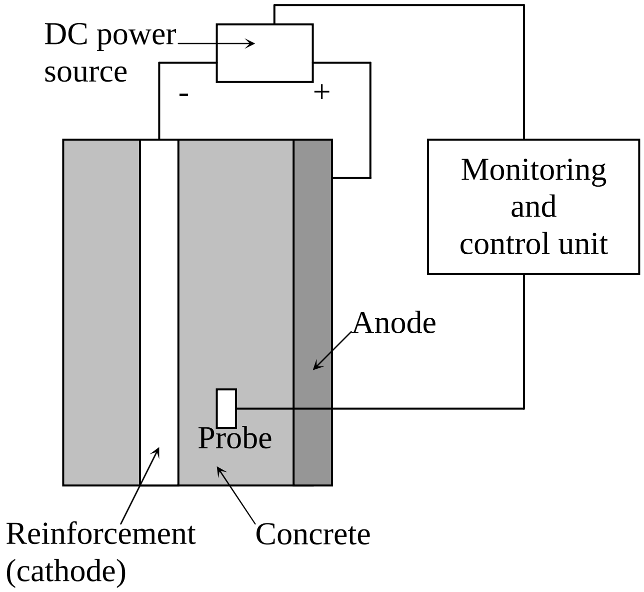


**Figure 4.1 Typical Cathodic Protection System Setup**
(*Cathodic* 1998)

Cathodic protection works by using current to shift the potential of the reinforcing steel in the negative direction. If the potential is shifted far enough so that all of the steel reinforcement becomes cathodic, corrosion will be stopped (Locke 2000). The standard protection criteria is generally to polarize the steel to an "instantaneous-off" potential more negative than –850 mV (CSE) (Page and Sergi 2000). An instantaneous-off potential, also referred to as the IR-free potential, is measured immediately after the current to the system is switched off so that there is no ohmic resistance polarization ($E_\Omega$), which would mask the other potentials. $E_\Omega$ is a function of the current (I) and the effective resistance within the polarization cell (R) (Jones 1996).

In concrete with a pH between of 12.5 and 13.5, the potential of the reinforcement would need to be more negative than -820 mV (SHE) [-1130 mV (CSE)] to put it into the region where corrosion of iron is thermodynamically impossible. However, if used, these potentials are so low that hydrogen evolution can occur on the steel surface, which can cause a number of problems, such as large current draws, anode degradation, bond loss, and hydrogen embrittlement (*Cathodic* 1998). Bond loss and hydrogen embrittlement are discussed further in sections 4.2.2 and 4.2.3.

The current for cathodic protection can be supplied to a bridge deck by one of two different methods: DC can be supplied by an external power source (impressed current), or current can be supplied by an anode that is made from a material that is more active than reinforcing steel (sacrificial anode). The DC power source for an impressed current system is external to the structure and can either consist of utility lines or generators, some of which are powered by wind or solar energy (*Cathodic* 1993). When a sacrificial anode, usually made of zinc, magnesium or aluminum, is coupled to steel, it corrodes galvanically (current flow is spontaneous due to the potential differences in the metals), giving up electrons to protect the steel (Virmani and Clemena 1998).

Chemical reactions occur adjacent to the anode in impressed current cathodic protection systems, usually involving is either the release of oxygen [Eq. (4-1)] or the evolution of chlorine [Eq. (4-2)]. Oxygen evolution is the most common reaction in well-designed systems (*Cathodic* 1998).

$$4\ H_2O \rightarrow 4\ H^+ + 4\ OH^- \rightarrow 4\ H^+ + 2\ H_2O + O_2 + 4\ e^- \qquad (4\text{-}1)$$

$$2\ NaCl \rightarrow 2\ Na^+ + 2\ Cl^- \rightarrow 2\ Na^+ + Cl_2 + 2\ e^- \qquad (4\text{-}2)$$

The $H^+$ ions formed during the anodic reaction reduce the pH of the concrete and can lead to the generation of acid. Depending on the anode material and the surface area to concrete surface area ratio, maximum current density outputs for anodes are limited to minimize acid generation. Acid is more likely to be generated on systems that output high current densities, such as embedded wires, meshes, or ribbons than on surface coatings or sprays (*Cathodic* 1998).

Both impressed current and sacrificial anode, or galvanic, systems have been used successfully on bridges in the United States. Each method has specific characteristics that make it more effective than the others in a given situation. Table 4.1 lists some of the general characteristics of each method. Impressed current systems are used most often on bridge decks, but there are some impressed current anodes that can be used on bridge substructure members as well. The use of galvanic anodes is generally limited to substructure members.

**Table 4.1 Comparison of Characteristics of Cathodic Protection Methods**
(Virmani and Clemena 1998)

| Impressed-Current | Sacrificial Anode |
|---|---|
| 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 |

Corrosion rates are effected by a number of variables. Therefore, there is no fixed value for either the potential or the current that a cathodic protection system must supply. Each system must be designed individually. Protection will increase over time as chloride ions are forced away from, and hydroxide ions are generated at the reinforcement. The increase in hydroxyl ions and the decrease in chloride ions around the steel surface reduce the risk of corrosion, so the cathodic protection current can be reduced. This also means that if current is interrupted, protection will be maintained for a time. This allows cathodic protection systems to be turned off, as is done regularly for short periods of time to check the polarization of the system, or for longer periods of time, as in intermittent cathodic protection systems (Polder 1998, Page and Sergi 2000).

It is important to determine the amount of current or potential difference that needs to be applied to protect a structure and to make sure that the anode can provide that current uniformly across the structure at a reasonable DC output voltage (Virmani and Clemena 1998, *Cathodic* 1998). Table 4.2 lists some estimated current density requirements for cathodic protection in various material and environmental conditions.

**Table 4.2 Practical Cathodic Protection Current Density Requirements for Varying Steel Conditions** (*Cathodic* 1998)

| **Environment surrounding steel reinforcement** | **Current density mA per m$^2$ of reinforcement** |
|---|---|
| Alkaline, no corrosion occurring, low oxygen resupply | 0.1 |
| Alkaline, no corrosion occurring, exposed structure | 1-3 |
| Alkaline, chloride present, dry, good quality concrete, high cover, light corrosion observed on reinforcement | 3-7 |
| Chloride present, wet, poor quality concrete, medium-low cover, widespread pitting and general corrosion on steel | 8-20 |
| High chloride levels, wet fluctuating environment, high oxygen level, hot, severe corrosion on steel, low cover | 30-50 |

Selection of the proper anode is important no matter which method of cathodic protection is used. When applied to a bridge deck, the selected anode must be able to resist deterioration under the expected environmental conditions and traffic loads. The structure must be capable of carrying the additional dead load that the anode will place on it. The anode must be durable enough to withstand installation demands and to reach its design life, and the anode must also have a large enough surface area to minimize deterioration in the surrounding concrete. An anode that is to be used on substructure members must also be able to stay in place on vertical surfaces and overhangs (Virmani and Clemena 1998).

### 4.2.2 Cathodic Protection of Prestressed Concrete Bridge Members

Significant advances have been made in the cathodic protection of reinforced concrete bridge decks since the early 1970s, but the development of cathodic protection for prestressed concrete bridge members has lagged behind (Virmani and Clemena 1998).

The FHWA initiated a study in 1988 to identify potential problems and limitations to the application of cathodic protection to prestressed concrete bridge members. There are two main concerns with cathodic protection of prestressed concrete structures (Virmani and Clemena 1998). The first is hydrogen embrittlement, caused when hydrogen generated by a cathodic reaction at the surface of the steel migrates into the steel rather than being released into the surrounding electrolyte. The initial cathodic reaction in cathodic protection systems is oxygen reduction [Eq. (4-3)] (Enos et al. 1997).

$$O_2 + 2\ H_2O + 4\ e^- \rightarrow 4\ OH^- \qquad (4\text{-}3)$$

However, the oxygen level at the steel-concrete interface can be depleted over time when subject to a constant current density, causing the potential to become more negative. If the potential drops enough, water reduction takes place [Eq. (4-4)], ($H_{ads}$ is surface-adsorbed hydrogen) (Enos et al. 1997).

$$H_2O + e^- \rightarrow OH^- + H_{ads} \qquad (4\text{-}4)$$

When hydrogen atoms migrate (are adsorbed) into the steel, they can become temporarily trapped at dislocations, grain boundaries, voids, and non-metallic inclusions, causing metal-to-metal bonds to weaken. If the hydrogen atoms pin the dislocations within the steel matrix, they will not allow plastic deformation, causing the steel to fail prematurely under load (*Cathodic* 1998).

Laboratory studies have found that the risk of hydrogen embrittlement in prestressing steel appears to be acceptably low when the potential is kept less negative than –0.90 V versus the saturated calomel electrode (SCE). Adequate protection can be achieved at this potential (Hartt et al. 1996). A 1988 study indicated that hydrogen embrittlement of embedded prestressing steel due to cathodic protection current could be avoided through careful monitoring and control of the amount of steel polarization. This could be accomplished by using embedded reference cells and a current and voltage-limiting rectifier (Virmani and Clemena 1998).

The second concern is partial loss of the bond between the prestressing strands and the concrete due to the migration of cations towards the strand, resulting in the softening of the concrete. Through the use of pull-out tests, it was concluded that there was little to no decrease in the bond between prestressing strands and concrete after a charge transfer equivalent to as much as 160 years of cathodic protection; so loss of tendon bond due to cathodic protection is not a problem for pre-tensioned concrete members (Hartt et al. 1996).

### 4.2.3 Potential Side Effects of Cathodic Protection

While polarization must be high enough to prevent corrosion, overprotection must be avoided because exposure to excessive current has been shown to lower the bond strength between reinforcement and concrete and to cause hydrogen embrittlement (Nash et al. 1994). As discussed above, hydrogen embrittlement and bond loss are significant concerns in prestressed steel, but they can also affect conventional steel reinforcement.

A study by Vrable (1977) indicated the tensile strength of concrete cylinders was not affected by cathodic protection current. The study also indicated that the decrease in bond strength between reinforcement and concrete was affected more by total ampere hour/ft$^2$ of applied current than by current density. To avoid problems with decreased bond, it is recommended that electrical potentials be less negative than –1.10 volts versus a copper-copper sulfate reference electrode (CSE) [-1.02 volts (SCE)]. However, potentials high enough to create a decrease in bond strength should never be reached because they are more negative than the recommended maximum potentials to prevent hydrogen embrittlement.

Hydrogen was found to evolve on steel at a potential more negative than -1.05 volts (CSE) [-0.97 volts (SCE)] in concrete with a pH of 12.5. This hydrogen evolution can result in hydrogen embrittlement of the steel (*Cathodic* 1993). It should also be noted that hydrogen can be generated at lower potentials when the pH has been reduced by corrosion or other influences (Funahashi and Young 1996).

Research by Ali (1993) suggests that the hydroxyl ions generated at cathodic sites, can cause alkali-silica reactions, resulting in concrete cracking. According to Ali, cathodic protection should not be used on structures that may be susceptible to alkali-silica reactions.

It is important for all of the reinforcing steel to be electrically connected in a structure that is to have a cathodic protection system installed to avoid stray currents, which will cause reinforcement that is not part of the circuit to corrode at an accelerated rate (*Cathodic* 1993).

### 4.2.4 Monitoring of Cathodic Protection Systems

Evaluation and control devices, such as probes, reference cells, and controllers are used to monitor and control cathodic protection systems (*Cathodic* 1993). Samples of steel can be embedded in the concrete as probes. The probes are monitored for corrosion potential and the current supplied by the cathodic protection system, and can indicate whether the system is operating, but provide limited information on its effectiveness. A reference electrode is required to determine instantaneous-off potentials and depolarizations, which are common criteria used to monitor cathodic protection systems (*Cathodic* 1998).

The objective of cathodic protection is usually to polarize the reinforcement to an instantaneous-off potential more negative than -850 mV (CSE) [-770 mV (SCE)]. This potential should decay (become less negative) by at least 100 mV from the instantaneous-off potential within 24 hours after the system is disconnected (Page and Sergi 2000). Depolarization tests are often performed over 4 hours, and the current to the system is adjusted until the polarization drops 100 mV in that period (*Cathodic* 1998).

If there is a significant change in environment within a structure, such as moisture, chloride content, or geometry, individual areas should be protected by separate cathodic protection circuits, or zones. Cathodic protection systems are generally applied in zones of 50 to 500 $m^2$, depending on the environment of the structure and the current requirements of the anode. For example, on marine substructures, there are generally several zones, such as underwater, between the low and high water level, in the splash zone, and above the splash zone (*Cathodic* 1998). Cathodic protection systems for bridge decks in Missouri are generally divided into zones of 185 $m^2$ (222 $yd^2$) (Wenzlick 2000).

The Missouri Department of Transportation takes voltage, current, and half-cell potential measurements in each zone for all cathodically protected bridges every two months. At this time, any electrical problems or power outages are repaired. Once a year, each system is shut down for a 4-hour depolarization test. The current is turned off and the half-cell potential of the reinforcement is measured (instant-off potential), if the potential decays 100 mV or more in 4 hours, the system is assumed to be operating properly. If the potential drop in the 4 hours is less than about 80 mV, the current to the system is increased (Wenzlick 1999).

### 4.2.5 Impressed-Current Systems

Impressed current systems are usually more appropriate for cathodic protection than sacrificial anode systems for atmospherically exposed concrete. The power for an impressed current system usually comes from a transformer rectifier with low voltage (< 24 V) and current (< 10 A). The current is passed through the anode and into the concrete, where the current flow from the rectifier becomes ionic transfer flow through the pore solution. “The material composition, shape, type and orientation of the anodes are all of fundamental importance to the performance of the cathodic protection system.” (*Cathodic* 1998). There are a number of different types of anodes that are used in impressed current cathodic protection systems. A description of these systems follows, beginning with impressed current anodes for bridge decks.

**Coke-breeze overlay anode systems —** The first conductive overlays were asphalt mixtures in which metallurgical coke-breeze was substituted for natural aggregate. The method was developed by Stratfall (1974) and Caltrans for the Sly Park Road Overcrossing bridge deck in California in 1973. The primary anodes consisted of rows of iron alloy discs, 32 mm (1.25 in.) thick and 254 mm (10 in.) in diameter, spaced at 3.6 m (12 ft) center-to-center. The discs were attached to the concrete with fast setting epoxy adhesive and connected to the rectifier by a cable. The secondary anode system, consisting of a coke-breeze asphalt mix, was placed in a 50 mm (2 in.) thick layer on top of the anodes over the entire deck to promote current flow and to lengthen the service lives of the anodes. A 50 mm (2 in.) thick wearing course of asphalt with conventional aggregate was placed on top of the coke-breeze asphalt layer. While coke breeze is a good electrical conductor, it is weak structurally. This overlay was still functioning after 11 years without any major changes, but did exhibit some signs of structural degradation.

The Ontario Ministry of Transportation later modified the conductive overlay system by replacing a portion of the coke breeze in the conductive overlay with conventional aggregate to improve stability and load resistance. In this system, a 40 mm (1.5 in.) conductive layer of coke-breeze asphalt is placed underneath a conventional asphalt wearing course of the same thickness. Cables to the primary anodes are placed in slots cut into the concrete deck (Nash et al. 1994).

The Ontario system has functioned well in several installations and is relatively inexpensive, but it does have some disadvantages. The overlay adds dead weight to the structure, and modifications are required to approaches, expansion joints, drains and curbs because of the increase in pavement height. Problems have included structural degradation of the overlay itself and freeze-thaw damage caused by moisture trapped underneath the overlay (Virmani and Clemena 1998).

**Slotted anode systems —** Slotted anode systems were developed in the mid 1970's and consisted of closely spaced platinized wires placed in slots in the deck and covered with a conductive backfill material. The cores of the wires were made of different materials, usually had a diameter of 0.78 or 1.6 mm (0.031 or 0.062 in.), and were covered with a platinum layer of 0.06 to 0.13 mm (0.025 to 0.050 in.). Slots were no more than 305 mm (1 ft) apart. A polymer grout has since replaced the backfill material as a cover for the anode wires (*Cathodic* 1993). Slotted anode systems have an advantage over conductive overlay systems because they do not require a thick overlay, which makes installation easier and reduces additional dead load on the structure (Virmani and Clemena 1998).

One of the first cathodic protection systems to use the slotted anode system was installed on a bridge deck in Virginia in 1983 (Clemena 1985). A rectifier supplied the protective current to each span via two primary anodes made of 0.78 mm (0.031 in.) diameter platinized niobium-copper wires, laid transversely in the deck. Secondary anodes, made of carbon strands, were spaced at 305 mm (1 ft) intervals across the width of the span to distribute the current longitudinally. The slots for both types of anodes, 13 mm (0.5 in.) wide and 19 mm (0.75 in.) deep, were sawed into the deck and filled with a conductive polymer grout. Problems encountered included added costs for cutting the 19 mm (0.75 in.) deep slots in the concrete, because the slots exposed some of the reinforcement in decks with shallow covers. Over time, the grout was found to break down as a result of acids produced by the anodic reactions [Eq. (4-1) and (4-2)].

Some of the grout eventually disbonded from the surrounding concrete deck. After 18 months at an average current density of about 17 $mA/m^2$ (1.6 $mA/ft^2$), a few small delaminations were detected in the deck, indicating that a higher current might have been needed.

Newer changes in slotted anodes include running the wire in both directions to form a grid to establish redundancy. Multi-filament carbon strands have a lower cost and higher tensile strength; titanium ribbon has also been tested (*Cathodic* 1993).

The Missouri Department of Transportation installed some cathodic protection systems using carbon fiber strands as secondary anodes with platinum primary anodes in the 1990s, but most of the systems “burned up” and were no longer working after only five years. Slotted anode systems using platinum primary anodes with a coke breeze epoxy secondary anode have performed better (Wenzlick 1999).

**Conductive polymer anode systems —** Another method of distributing impressed current cathodic protection to bridge decks involves placing platinized niobium-copper wires directly on the deck, in the same grid pattern as used for the slotted anode system, and then covering (mounding) the wires with conductive polymer concrete. The mound is usually about 33 mm (1.25 in.) wide and 12.5 mm (0.5 in.) high. Calcined petroleum coke breeze is broadcast over the mounds before the polymer concrete has cured to increase the conductivity of the polymer concrete overlay and to provide a good bonding surface. A non-conducting concrete or latex-modified concrete overlay is then placed over the entire deck (Virmani and Clemena 1998). This method was developed in hopes of preventing acid attack at the anodes and improving the distribution of current (*Cathodic* 1993). It is easier to install than slotted anode systems, but problems have been encountered at several installations with cracking of the overlays directly over the mounds (Swait and Rog 1987).

Conductive polymer anode overlays were developed to provide a system that could be applied directly on a bridge deck without requiring it to be scarified. A multi-layer resin-filled overlay system for impressed current cathodic protection was investigated in an FHWA study (Webster et al. 1987). The first two or three layers were made conductive by electrically conductive fillers and aggregates. The top layer contained regular aggregate to make it skid and abrasion resistant. Different combinations, using 16 different fillers and 18 different resins, were tested for electrical resistivity and durability, freeze-thaw durability, thermal coefficient, compressive and flexural strength, shear bond strength with concrete, and permeability to both gas and water. From these preliminary tests, 8 resins: 3 ortho-phthalic polyester, 3 iso-phthalic polyester, and 2 vinyl ester resins, and one filler: calcined coke breeze, were selected for extensive evaluation.

In the next stage of the study, overlays made with the selected resins were evaluated for their performance as a cathodic protection system, as well as for their weatherability. The results indicated that premixed polymers had more promise than the built-up systems as cathodic protection systems because early deterioration of the bond strength was detected in the built-up systems. The premixed systems demonstrated better bond and freeze-thaw durability, cured better, and were more permeable to gasses than the built-up overlays (Webster et al. 1987).

A field study was begun to test the premixed conductive polymer concrete overlay on a bridge deck. Two overlays were designed and installed on separate sections of a cathodically protected bridge deck in Virginia. Both overlays were 12.7 mm (0.5 in) thick. The first mixture consisted of 49.26% of both calcined coke breeze and silica sand by weight, a wetting agent, an initiator, and a resin blend consisting of 98.6% Hetron Q6305 (from Ashland Chemical Co.), a modified vinyl ester resin, by weight, a promoter, and a saline coupling agent. The second overlay consisted of 49.26% of both calcined coke breeze and crushed basalt by weight, a wetting agent, an initiator, and a resin blend consisting of 98% Polylite (from Reichold Chemical Co.) by weight, a promoter, a saline coupling agent, and a wetting agent (Webster et al. 1987).

The first overlay mix was torn out and replaced with the second mix within a year of installation. Depolarization tests indicated that the second system was functioning properly as a secondary anode after 18 months. At this time, the second overlay was found to have 5% delaminations. The average current density for the system was 0.9 mA/m$^2$ (1.05 mA/ft$^2$). This system has not been used in other applications, probably because of the development of commercial anodes that have been shown to be more durable (Virmani and Clemena 1998).

Missouri has used platinum anodes mounded with conductive polymer as an alternative to slotted anode cathodic protection systems. These anodes are overlaid when used on bridge decks, but some problems have been encountered on sidewalls, where overlays are not used. Over time, the anodes have tended to shrink away from the concrete, reducing the area of anode to concrete contact, and thus requiring the system to draw more current to protect the reinforcement. This problem has not been observed on bridge decks, probably because the overlay helps to hold the anodes down, but the current has needed to be increased on some bridge decks with systems using this type of anode (Wenzlick 2000).

**Grid anodes —** The first commercially available grid-type anode, the Raychem Farex 100 anode consisted of copper wires surrounded by a flexible, electrically conductive polymer to form an anode cable about 8 mm (0.31 in.) in diameter. These cables were woven into a panel, secured onto a scarified bridge deck and covered with a rigid overlay. The grids could also be used on vertical faces of piers and covered with shotcrete. However, difficulties were encountered when attempting to cover the anode cables on substructure members without getting any voids in the shotcrete, and disbondment of the anode with the shotcrete had been reported. The copper wire was also susceptible to acid attack, and exposed copper wire corroded, leaving an open circuit. This product was no longer manufactured after the late 1980s or early 1990s (Virmani and Clemena 1998).

Titanium mesh anodes consist of a titanium grid, coated with a metal oxide catalyst. 1.2 m (4 ft) wide panels of the mesh are secured to a scarified deck with plastic anchor bolts. The panels of titanium mesh are then electrically connected by spot welding a titanium strip across adjacent panels and then connecting the strip to a lead wire connected to the rectifier. A conventional or latex-modified concrete overlay is then placed over the deck (Virmani and Clemena 1998).

Titanium mesh anodes are expected to have long service lives in reinforced concrete because they have worked well on other applications that require much higher current outputs than needed for reinforced concrete. The anodes have relatively high anode/concrete area ratios, which means that they can distribute current across a structure more uniformly. Like the Ferex anode, titanium mesh anodes can also be used on substructure members, but they face the same problems with voids and disbondment of the shotcrete as the Farex anode. However, these problems may be less severe for a titanium mesh because it is thinner than the Farex anode. When used with a rigid overlay, titanium mesh anodes have generally performed better for reinforced concrete bridge decks than any other anode tested (Virmani and Clemena 1998).

Florida currently has a number of titanium mesh anode systems in operation on pilings, pile bents and footers on coastal bridges. Most of these systems have been performing as expected (Lasa 2000). In most cases, the titanium mesh is attached to the existing concrete and encapsulated with mortar or structural concrete. As a standard rehabilitation method for concrete pilings in Florida, a titanium mesh anode is placed inside a fiberglass pile jacket, fabricated to provide 76 mm (3 in.) of clearance between the jacket and the pile. The mesh is centered half way between the pile and the jacket. The pile jacket is filled with portland cement-sand mortar, and connection wires are routed to a rectifier. The first installation of this system has been in service, maintaining cathodic protection to the piles, since 1993. After energizing, polarized potentials of the steel reinforcement ranged from –0.780 to –0.990 V (CSE) with a steady state current of 6.4 mA/m$^2$ (0.6 mA/ft$^2$) (Kessler et al. 1999). Following the success of the first system, the system has been used successfully on at least 10 bridges in Florida (Lasa 2000).

Florida has also used titanium mesh anodes to provide cathodic protection to reinforcing steel when structural repairs are needed. The titanium anode is installed on the surface of the existing concrete and surrounded by a new layer of reinforcing steel, making sure that there is no physical contact between the new reinforcement and the anode. The old and the new reinforcement, along with the anode, are connected to wires for connection to the rectifier, forms are installed, concrete is placed around the anode and the new reinforcement, the wires are connected to the rectifier, and the system is energized. The first installation of this system was on eight pier footers in which significant chloride ion concentrations had caused severe corrosion and structural deficiencies. After nine years of service, no corrosion or deterioration was observed on the protected structure (Kessler et al. 1999).

The Missouri Department of Transportation uses the Elgard system, a titanium mesh coated with a metal oxide catalyst, on many of its cathodically protected bridge decks. The anode is fastened to the deck and then covered with an overlay, generally a dense low-slump concrete overlay. This system has performed well to date, and is currently cheaper than the platinum slotted anodes that are also used in the state. The manufacturer claims that the systems will have a 50 year service life, and Missouri expects to get at least 20 years of service life out of the systems that are in place (Wenzlick 2000).

**Conductive coatings as anodes for bridge substructure members —** Deterioration of concrete piers and other bridge substructure members due to corrosion caused by deicing salts leaking through joints or being splashed by traffic can also be prevented by cathodic protection. However, these situations usually require different types of anodes. The anodes on substructure members do not have to hold up under traffic wear, but they do have to be relatively easy to install on vertical surfaces, and liquid systems (systems that are sprayed or painted onto the structure) must have a consistency so that they do not to run down the surface of the members. The copper anode grid and titanium mesh discussed above have both been marketed for use on bridge substructure members, but they are better suited for use on bridge decks (Virmani and Clemena 1998).

A number of conductive sprayable secondary anodes have been developed for use in impressed current cathodic protection systems on bridge substructures. Primary anodes need to be fixed to the concrete to establish a connection between the DC cables and the anodes. The first primary anodes consisted of brass or copper plates fixed to the concrete with an epoxy that also acted as an insulator to prevent the plates from directly supplying current to the structure, which would cause localized high current areas and degradation of the plate (*Cathodic* 1998). Platinized wire has also been placed on the surface of the concrete as the primary anode (Nash et al. 1994). After the primary anode is fixed to the concrete, the entire surface was covered with a conductive secondary anode.

A sprayable conductive polymer coating was field-tested on two bridge piers in Virginia for an FHWA study (Fontana et al. 1987). After the primary anode was installed, a resin, coke filler, and additives were mixed and pumped to a spray nozzle through a hose while a catalyst was pumped to the same nozzle through another hose. The two components were mixed during the spraying operation, which was done until a thickness of 0.25 to 0.38 mm (10 to 15 mils) was reached. The coating remained suspended for several days and did not drip or sag when sprayed on vertical surfaces or overhangs. This system functioned satisfactorily, but has not been used outside of the field test in Virginia because equipment to mix and spray the resin mixture is not readily available (Virmani and Clemena 1998). This system only lasted about 5 years (Clemena 2000).

In 1983, metallized zinc was first used as a coating for cathodic protection of reinforced concrete piers. Zinc has a number of advantages as a coating for cathodic protection. Zinc coatings are relatively easy to apply to vertical surfaces, corners, and undersides of structures because the process is similar to spray painting. Because zinc coatings are very conductive, only a limited number of primary anodes are required, and the current is distributed efficiently over the structure. In field trials, this system had a higher tendency to create electrical shorts with chairs and tie wires than the other conductive coatings (Virmani and Clemena 1998). Thermally sprayed zinc is more tolerant than organic conductive coatings to moisture during both application and service; however, problems with adhesion can arise if the coating thickness is more than 0.2 mm (0.008 in.). Consumption rate calculations suggest that a 0.2 mm coating would completely oxidize in less than 6 years with a sustained current density of 20 mA/m$^2$ (Wyatt 1993). Currently, research is being conducted on a coating material that may rejuvenate metallized zinc anodes; this could increase the service life of these anodes to 20 years or more. No field studies have been performed yet (Clemena 2000).

As an alternative to thermal spray zinc anodes, thermal spray titanium anodes for impressed current cathodic protection have been studied in the laboratory and are currently being tested in several field trials. The biggest difference between zinc and titanium anodes is that zinc is a consumable anode. Zinc can be used as either an impressed current or a sacrificial anode, while titanium can only be used as an impressed current anode. Even though thermal sprayed titanium anodes are more expensive than thermal sprayed zinc anodes, they may prove to be more cost effective because the estimated service life of zinc anodes is 20 years while the estimated service life of titanium anodes is 40 years. Titanium anodes are also less sensitive than zinc anodes to low levels of relative humidity. The application of thermal spray titanium is similar to that of thermal spray zinc, except that titanium has a higher melting point, so voltage and current requirements to melt the metal for application are higher. The metallic coating formed by thermal sprayed titanium is not as uniform or as metallic as the coating formed by thermal sprayed zinc. While zinc is grayish silver and blends in well with concrete, titanium has a dark brownish gold color. This does not affect the operation of the system, but it does affect the aesthetics of the structure (Covino et al. 1999).

Paints, made conductive by the addition of carbon particles, are relatively inexpensive and can be applied with common tools including brushes, rollers, and sprayers. Field testing of these paints has indicated that they could have a service life of up to 15 years, as long as any deterioration that may occur is touched up shortly after it occurs (Virmani and Clemena 1998).

The fracture and separation of the top layer of the concrete next to the coating was identified as a common mode of failure for all of the conductive coatings discussed above. This occurs because acid is formed by the anodic reaction at the concrete-coating interface that decreases the adhesion of the coating. Corrosion products also contribute to coating failures by forming on the surface of metallic coatings (Funahashi et al. 1994).

### 4.2.6 Sacrificial Anode Systems

Some metals, such as aluminum, magnesium, and zinc, are anodic in comparison to steel. The electrical potential of these metals allows them to protect steel by galvanically supplying cathodic protection current without the aid of an outside power source. The driving voltage, and therefore the amount of protective current that is supplied by galvanic anodes in limited by the potential of that metal. For example, thermally sprayed zinc has been able to galvanically protect concrete in splash and tidal zones in seawater, but the anode must be periodically wet to supply adequate protection current. A problem with sacrificial anodes is that their protection current decreases with time, and they eventually become passive, so most systems have a relatively short useful life (Virmani and Clemena 1998).

Sacrificial anode cathodic protection systems are generally restricted to submerged structures, but have recently begun to be used on bridge substructures exposed to the atmosphere. Impressed current cathodic protection systems have an advantage in the fact that the power output can be controlled, but impressed current systems can sometimes overprotect a structure, reducing the life of the anode and causing hydrogen embrittlement. In contrast to impressed current systems, which require a relatively large number of electrical components and have increased monitoring and maintenance requirements, sacrificial anode systems do not require an external power source. The use of most sacrificial anode systems is limited to substructure members when used on bridges, but a few methods have been tested on bridge decks as well.

**Sacrificial anodes for bridge decks —** In 1977, sacrificial anodes were first tested on a bridge deck in Illinois as part of a National Cooperative Highway Research Program study. Two types of anodes were tested: perforated zinc sheets fastened to the deck with mortar and covered with a concrete overlay, and conventional zinc alloy ribbons embedded into grooves in the concrete surface. The concrete overlay on the zinc sheet system failed early on, and both overlays were replaced with free-draining asphalt overlays. These systems had to be removed in 1991 because of failure of the overlays. The researchers estimated that both anodes still had enough mass left to keep operating for many more years had the overlay held up (Virmani and Clemena 1998).

A laboratory study was conducted to continue the effort to identify the most promising anodes, using aluminum, magnesium, and zinc alloys (Whiting et al. 1995). After screening tests, three anodes were selected for further evaluation in concrete slabs. An aluminum mesh anode was tested on a bridge deck in a northern exposure area, but the system did not perform satisfactorily.

**Zinc-hydrogel anode systems —** A 1996 FHWA study investigated the ability of aluminum, aluminum alloy, and zinc sacrificial anodes in contact with several different commercial conductive hydrogel adhesives to deliver galvanic current to a cathodic protection system for bridge substructure members (Bennett and Shue 1996). The zinc anodes were included in the study because zinc has good current efficiency and is relatively inexpensive, while the aluminum and aluminum alloy anodes were included because they could, potentially, be 30% thinner, 74% lighter, and 65% less expensive than zinc. The study identified zinc as the best candidate for a hydrogel anode system. The tests indicated that the aluminum and aluminum alloy anodes either had unstable behavior or were not able to provide a high enough working potential when used with the hydrogel adhesives, and thus were not appropriate for this use.

The best combination of a conductive adhesive and a zinc sacrificial anode was installed in three structures for field testing. The three installations used zinc sheets, ranging in thickness from 10 to 20 mm (0.4 to 0.8 in.). The hydrogel used was a special formulation by 3M that had indicated in accelerated testing that it would be capable of passing a cathodic protection charge for about 11 years (Bennett and Shue 1996).

The first field system was installed in 1995 on the South Bridge Fishing Pier in Ft. Pierce, Florida. The zinc/hydrogel anode was placed on about 300 m$^2$ (360 yd$^2$) of the concrete surface, including 4 piles, 4 pile caps, and 2 double-tee beams. For the first 20 months of operation, the system met the cathodic protection polarization requirement of 100 mV, but between 9 and 20 months, the polarization of the system decreased from 120 mV to 108 mV. Another zinc/hydrogel anode system was installed on the Cape Perpetua Bridge in Yachats, Oregon in 1998. No data is available on this installation (Virmani and Clemena 1998).

In 1996, an experimental installation of zinc/hydrogel anodes was placed on 5 piers on the Long Key Bridge in the Florida Keys (Kessler et al. 1998a). The bridge was built in 1979 using epoxy-coated steel and had begun to exhibit signs of corrosion, especially on the pier caps, which sat on top of bearing pads that separated them from the piers. In most cases, it is impractical to provide cathodic protection to structures with epoxy-coated reinforcing steel, because electrical continuity is generally nonexistent. A sacrificial zinc sheet anode (zinc-hydrogel) was designed that could be slipped in-between the pier caps and the bearing pads to prevent further damage to the concrete in the pier caps. The pad was put into place by jacking up the pier caps about 1 inch. The bearing pads were replaced and a zinc anode was put in place between the new bearing pad and the pier cap concrete. A conductive adhesive gel was used to keep the sheet in place and to ensure contact with the concrete to be protected. Copper wires were run from the zinc sheet to the reinforcing bars to complete the circuit. The wires were coated with epoxy and the groves in the concrete were filled with mortar.

After about 5 months, the system appeared to be performing satisfactorily. The average current density was 10.8 $mA/m^2$ (1.01 $mA/ft^2$), calculated using the entire surface area of the protected reinforcement (Kessler et al. 1998a). According to Ivan Lasa with the Florida Department of Transportation (Lasa 2000), this system is still in place, but is no longer working. The failure of the system is attributed to errors made during installation. At that time, a good seal around the edges of the plates was not obtained, allowing water to get into the system and dissolve the hydrogel during a hurricane in 1999.

The Missouri Department of Transportation installed 3M zinc-hydrogel anodes to about 200 $m^2$ (240 $yd^2$) of concrete on four pier caps and three abutment beams on two bridges in Franklin County in November of 1999. As of February 2000, the measured current generated at each site was about 0.1 A (Wenzlick 2000). This translates to a current density of about 3.5 $mA/m^2$ (on the concrete surface), which is within the recommended range for good quality, dry, chloride contaminated concrete where the reinforcement is showing signs of light corrosion, as shown in Table 4.2 (*Cathodic* 1998).

Field studies of zinc/hydrogel anode systems show that the systems are relatively easy to install, without requiring a specialty contractor, and appear to be in good condition and well adhered to the concrete at ages up to 3 years. For the systems to work, the adhesive must be protected from direct exposure to water. This is achieved by caulking the edges of the plates (Virmani and Clemena 1998).

**Sprayed alloy anode systems —** A 1997 FHWA study concentrated on finding sacrificial anodes that could be applied by conventional metallization, flame or arc spraying (Funahashi and Young 1997). Fifteen different commercially available anodic materials including pure zinc, pure aluminum, and combinations of zinc, aluminum, and magnesium were tested for anode capacity and efficiency, cyclic polarization, atmospheric corrosion characteristics, susceptibility to temperature and humidity, and effect of pH on performance.

The anodes tested generally provided more current as temperature and relative humidity increased. All of the sacrificial anodes tested provided enough current to protect reinforcing steel at high temperatures (≥ 32° C) and high relative humidities (≥ 90%). However, the zinc alloys and most of the aluminum alloys did not produce enough current to protect the reinforcing steel in conditions of low temperature and low relative humidity. All of the anodes showed a decrease in current output and potential when the pH of the environment was decreased, especially when the pH was below 12. The decrease in current output and potential due to a decrease in the pH, temperature, or relative humidity was the least for the aluminum-zinc alloy.

The last part of the study included the development of a new alloy for this type of system, since none of the original materials had acceptable properties for use as a sacrificial anode for reinforced concrete, especially in low pH environments. Of the new anode materials, the best performance was provided by an aluminum-zinc-indium alloy. This alloy maintained relatively active static potentials in all environments and exhibited the lowest anode polarization of the alloys tested (Virmani and Clemena 1998). Low anode polarization is desirable because high anodic polarization means that the reaction is being driven by the anodic reaction, leading to faster dissolution of the anode (Jones 1996). Field trials of this system were performed in Florida, Pennsylvania, and Virginia (Virmani and Clemena 1998).

The Florida field trial consisted of two piles sprayed with Al-Zn-In and two piles sprayed with conventional pure zinc for comparison. Both anodes were applied at a thickness of 300 μm (12 mils). The system was monitored for two years, after which core samples were taken for evaluation. The field trial showed that the current output of the Al-Zn-In alloy anode was higher than that of the zinc anode and was probably more than necessary for protection of the reinforcing steel. Changes in temperature, relative humidity, and moisture content of the concrete caused the current output of both anodes to vary. The higher current output of the Al-Zn-In anode caused higher polarization, exceeding the 100 mV criteria for the entire 2 year test period. The zinc anode did not provide the required 100 mV polarization. Both anodes adhered well to the concrete for the duration of the test and were expected to have service lives of more than 15 years when applied with a thickness of 300 μm (12 mils) (Virmani and Clemena 1998).

**Zinc Mesh Pile Jacket Anodes —** This system was designed to provide cathodic protection to the splash area and submerged portion of reinforced concrete bridge pilings in marine environments. A zinc mesh pile jacket anode consists of a stay-in-place fiberglass form with an internal expanded zinc mesh anode. The anode is placed around the pile to be protected and the jacket is filled with a portland cement-sand mortar (Kessler et al. 1996). This system is currently in use and performing well with relatively low maintenance costs on a number of bridges in Florida (Lasa 2000).

The first experimental installation of this system was used on two substructure pilings on the Broward River Bridge in Jacksonville Florida in 1994 (Kessler et al. 1996). Before the anode was installed, corrosion deteriorated concrete, which was visible on both of the selected piles, was removed to expose the corroding steel. The exposed reinforcement was cleaned with a wire brush and the concrete surface was cleared of all marine growth and other accumulation within the jacket area.

The fiberglass jackets with the zinc mesh anode inside were placed around the piles, resting on temporary wooden platforms that had been placed at low tide elevation around each pile to support the jackets. A 76 mm (3 in.) deep seal of rapid-setting mortar was poured into the bottom of each jacket, the jackets were secured to the piles, and the forms were filled with a portland cement-sand grout with a minimum cement content of 558 kg/m$^3$ (940 lb/yd$^3$). A commercially available, 21.8 kg (48 lb) bulk zinc anode was then installed on the concrete 0.61 m (2 ft) below the low tide level on each pile. All connections to the steel reinforcement were made at a control panel located about 0.35 m (1 ft) above the top of the pile jacket.

The system is still operating, and the authors conclude that expanded zinc mesh pile jacket anodes provide an efficient method for protecting bridge pilings in marine environments. The cost of the system was comparable to that of standard pile jackets, yet the system was able to maintain cathodic protection to the reinforcement in the splash area as well as in the submerged portion of the piles. The authors estimated the service life of the system to be at least 45 years.

### 4.2.7 Field Studies

There have been a number of field studies performed to determine the effectiveness of various cathodic protection anode systems. Studies that used only one type of anode have been described in the discussion of the various anode systems. The following section discusses a study that investigated 5 different types of cathodic protection anode systems, a study that investigated the effectiveness of supplying the cathodic protection current with solar power, and a study of cathodic protection systems on prestressed concrete bridges.

**Big Spring, Texas —** For a 1994 FHWA study (Nash et al. 1994), 5 different impressed current cathodic protection systems were installed on a single structure in Big Spring, Texas in October of 1988. Three distributed anode systems were installed on the deck and two conductive coating systems were installed on the sidewalk and supporting bents. In each zone on the bridge deck, the concrete was milled 2 inches in traffic lanes, loose concrete was removed, and the reinforcing steel was checked to insure that it was all electrically connected prior to placement of the system. Exact amounts of salt applied to the bridge are not known, but in the period from December 1992 through December 1993, deicing salt was applied 19 times. Silver-silver chloride reference cells were used to take potential readings on the reinforcement. The systems were all controlled current systems, meaning that current was adjusted until readings from the reference cell indicated that the potential of the reinforcement was in the linear, all cathodic region of the E versus log I plot (see Figure 1.1). Currents were adjusted after 4 hour depolarization tests, performed after 90 days of operation of the systems.

The first distributed anode system used on the bridge deck was an expanded titanium mesh manufactured by Elgard (Elgard 150 titanium mesh). The titanium mesh was rolled out and tacked to the deck with insulated fasteners, which created a clearance of at least 6 mm (¼ in.) between imbedded metals and the anode. The mesh was reported to be very easy to install, but overlay placement was difficult because the mesh had a tendency to float in the concrete. This anode was still in place and working properly 5 years later.

The second distributed anode system installed on the deck was Raychem Farex 1000 Anode strand, manufactured by Raychem Corporation. The anode consisted of flexible conductive polymer strands that were connected to the concrete with special fasteners. This anode took a little longer to install than the titanium mesh anode, but presented no problems with the placement of the concrete overlay. After less than two years in service, the rectifier was no longer able to provide the anode with enough voltage to keep the current constant. After 1,100 days, significant current was not passing through any of the circuits. All of the circuits failed within 4 years of installation.

The third distributed anode system used on the bridge was a carbon strand system manufactured by Rescon (Rescon rigid conductive polymer concrete, carbon strand). The primary anode consisted of a platinized wire with carbon filament. The secondary anode was a carbon-based backfill material consisting of 65% coke-breeze that was mounded over the primary anode. The carbon strand system was difficult to install and the backfill produced toxic fumes, so placement took several days, but there were no problems installing the concrete overlay afterwards. In other tests, this method has led to discoloration of the concrete due to acid attack, but discoloration was not observed in this application. One strand was still in operation at the time of the report; the others failed after 4 to 5 years.

A hot sprayed zinc anode with a thickness of 500 μm ± 75 μm (20 ± 3 mils) was used on the sidewalks. Copper wires were used to distribute the current to the zinc. The entire system was then covered with a non-conductive surfacing, which cracked shortly after installation. The system failed in less than one year, probably due to disbondment and cracking of the zinc, but according to the authors, the failure could also be attributed to the prior condition of the sidewalk.

The final cathodic protection system used on the bridge, a conductive paint anode by Porter DAC-85, was installed on a pier cap. The primary anode was platinum wire, and the conductive coating was a fairly easy to install graphite/acrylic with a minimum thickness of 16 mils. This system also failed after about 4 years.

Calculations were performed to determine the cost effectiveness of each cathodic protection system used on the Big Spring bridge, compared to deck replacement or maintenance without using cathodic protection. Many assumptions had to be made regarding the maintenance costs of the cathodic protection systems and their service lives, but from the assumptions made, it was determined that in Big Spring, Texas, cathodic protection would not generally be a cost effective method for maintaining or protecting bridge decks. This conclusion may not hold true in all cases, however, because this bridge is on a low traffic volume road, where traffic control costs during repair would not be significant, thus making repair a viable option compared to cathodic protection (Nash et al. 1994).

**Solar Powered Cathodic Protection —** A study was performed to determine the effectiveness of an intermittent cathodic protection system using solar power (Kessler et al. 1998b). Rather than use batteries to store power to run the system during the night and periods of no sunlight, as done in previous systems, this system provided protection only when it was exposed to sunlight. If this method could be successful at preventing corrosion, it would be more attractive to owners and designers because of its lower cost and relative simplicity.

Two sets of laboratory specimens were tested. The first set consisted of rectangular concrete prisms with No. 13 (No. 4) bars as reinforcement. The second set was fabricated after observing the behavior of the first set and consisted of two scaled reinforced concrete piles. The specimens were tested over a period of three years. Over this time, the potentials gradually became more negative and no corrosion was observed in samples with cathodic protection, while control samples exhibited significant signs of corrosion. When measured, the chloride content of the cathodically protected samples was 29,000 and 24,000 ppm (about 70 and 55 kg/m$^3$) versus 59,000 ppm (about 140 kg/m$^3$) for the control specimen [the original chloride content of the concrete was 20,000 ppm (about 50 kg/m$^3$) for both cases]. Currents of up to 3 times that of regular corroding specimens were observed in the samples immediately after disconnection from the current source, but since no corrosion was observed, it was assumed that this current was driven by the diminishment of polarization films that form around the bars during periods of cathodic protection. These currents diminished and were considered negligible after 1 to 2 hours (Kessler et al. 1998b).

As part of the study, field tests were run on 10 pilings on a bridge in an aggressive marine environment in northern Florida. The bridge had an existing impressed current titanium mesh cathodic protection system that was disconnected for 8 weeks and then reenergized using photovoltaic cells. After about 175 days of service, depolarization was measured at 131 and 145 mV, using two embedded silver/silver chloride reference electrodes, indicating adequate protection. Kessler et al. (1996b) concluded that, although depolarization will occur during periods of no cathodic protection, adequate polarization levels can be maintained when sufficient current is supplied to the system. However, it is important for the system to be calibrated when exposed to maximum sunlight to avoid overprotection from increases in current, and measures should be taken to impede the depolarization of the system during times of no sunlight.

**Cathodic Protection of Prestressed Concrete Structures —** In an FHWA study published in 1998 (Bennett and Schue), selected anode systems were tested for cathodic protection in prestressed concrete. Different anodes were tested on prestressed box-beams located in snow belt states and in prestressed concrete beams and piles in marine environments. The cathodic protection systems used included: galvanic and impressed current metallized zinc coating systems on prestressed beams, an impressed current titanium mesh with conductive rubber or with a cement grout jacket on prestressed piles, and an impressed current conductive paint system and a metallized zinc coating that functioned galvanically, and then as an impressed current system, on the soffit of prestressed box beams.

The systems and bridge components were evaluated after the cathodic protection systems had been operated and monitored for 32 and 37 months. When steel and concrete specimens from the concrete bridge members were examined after of treatment, no adverse effects on the structural integrity of the concrete adjacent to the steel and no alteration in the chemistry or mineralogy of the concrete in contact with the steel were found. This led to the conclusion that cathodic protection did not effect the bond between the prestressing steel and the concrete. Even though some specimens were selected from sites where steel potentials were frequently very negative (less than -900 mV versus SCE), the ductility of all of the prestressed steel specimens tested after cathodic protection was relatively high, consistent with little or no occurrence of hydrogen embrilttlement. Examination of the specimens after cathodic protection found a reduction in the concentration of chloride ions around the surface of the embedded steel reinforcement due to the application of cathodic protection current, which has also been observed in other studies.

The authors concluded that, while applied current and voltage must be monitored carefully, cathodic protection could effectively control the corrosion of prestressing steel in pretensioned concrete bridges. For structures with large variances in concrete resistance, it might be difficult to achieve sufficient protection in areas of high resistivity while preventing conditions for hydrogen generation from occurring in areas of low resistivity. Differences in concrete resistivity are usually caused by variations in moisture content, but can also be influenced by depth-of-cover, chloride content, anode contact resistance, and distance from the anode power feed. The study also concluded that cathodic protection systems were best operated in the constant voltage mode, with the operating voltage selected so that the potential required for hydrogen generation would never be reached at the surface of the prestressing steel. This can also be accomplished in the constant current mode by limiting the rectifier voltage.

### 4.2.8 Summary

Conclusions on the performance of various cathodic protection systems based on the studies reviewed include:

- Cathodic protection can effectively stop corrosion in contaminated reinforced concrete structures and can reduce the concentration of chloride ions at the surface of protected reinforcement.
- Currently, the most common impressed-current anode in use for cathodic protection of reinforced concrete bridge decks is the titanium mesh anode, used in conjunction with a concrete overlay. This anode fills the need for a durable impressed-current cathodic protection anode on bridge decks and pilings.
- Zinc mesh pile jacket anodes show promise as sacrificial anodes for the splash zone of bridge piles in marine environments. Zinc-hydrogel anodes can provide protection for substructure members in marine or inland environments, as long as water can be kept out of the system.
- Cathodic protection can be applied effectively and safely to prestressed concrete bridge members. However, if the resistivity of the concrete is not uniform, it may be difficult to obtain sufficient protection at locations where resistivity is high

without generating hydrogen in areas of low resistance. Cathodic protection is not recommended for prestressed concrete structures with very diverse resistivity, which is often caused by large variations in moisture content.

- When applying cathodic protection to prestressed concrete, the voltage should be kept low enough that the potential required for hydrogen generation at the surface of the prestressing steel (<900 mV versus SCE) is never reached. This is best done by operating the cathodic protection system in the constant voltage mode.

## 4.3 ELECTROCHEMICAL CHLORIDE EXTRACTION

A common repair method for chloride contaminated concrete structures is to remove and replace all of the contaminated concrete from around the reinforcement. There are several disadvantages to this method. Partial replacement of the concrete can lead to changes in the strength of the structure and set up potential differences in the reinforcement, which can increase the rate of corrosion. Removal of the old concrete can also cause damage to the reinforcement and expose workers to dust and noise. Electrochemical chloride extraction (also called desalination) has been developed as an alternative repair method in which chloride ions are pulled out of contaminated concrete, allowing the concrete to stay in place (*Electrochemical* 1998).

### 4.3.1 Description

Electrochemical chloride removal is the process of removing chloride ions from contaminated concrete by electrochemical means (Virmani and Clemena 1998). This method was first studied in the mid-1970s by the Kansas Department of Transportation (Morrison et al. 1976). However, electrochemical chloride removal methods were not implemented at that time because the high levels of current that were used in the early studies were found to increase the permeability of the concrete, decrease the concrete-to-steel bond, and cause cracking in the concrete. More recent studies have shown that electrochemical chloride removal can be applied effectively at current levels on the concrete surface below 5 $A/m^2$, for which no adverse effects on the concrete are observed. These treatments removed 20 to 50% of the chloride ions from the concrete, and redistributed the rest away from the reinforcement (Clemena and Jackson 1997).

Electrochemical chloride extraction is similar to cathodic protection. The reinforcement is connected to the negative pole of a DC power source, while a temporary external anode, placed within an electrolyte covering the concrete surface, is connected to the positive pole. Current is run through the reinforcement to the anode, creating an electrical field in which chloride ions are pulled away from the reinforcement and towards the electrolyte where they are absorbed for removal. The chloride ions move through the concrete by migration at the relatively high current densities that are applied to achieve electrochemical chloride removal [1 $A/m^2$ of concrete surface (typically 0.4 to 2.0 $A/m^2$ of reinforcement) versus a range of 0.0001 to 0.05 $A/m^2$ of reinforcement for cathodic protection]. The rate of removal of chloride ions depends on the current flow, which varies with both the condition of the concrete and the location of the reinforcement. Chloride ions migrate faster in the zone directly between the reinforcement and the anode, and migrate progressively slower as the distance from that band increases. Therefore, chloride migration is slowest within the cover region at the midpoint between reinforcing bars, and relatively little chloride is removed from the concrete beneath the reinforcement (*Electrochemical* 1998).

The potential difference between the cathode (the reinforcement) and the anode causes a number of electrochemical reactions (*Electrochemical* 1998). The electrolyte can be any of a number of different solutions, including saturated calcium hydroxide, sodium borate, sodium hydroxide, or tap water. If the electrolyte is an alkaline solution, the hydroxyl ions are converted to oxygen gas and water at the anode according to Eq. (4-5).

$$4\ OH^- \rightarrow O_2 + 2\ H_2O + 4\ e^- \qquad (4\text{-}5)$$

If the electrolyte is a relatively neutral solution, such as water, the water is converted to oxygen gas and hydrogen ions at the anode, as shown in Eq. (4-6).

$$2\ H_2O \rightarrow O_2 + 4\ H^+ + 4\ e^- \qquad (4\text{-}6)$$

In this case, if the electrolyte does not contain any salts, the pH of the solution increases as the hydrogen ions migrate towards the reinforcement until they meet up with the hydroxyl and chloride ions that are moving towards the anode (Polder 1994). The hydrogen ions combine with the hydroxide ions to form water, and with the chloride ions to form hydrochloric acid.

Excess chloride ions are released in the form of chlorine gas at the anode [Eq. (4-7)].

$$2\ Cl^- \rightarrow Cl_2 + 2\ e^- \qquad (4\text{-}7)$$

The reactions in Eq. (4-6) and (4-7) are not desirable because hydrochloric acid attacks the concrete and because chlorine gas constitutes a health hazard. As a result, an alkaline electrolyte is usually used (*Electrochemical* 1998).

At the steel reinforcement, hydroxyl ions are formed in the presence of oxygen and water [Eq. (4-8)].

$$O_2 + 2\ H_2O + 4\ e^- \rightarrow 4\ OH^- \qquad (4\text{-}8)$$

If oxygen is in limited supply, hydrogen gas is generated in addition to the hydroxyl ions [Eq. (4-9)].

$$2\ H_2O + 2\ e^- \rightarrow H_2 + 2\ OH^- \qquad (4\text{-}9)$$

If enough chlorides are removed from the concrete to drop the chloride concentration below the threshold level at the steel, the increase in hydroxyl ions will help the steel surface to repassivate (*Electrochemical* 1998).

Electrochemical chloride extraction does have some limitations. It can only be used in cases where there is little damage (spalls and delamination) to the concrete surface, and where the corrosion of the reinforcement does not yet effect the safety of the structure (Elsener et al. 1997b). Spalls and cracks need to be repaired conventionally or sealed prior to the application of electrochemical chloride extraction to prevent the electrolyte from coming into contact with the reinforcement and causing a short circuit (*Electrochemical* 1998). A number of side effects are also possible with electrochemical chloride extraction.

### 4.3.2 Possible Side Effects

With electrochemical chloride extraction, the possibility exists of initiating the alkali-silica reaction as a side effect, as alkali metal ions are rearranged and hydroxyl ions are generated at the reinforcement. The pH necessary to initiate alkali-silica reaction is not known, but it can be assumed that if a structure is suffering from corrosion due to salt water ingress, the concentration of sodium ions may be above the threshold for sustaining alkali-silica reaction. Studies have indicated that the alkali-silica reaction can be mitigated by lithium (Velivasakis et al. 1997).

Trials have shown that high cathodic current densities (above 5 $A/m^2$) applied to the reinforcement during electrochemical chloride removal may move the potential of the steel into the hydrogen evolution region. When hydrogen is generated around the steel, it can either form gas molecules, or be absorbed back into the steel, causing hydrogen embrittlement. Because hydrogen embrittlement can cause brittle failure (principally in high strength steels), electrochemical chloride removal should not be used on structures with prestressing steel that is in direct contact with the concrete. There is still some debate as to whether electrochemical techniques can be used on post-tensioned structures, where some research has indicated that a metallic duct can shield the reinforcement from the applied current. “Because of the high risks involved [electrochemical chloride extraction and electrochemical realkalisation] on post-tensioned structures should be applied today only in collaboration with an expert.” (Elsener et al. 1997b).

### 4.3.3 Field Studies

Electrochemical chloride extraction was tested on a 28-year old bridge deck in Arlington, Virginia as part of a pilot study (Clemena and Jackson 1997). All damaged concrete on the bridge was repaired, and the system was installed, first on two spans in the northbound direction (areas 1 and 2), and then on the same two spans in the southbound direction (areas 3 and 4). Traffic was maintained, running both directions on the half of the bridge that was not being treated. Electrical continuity was established between the reinforcing bars in each span, which were then attached to four ground connections. To prevent electrical shorts, cementitious grout was applied to all surface cracks and all areas where the cover over the reinforcement might have been thin.

Four different electrolytes were used in the four different test areas of the study (see Table 4.3). The anode was a layer of catalyzed titanium mesh between two layers of synthetic felt, surrounded by a wooden dam, and kept wet with electrolyte. The felt was laid out over each 7.3 m (24 ft) by 24.8 m (81.3 ft) test area. Five 1.2 m (48 in.) and one 0.6 m (24 in.) wide titanium mesh strips were laid out along each test area with a gap of 80 to 110 mm between the strips, and then another layer of felt was laid over the top of the strips. Each test area was covered with a 250 μm (10 mil) thick black plastic sheet to protect the anode system and to minimize the evaporation of the electrolyte. A soaker hose was routed around each area and connected to a small pump to circulate the electrolyte. Each area was divided into two zones, each with 3 anode strips. The ends of the titanium mesh strips in each zone were connected with the positive terminal of a rectifier using copper lead wires. Two rectifiers, each having a maximum output rating of 150 A, at 40 V, and requiring a single-phase 220 V AC line, were used for each area. The four ground connections in each area were then connected to the negative terminals of the rectifiers with copper lead wires. When the rectifiers were switched on, they were operated in the constant-voltage mode, with the DC output set as high as possible without exceeding 40 V and a current density of 1 A/m$^2$ of concrete.

In most cases, the voltage was kept constant, and the current density passing through each area decreased as treatment progressed. Table 4.3 presents the average voltages and initial and final current densities, as well as the electrolyte used, in each test area on the bridge. The decreases in current density observed in this study also occur in cathodic protection systems, and result from increasing resistance caused by the electromigration of different ions into and out of the concrete (Clemena and Jackson 1997).

**Table 4.3 Electrochemical Chloride Extraction** (Clemena and Jackson 1997)

| Area | Size (m$^2$) | Electrolyte Used | Time (days) | Average Voltage (V) | Current Density (mA/m$^2$ of concrete surface) | | Total Charge (A-hr/m$^2$) |
|---|---|---|---|---|---|---|---|
| | | | | | Initial | Final | |
| 1 | 174 | $LiBO_3$, $H_2O$ | 57 | 30.2 | 925 | 662 | 1,077 |
| 2 | 183 | $LiBO_3$, $(C_6H_5)_4PCl$, $H_2O$ | 57 | 32.5 | 914 | 593 | 1,033 |
| 3 | 180 | $Ca(OH)_2$, $H_2O$ | 58 | 29.3 | 854 | 541 | 1,019 |
| 4 | 183 | $Ca(OH)_2$, $H_2O$ | 58 | 36.6 | 730 | 427 | 741 |

Once the system was turned on, treatment lasted for about 8 weeks. During this time, the dams were monitored for possible leakage. The output voltages and currents of the rectifiers, the current passing through each of the lead wires, and the pH of the electrolytes were measured about once a week. The pH of the electrolyte needed to be adjusted during the first few weeks, when the formation of acid around the anode, due to an accumulation of $H^+$ ions produced at the anode and chloride ions that had migrated out of the concrete, was the most noticeable. During this time, the pH of some of the electrolytes fell as low as 3 because of the accumulation of $H^+$ ions. This is a problem because, if the pH of the electrolyte is acidic for too long, it can cause softening of the cement paste on the surface of the concrete. The pH was adjusted by spreading lime over the areas, and in addition, on areas 3 and 4, water was fed continuously into each span instead of containing and recirculating the electrolyte within the dams, as was done for areas 1 and 2.

Measurements of concrete surface hardness indicated that some softening occurred during treatment, but the decrease was statistically significant in only area 2, where the measurements indicated that the relative surface hardness had been reduced by 8.9%.

Concrete samples were taken from each test area both before and after treatment and used to estimate the quantity of chloride ions that had been removed by the electrochemical chloride extraction. Twelve sampling points were selected in each area, and concrete samples were taken from two depths at each point, 6 to 19 mm (0.25 to 0.75 in.) and 19 to 32 mm (0.75 to 1.25 in.). No measurements were taken below 32 mm (1.25 in.) because the depth of the transverse reinforcement ranged from 19 to 44 mm (0.75 to 1.75 in.) and electrochemical chloride extraction is not expected to remove chloride ions from under reinforcement.

Comparisons of the chloride concentration in the spans both before and after treatment show a significant decrease at both sampling depths. The amount of chloride removed from the concrete varied from 75.8 to 82.1% at the 6 to 19 mm depth and from 72.2 to 81.7% at the 19 to 32 mm depth. Current efficiencies ranged from 11 to 15%. Current efficiency is defined as the fraction of the total applied charge carried by the amount of chloride ions that were removed (the rest of the current was carried by other mobile ions in the concrete and the electrolyte). The results indicate that current efficiency was probably effected more by differences between the concretes in the two spans (areas1 and 3 versus 2 and 4) than it was by the different electrolytes. Table 4.4 presents the chloride ion concentrations in the deck before and after treatment.

It is not practical to remove all of the chloride ions in a structure using electrochemical chloride extraction, especially if the initial level is high, but as can be seen in Table 4.4, the chloride concentration in a structure can be substantially reduced. Before treatment, all of the concrete samples from the bridge deck had chloride ion concentrations above the threshold level for corrosion used for this study [0.77 kg/m$^3$ (1.3 lb/yd$^3$)] at the 19 to 32 mm level, but after treatment, only 20 to 33% of the samples still had chloride ion concentrations above the threshold at that level. Even reinforcement at locations with chloride concentrations above the threshold would not necessarily be in immediate danger of corroding because, in addition to reducing Cl$^-$ ions, electrochemical treatment increases the concentration of OH$^-$ ions at the reinforcement, which helps to passivate the bars.

**Table 4.4 Average Chloride Contents in the Concrete Before and After Electrochemical Chloride Extraction Treatment** (Clemena and Jackson 1997)

| Area | Average Cl$^-$ Content (kg/m$^3$) at depth | | | | | Change (%) | Efficiency (%) |
|---|---|---|---|---|---|---|---|
| | 6 to 19 mm | | Change (%) | 19 to 32 mm | | | |
| | Before | After | | Before | After | | |
| 1 | 5.20 | 1.04 | -80.0 | 2.68 | 0.59 | -78.0 | 11.2 |
| 2 | 5.92 | 1.06 | -82.1 | 3.78 | 0.69 | -81.7 | 15.0 |
| 3 | 5.03 | 1.07 | -78.7 | 3.05 | 0.71 | -76.7 | 11.7 |
| 4 | 4.97 | 1.20 | -75.8 | 2.34 | 0.65 | -72.2 | 14.2 |

After treatment, core samples were taken from each area and examined petrographically for signs of abnormality in the concrete. The concrete was in good condition and free of cracks, with the aggregate tightly bonded to the cement paste. The wearing surfaces (the top of the pavement that had been exposed to traffic wear) of the cores taken from areas 3 and 4 had a slight loss of paste around the fine aggregate particles that was likely caused when the electrolyte became acidic, before the problem was solved by flushing some of the electrolyte and adding fresh water and lime to the systems. A limited amount of alkali-silica reaction was observed in the cores from three of the zones, but it is likely that the alkali-silica reaction activity had taken place before the treatment. Although lithium ions had migrated into the concrete (in areas 1 and 2), it was not known if they mitigated the alkali-silica reaction, or if they were even necessary (Clemena and Jackson 1997). A SHRP study (Bennett et al. 1993) found that applying electrochemical chloride extraction to a pier containing alkali-silica reaction-susceptible aggregate without using lithium did not aggravate alkali-silica reaction activity.

The cost of treatment for the four areas was $92,412. A total of 720 $m^2$ of concrete was treated, for a unit cost of $128.35/$m^2$. The high cost was attributed to a lack of competitive, experienced applicators of the method, since there was only one contractor qualified for the job at the time of the study (Clemena and Jackson 1997).

According to Clemena (2000), Virginia stopped using electrochemical after 5 trial applications, 2 on bridge decks and 3 on piers, including the one described above. The systems worked best on the bridge decks, but most agencies can not afford to close a bridge deck, or to limit traffic flow for 8 weeks at a time, which is required for treatment, and the treatment is not as effective on the piers as it is on the decks. Once these problems are overcome, electrochemical chloride extraction will probably see more use.

Other trials of the electrochemical chloride extraction treatment method have had similar results. In the summer of 1989, the Ontario Ministry of Transportation conducted an electrochemical chloride extraction trial on the Burlington Skyway, constructed in 1955 (Manning and Ip 1994). The treated area was a 30.8 $m^2$ section near the base of a column in one of the piers. Three faces of the column were treated using the NORCURE™ process, while the fourth face served as a control. NORCURE™ is a commercial electrochemical migration process developed in Europe. The anode was steel mesh encased in an electrolyte of cellulose fiber moistened with lime-water. The rectifier output was limited to 40 V, which resulted in an initial current density of 0.77 A/m$^2$ of concrete surface and a final current density of 0.26 A/m$^2$ of concrete surface at the end of the eight week treatment. After the treatment was finished, the surface of the concrete was cleaned by blasting with an abrasive but was not sealed because the source of chloride ions had been blocked.

The treatment lowered the chloride content of the concrete to below the corrosion threshold level at the reinforcing steel and shifted corrosion potentials into the passive range. Samples taken at the end of the treatment indicated that between 42% and 87% of the available chloride ions had been removed from the concrete. When measured again one year later, the chloride ion concentrations were found to be the same as they had been at the end of the treatment, within the anticipated sampling error. The potential measurements were divided into three ranges: passive [less negative than –200 mV versus a copper-copper sulfate electrode (CSE)], uncertain (-200 to –350 mV CSE), and active corrosion (more negative than –350 mV CSE) (Manning and Ip 1994). Potential measurements indicated that the corrosion potential of the reinforcement had been reduced to the passive range over most of the treated faces during the first year after treatment. There was even a reduction in the potential on the control face, indicating that it was also effected by the treatment. In the seven years following treatment, the potential readings remained stable, with no significant changes and no potentials shifting back into the active corrosion range within the treated areas. Table 4.5 presents the measured corrosion potentials before and for 7 years following treatment of the column. The west, south, and east faces received the electrochemical chloride extraction treatment, while the north face was not treated (Velivasakis et al. 1997).

**Table 4.5 Corrosion Potential Measurements on the Burlington Skyway** (Velivasakis et al. 1997)

| Time from treatment | mV versus copper-copper sulfate electrode, negative sign omitted, percentage of readings within range | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | North (not treated) | | | West (treated) | | | South (treated) | | | East (treated) | | |
| | < 200 | 200 to 350 | > 350 | < 200 | 200 to 350 | > 350 | < 200 | 200 to 350 | > 350 | < 200 | 200 to 350 | > 350 |
| Before | 0 | 85 | 15 | 0 | 96 | 4 | 0 | 96 | 4 | 0 | 84 | 16 |
| 1 year | 41 | 59 | 0 | 98 | 2 | 0 | 100 | 0 | 0 | 100 | 0 | 0 |
| 2 years | 41 | 59 | 0 | 100 | 0 | 0 | 100 | 0 | 0 | 98 | 2 | 0 |
| 3 years | 26 | 74 | 0 | 96 | 4 | 0 | 100 | 0 | 0 | 96 | 4 | 0 |
| 4 years | 26 | 70 | 4 | 98 | 2 | 0 | 100 | 0 | 0 | 96 | 4 | 0 |
| 5 years | 19 | 74 | 7 | 96 | 4 | 0 | 96 | 4 | 0 | 96 | 4 | 0 |
| 6 years | 26 | 59 | 15 | 96 | 4 | 0 | 93 | 7 | 0 | 93 | 7 | 0 |
| 7 years | 30 | 63 | 7 | 96 | 4 | 0 | 96 | 4 | 0 | 91 | 9 | 0 |

In another study, electrochemical chloride extraction was completed on 24 reinforced concrete columns that formed the substructure of an overpass in Saskatchewan. After treatment, the chloride content in the concrete was reduced by 62 to 89% in areas where significant chlorides had been present. The chloride content at the reinforcement was reduced to below the corrosion threshold, and the corrosion potential readings had been significantly reduced. Before treatment, the corrosion potentials in the most severely contaminated areas of the columns had been as high as –500 mV (CSE), but after treatment, none of the measurements were more negative than –214 mV (CSE) (Velivasakis et al. 1997).

### 4.3.4 Summary

The studies described above indicate that electrochemical chloride extraction can successfully remove substantial amounts of chloride from contaminated concrete, and lead to an increase in the pH of the concrete and repassivation of corroding reinforcing steel. Studies have indicated that adverse side-effects due to electrochemical chloride extraction can be avoided as long as current densities are kept below 5 A/m$^2$ of concrete surface (Bennett et al. 1993). Studies have demonstrated that a current density of less than 1 A/m$^2$ of concrete surface is sufficient for treatment (Clemena and Jackson 1997, Manning and Ip 1994). However, because of the risk of hydrogen generation adjacent to the reinforcement, electrochemical chloride extraction should not be used on prestressed concrete structures (Velivasakis et al. 1997). Uncertainty exists as to how long the benefits of electrochemical chloride extraction on a structure will last, so estimating the life cycle cost of treatment is difficult (Virmani and Clemena 1998).

# CHAPTER 5
# CORROSION INHIBITORS

## 5.1 GENERAL

Corrosion inhibitors are chemicals that can slow down or prevent corrosion of reinforcing steel in concrete. Corrosion inhibitors were first investigated in the 1960’s. Some early inhibitors included sodium nitrite and the sodium and potassium salts of chromate and benzoate. Studies found that the sodium and potassium salts reduced the strength of the concrete and gave mixed results on corrosion inhibition. However, other inhibitors have shown promise as methods for protecting reinforced concrete from corrosion damage (Virmani and Clemena 1998). A common inhibitor used today, calcium nitrite, was developed to be used in concrete as a noncorrosive set accelerator (Berke and Rosenberg 1989).

Today, corrosion inhibitors are often used in conjunction with other corrosion protection systems, such as epoxy coated steel. In places, such as the Florida Keys, that have experienced problems with epoxy-coated steel, inhibitors are used as an alternate corrosion protection system. Inhibitors are also used in prestressed concrete applications, where epoxy-coated steel is not often used. Corrosion inhibitors are added to the grout used to fill post-tension ducts (Virmani and Clemena 1998).

Corrosion inhibiting admixtures are unique among the methods to protect reinforced concrete from corrosion damage due to the fact that their protection mechanism becomes an integral part of the concrete matrix (Berke et al. 1988). A corrosion inhibitor is defined as a substance that will prevent or minimize corrosion. Most inhibitors act by chemically stabilizing the steel surface, although some also act to reduce the permeability of the concrete. Corrosion inhibitors are generally used as admixtures in concrete for new construction, but they can also be used for repairs by being admixed into concrete for patches, sprayed or painted onto the surface of the concrete or applied by saturation treatment (Virmani and Clemena 1998).

Corrosion inhibitors are typically divided into three categories: anodic inhibitors, cathodic inhibitors, and organic inhibitors. Anodic inhibitors, made up of chromates, nitrites, molybdates, alkali phosphates, silicates, and carbonates, act by minimizing the anodic part of the corrosion reaction. These inhibitors form an insoluble protective film on anodic surfaces to passivate the steel. Some anodic inhibitors, such as nitrites, can cause accelerated corrosion and pitting if they are not used in large enough quantities. Cathodic inhibitors, consisting of zinc, salts of antimony, magnesium, manganese, and nickel, form an insoluble film on the cathodic surfaces of the steel. They are usually less effective than anodic inhibitors, but are also safer. Organic inhibitors, including amines, esters, and sulfonates, block both the anodic and cathodic reaction on the entire surface of the metal (Virmani and Clemena 1998). Organic corrosion inhibitors have been used in the oil and gas industries for years, but they were not developed for use in reinforced concrete until the early 1990s. (Nmai et al. 1992) Organic inhibitors can also act as pore blockers, reducing the permeability of the concrete.

## 5.2 CALCIUM NITRITE

Currently, there are only a few commercially available corrosion inhibiting admixtures. The most common example of an anodic inhibitor is calcium nitrite, which has been commercially available as Darex Corrosion Inhibitor (DCI and DCI-S) from W. R. Grace since 1978. DCI-S is simply DCI plus a concrete set retarder (Pyc et al. 1999). Calcium nitrite is now also available from Master Builders as Rheocrete CNI. Both products are approximately 30% calcium nitrite and 70% water. To function as an inhibitor, enough calcium nitrite must be added to the concrete mix to counteract both the effect of chloride ions and the fact that calcium nitrite is water soluble and will leach out of the concrete over time. The typical dosage is 10 to 30 $l/m^3$ (2 to 6 $gal/yd^3$) of concrete, depending on the expected chloride exposure level (Virmani and Clemena 1998). Table 5.1 gives the recommended calcium nitrite dosages needed to protect reinforcement from corrosion for different chloride ion contents.

Because calcium nitrite raises the corrosion threshold at the reinforcement rather than blocking chloride ions from entering concrete, it is important that quality concrete be used to keep the chloride level at the reinforcement from getting too high. A maximum w/c ratio of 0.45 is recommended for concrete with calcium nitrite (Kaiser 2000).

**Table 5.1 Calcium Nitrite Dosage Required for Protection Against Chloride-Induced Corrosion** (Bentur et al. 1997)

| Calcium Nitrite Dosage ($l/m^3$) | Chloride Ion Concentration at Level of Reinforcement ($kg/m^3$) |
|---|---|
| 10 | 3.6 |
| 15 | 5.9 |
| 20 | 7.7 |
| 25 | 8.9 |
| 30 | 9.5 |

Calcium nitrite is also used during repair of corrosion-damaged structures. When concrete is removed, calcium nitrite is applied directly to exposed reinforcing steel and concrete, as well as being used in the replacement concrete.

Calcium nitrite reduces corrosion in the presence of chlorides by competing with the chloride ions reacting with the steel, stabilizing the passive film and increasing the concentration of chloride ions necessary to initiate corrosion in the concrete by the following reaction:

$$2Fe^{2+} + 2OH^- + 2\ NO_2^- \rightarrow 2\ NO + Fe_2O_3 + H_2O \qquad (5\text{-}1)$$

A disadvantage of calcium nitrite is that it acts as a set accelerator. Most of the time, a set retarder must be added to the concrete mix to counteract the acceleration of the calcium nitrite (Sherman et al. 1993, Elsener et al. 1997a).

## 5.3 ORGANIC CORROSION INHIBITORS

Master Builders manufactures the organic inhibitor Rheocrete 222$^+$ (previously Rheocrete 222). The active ingredient of Rheocrete 222$^+$ is a water-based combination of amines and esters. According to the manufacturer, this mixture forms a protective film on reinforcing steel, which serves as a physical barrier to both the anodic and cathodic corrosion reactions. This mixture also reduces the ingress of chloride ions into the concrete by lining the concrete pores with hydrophobic chemical compounds. The recommend dosage of Rheocrete 222$^+$ for any chloride exposure level is 5 l/m$^3$ (l gal/yd$^3$) of concrete (Virmani and Clemena 1998).

Studies have shown that the addition of organic corrosion inhibitors does have an effect on some of the properties of concrete. The addition of Rheocrete 222$^+$ to a mix may require more mixing time or the addition of more air-entraining agent to achieve the desired air content. The compressive strength of concrete is also affected by the addition of Rheocrete 222$^+$ (Nmai et al. 1992). Darwin and Hadje-Ghaffari (1990) found that the addition of Rheocrete 222 reduced the compressive strength of concrete cylinders by 15 to 20% after 7 days. This reduction in compressive strength can be corrected by increasing the cement content of the concrete.

FerroGard 901 and 903, manufactured by SIKA, are water-based blends of surfactants and dimethyl ethanolamine, an amino alcohol. FerroGard 901 is a liquid admixture, and FerroGard 903 is designed to be applied to and to penetrate hardened concrete surfaces. SIKA has shown through x-ray photoelectron spectroscopy and secondary ion mass spectroscopy that the amino acids absorb on the steel bars when a bond is formed between its amino functional group and the hydroxide group on the steel surface. This causes the formation of insoluble iron oxide complexes that inhibit corrosion by stabilizing the oxide surface (Virmani and Clemena 1998). The recommended dosage of Ferrogard 901 is 10 $l/m^3$ (2 $gal/yd^3$) of concrete, except in areas of severe marine exposure, where the recommended dosage is 15 $l/m^3$ (3 $gal/yd^3$). The recommended application rate for FerroGard 903 is 0.27 - 0.41 $l/m^2$ (0.06 – 0.09 $gal/yd^2$) for structures with chloride ion concentrations at the reinforcement level up to 3.5 $kg/m^3$ (6 $lb/yd^3$).

Another corrosion inhibitor from SIKA is Armatec 2000, an organic inhibitor made up of a mixture of alcohol and amine. The recommended dosage for Armatec 2000 is 2.5 $l/m^3$ (0.5 $gal/yd^3$) of concrete (Pyc et al. 1999). Both Ferrogard and Amaratec 2000 are considered to be migrating inhibitors. Although the measured diffusion rates of these inhibitors have varied, these inhibitors have been shown to migrate through concrete to the reinforcing steel when applied to the concrete surface. These inhibitors act by forming a film on the steel surface, blocking both the anodic and cathodic corrosion reactions (Elsener et al. 1997a).

The last commercially available corrosion inhibiting admixture to be discussed is Catexol 1000CI, manufactured by Axim Concrete Technologies. Catexol is probably a combination of both an organic and a nitrite inhibitor, but specific data has not been released by the manufacturer (Virmani and Clemena 1998).

## 5.4 PREVIOUS STUDIES

With the exception of DCI, the inhibitors described above were introduced during the 1990s. As a result, the time since the introduction of these corrosion inhibiting admixtures has not been adequate to provide for an assessment of their field performance. Most of the data on the inhibitors comes from laboratory studies (Virmani and Clemena 1998).

The FHWA started an outdoor exposure study to investigate the effectiveness of calcium nitrite as a corrosion inhibiting admixture in 1980 (Virmani et al. 1983). Thirteen slabs were made containing two mats of black steel bars and varying amounts of admixed chloride and 2.75% calcium nitrite (by weight of cement) in the top lift. Five control slabs were made containing no calcium nitrite. The reinforcement was evaluated by measuring the macrocell corrosion current, half-cell potential, driving voltage, concrete electrical resistivity, and by visual inspection.

The study concluded that calcium nitrite seemed to be functioning as a corrosion inhibitor by preventing a large electrical potential difference from developing between the bars. Calcium nitrite was effective in reducing the corrosion rate in black steel bars at chloride-to-nitrite ratios of 1.79 and less. The study determined that at a chloride-to-nitrite ratio of 1.25, it would take at least 10 years for the same amount of iron to be consumed as would be consumed in 1 year in concrete with the same chloride concentration and no inhibitor (Virmani et al. 1983).

After 7 years of observation, the maximum ratio of chloride to nitrite ions necessary to reduce the rate of corrosion in the steel was reduced to 0.90. Specimens with ratios of 1.11 showed signs of cracking and rust spots, even though the corrosion current was still about 10 times lower than that of the control specimens (Virmani 1990). These ratios could have been affected by the fact that the chloride ions were added to the concrete at the same time as the calcium nitrite, but the authors still concluded that protection could be provided by maintaining the ratio of chloride-to-nitrite ions at less than 1.0 at the level of the steel throughout the life of the structure (Virmani and Clemena 1998).

Calcium nitrite (DCI-S) and an organic corrosion inhibitor (Rheocrete 222) were evaluated as part of a 1995 study by Senecal et al.. The inhibitors were evaluated for two steel alloys using Southern Exposure and cracked beam tests, as well as rapid corrosion potential and macrocell tests. To increase the permeability of the concrete, the water-cement ratio used in the study was 0.5, which is higher than the maximum ratio recommended for use with calcium nitrite but considered to be acceptable for the organic corrosion inhibitor (Berke et al. 1993). The average macrocell corrosion rates in the Southern Exposure tests were the same for both inhibitors for one type of steel, while for the other type, the average corrosion rate was five times higher in the specimen with calcium nitrite than in the specimen with the organic inhibitor. The results from the cracked beam tests were similar.

Specimens with either inhibitor had lower corrosion rates than regular concrete specimens, but overall, the organic inhibitor performed better than the calcium nitrite in these tests (Senecal et al. 1995).

A study of the effectiveness of corrosion inhibiting admixtures was published by the Virginia Transportation Research Council in 1999 (Zemajatis et al.). Three types of commercially available corrosion inhibiting admixtures (DCI-S, Armatec 2000, and Rheocrete 222) and one type of galvanized steel were tested in concrete specimens. The specimens were subjected to weekly cycles consisting of 3 ½ days ponding followed by 3 ½ days drying with a 6% sodium chloride solution.

As the tests progressed, chloride samples showed that chloride ion concentrations at the reinforcement level exceeded the corrosion threshold in all of the specimens and that there was no significant difference in the rate of chloride ingress between the control specimens and specimens containing corrosion inhibitors (Zemajatis et al. 1999). Potential measurements and measured chloride concentration levels indicated that active corrosion began in the samples containing Armatec 2000 and Rheocrete 222 after about 33 weeks of exposure. According to the same criteria, active corrosion began in the control specimens after about 40 weeks of exposure, and in the DCI-S specimens after about 65 weeks (Pyc et al. 1999).

The study concluded that DCI had increased the concentration of chloride ions necessary to initiate corrosion of reinforcement in concrete, but that Rheocrete 222 and Armatec 2000 did not appear to provide any corrosion inhibition to the specimens (Zemajatis et al. 1999).

A paper, published in 1992 by three engineers at Master Builders (Nmai et al. 1992) discussed the results of ponding tests on concrete slabs containing an organic corrosion inhibiting admixture and calcium nitrite. Control slabs, with no corrosion inhibitors, were also evaluated.

The test specimens consisted of 76 x 102 x 356 mm (3 x 4 x 14 in.) concrete beams reinforced with a single No. 13 [No. 4] steel bar. Three mixes were used, containing the equivalent of 5.0 $l/m^3$ (1.0 $gal/yd^3$) of organic corrosion inhibiting admixture, 20.0 l/m$^3$ (4 gal/yd$^3$) of calcium nitrite, or no corrosion inhibitor. All mixes had a water/cement ratio of 0.50. The specimens were notched across the top width at the midpoint and stressed flexurally to induce cracks to the level of the reinforcement [about 38 mm (1.5 in.)]. The tops of the beams were diked and continuously ponded with a 6% sodium chloride solution.

Half-cell potentials were measured periodically with copper-copper sulfate electrodes (CSE). Active corrosion was considered to be occurring at potentials more negative than –240 mV CSE. Using this criteria, the control specimens began to corrode after about 5 days of exposure, the calcium nitrite specimens began to corrode after about 30 days, and the specimens containing the organic inhibitor showed no signs of corrosion activity after 180 days. The study concluded that while the organic corrosion inhibitor did not raise the corrosion threshold in concrete as calcium nitrite did, the organic inhibitor decreased the corrosion rate more because it reduced the ingress of chloride ions into the concrete.

## 5.5 SUMMARY

Research has shown that the corrosion inhibitor calcium nitrite can extend the service life of concrete structures. However, the tendency of calcium nitrite to act as a set accelerator should be taken into account in the design process. Because calcium nitrite reduces corrosion by chemically reacting with the steel, the effectiveness of calcium nitrite is dependent on the ratio of chloride-to-nitrite ions, which should be kept below 1.0 for the life of the structure. Enough calcium nitrite must be added to the concrete mix to account for the fact that calcium nitrite is water soluble and will leach out of the concrete over time and the fact that chloride ion concentrations will generally increase over time. Organic inhibitors have been shown to limit the number of chloride ions that reach the steel by providing a physical barrier, and therefore, the same dosage is recommended regardless of the anticipated maximum chloride concentration in the concrete. Information on the inhibitors other than calcium nitrite and Rheocrete 222$^+$ (FerroGard 901 and 903, Ameratec 2000 and Catexol 1000CI) is limited (Virmani and Clemena 1998).

# CHAPTER 6

# ECONOMIC ANALYSIS

## 6.1 GENERAL

The cost of any corrosion protection system for a concrete structure is a combination of the initial cost of the system, and any maintenance, operation, and/or repair costs that occur within the service life of the structure (Bentur et al. 1997).

## 6.2 LIFE-CYCLE COST ANALYSIS

Most bridges are designed for 40 year service lives; however, for the purpose of economic evaluation, an economic life of 75 years is used to compare systems with useful lifetimes in excess of 40 years. For the economic analysis in this study, a typical 230 mm (9 in.) thick bridge deck with 50 mm (2 in.) of concrete cover over the top mat of steel is used, with the following exceptions: a thickness of 205 mm (8 in.) is used for bridges with no corrosion protection and black steel reinforcement, and a thickness of 216 mm (8.5 in.) is used for bridges with epoxy-coated steel reinforcement as the only corrosion protection method. When overlays are included, the total deck thickness with the overlay remains 230 mm (9 in.).

Times to repair and current costs were obtained for the various corrosion protection systems from recent literature and by talking to engineers at state and province transportation agencies. Engineers at the Kansas Department of Transportation were consulted about repair times and the costs of materials for many of the methods.

Some of the times to repair and replacement of the decks are estimates because many of the corrosion protection methods evaluated have not been in use long enough for chloride ions to diffuse into the structure and initiate corrosion at the reinforcement.

### 6.2.1 Repair

The time to repair of a concrete structure due to corrosion is dependent on both the time required for chloride ions to reach the corrosion threshold in the concrete adjacent to the reinforcement and the time required for damage to occur after corrosion is initiated. For bridges in the United States [assuming 50 mm (2 in.) cover], it usually takes 2 to 5 years from the time corrosion is initiated before significant damage occurs and needs to be repaired (Bentur et al. 1997).

Bridge deck repairs generally consist of removing contaminated concrete to below the top mat of reinforcement, cleaning the bars (and sometimes coating them with epoxy or a corrosion inhibitor), and placing new concrete. In cases of severe damage, full depth repairs are made, in which all of the concrete in an area is removed, the reinforcement is cleaned and possibly coated with epoxy or a corrosion inhibitor, a form is positioned under the deck, and new concrete is placed in the repair area. It is standard practice for the Kansas Department of Transportation to place a silica fume overlay over bridge decks as part of the repair operation.

Bid tabulations and information on 27 bridges in the state of Kansas that received repairs and overlays in 1999 were reviewed. The bridges averaged 36 years in age at the time of repair. Twelve of the bridges were reinforced concrete slabs on girders, and the other 15 were haunched slabs. Review of these records found that an average of 23% of the total deck area received partial depth repair, and an average of 6% of the deck area received full depth repair. These numbers were fairly consistent between the two deck types, with average partial and full depth repairs of 22% and 6%, respectively, for slab on girder bridges, and 25% and 6%, respectively, for haunched slab bridges. The average costs for partial and full depth repairs were about 20% and 10% higher, respectively, on the haunched slab bridges than on the bridges with slabs on girders. The costs for repairs on slabs on girders are used in the current economic analysis. The repair costs for the 27 bridges are summarized in Table A.1.

Whenever a repair is necessary on a bridge, a significant portion of the total cost of the repair comes from incidental costs, rather than the actual repair or material costs. Incidental costs include mobilization, traffic control, and repairs and improvements to other parts of the bridge, such as drains, barrier rails, and approaches. An average cost for these incidentals was determined for this study using the bid tabulations. The material replacement costs, including machine preparation, area prepared for patching (full and partial depth), and a 40 mm thick silica fume overlay, were subtracted from the total cost of the repairs to determine the cost of the incidentals. Costs for other major repairs done at the same time, such as painting, expansion joints, or rockers, were not included in the total costs.

The incidental costs for the 27 bridges reviewed in Kansas for 1999 averaged \$130/m$^2$, with a minimum of \$31/m$^2$ and a maximum of \$280/m$^2$. The average incidental costs for girder and haunched slab bridges, respectively, were \$ 126/m$^2$ and \$133/m$^2$. Material replacement costs ranged from 32% to 80% of the total repair cost, with an average of 53%, regardless of the bridge type (girder or haunched slab). Incidental costs are not as dependent as material replacement costs on bridge size. The two bridges on which the percentage of the material replacement cost was the highest and the incidental costs were the lowest were both significantly (about 2.5 x) larger than the average deck size. On all but one of the two largest bridges, the cost to repair and overlay the bridge deck exceeded the average cost for a new bridge deck with black steel in Kansas.

There is another cost associated with repairs on bridges that is not considered in this economic analysis. This is the cost to users when a bridge is closed for repair or replacement, which includes time lost due to delays or detours, accidents, and other resources used. These costs can make the price of repairing or replacing a bridge considerably higher than the price used in this analysis.

### 6.2.2 Current Costs

The costs used in the economic analysis are presented in Tables 6.1 – 6.4, along with the calculations that were used to determine current costs for each corrosion protection system. The present value, along with new, repair, and replacement costs for each method is illustrated in Figure 6.1. All figures are for in-place costs, in 1999 U.S. dollars.

The base cost for a 230 mm (9 in.) thick conventional concrete deck with black steel reinforcement was calculated by converting the in-place cost of concrete for superstructures in Kansas, \$425 per $m^3$, to a cost of \$97/$m^2$ [Calc. 6.1 (A) – the number refers to the table], and converting the in-place cost of black steel reinforcement, \$1.30/kg, to a cost of \$43/$m^2$ [Calc. 6.3 (G)], based on an average density of reinforcement of 32.7 kg/$m^2$, calculated using two Kansas bridge decks [Calc. 6.3 (A and D)]. These values are added to obtain a cost of \$140/$m^2$ for a new, 230 mm (9 in.) thick bridge deck with black steel reinforcement [Calc. 6.3(H)].

**Table 6.1 Concrete Prices and Calculations for Concrete Costs**

| Concrete type | Price obtained from: | Price |
|---|---|---|
| Class AAA (substructure) | Kansas | \$400/$m^3$ |
| Class C (abut.) | Texas | \$544-612/$m^3$ |
| Class C HPC (bent) | Texas | \$892-974/$m^3$ |
| Grade 30 (superstructure) | Kansas | \$425/$m^3$ |
| Class C (0.53 w/c) (slab) | Texas | \$360/$m^3$ |
| Class S (0.44 w/c) (slab) | Texas | \$460/$m^3$ |
| Class B | Vermont | \$333-405/$m^3$ |
| Silica fume concrete | Vermont | \$650-980/$m^3$ |
| Silica fume | W.R. Grace | \$1.10/kg |

A — The concrete in a standard deck in KS [230 mm (9 in.) thick] will cost:

→ $0.230\ m^3/m^2 \times \$425/m^3 = \$97/m^2$

B — Deck with black steel, no protection, and minimum (50 mm) cover will be 205 mm (8 in.) thick:

→ $0.205\ m^3/m^2 \times \$425/m^3 = \$87/m^2$

### Table 6.1 Continued — Concrete Prices and Calculations for Concrete Costs

C — Deck with epoxy-coated steel will be 216 mm (8.5 in.) thick:

→ 0.216 $m^3/m^2$ x $\$425/m^3$ = $\$92/m^2$

D — Standard overlay in Kansas is 40 mm (1.5 in.), over 190 mm (7.5 in.) of conventional concrete. The 190 mm of conventional concrete will cost:

→ 0.190 $m^3/m^2$ x $\$425/m^3$ = $\$81/m^2$

Overlay costs are presented in table 6.2.

E — Silica fume concrete for a 230 mm (9 in.) thick bridge deck will cost:

→ 0.230 $m^3/m^2$ x $\$700/m^3$ = $\$161/m^2$

### Table 6.2 Overlay Prices and Calculations for Overlay Costs

| Type of overlay | Price obtained from: | Thickness | Price |
|---|---|---|---|
| Low-slump concrete overlay | Missouri | 45 mm | $\$58/m^2$ |
| | Iowa | 50 mm | $\$23\text{-}132/m^2$ |
| Silica fume concrete overlay | Kansas | 40 mm | $\$54/m^2$ |
| | Missouri | 45 mm | $\$32\text{-}54/m^2$ |
| Latex-modified concrete overlay | Missouri | 45 mm | $\$66/m^2$ |
| Epoxy-Polymer overlay | Missouri | 7 mm | $\$47/m^2$ |
| Bituminous concrete pavement | Ontario | | $28/metric ton |
| | New Hampshire | | $35/ton |
| | New Hampshire | 50mm | $\$3.85/m^2$ |

For this study:

— Cost used for low slump concrete overlay is $\$30/m^2$ for a 50 mm thick overlay

— Cost used for a silica fume concrete overlay is $\$54/m^2$ for a 40 mm thick overlay

**Table 6.1 Continued — Overlay Prices and Calculations for Overlay Costs**

- Cost used for a latex-modified concrete overlay is \$66/m$^2$ for a 45 mm thick overlay
- Cost used for an epoxy-polymer overlay is \$47/m$^2$ for a 7 mm thick overlay

A — Assuming that asphalt has a specific gravity of 2.4 (this is conservative), an 80 mm (3 in.) asphalt concrete overlay (used in Canada) will cost:

$\rightarrow$ 2.4 x 1000 kg/m$^3$ = 2400 kg/m$^3$

$\rightarrow$ 2400 kg/m$^3$ x \$0.028/kg = \$66/m$^3$

$\rightarrow$ \$66/m$^3$ x 0.08 m$^3$/m$^2$ = \$5.32/m$^2$

**Table 6.3 Reinforcement Prices and Calculations for Slab Costs**

| **Type of reinforcement** | **Price obtained from:** | **Cost (per kg)** |
|---|---|---|
| Grade 420 (black) | Kansas | \$1.30 |
| | Vermont | \$1.21-1.54 |
| Grade 420 (epoxy-coated) | Kansas | \$1.45 |
| | Vermont | \$1.32-1.76 |
| Stainless steel | Virginia | \$5.51 |
| | Ontario | \$3.45-6.90 |
| Stainless steel-clad | Virginia | \$1.76 |
| Galvanized | North American Galvanizing Co. | \$1.63 |

A — Two bridge decks in Kansas:

- Deck 1$\rightarrow$ 356.7 m$^2$ concrete and 52,358 kg of steel $\rightarrow$ 147 kg/m$^3$
- Deck 2 $\rightarrow$ 358.5 m$^2$ of concrete and 50,421 kg of steel $\rightarrow$ 141 kg/m$^3$
- Standard deck thickness in KS is 230 mm (9 in.)
  $\rightarrow$ 0.230 m$^3$/m$^2$
- Average 143 kg of reinforcement per m$^3$ of concrete

**Table 6.3 Continued — Reinforcement Prices and Calculations for Slab Costs**

B — For a 205 mm deck with 143 kg of reinforcement per $m^3$ of concrete:

→ 0.205 $m^3/m^2$ x 143 $kg/m^3$ = 29.3 $kg/m^2$

C — For a 216 mm deck with 143 kg of reinforcement per $m^3$ of concrete:

→ 0.216 $m^3/m^2$ x 143 $kg/m^3$ = 30.9 $kg/m^2$

D — For a 230 mm deck with 143 kg of reinforcement per $m^3$ of concrete:

→ 0.230 $m^3/m^2$ x 143 $kg/m^3$ = 32.7 $kg/m^2$

E — Cost of black steel reinforcement in a 205 mm deck with 29.3 kg of reinforcement per $m^2$ of concrete

→ 29.3 $kg/m^2$ x\$1.30/kg =\$38/$m^2$

F — Standard 205 mm (8 in.) slab with black steel reinforcement in Kansas

→ \$87/$m^2$ + \$38/$m^2$ = \$125/$m^2$

G — Cost of black steel reinforcement in a 230 mm deck with 32.7 kg of reinforcement per $m^2$ of concrete

→ 32.7 $kg/m^2$ x \$1.30/kg = \$43/$m^2$

H — Standard 230 mm (9 in.) slab with black steel reinforcement in Kansas

→ \$97/$m^2$ + \$43/$m^2$ = \$140/$m^2$

I — Cost of epoxy-coated steel reinforcement in a 216 mm deck with 30.9 kg of reinforcement per $m^2$ of concrete

→ 30.9 $kg/m^2$ x \$1.45/kg = \$45/$m^2$

J — Standard 216 mm (8.5 in.) slab with epoxy-coated steel reinforcement in Kansas

→ \$92/$m^2$ + \$45/$m^2$ = \$135/$m^2$

K — Cost of epoxy-coated steel reinforcement in a 230 mm deck with 32.7 kg of reinforcement per $m^2$ of concrete

→ 32.7 $kg/m^2$ x \$1.45/kg = \$47/$m^2$

L — Standard 230 mm (9 in.) slab with epoxy-coated steel reinforcement in Kansas

→ \$97/$m^2$ + \$47/$m^2$ = \$144/$m^2$

**Table 6.3  Continued — Reinforcement Prices and Calculations for Slab Costs**

M — Cost of stainless steel reinforcement in a 230 mm deck with 32.7 kg of reinforcement per $m^2$ of concrete

$\rightarrow$ 32.7 kg/$m^2$ x \$5.51/kg = \$180/$m^2$

N — Standard 230 mm (9 in.) slab with stainless steel reinforcement in Kansas

$\rightarrow$ \$97/$m^2$ + \$180/$m^2$ = \$277/$m^2$

O — Cost of stainless steel-clad reinforcement in a 230 mm deck with 32.7 kg of reinforcement per $m^2$ of concrete

$\rightarrow$ 32.7 kg/$m^2$ x \$1.76/kg = \$58/$m^2$

P — Standard 230 mm (9 in.) slab with stainless steel-clad reinforcement in Kansas

$\rightarrow$ \$97/$m^2$ + \$58/$m^2$ = \$155/$m^2$

Q — Cost of galvanized reinforcement in a 230 mm deck with 32.7 kg of reinforcement per $m^2$ of concrete

$\rightarrow$ 32.7 kg/$m^2$ x \$1.63/kg = \$53/$m^2$

R — Standard 230 mm (9 in.) slab with galvanized reinforcement in Kansas

$\rightarrow$ \$97/$m^2$ + \$53/$m^2$ = \$150/$m^2$

**Table 6.4 Repair Prices and Calculations for Deck Repair Costs**

| Type of repair | Price obtained from: | Cost |
|---|---|---|
| Remove concrete (slab) | Texas | $\$7/m^2$ |
| Remove bridge pavement | Vermont | $\$6\text{-}12/m^2$ |
| Machine preparation of area | Kansas | $\$21/m^2$ |
| Cold plane bituminous concrete | Vermont | $\$24/m^2$ |
| Full depth repair (about 6% of deck) | Kansas | $\$267/m^2$ |
| | Iowa | $\$418/m^2$ |
| | Missouri | $\$335/m^2$ |
| | Texas | $\$400/m^2$ |
| Partial depth repair (about 22% of deck) | Kansas | $\$190/m^2$ |
| | Iowa | $\$101/m^2$ |
| | Missouri | $\$309/m^2$ |
| | Texas | $\$430/m^2$ |

A — Deck repair (no overlay on original deck): 22% partial depth + 6% full depth + machine preparation + silica fume overlay + incidental costs

$\rightarrow \$190/m^2 \times 0.22 + \$267/m^2 \times 0.06 + \$21/m^2 + \$54/m^2 + \$126/m^2 = \$259/m^2$

B — Replace overlay (includes repair): remove concrete + 22% partial depth + 6% full depth + machine preparation + replace overlay + incidental costs

$\rightarrow \$7/m^2 + \$190/m^2 \times 0.22 + \$267/m^2 \times 0.06 + \$21/m^2 + \$126/m^2 +$ overlay

$\rightarrow \$212/m^2$ + overlay

C — Asphalt replacement over membrane: cold plane old asphalt and replace top 40 mm

$\rightarrow \$24/m^2 + \$5.32/m^2 \times 40\text{ mm}/80\text{ mm} = \$27/m^2$

**Table 6.4 Continued — Repair Prices and Calculations for Deck Repair Costs**

D — Membrane replacement: cold plane old asphalt + cost of membrane + cost of asphalt overlay + incidental costs

→ \$24/m$^2$ + cost of membrane + cost of asphalt + incidental costs

### 6.2.3 Cost Analysis

This section contains the prices, calculations, people and research consulted, and estimates made to determine the construction and repair costs for standard bridge decks and times to repair and/or replacement. The number at the beginning of each section corresponds with the number for the same corrosion protection option illustrated in Figure 6.1 and Table A.2, which show construction and repair costs and times, and present value costs for each system. The calculations used to establish costs are cited (from tables 6.1, 6.2, and 6.3) for each case and shown in detail for selected deck/corrosion protection systems.

1. NO PROTECTION – BLACK STEEL REINFORCEMENT

According to McCrumb and Arnold (1993), corrosion damage in unprotected decks usually appears after 10 to 15 years in service. Bentur et al. (1997) predicted that a deck with 50 mm (2 in.) of cover over black steel would need repair after 20 years. According to Dan Scherschligt and Ken Hurst at the Kansas Department of Transportation, most bridges with black steel reinforcement (50 mm cover) and no corrosion protection do not need to be repaired until after they have been in service for at least 25 years. Twenty-five years is used as the time to repair for this study.

The average low bid cost for in-place black steel in Kansas for 1999 was \$1.30/kg. The price for concrete was \$425/m$^3$ (Rezayazdi 2000). The total cost for a new 205 mm (8 in.) thick bridge deck was calculated as \$125/m$^2$, using calculations 6.1(B), and 6.3(D, E, and F).

6.1(B) — Concrete price for a 205 mm (8 in.) thick deck with black steel, no protection, and minimum (50 mm) cover

→ 0.205 $m^3/m^2$ x $425/$m^3$ = $87/$m^2$

6.3(D) — Density of black steel reinforcement in a 205 mm deck with 143 kg of reinforcement per $m^3$ of concrete

→ 0.205 $m^3/m^2$ x 143 kg/$m^3$ = 29.3 kg/$m^2$

6.3(E) — Cost of black steel reinforcement in a 205 mm deck with 29.3 kg of reinforcement per $m^2$ of concrete

→ 29.3 kg/$m^2$ x $1.30/kg =$38/$m^2$

6.3(F) — Standard 205 mm (9 in.) slab with black steel reinforcement in Kansas

→$87/$m^2$ +$38/$m^2$ =$125/$m^2$

When the thickness of the deck is increased to 230 mm (9 in.), as is done when a corrosion protection method, such as an overlay, is used, the base cost for a bridge deck with black steel reinforcement increases to $140/$m^2$, and can be determined from calculations 6.1(A), and 6.3(D, G, and H).

The repair cost was determined in calculation 6.4(A) and is used for all repairs. Whenever a bridge deck in Kansas is repaired, a 40 mm silica fume overlay is also placed on the deck, so unless otherwise noted, a 40 mm silica fume overlay is included in all repair costs for this study.

6.4(A) — Deck repair: 22% partial depth + 6% full depth + machine prep. + silica fume overlay + incidental costs

→ $190/$m^2$ x 0.22 + $267/$m^2$ x 0.06 +$ 21/$m^2$ +$54/$m^2$ + $126/$m^2$ = $259/$m^2$

## 2. EPOXY-COATED STEEL REINFORCEMENT

Three bridges with epoxy-coated reinforcement in Indiana were still in good condition after 20-23 years (the oldest had 0.68 % distressed area). Bentur et al.

(1997) predict that the first repair will be required after 32 years. Tests in Virginia showed that coating disbonds after 15 years (Pyc et al. 2000). For this study, it was estimated that repairs will be required after 30 years.

The average low bid cost for in-place epoxy-coated steel in Kansas for 1999 was \$1.45/kg (Rezayazdi 2000). Calculations 6.1(C) and 6.3(C, I and J) were used to arrive at a cost of \$135/m$^2$ for a 216 mm (8.5 in.) bridge deck with conventional concrete and epoxy-coated steel.

When other corrosion protection methods, such as overlays are combined with epoxy-coated reinforcement, the thickness of the deck is increased to 230 mm (9 in.). In this case, the base cost increases to \$144/m$^2$, as can be determined from calculations 6.1(A), and 6.3(D, K, and L).

## 3. STAINLESS STEEL REINFORCEMENT

Both solid stainless steel and stainless steel-clad reinforcement have service lives of more than 40 years [75-100 years for solid bars, according to McDonald et al. (1998)]. According to Gerry Clemena at the Virginia Department of Transportation, the in-place cost for solid stainless steel reinforcing bars is \$5.50/kg (this is the number used in the cost calculations). According to David Manning at the Ontario Ministry of Transportation, the in-place cost can drop as low as \$3.45/kg if large quantities are ordered (> 10 tons), but can be up to two times that amount if smaller quantities are used.

Calculations 6.1(A), and 6.3(D, M, and N) are used to obtain a cost of \$277/m$^2$ for a 230 mm (9in.) bridge deck with conventional concrete and solid stainless steel. Decks with solid stainless steel reinforcement are not expected to require any repairs for more than 75 years.

## 4. STAINLESS STEEL-CLAD REINFORCEMENT

Stainless steel-clad reinforcement costs \$1.76/kg (in-place) according to Clemena (2000). The corrosion rate of stainless steel-clad reinforcement is 2 orders of magnitude lower than the corrosion rate for black steel (Darwin et al. 1999). Decks with stainless steel-clad reinforcement are estimated to have a service life of 75 years.

Calculations 6.1(A), and 6.3(D, O, and P) are used to obtain a cost of \$155/m$^2$ for a 230 mm (9 in.) bridge deck with conventional concrete and stainless steel-clad reinforcement.

## 5. GALVANIZED REINFORCEMENT

According to McCrumb and Arnold (1993), galvanized reinforcement will add 5 years to the time to corrosion damage compared to black steel. For this study, the time to repair for bridges with galvanized reinforcement is estimated to be 27 years.

The North American Galvanizing Company charges \$0.33/kg to galvanize wire reinforcement, which would bring the in-place cost for galvanized reinforcement to \$1.63/kg.

Calculations 6.3(A), and 6.3(D, Q, and R) are used to arrive at a cost of \$150/m$^2$ for a 230 mm (9 in.) bridge deck with conventional concrete and galvanized steel reinforcement.

## 6. SILICA FUME CONCRETE

According to Bentur et al. (1997), a bridge deck constructed with full depth silica fume concrete will need its first repair after 25 years, assuming 50 mm (2 in.) cover and a diffusion coefficient of 1.03 x 10$^{-12}$ m$^2$/s. Miller and Darwin (2000) found the mean effective diffusion coefficient of silica fume concrete overlays in Kansas to be between 1.04 x 10$^{-12}$ and 1.97 x 10$^{-12}$ m$^2$/s, so the time to repair could be shorter. Because this study assumes 75 mm (3 in.) cover, a time to repair of 30 years was estimated.

According to Kaiser (2000), silica fume costs \$1.10/kg. Assuming a 238 kg/m$^3$ concrete mix with 7.8% silica fume by mass of cement, the material price of the concrete would increase by \$19.60/m$^3$. Vermont bid sheets listed the price of silica fume concrete from \$650 to \$980/m$^3$, so a price of \$700/m$^3$ was assumed for calculations.

Calculations 6.1(E), and 6.3(D and G) are used to determine a price of \$161/m$^2$ for the silica fume concrete and \$43/m$^2$ for the black steel reinforcement, for a total cost of \$204/m$^2$ for a 230 mm (9 in.) thick deck.

## 7. LOW-SLUMP OVERLAY

According to Jim Grove at the Iowa DOT, low-slump overlays have been in place for up to 30 years and are still performing well (the first one placed was removed after 23 years). The service lives of these overlays was estimated to be 25 years when, during repairs, about half of the surface of the deck was excavated to beneath the reinforcement before the overlay was placed (Chamberlin 1988). A time to repair of 30 years is used for this study because the overlay is applied to new decks, before chloride ions have a chance to infiltrate the concrete.

The 1999 bid costs for low-slump concrete overlays (2 in.) in Iowa ranged from a low of \$23/m$^2$ (\$19/yd$^2$) to a high of \$132/m$^2$ (\$110/yd$^2$) with an average price of \$44/m$^2$ (\$37/yd$^2$). A total of 37,320 m$^2$ (44,630 yd$^2$) of low-slump concrete overlays were placed on bridge decks in Iowa in 1999. The large difference between the low and high costs is a matter of “economies of scale”. The high costs likely come from small bridges on county roads where the mobilization costs are high and the bridge is relatively small, i.e.: 20 ft long, versus a 1 mile long interstate bridge. However, the small bridge will have advantages in other costs, because traffic control costs will likely be much lower (Jack 2000). Because this analysis is based on all new construction, mobilization costs are not as significant, so a price of \$30/m$^2$, which is below the average, is used.

These overlays are 50 mm (2 in.) thick, so the cost of the conventional concrete underneath has to be recalculated for 180 mm (7 in.) of concrete:

$$\rightarrow 0.180 \text{ m}^3/\text{m}^2 \times \$425/\text{m}^3 = \$76.5/\text{m}^2$$

Adding the new concrete cost to the cost for black steel reinforcement of $43/m$^2$ found in Calc. 6.3(G), the cost for the 180 mm deck with black steel reinforcement is:

→ $76.5/m$^2$ + $43/m$^2$ = $120/m$^2$

Adding $30/m$^2$ for the low-slump overlay, the total price for the deck is $150/m$^2$. Overlay replacement is assumed to be accompanied by repair, and calculated according to Calc. 6.4(B):

→ $212/m$^2$ + $30/m$^2$ = $242/m$^2$

## 8. SILICA FUME OVERLAY

The service life of silica fume overlays on a deck with black steel reinforcement is estimated to be 30 years because, according to Miller and Darwin (2000), silica fume overlays may not perform any better than low-slump overlays.

The cost for the bottom 190 mm (7.5 in.) of conventional concrete $81/m$^2$, [see Calc. 6.1(D)] and for black steel reinforcement $43/m$^2$ [see Calc. 6.3(G)] are added to get the cost of the conventional concrete deck:

→ $81/m$^2$ + $43/m$^2$ = $124/m$^2$

The deck cost is added to the average low bid cost for a 40 mm (1.5 in.) silica fume overlay in Kansas in 1999, $54/m$^2$, to get a total cost of:

→ $124/m$^2$ + $54/m$^2$ = $178/m$^2$

Overlay replacement is assumed to be accompanied by repair, and calculated according to Calc. 6.4(B):

→ $212/m$^2$ +$54/m$^2$ = $266/m$^2$

## 9. LATEX-MODIFIED OVERLAY

Latex-modified overlays can be expected to last up to 25 years, after which they usually need to be replaced due to rutting and general wear (Babaei and Hawkins 1990).

The 1999 average cost for a 45 mm latex-modified concrete overlay in Missouri was \$66/m$^2$.

These overlays are 45 mm thick, so the cost of the conventional concrete underneath has to be recalculated for 185 mm of concrete:

$\rightarrow$ 0.185 m$^3$/m$^2$ x \$425/m$^3$ = \$79/m$^2$

Adding the new concrete cost to the cost for black steel reinforcement of \$43/m$^2$ found in Calc. 6.3(G), the cost for the 185 mm deck with black steel reinforcement is:

$\rightarrow$ \$79/m$^2$ + \$43/m$^2$ = \$122/m$^2$

The cost of the latex-modified concrete overlay is \$66/m$^2$, so the total price for the deck is:

$\rightarrow$ \$122/m$^2$ + \$66/m$^2$ = \$188/m$^2$

Overlay replacement is assumed to be accompanied by repair, and calculated according to Calc. 6.4(B):

$\rightarrow$ \$212/m$^2$ + \$66/m$^2$ = \$278/m$^2$

## 10. POLYMER CONCRETE OVERLAY REPAIR OF UNPROTECTED DECK

Polymer concrete overlays last 10-15 years and cost \$47/m$^2$ (Wenzlick 1999). Because these overlays are only used for repair, the new deck is assumed to be a 205 mm (8 in.) thick, unprotected deck (see item 1 in this section). The deck is assumed to be repaired and overlayed with a 7 mm (¼ in.) polymer concrete overlay after 25 years in service. It is estimated that this overlay will need to be replaced every 15 years, with further repair at 55 years.

The cost for the 205 mm deck with black steel reinforcement is \$125/m$^2$ [Calc 6.3(F)]. The cost to repair the deck and place a polymer concrete overlay includes 22% partial depth repair, 6% full depth repair, machine preparation, the polymer concrete overlay, and incidental costs:

$\rightarrow$ \$190/m$^2$ x 0.22 + \$267/m$^2$ x 0.06 + \$21/m$^2$ + \$47/m$^2$ + \$126/m$^2$ = \$252/m$^2$

In this case, overlay replacement is not always assumed to be accompanied by repair, so replacement costs without repair are based on removal of the old overlay, deck preparation, materials, and incidental costs:

$\rightarrow \$7/m^2 + \$21/m^2 + \$47/m^2 + \$126/m^2 = \$201/m^2$

## 11. PREFORMED SHEET MEMBRANE

Preformed sheet membranes generally last 10-20 years (Roberts 2000), 15-20 years according to Manning (1998). It is assumed that these membranes will keep chloride ions from reaching the concrete, so there are no repair costs within the 75 year economic evaluation period. Replacement is assumed every 20 years for this study.

The cost for a sheet membrane in Vermont is $\$18\text{-}22/m^2$ (Graham 2000) + $\$4/m^2$ for a 50 mm (2 in.) asphalt overlay. A price of $\$20/m^2$ for the membrane is used for this study. This is added to the total cost of the deck, $\$140/m^2$ for a 230 mm thick conventional concrete deck with black steel reinforcement [Calc. 6.3(H)]. Asphalt overlay replacement costs are $\$27/m^2$ [Calc.6.4(C)]. No incidental costs are included with the asphalt overlay cost because this is done along with the rest of the road when it is resurfaced. From calculation 6.4(D), membrane replacement costs are:

$\rightarrow \$24/m^2 + \$15/m^2 + \$4/m^2 + \$126/m^2 = \$169/m^2$

## 12. SPRAY APPLIED LIQUID MEMBRANE

Spray applied liquid membranes are expected to last for 50 years in New Hampshire (Roberts 2000). This is the time used for this study. It is assumed that these membranes will keep chloride ions from reaching the concrete, so there are no repair cost within the 75 year economic evaluation period.

Spray applied liquid membranes cost $\$60/m^2$ + $\$4/m^2$ for a 50 mm (2 in.) asphalt overlay (Roberts 2000). This is in addition to the cost of a new bridge deck [$\$140/m^2$, Calc. 6.3(H)]. The top 40 mm of asphalt may be overlaid (without replacing the membrane) every 15-20 years at a cost of \$27/m$^2$ [calculation 6.4(C)]. No incidental costs were included in the cost of the asphalt overlay because it is done along with the road when it is resurfaced. From calculation 6.4(D), membrane replacement costs (at 50 years) are:

$$\rightarrow \$24/\text{m}^2 + \$60/\text{m}^2 + \$4/\text{m}^2 + \$126/\text{m}^2 = \$214/\text{m}^2$$

### 13. TORCH APPLIED MEMBRANE

Torch applied membranes are expected to last for 20+ years. They cost \$29/m$^2$ + \$4/m$^2$ for a 50 mm (2 in.) asphalt overlay (Roberts 2000). This is in addition to the cost of a new bridge deck [\$140/m$^2$, Calc. 6.3(H)]. For this analysis, it is assumed that they will last 20 years, because that is the limit for the asphalt overlay. Costs for asphalt overlay replacement are from Calc. 6.4(C), and from Calc. 6.4(D), membrane replacement costs are:

$$\rightarrow \$24/\text{m}^2 + \$29/\text{m}^2 + \$4/\text{m}^2 + \$126/\text{m}^2 = \$183/\text{m}^2$$

### 14. HOT RUBBERIZED ASPHALT MEMBRANE

According to David Manning (2000), hot applied liquid asphalt membranes cost \$2/ft$^2$ (Canadian) (about \$15/m$^2$ US), and generally last for 30 to 40 years. The cost does not include the hot mix overlay, which costs about \$40/metric ton (Canadian). This is about \$5.32/m$^2$ US for two, 40 mm (1.5 in.) layers [Calc. 6.2(A)]. The top 40 mm of the asphalt overlay is replaced every 15-20 years (20 years is used for this study), at a cost of about \$27/m$^2$ [Calc. 6.4(C)], and from Calc. 6.4(D), membrane replacement costs are:

$$\rightarrow \$24/\text{m}^2 + \$15/\text{m}^2 + \$5/\text{m}^2 + \$126/\text{m}^2 = \$170/\text{m}^2$$

### 15. LINSEED OIL

Linseed oil costs \$0.90/m$^2$ according to Randy Cox with the Texas DOT. Sherman et al. (1993) limits the service life of treatments to 6 months to 3 years. For this study, it is assumed that linseed oil is reapplied every three years, but according to Wenzlick (2000), linseed oil is not generally reapplied in practice.

There is no specific information on how long linseed oil will extend the service life of a bridge deck, so a time to repair of 25 years (the same as for an unprotected bridge) is estimated.

## 16. SILANE

Silane costs \$4.26/m$^2$ according to Randy Cox (2000) with the Texas DOT. According to Sherman et al. (1993), the sealer should be reapplied every 5 years [7 years according to Zemajtis and Weyers (1996)]. A reapplication period of 7 years is used for this study, although this would probably not be done in the field. According to Zemajtis and Weyers (1996), silane will extend the service life of a deck to almost 40 years. The time to first repair is estimated to be 35 years.

## 17. IMPRESSED CURRENT CATHODIC PROTECTION — TITANIUM MESH

The cost for titanium mesh anodes for bridge decks in Virginia in 1999 ranged from \$57 to 97/m$^2$. In December 1999, the bid to install cathodic protection, using the Elgard (titanium mesh) system, on 5 bridges in Kansas City, Missouri, was \$68/m$^2$ (6.36/ft$^2$), including materials, labor, and rectifiers. This is the cost used in the economic calculations. The only additional cost is that of placing a low-slump dense concrete overlay over the system (overlays are required over these systems on bridge decks), which is \$30/m$^2$ (see item 7 above).

The manufacturer claims a 50 year service life, while the DOTs expect to get at least 20 year service lives from the anodes. For this study, replacement of the anode and overlay are estimated at 30 and 60 years because the overlays are not expected to last 50 years, and the anode will be damaged when the overlay is replaced. No repair of the deck will be required as long as the cathodic protection system continues to protect the reinforcement. Therefore, the cost to replace the overlay would be based on removal of the old overlay, deck preparation, materials, and incidental costs:

$$\rightarrow \$7/\text{m}^2 + \$21/\text{m}^2 + \$30/\text{m}^2 + \$12/\text{m}^2 = \$184/\text{m}^2$$

It is assumed that the cost to replace the anode system would be the same as the new price, even though the new price includes rectifiers, which probably would not need to be replaced. No additional costs are included for removal of the old system.

Maintenance and operation costs for cathodic protection systems on bridges in Florida ranged from \$1.30 to \$6.90/m$^2$. The higher prices were from systems that included remote monitoring. *For this study, the maintenance and operation cost for all cathodic protection systems is assumed to be \$1/m$^2$/year.*

## 18. IMPRESSED CURRENT CATHODIC PROTECTION — SLOTTED ANODE

Slotted anode cathodic protection systems using platinum primary anodes with a coke breeze epoxy secondary anode are expected to last for 20 years (Wenzlick 2000). No major repairs of the deck will be required as long as the cathodic protection system continues to protect the reinforcement, but for this study, it was estimated that the anode system and overlay would be replaced every 30 years.

The current cost for this system is about \$107/m$^2$ (\$10/ft$^2$) (Wenzlick 2000), with an additional \$30/m$^2$ for a low-slump concrete overlay.

Maintenance and operation costs are assumed to be \$1/m$^2$/year. Calculations are given above in item 17.

## 19. IMPRESSED CURRENT CATHODIC PROTECTION — SPRAYED ZINC

According to Gerry Clemena (2000) at the Virginia DOT, thermal sprayed zinc cathodic protection anodes cost between \$86-108/m$^2$, and can be expected to last 7-10 years, depending on the current of the system. For this study, replacement of the system was estimated every 15 years. These systems are not overlayed; they are used on substructure members and bridge deck undersides.

Maintenance and operation costs are assumed to be $1/m$^2$/year. Replacement costs are assumed to be the same as the original costs.

## 20. ELECTROCHEMICAL CHLORIDE EXTRACTION

According to Burns (2000), the cost for ECE on bridge decks is usually between $95-150/m$^2$, and the cost on substructures is usually between $230-290/m$^2$.

The cost for ECE varies for each case. The 1997 cost to treat 720 m$^2$ in Virginia was $92,412, for a unit cost of $128.35/m$^2$. The high cost was attributed to a lack of competitive, experienced applicators of the method, since there was only one contractor qualified for the job at the time of the study (Clemena and Jackson 1997).

In this study, the cost of ECE is assumed to be $120/m$^2$. This treatment is assumed to be used at the time of repairs (25 years for an unprotected deck with black steel reinforcement), and would be followed up with the placement of an overlay or a sealer on the deck. For this analysis, the cost of repair and a low-slump overlay (machine preparation + overlay + incidental costs) is added to the treatment cost. The cost to repair and overlay the deck is:

$$\rightarrow \$21/\text{m}^2 + (\$190/\text{m}^2 \text{ x } 0.22) + (\$267/\text{m}^2 \text{ x } 0.06) + \$30/\text{m}^2 + \$126/\text{m}^2 = \$235/\text{m}^2$$

The original bridge deck is assumed to be 205 mm (8 in.) thick with black steel reinforcement, for a basic cost of $125/m$^2$ [Calc. 6.3(F)].

## 21. CALCIUM NITRITE

The cost for DCI (W.R.Grace) is $1.85/liter (Kaiser 2000). The cost for Rheocrete CNI is $1.63/liter (Master Builders). The recommended dosage is 10 to 30 liters/m$^3$, giving a range in cost of $16-55/m$^3$. According to Bentur et al. (1997), dosages of 10 and 30 liters/m$^3$ will raise the corrosion threshold for chloride ions to 3.6 and 9.5 kg/m$^3$, respectively. Calculations by Bentur et al. (1997) indicate that, at a concentration of 10 and 15 l/m$^3$, it would take 37 and 75+ years, respectively, for chlorides to reach the corrosion threshold in uncracked concrete with a diffusion coefficient of 1.63 x 10$^{-12}$ m$^2$/s. Miller and Darwin (2000) found the effective diffusion coefficient for conventional concrete bridges in Kansas to be between 0.93 x $10^{-12}$ and 1.27 x $10^{-12}$ $m^2/s$. Repairs are expected to be required after 40 years for a concentration of 10 $l/m^3$.

When 10 $l/m^3$ of calcium nitrite is added to a bridge deck, the concrete price is increased by:

→ \$1.85/l x 10 $l/m^3$ = \$18.5/$m^3$

→ \$425/ $m^3$ + \$19/ $m^3$ = \$444/$m^3$

The concrete price for a 230 mm (9 in.) thick deck is:

→ 0.0230 $m^3/m^2$ x \$444/$m^3$ = \$102/$m^2$

The cost for a 230 mm (9 in.) thick deck with calcium nitrite and black steel reinforcement is:

→ \$102/$m^2$ + \$43/$m^2$ [Calc. 6.3(G)] = \$145/$m^2$

## 22. ORGANIC CORROSION INHIBITOR

Organic corrosion inhibitor (Rheocrete $222^+$) costs \$3.57/liter and is applied at a rate of 5 $l/m^2$. According to Bentur et al. (1997), this inhibitor will create a barrier to chloride ions on the reinforcement, and result in a time to corrosion threshold of 32 years, with first repair after 36 years.

To add 5 $l/m^3$ of organic corrosion inhibitor to a bridge deck, the concrete price is increased by:

→ \$3.57/l x 5 $l/m^3$ = \$18/$m^3$

→ \$425/ $m^3$ + \$18/ $m^3$ = \$443/$m^3$

The concrete price for a 230 mm (9 in.) thick deck is:

→ 0.230 $m^3/m^2$ x \$443/$m^3$ = \$102/$m^2$

The cost for a 230 mm (9 in.) thick deck with organic corrosion inhibitor and black steel reinforcement is:

→ \$102/$m^2$ + \$43/$m^2$ [Calc. 6.3(G)] = \$145/$m^2$

23 +. COMBINATIONS:

— For combinations of other methods with epoxy-coated reinforcement, \$4/m$^2$ is added to the base price of the bridge deck

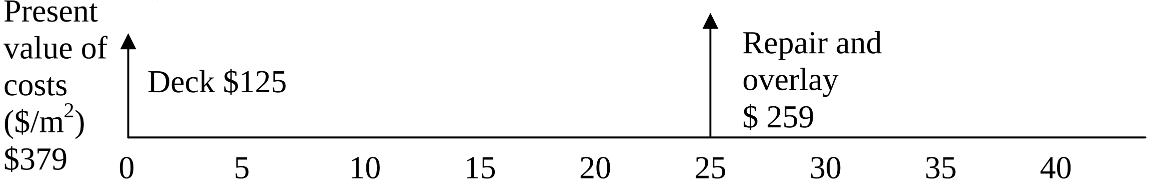
→ \$144/m$^2$ instead of \$140/m$^2$ for a conventional 230 mm (9 in.) thick deck

Five years is added to the time-to-repair when epoxy-coated reinforcement is combined with silica fume concrete and silica fume, low slump, and polymer concrete overlays because epoxy-coated reinforcement is estimated to add 5 years to the time-to-repair for conventional concrete decks. No time is added to the time-to-repair of bridge decks using epoxy-coated reinforcement in combination with latex-modified overlays, waterproof membranes, or corrosion inhibitors over the times for decks with the same systems and black steel. The time to repair of epoxy-coated steel combined with latex-modified overlays and waterproof membranes is not increased because the purpose of these systems is to keep chloride ions and water out of the concrete. Therefore, the susceptibility of the reinforcement to water and chloride ions is not as critical as it is in cases where the purpose of the cover is to slow the ingress of these contaminants. The time-to-repair of decks using a combination of epoxy-coated steel and corrosion inhibitors are not longer than the times for decks using only corrosion inhibitors because these times are already relatively long. It is estimated that epoxy-coated reinforcement will have suffered from damage or adhesion loss and will provide no extra protection by the time the first repairs become necessary, at 37 and 40 years for organic corrosion inhibitor and calcium nitrite, respectively.

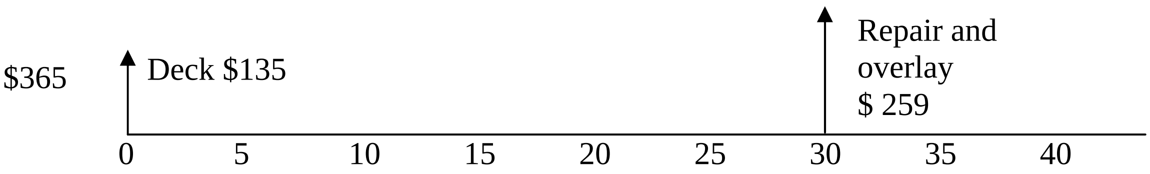
## Figure 6.1 Current Costs for Corrosion Protection Methods Over the First 40 Years

This figure presents the costs and times over the first 40 years for new construction and repair for the selected corrosion protection systems and combinations discussed in Section 6.2.3. The repair cycles are continued through 75 years to obtain a present value for total costs of each system. Dates and costs for both construction costs and repairs are presented in Table A.2. The costs and times presented in this figure are for a new, 230 mm (9 in.) thick (unless marked otherwise) bridge deck, in dollars per square meter.

1. NO PROTECTION – BLACK STEEL REINFORCEMENT – [205 mm (8 in.) deck]



2. EPOXY-COATED STEEL REINFORCEMENT – [216 mm (8.5 in.) deck]



3. STAINLESS STEEL REINFORCEMENT

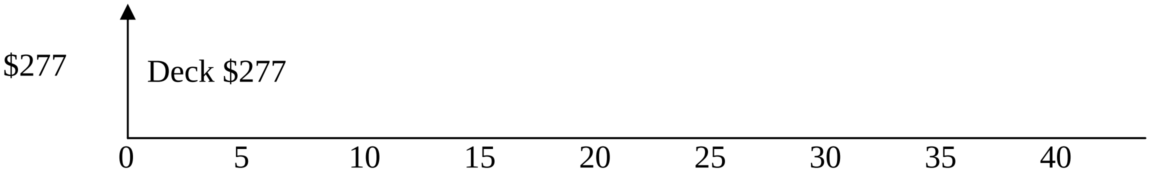
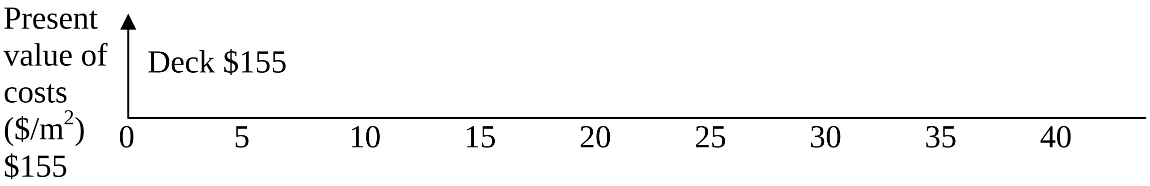
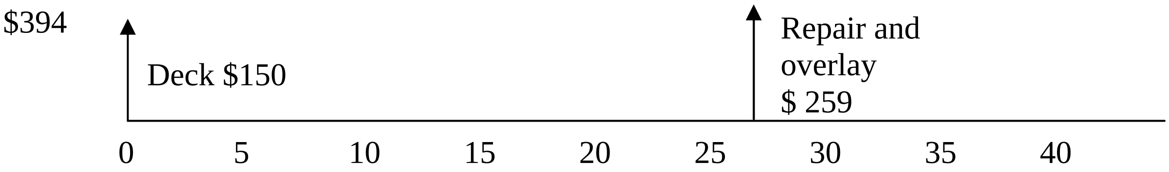


**Figure 6.1 Continued — Current Costs for Corrosion Protection Methods Over the First 40 Years**

4. STAINLESS STEEL-CLAD REINFORCEMENT

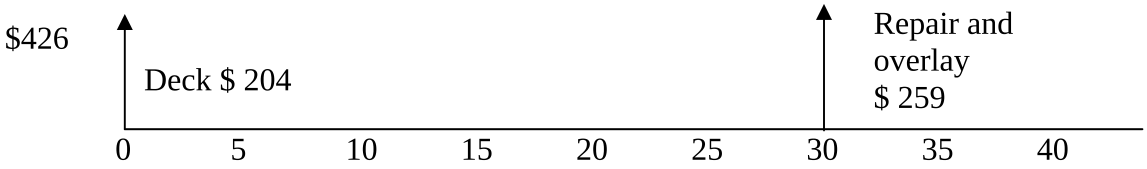


5. GALVANIZED REINFORCEMENT

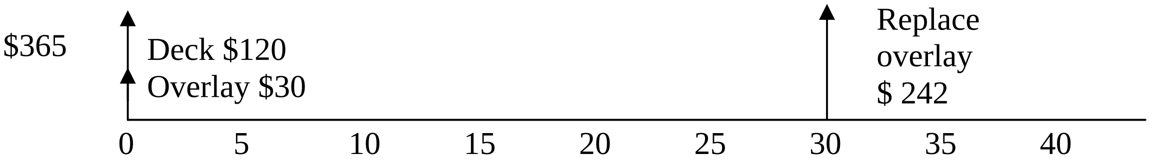


**BLACK STEEL REINFORCEMENT WITH:**

6. SILICA FUME CONCRETE



7. LOW-SLUMP OVERLAY



8. SILICA FUME OVERLAY

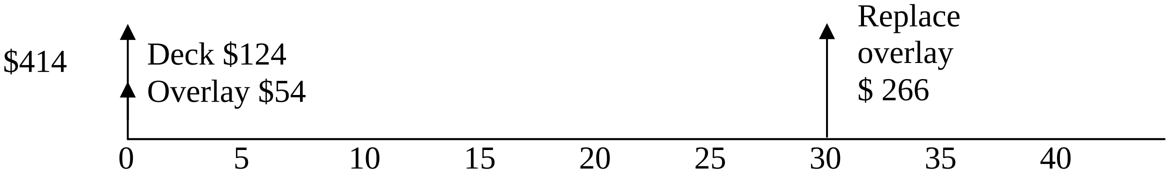
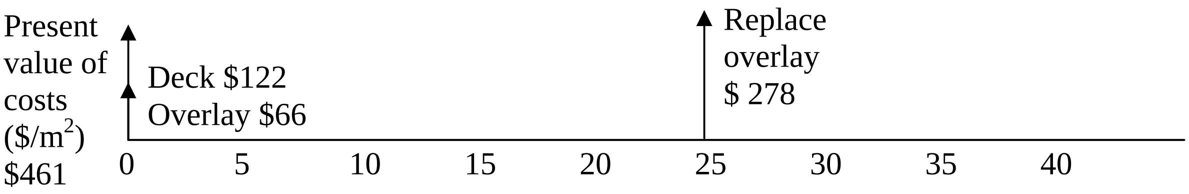
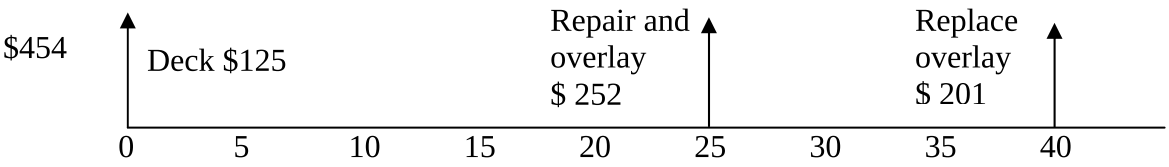


**Figure 6.1 Continued — Current Costs for Corrosion Protection Methods Over the First 40 Years**

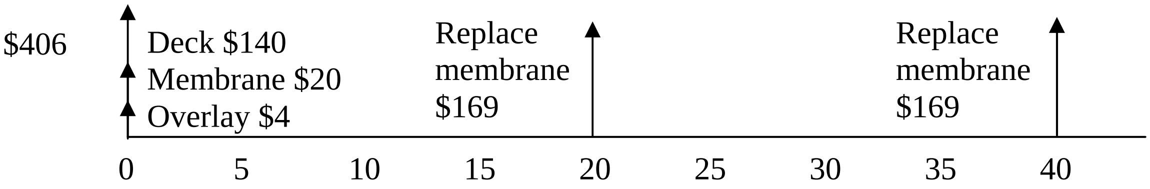
9. LATEX-MODIFIED CONCRETE OVERLAY



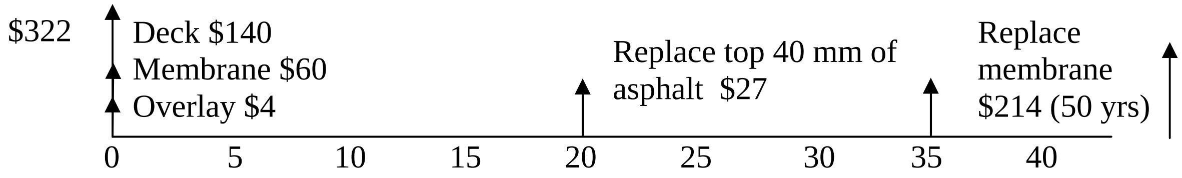
10. POLYMER CONCRETE OVERLAY OF UNPROTECTED DECK – [205 mm (8 in.) deck]



11. PREFORMED SHEET MEMBRANE



12. SPRAY APPLIED LIQUID MEMBRANE



13. TORCH APPLIED MEMBRANE

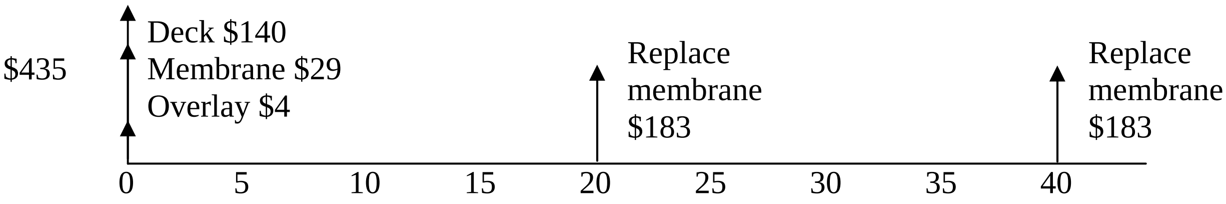
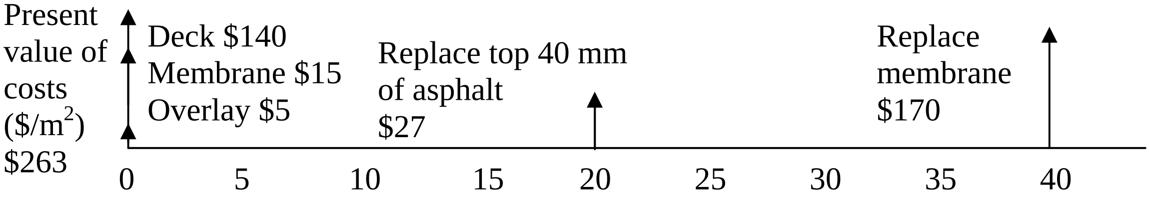
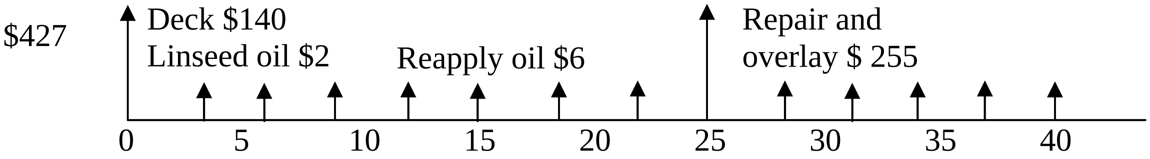


**Figure 6.1 Continued — Current Costs for Corrosion Protection Methods Over the First 40 Years**

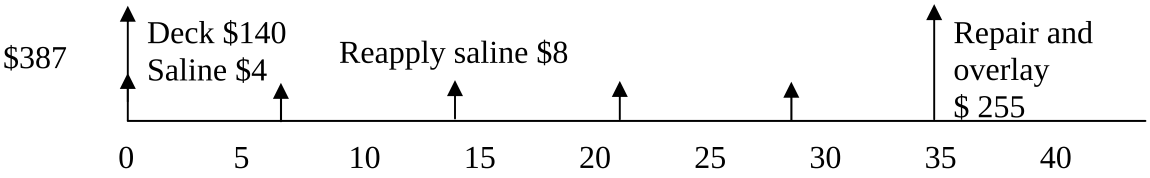
14. HOT RUBBERIZED ASPHALT MEMBRANE



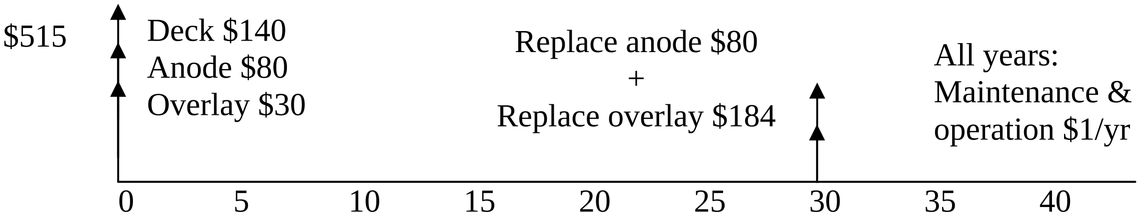
15. LINSEED OIL



16. SILANE



17. IMPRESSED CURRENT CATHODIC PROTECTION — TITANIUM MESH



18. IMPRESSED CURRENT CATHODIC PROTECTION — SLOTTED ANODE

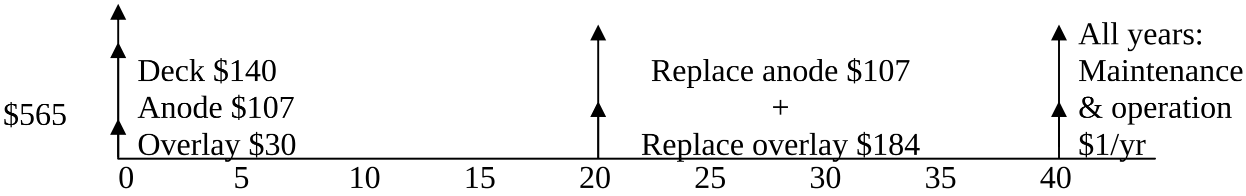
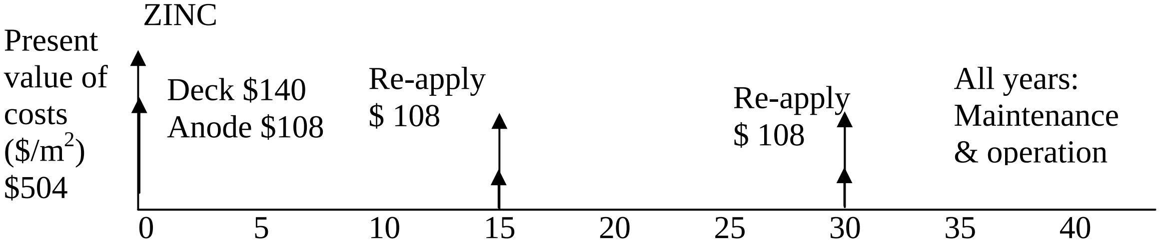
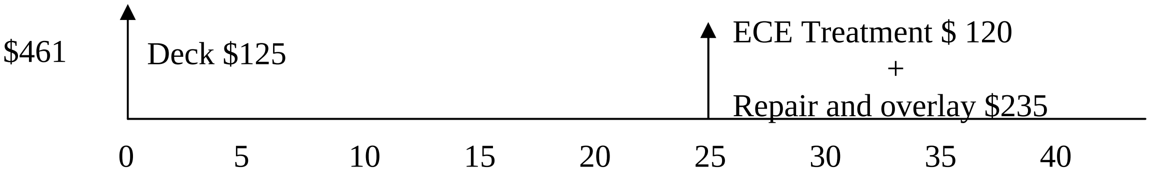


**Figure 6.1 Continued — Current Costs for Corrosion Protection Methods Over the First 40 Years**

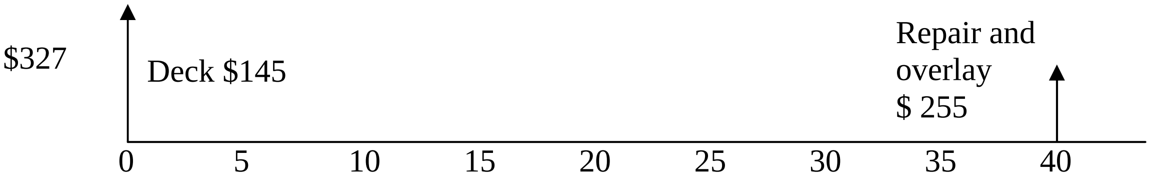
19. IMPRESSED CURRENT CATHODIC PROTECTION — SPRAYED ZINC



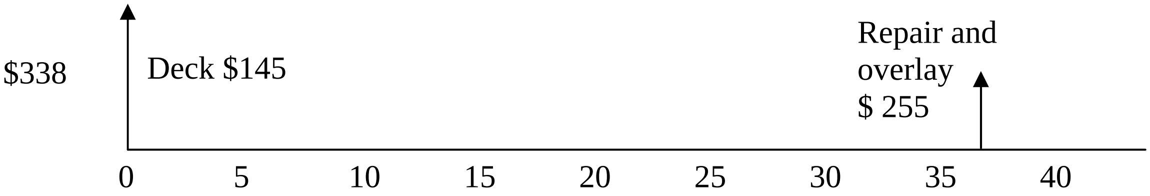
20. ELECTROCHEMICAL CHLORIDE EXTRACTION – [205 mm (8 in.) deck]



21. CALCIUM NITRITE



22. ORGANIC CORROSION INHIBITOR



**COMBINATIONS:**

23. EPOXY-COATED STEEL + SILICA FUME CONCRETE

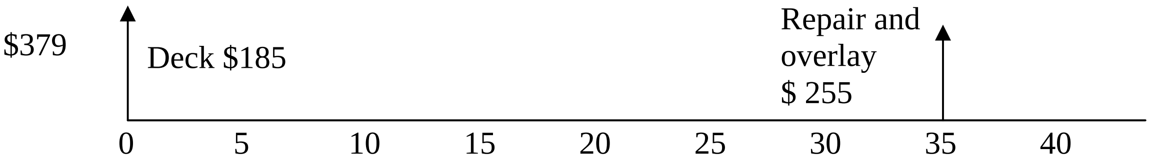
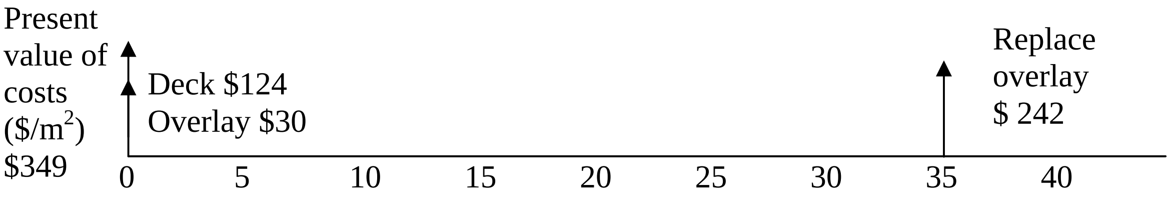
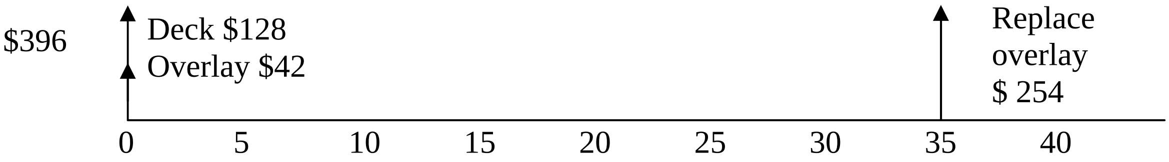


**Figure 6.1 Continued — Current Costs for Corrosion Protection Methods Over the First 40 Years**

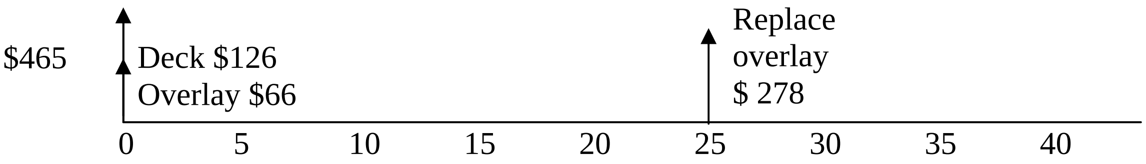
24. EPOXY-COATED STEEL + LOW-SLUMP OVERLAY



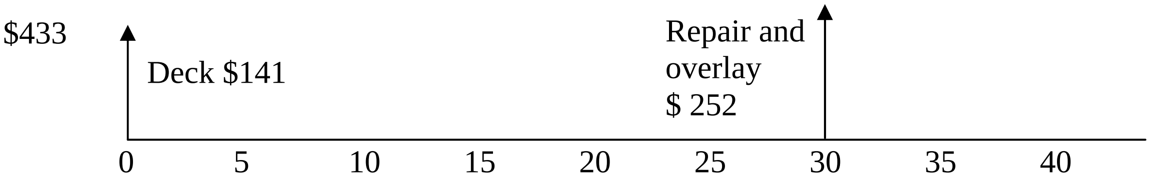
25. EPOXY-COATED STEEL + SILICA FUME OVERLAY



26. EPOXY-COATED STEEL + LATEX-MODIFIED CONCRETE OVERLAY



27. EPOXY-COATED STEEL + POLYMER CONCRETE OVERLAY – [216 mm (8.5 in.) deck]



28. EPOXY-COATED STEEL + PREFORMED SHEET MEMBRANE

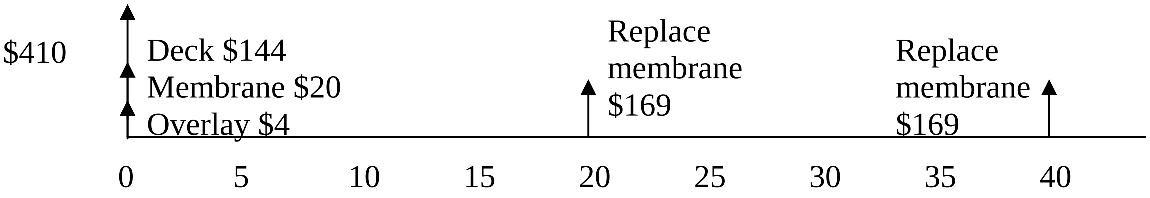
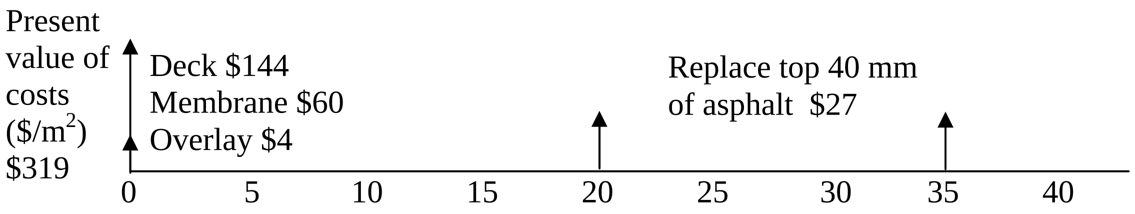
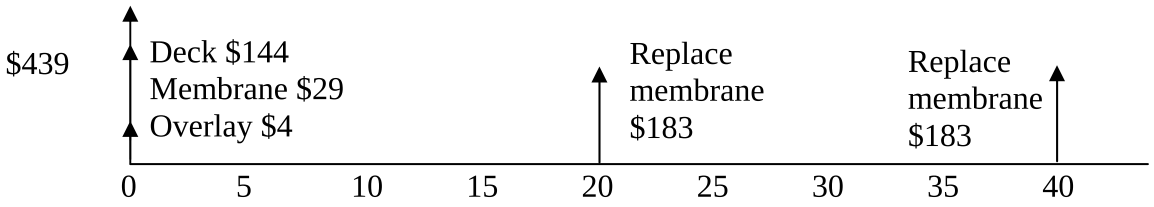


**Figure 6.1 Continued — Current Costs for Corrosion Protection Methods Over the First 40 Years**

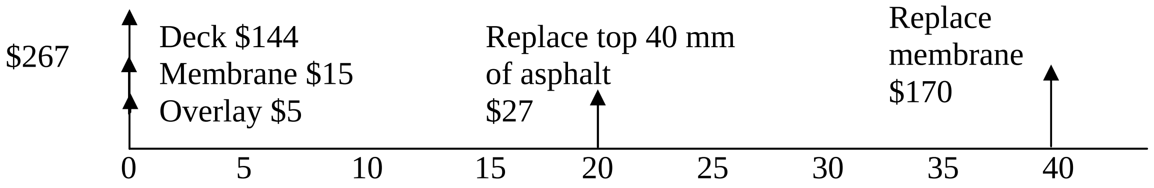
29. EPOXY-COATED STEEL + SPRAY APPLIED LIQUID MEMBRANE



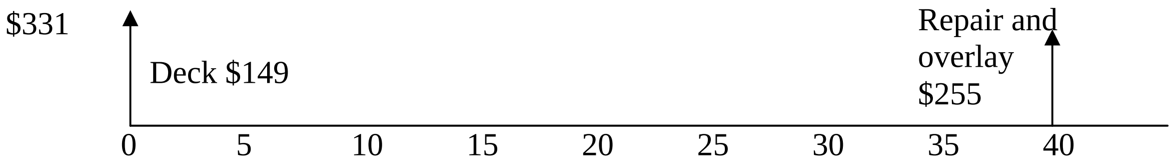
30. EPOXY-COATED STEEL + TORCH APPLIED MEMBRANE



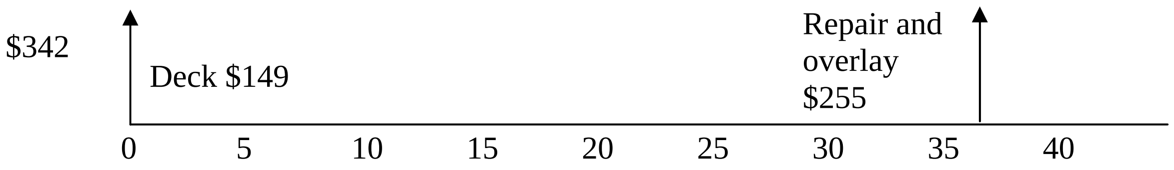
31. EPOXY-COATED STEEL + HOT RUBBERIZED ASPHALT MEMBRANE



32. EPOXY-COATED STEEL + CALCIUM NITRITE



33. EPOXY-COATED STEEL + ORGANIC CORROSION INHIBITOR



### 6.2.4 Present Value of Costs

The present value for the costs of each corrosion protection method is calculated assuming discount rates of 2, 4, and 6%. The lowest rate was selected because the discount rate used by the Kansas Department of Transportation for economic comparisons in 1999 was 2.04% (Scherschligt 2000). The present worth of repair and replacement costs is calculated using the Eq. (6-1):

$$P = F * (1 + i)^{-n} \qquad (6\text{-}1)$$

where:

P = present worth
F = cost of repair or replacement
i = discount rate (4%)
n = time to repair or replacement (in years)

The present worth of maintenance and operation costs, which are considered on an annual basis, are calculated using Eq. (6-2):

$$P = A * \frac{(1 + i)^{n} - 1}{i * (1 + i)^{n}} \qquad (6\text{-}2)$$

where:

A = annual (maintenance and operation) cost
n = total time for maintenance and operation of bridge (in years)

The total present worth of each corrosion protection method is calculated by adding the initial cost to the present values of the repair and replacement costs, and the maintenance and operation costs. The individual costs and present value for each method are shown in Figure 6.1 for the 2% discount rate. Present values calculated using all three rates are presented in Table A.2. Inflation is not included in these analyses.

## 6.3 OVERALL COMPARISON

Figure 6.2 compares the 15 corrosion protection systems with the lowest present value of costs based on a 2% discount rate. Figure 6.3 presents present value costs of the other corrosion protection methods. At a 2% discount rate, the present value cost for the first 13 methods is lower than the present value cost of a deck with no protection. The present value costs are also given for discount rates of 4 and 6%.

The discount rate chosen for comparison is important because, at a lower discount rate, the first cost is relatively less important, maintenance costs are more important, and the present value for future cost is influenced less by time than at a higher discount rate.

According to the present value cost analysis performed for this study, the corrosion protection system with the lowest present value cost consists of stainless steel-clad reinforcement, with a present value cost of \$155/m$^2$. This cost does not change with changes in the discount rate because it assumes that no repair or maintenance work is required on the deck throughout its 75 year economic life. The most expensive option uses a slotted anode impressed current cathodic protection system, with a present value cost of 565/m$^2$ (2% discount rate).

The present value of the costs for a 205 mm (8 in.) thick conventional concrete bridge deck with black steel reinforcement and 50 mm (2 in.) of cover is \$379/m$^2$ (2% discount rate). The combination of epoxy-coated steel and a silica fume overlay, which is common in the state of Kansas, has a present value cost of \$414/m$^2$ (2% discount rate), which is slightly higher than the present value cost of an unprotected deck.

Figure 6.2 Present Values of 15 Lowest Cost Corrosion Protection Options Based on a 2% Discount Rate in Dollars per Square Meter

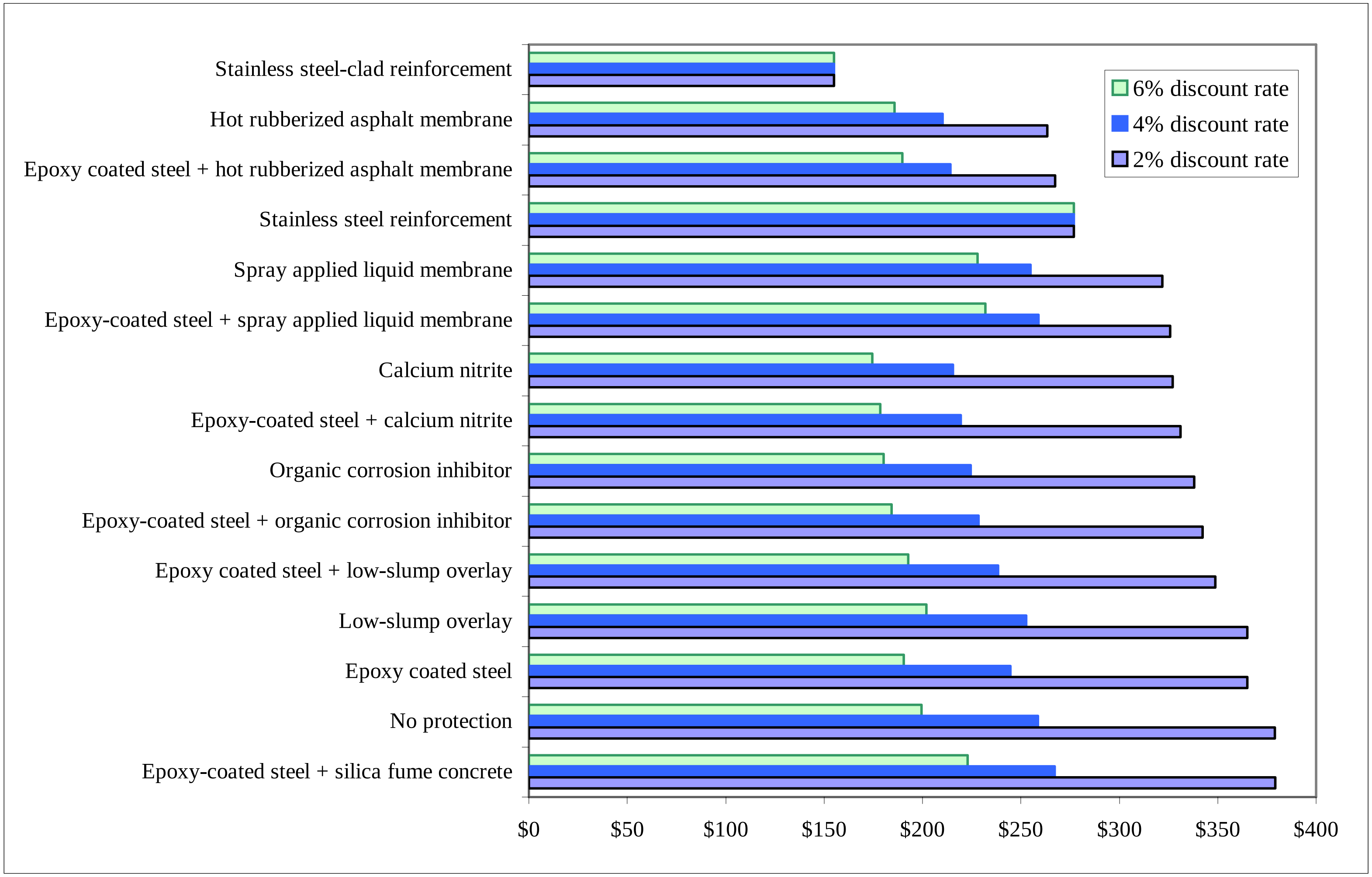
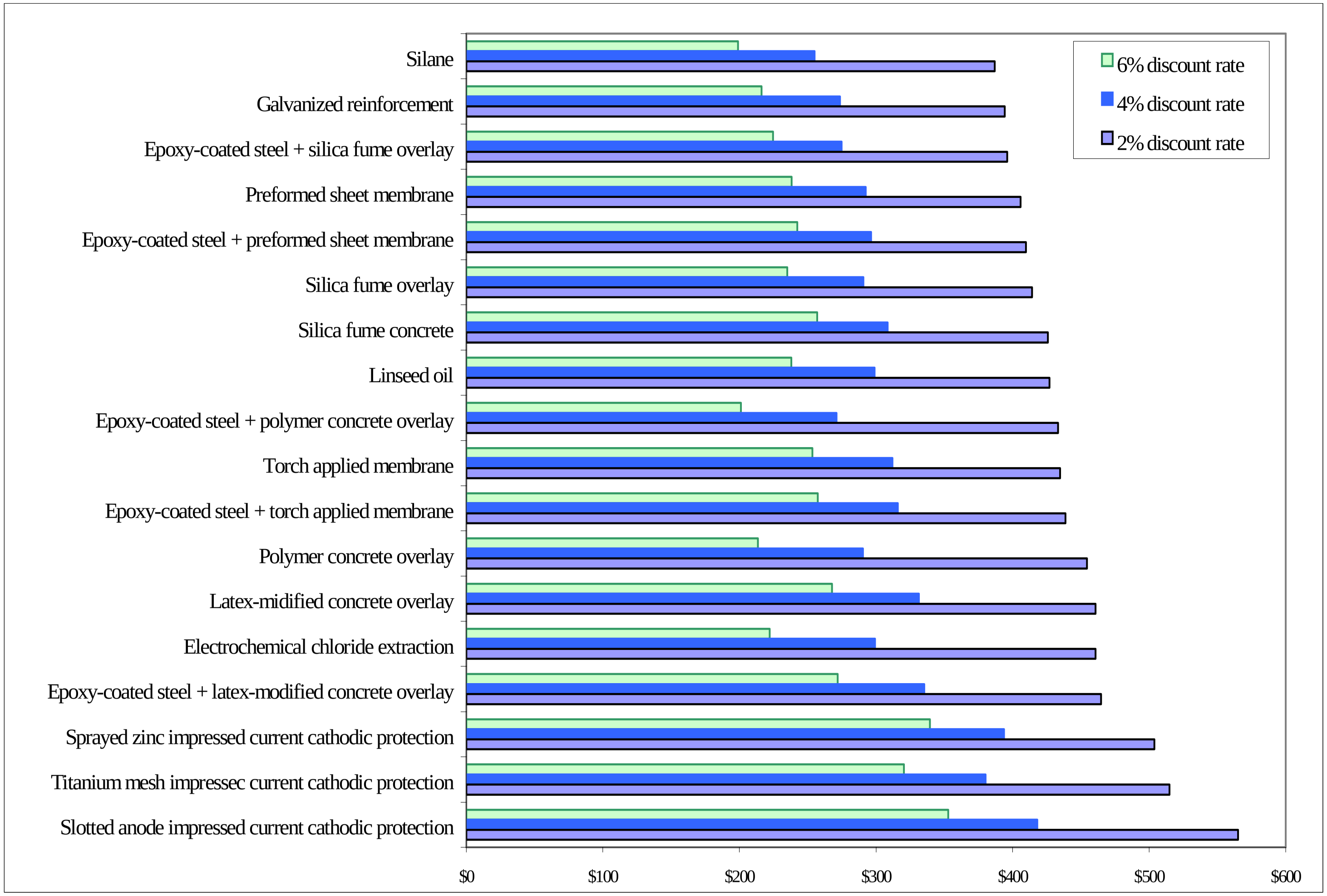


Figure 6.3  Present Values of the 18 Highest Cost Corrosion Protection Options Based on a 2% Discount Rate in Dollars per Square Meter



Other options that have low present value costs at a 2% discount rate include hot rubberized asphalt membranes, solid stainless steel reinforcement, spray applied liquid membranes, calcium nitrite, low slump overlays, and epoxy coated steel. Epoxy-coated steel can be combined with membranes, overlays, corrosion inhibitors, or silica fume concrete, and is a low cost backup to these corrosion protection methods.

At a 4% discount rate (Figures 6.4 and 6.5), the corrosion protection system with the lowest present value cost is still stainless steel-clad reinforcement, followed by hot rubberized asphalt membranes and calcium nitrite. The present value cost of spray-applied liquid membranes is still lower than that of an unprotected deck, but higher than the present value of the cost for a deck with organic corrosion inhibitor, epoxy-coated steel, low slump overlay, or saline. At a 4% discount rate, the present value of the cost for a deck with solid stainless steel reinforcement is higher than for a deck with no corrosion protection.

At a 6% discount rate (Figures 6.6 and 6.7), the corrosion protection system with the lowest present value cost, following stainless steel-clad reinforcement, is the use of calcium nitrite, followed by the use of an organic inhibitor. The other corrosion protection systems with present value costs lower than those of an unprotected deck at a 6% discount rate are hot rubberized asphalt membranes, epoxy-coated steel, epoxy-coated steel plus a low slump overlay, and saline. At a 6% discount rate, the deck with solid stainless steel reinforcement has one of the highest present values.

Cathodic protection systems, installed on new decks, had the highest present value costs for this analysis; however, these systems are often used as repair methods when a deck would otherwise need to be replaced, which would make the system much more cost-effective. Cathodic protection systems are also often used on bridge substructure members, where there are fewer options for corrosion protection.

**Figure 6.4 Present Values of 15 Lowest Cost Corrosion Protection Options Based on a 4% Discount Rate in Dollars per Square Meter**

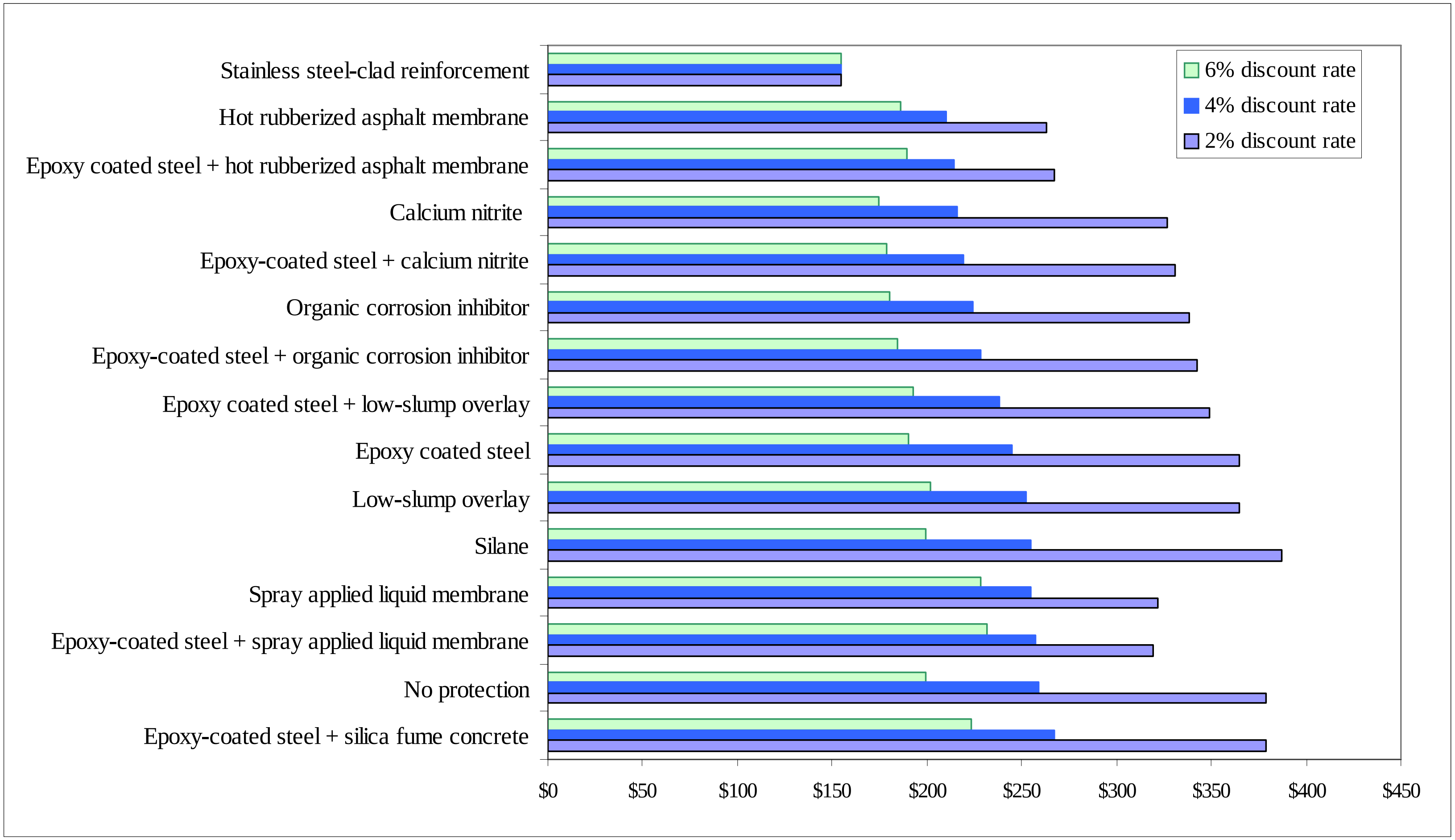


Figure 6.5  Present Values of 18 Highest Cost Corrosion Protection Options Based on a 4% Discount Rate in Dollars per Square Meter

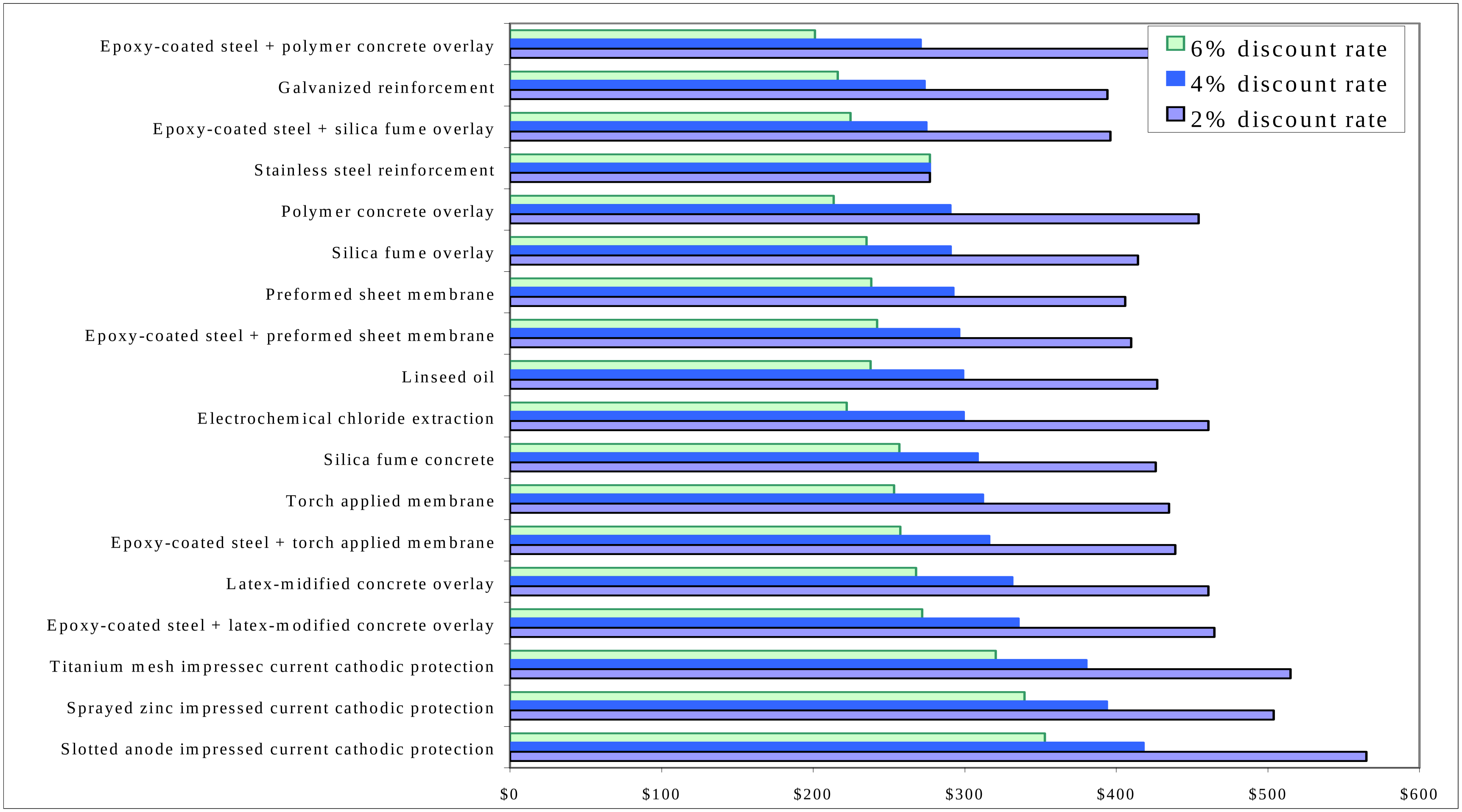


**Figure 6.6 Present Values of 15 Lowest Cost Corrosion Protection Options Based on a 6% Discount Rate in Dollars per Square Meter**

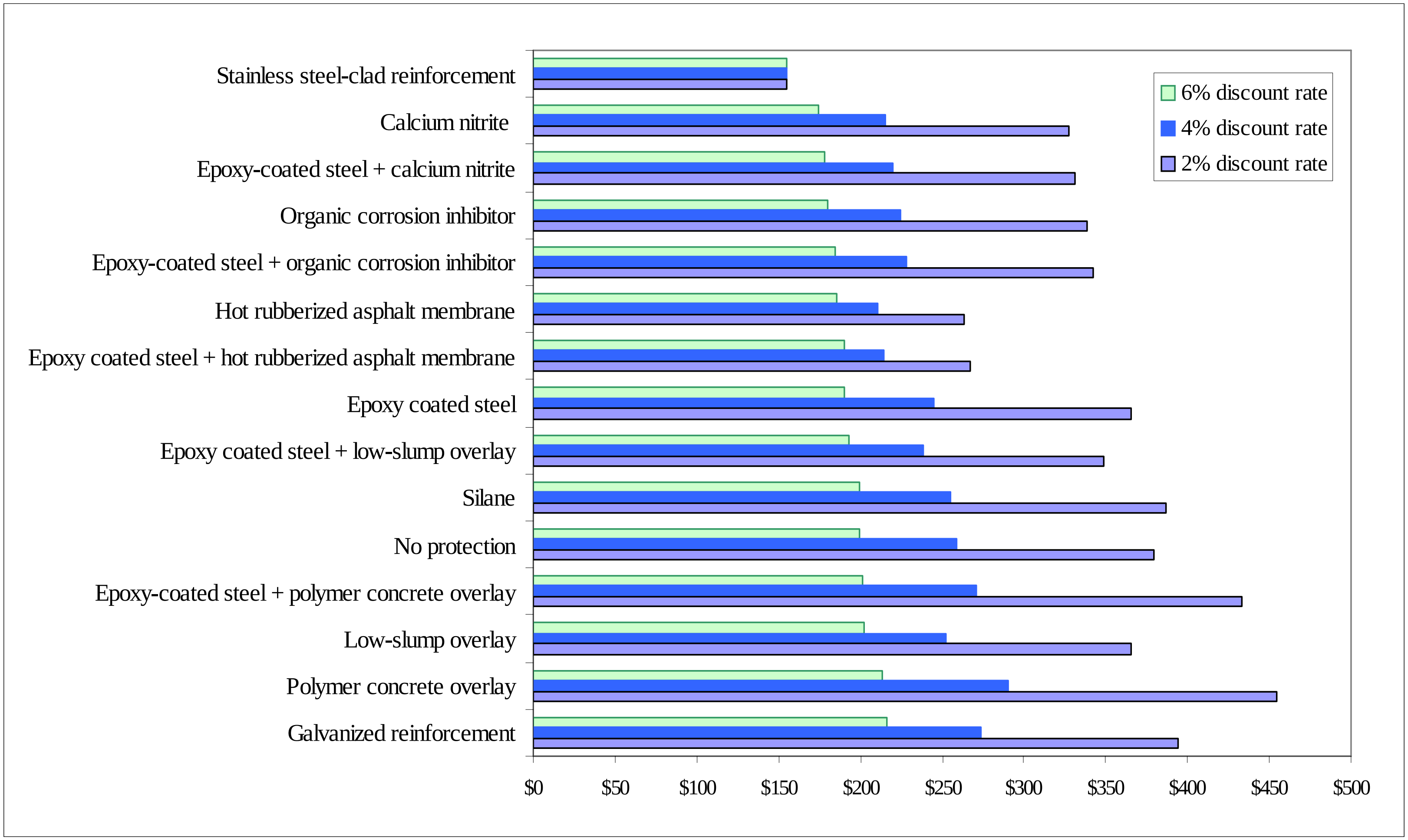
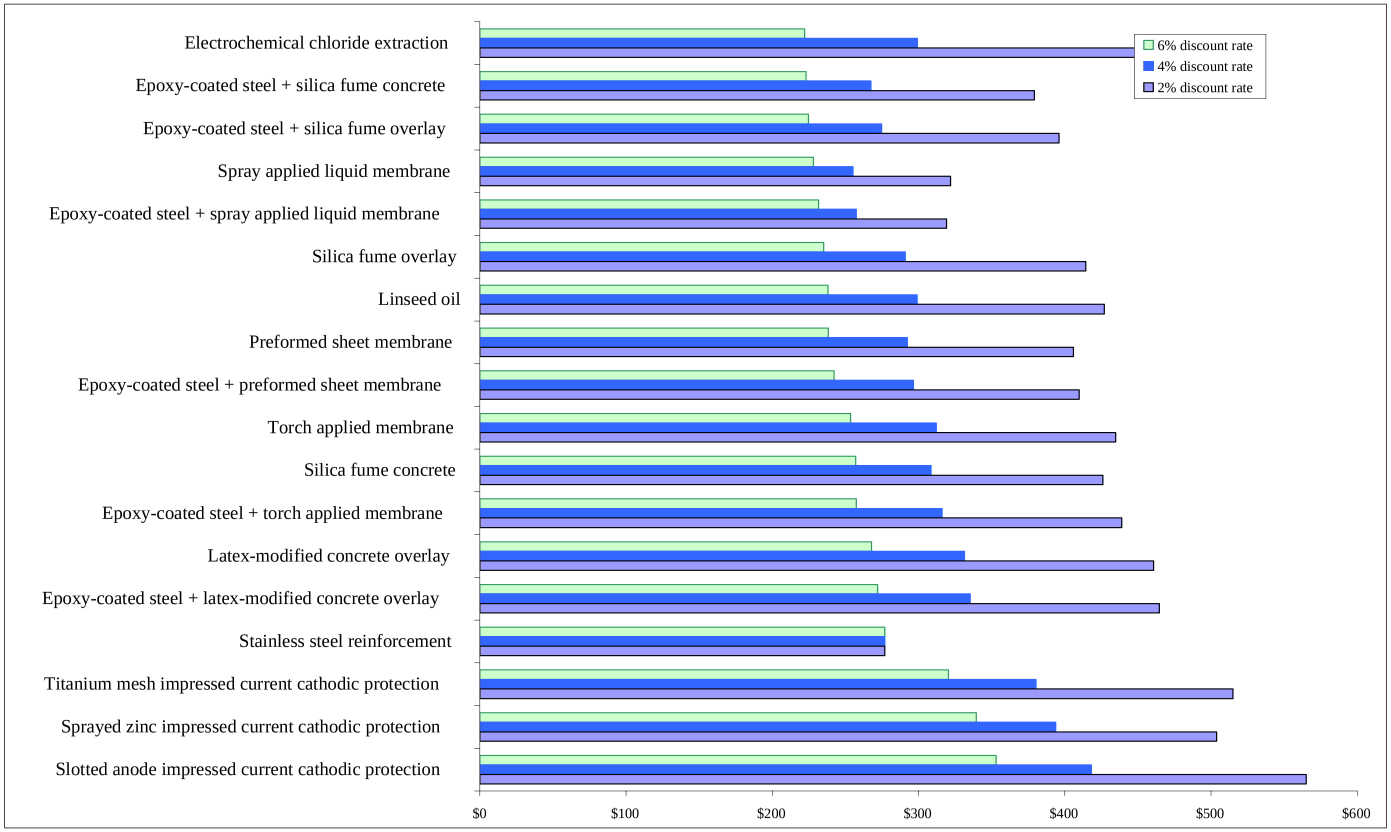


Figure 6.7 Present Values of 18 Highest Cost Corrosion Protection Options Based on a 6% Discount Rate in Dollars per Square Meter



Some of the systems included in this analysis have long-term performance records on bridges, while others have only been used for a short time, and may have no field performance record. The option with the lowest present value cost, stainless steel-clad reinforcement, is new enough that no field performance data, and only limited laboratory research data is available. If stainless steel-clad reinforcement performs as well as expected, it will be hard to find a more cost-effective corrosion protection system. Other new systems with low present value costs include spray-applied liquid membranes and corrosion inhibitors. These systems should be considered for use on an experimental basis.

The system with the second lowest present value cost (at 2 and 4% discount rates), the hot rubberized asphalt membrane, has been in use since the 1970s and has a good performance record. Low slump overlays have been used successfully since the 1960s and have present value costs lower than those for a deck with no corrosion protection. These systems should be considered for use as standard bridge deck corrosion protection methods.

# CHAPTER 7

# SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

## 7.1 SUMMARY

Corrosion of reinforcing steel in bridges is a significant economic and safety problem for transportation agencies, preventing many bridges from attaining their design lives. Since the 1970s, there have been many research projects and field studies conducted to evaluate methods for protecting reinforced concrete bridges from corrosion damage. This study provides a review of literature on the research and field trials.

Each method and its underlying principles are described, performance results of laboratory and/or field trials are reviewed, and systems are evaluated based on the results of the trials. Using performance results from the studies reviewed and costs obtained from various transportation agencies, an economic analysis is used to estimate the cost-effectiveness of each system over a 75 year economic life for discount rates of 2, 4, and 6%.

The methods discussed in this report include: alternative reinforcement and slab design, including epoxy-coated steel, galvanized steel, solid stainless steel, stainless steel-clad, nickel-clad, titanium, copper-clad, and fiber reinforced plastic reinforcement, and steel-free slabs; barrier methods, including the use of low permeability concrete, low water/cement ratio mix designs, the use of mineral admixtures, overlays, waterproof membranes, sealers, and deep polymer impregnation; electrochemical methods, including cathodic protection and electrochemical chloride extraction; and corrosion inhibitors.

## 7.2 CONCLUSIONS

Many different methods have been used to help improve the durability of concrete bridges throughout North America. Some of these methods have performed better than others, but there is no one perfect method for every type of application. In some cases, simply ensuring adequate concrete cover over conventional steel reinforcement and using quality concrete with a water cement ratio below 0.45 has been sufficient, but on structures that are exposed to corrosive environments in coastal or snow belt regions, further protection is generally required.

The most common corrosion protection method in the United States today is the use of epoxy-coated reinforcing steel. Although controversial in many areas, epoxy-coated reinforcement has performed well in many states, including Kansas, since it was introduced in the early 1970s.

Based on the economic analysis performed in this study, the use of epoxy-coated reinforcement alone is more cost-effective than building a bridge with black steel reinforcement and no other corrosion protection method, but there are other methods that now appear to be more cost-effective than either of these options. The more cost-effective corrosion protection options include epoxy-coated reinforcement in combination with another system such as a membrane, overlay, or corrosion inhibitor. The increase in the cost of a new deck with epoxy-coated steel over black steel reinforcement is very small, so even if it is not expected to improve the service life of a structure, epoxy-coated steel is a low cost backup to other corrosion protection methods.

Research on stainless steel reinforcement indicates that it may remain free of corrosion in chloride contaminated concrete for more than 75 years, but the relatively high cost of stainless steel has prohibited it from being used for entire structures. Rather, stainless steel reinforcement is usually limited to areas where contamination is expected to be the highest. The present value cost of a deck with solid stainless steel reinforcement does not vary with changes in the discount rate in this analysis because no repairs will be required during the 75 year economic life. At a low discount rate (2%), solid stainless steel reinforcement is a cost-effective option compared to other options, but at higher discount rates (4%+), the present value cost of a deck with solid stainless steel is significantly higher than that of an unprotected deck for the 75 year economic life considered in this study.

Stainless steel clad reinforcement is much less expensive than solid stainless steel reinforcement. In fact, its in-place cost is not much more than that of epoxy-coated steel. The performance of stainless steel-clad reinforcement should be similar to that of solid stainless steel bars if the stainless steel coating is continuous and the black steel core, exposed at the bar ends, is covered so that it does not come into contact with the concrete pore solution. Assuming that a bridge deck, constructed using stainless steel-clad reinforcement with the ends well covered, does not require any repairs within its 75 year economic life, the present value of the cost of a bridge deck built with stainless steel-clad reinforcement is significantly lower than the present value for the cost of any other corrosion protection system.

Low permeability concrete and concrete overlays have been successful in delaying the penetration of chloride ions into bridge decks as long as cracking is kept to a minimum, generally through the use of proper curing techniques. Research has indicated that low-slump, silica fume, and latex-modified overlays have similar service lives, as long as they are constructed and cured properly. Of the three, low-slump overlays are the least expensive.

Many transportation agencies in the northeastern United States and Canada use waterproof membranes with asphalt overlays as the standard method for protecting bridge decks from corrosion. Agencies are divided on the effectiveness of membranes. Some do not like to place asphalt on bridge decks because it traps water and hides deterioration in the concrete deck beneath, but many researchers feel that membranes offer excellent protection if they are properly installed, and they have performed well in many areas. Spray-applied liquid methacrylate membranes are expected to prevent the ingress of water and chloride ions into bridge decks for up to 50 years. Hot rubberized asphalt membranes are expected to last for up to 40 years, and are the least expensive option, other than stainless steel-clad reinforcement, in the present value economic analysis.

Once the chloride concentration in a bridge has exceeded the corrosion threshold, the only way to prevent the corrosion of bare steel may be through electrochemical methods.

Cathodic protection systems can be installed permanently on both bridge decks and substructure members. They have been successful at stopping corrosion, even in severely contaminated structures. However, these systems require regular maintenance, which can be expensive. Currently, the most common impressed-current anode in use for the cathodic protection of reinforced concrete bridge decks is the titanium mesh anode, used in conjunction with a concrete overlay. The system fills the need for a durable impressed-current cathodic protection anode for bridge decks and pilings.

Cathodic protection systems are the most expensive corrosion protection options for new bridge decks in the present value cost analysis. However, cathodic protection systems are often installed on bridges that would otherwise need to be replaced, an option that is not addressed in this study. If they can prevent a bridge from needing to be replaced, these systems could still be cost-effective.

Zinc mesh pile jacket anodes show promise as sacrificial anodes for the splash zone of bridge piles in marine environments. Zinc-hydrogel anodes can provide protection for substructure members in marine or inland environments, as long as water can be kept out of the system.

Cathodic protection can be applied effectively and safely to prestressed concrete bridge members. However, if the resistivity of the concrete is not uniform, it may be difficult to obtain sufficient protection at areas of high resistance without generating hydrogen in areas of low resistance. Cathodic protection is not recommended for prestressed concrete structures with highly variable resistivity, which is often caused by large variations in moisture content.

Electrochemical chloride extraction has been shown to remove a significant fraction of chloride ions from contaminated structures. The procedure is not yet used as a standard corrosion protection method because most agencies cannot afford to close down even part of a bridge deck for the 6 to 8 weeks required for the process. However, this method has the potential to be an effective tool in the corrosion protection arsenal.

Corrosion inhibitors have been shown to protect against corrosion in chloride contaminated concrete in laboratory tests, but information on their performance in the field is limited. Research has shown that the corrosion inhibitor calcium nitrite can extend the service life of concrete structures. Because calcium nitrite reduces corrosion by chemically reacting with the steel, the effectiveness of calcium nitrite is dependent on the ratio of chloride-to-nitrite ions, and calcium nitrite is used up over time as it protects the reinforcing steel. Organic inhibitors provide a physical barrier that limits the number of chloride ions that reach the surface of the steel reinforcement. Both types of corrosion inhibitor have the potential to be cost-effective, if they perform as well in the field as they have in the laboratory.

## 7.3 IMPLEMENTATION PLAN

Based on the results of this review, it is recommended that the focus of corrosion protection for bridges remain the use of good quality, low permeability concrete and adequate cover over the reinforcement. Of the corrosion protection methods that have been proven to be effective in the field, the method with the lowest present value cost is the use of a hot rubberized asphalt membrane system, followed by the use of a spray-applied liquid membrane system, the use of a low-slump overlay, and the use of epoxy-coated reinforcement.

Hot rubberized asphalt and spray-applied liquid membranes should be considered for use on future projects. When concrete overlays are placed, low slump concrete should be considered in place of silica fume concrete. Epoxy-coated reinforcement should still be used as an inexpensive backup to these options.

According to the results of the economic analysis performed for this study, the most cost-effective corrosion protection option is the use of stainless steel-clad reinforcement. This method should be considered for experimental use. Solid stainless steel should be considered as well if a low discount rate (around 2%) is used to figure present value costs.

Calcium nitrite and organic corrosion inhibitors were also among the lowest cost corrosion protection options for all three discount rates used, and should be considered for experimental use.

## 7.4 RECOMMENDATIONS FOR FUTURE STUDY

It is recommended that future work in this area include quantitative comparisons. Most of the corrosion protection systems discussed in this report have never been directly compared in either laboratory or field studies. Such research would make service life comparisons much easier.

Further analysis should be performed on silica fume overlays to confirm recent results, that the extra cost is not justified based on corrosion protection performance.

Trials should be performed with stainless steel-clad reinforcement in concrete. If the ends of the bars can be effectively covered, and if the bars perform as well as they have in limited laboratory studies, this method could be the most cost-effective corrosion protection method available.

Other corrosion protection methods that have shown promise in laboratory studies, and have some of the lowest present value costs following stainless steel-clad bars, solid stainless steel reinforcement, and hot rubberized asphalt or spray applied liquid membranes (2% discount rate), are calcium nitrite and organic corrosion inhibitors. Field studies should be performed to confirm promising laboratory results.

Future economic evaluations should be considered to determine the costs of the disruption to society caused by bridge repair.

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## Table A.1 Summary of Repair Costs for Bridges in Kansas

| Bridge Number | Bridge type | Year built | Age at repair (years) | Machine preparation | Area prep. for patching | Area prep. for patching (full depth) | Silica fume overlay | Deck area ($m^2$) | Material replacement cost | Total incidental cost | Incidental cost ($/$m^2$) | Total repair cost | % of total cost for materials |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 75-42 | SWGC | 1972 | 27 | $7,808 | $45,920 | $18,040 | $25,116 | 546 | $96,884 | $89,881 | $165 | $186,765 | 52% |
| 45-16 | SBMC | 1941 | 58 | $6,149 | $72,285 | $20,304 | $35,910 | 630 | $134,648 | $92,076 | $146 | $226,724 | 59% |
| 11(60) | SBMC | 1934 | 65 | $13,146 | $4,080 | $1,500 | $25,860 | 431 | $44,586 | $48,353 | $112 | $92,939 | 48% |
| 104(21) | SBMC | 1949 | 50 | $14,148 | $16,920 | $42,600 | $23,580 | 472 | $97,248 | $128,025 | $271 | $225,273 | 43% |
| 78-55 | SMCC | 1954 | 45 | $19,638 | $83,898 | $18,414 | $78,894 | 1,655 | $200,844 | $164,033 | $99 | $364,877 | 55% |
| 13-21 | SBMC | 1963 | 36 | $16,484 | $15,675 | $8,400 | $37,720 | 472 | $78,279 | $93,522 | $198 | $171,801 | 46% |
| 39-49 | PDGC | 1975 | 23 | $6,852 | $9,114 | $2,720 | $26,508 | 702 | $45,194 | $51,290 | $73 | $96,484 | 47% |
| 39-48 | PDGC | 1975 | 23 | $13,016 | $43,183 | $2,720 | $66,927 | 1,725 | $125,846 | $52,670 | $31 | $178,516 | 70% |
| 48-65 | SBMC | 1976 | 23 | $21,775 | $26,100 | $1,250 | $39,195 | 871 | $88,320 | $74,537 | $86 | $162,857 | 54% |
| 48-66 | SBMC | 1975 | 24 | $21,775 | $30,500 | $1,250 | $39,195 | 871 | $92,720 | $74,537 | $86 | $167,257 | 55% |
| 46-127 | RCSH | 1973 | 26 | $6,867 | $11,800 | $4,810 | $39,676 | 763 | $63,153 | $114,472 | $150 | $177,625 | 36% |
| 46-130 | RCSH | 1973 | 26 | $8,100 | $54,575 | $29,600 | $46,800 | 900 | $139,075 | $113,692 | $126 | $252,767 | 55% |
| 21-37 | RCSH | 1961 | 38 | $15,155 | $41,410 | $3,150 | $29,145 | 583 | $88,860 | $64,060 | $110 | $152,920 | 58% |
| 92-001 | RCSH | 1963 | 36 | $10,475 | $6,975 | $2,813 | $27,235 | 419 | $47,498 | $62,882 | $150 | $110,380 | 43% |
| 84-57 | RCSH | 1965 | 34 | $12,459 | $8,525 | $3,500 | $29,794 | 542 | $54,278 | $43,190 | $80 | $97,468 | 56% |
| 92-16 | RCSH | 1957 | 42 | $10,443 | $54,590 | $3,850 | $27,058 | 474 | $95,941 | $45,167 | $95 | $141,108 | 68% |
| 92-18 | RCSH | 1957 | 42 | $5,865 | $32,065 | $3,500 | $16,796 | 267 | $58,226 | $45,717 | $171 | $103,943 | 56% |
| 74-11 | RDGH | 1962 | 37 | $10,769 | $30,150 | $8,400 | $24,679 | 449 | $73,997 | $62,793 | $140 | $136,790 | 54% |
| 74-13 | RDGH | 1962 | 37 | $10,769 | $20,700 | $5,600 | $24,679 | 449 | $61,747 | $62,543 | $139 | $124,290 | 50% |
| 2(19) | RDGH | 1960 | 39 | $20,880 | $55,800 | $6,080 | $34,800 | 696 | $117,560 | $94,996 | $136 | $212,556 | 55% |
| 39-27 | RDGH | 1969 | 30 | $25,770 | $92,700 | $64,500 | $51,540 | 1,718 | $234,510 | $59,575 | $35 | $294,085 | 80% |
| 6(31) | RDGH | 1961 | 38 | $15,650 | $29,400 | $27,250 | $41,779 | 626 | $114,079 | $103,335 | $165 | $217,414 | 52% |
| 6(45) | RDGH | 1955 | 44 | $12,510 | $6,000 | $9,375 | $27,647 | 417 | $55,532 | $116,347 | $279 | $171,879 | 32% |
| 92-2 | RDGH | 1963 | 36 | $11,218 | $18,450 | $3,938 | $29,166 | 448 | $62,772 | $62,362 | $139 | $125,134 | 50% |
| 84-59 | RDGH | 1965 | 34 | $12,353 | $36,750 | $9,100 | $29,541 | 537 | $87,744 | $46,985 | $87 | $134,729 | 65% |

# Table A.2 Present Value Costs For Corrosion Protection Options

| Corrosion protection method | New cost | Repair 1 cost | Time to repair 1 | Repair 2 cost | Time to repair 2 | Repair 3 cost | Time to repair 3 | Repair 4 cost | Time to repair 4 | Maint. and opp. cost | Present value of costs at 2% | Present value of costs at 4% | Present value of costs at 6% |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | $\$/m^2$ | $\$/m^2$ | years | $\$/m^2$ | years | $\$/m^2$ | years | $\$/m^2$ | years | $\$/m^2/yr$ | $\$/m^2$ | $\$/m^2$ | $\$/m^2$ |
| 1. No protection | $125 | $259 | 25 | $259 | 50 | | | | | | $379 | $259 | $199 |
| 2. Epoxy coated steel | $135 | $259 | 30 | $259 | 55 | | | | | | $365 | $245 | $191 |
| 3. Stainless steel reinforcement | $277 | | | | | | | | | | $277 | $277 | $277 |
| 4. Stainless steel-clad reinforcement | $155 | | | | | | | | | | $155 | $155 | $155 |
| 5. Galvanized reinforcement | $150 | $259 | 27 | $259 | 52 | | | | | | $394 | $274 | $216 |
| 6. Silica fume concrete | $204 | $259 | 30 | $259 | 60 | | | | | | $426 | $308 | $257 |
| 7. Low-slump overlay | $150 | $242 | 30 | $242 | 55 | | | | | | $365 | $253 | $202 |
| 8. Silica fume overlay | $178 | $266 | 30 | $266 | 55 | | | | | | $414 | $291 | $235 |
| 9. Latex-modified concrete overlay | $188 | $278 | 25 | $278 | 50 | | | | | | $461 | $331 | $268 |
| 10. Polymer concrete overlay | $125 | $252 | 25 | $201 | 40 | $252 | 55 | | | | $454 | $291 | $213 |
| 11. Preformed sheet membrane | $164 | $169 | 20 | $169 | 40 | $169 | 60 | | | | $406 | $292 | $238 |
| 12. Spray applied liquid membrane | $204 | $27 | 20 | $27 | 35 | $214 | 50 | $27 | 70 | | $322 | $255 | $228 |
| 13. Torch applied membrane | $173 | $183 | 20 | $183 | 40 | $183 | 60 | | | | $435 | $312 | $253 |
| 14. Hot rubberized asphalt membrane | $160 | $27 | 20 | $170 | 40 | $27 | 60 | | | | $263 | $210 | $186 |
| 15. Linseed oil | $142 | $6 | 3 | $6 | 6 | $255 | 25 | $255 | 55 | | $427 | $299 | $238 |
| 16. Silane | $144 | $8 | 7 | $8 | 14 | $255 | 35 | $250 | 60 | | $387 | $255 | $199 |

## Table A.2 Continued — Present Value Costs For Corrosion Protection Options

| Corrosion protection method | New cost | Repair 1 cost | Time to repair 1 | Repair 2 cost | Time to repair 2 | Repair 3 cost | Time to repair 3 | Repair 4 cost | Time to repair 4 | Maint. and opp. cost | Present value of costs at 2% | Present value of costs at 4% | Present value of costs at 6% |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
|  | $\$/m^2$ | $\$/m^2$ | years | $\$/m^2$ | years | $\$/m^2$ | years | $\$/m^2$ | years | $\$/m^2/yr$ | $\$/m^2$ | $\$/m^2$ | $\$/m^2$ |
| 17. Titanium mesh cathodic protection | $250 | $264 | 30 | $264 | 60 |  |  |  |  | $1 | $515 | $380 | $320 |
| 18. Slotted anode cathodic protection | $277 | $291 | 30 | $291 | 60 |  |  |  |  | $1 | $565 | $418 | $353 |
| 19. Sprayed zinc cathodic protection | $248 | $108 | 15 | $108 | 30 | $108 | 45 | $108 | 60 | $1 | $504 | $394 | $339 |
| 20. Electrochemical chloride extraction | $125 | $355 | 25 | $355 | 55 |  |  |  |  |  | $461 | $299 | $222 |
| 21. Calcium nitrite | $145 | $259 | 40 | $259 | 70 |  |  |  |  |  | $327 | $216 | $175 |
| 22. Organic corrosion inhibitor | $145 | $259 | 37 | $259 | 67 |  |  |  |  |  | $338 | $224 | $180 |
| 23. Epoxy-coated steel + silica fume concrete | $185 | $259 | 35 | $259 | 70 |  |  |  |  |  | $379 | $267 | $223 |
| 24. Epoxy coated steel + low-slump overlay | $154 | $242 | 35 | $242 | 60 |  |  |  |  |  | $349 | $238 | $193 |
| 25. Epoxy-coated steel + silica fume overlay | $182 | $266 | 35 | $266 | 60 |  |  |  |  |  | $396 | $275 | $225 |
| 26. Epoxy-coated steel + latex-modified concrete overlay | $192 | $278 | 25 | $278 | 50 |  |  |  |  |  | $465 | $335 | $272 |
| 27. Epoxy-coated steel + polymer concrete overlay | $135 | $252 | 30 | $201 | 45 | $252 | 60 |  |  |  | $433 | $271 | $201 |
| 28. Epoxy-coated steel + preformed sheet membrane | $168 | $169 | 20 | $169 | 40 | $169 | 60 |  |  |  | $410 | $296 | $242 |
| 29. Epoxy-coated steel + spray applied liquid membrane | $208 | $27 | 20 | $27 | 35 | $214 | 50 | $27 | 70 |  | $326 | $259 | $232 |
| 30. Epoxy-coated steel + torch applied membrane | $177 | $183 | 20 | $183 | 40 | $183 | 60 |  |  |  | $439 | $316 | $257 |
| 31. Epoxy coated steel + hot rubberized asphalt membrane | $164 | $27 | 20 | $170 | 40 | $27 | 60 |  |  |  | $267 | $214 | $190 |
| 32. Epoxy-coated steel + calcium nitrite | $149 | $259 | 40 | $259 | 70 |  |  |  |  |  | $331 | $220 | $179 |
| 33. Epoxy-coated steel + organic corrosion inhibitor | $149 | $259 | 37 | $259 | 67 |  |  |  |  |  | $342 | $228 | $184 |