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Final Report

METHODS OF CORROSION PROTECTION AND DURABILITY OF CONCRETE BRIDGE DECKS REINFORCED WITH EPOXY-COATED BARS - PHASE I

Lisa M. Samples
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16. Abstract

In this study, the results of an extensive field and laboratory investigation conducted to evaluate the corrosion performance of epoxy-coated reinforcement are reported. One hundred and twenty three concrete bridge decks containing epoxy-coated reinforcement, uncoated reinforcement, and various other corrosion protection systems were included in a field investigation. An initial visual inspection of these bridge decks showed that 44% had signs of distress from corrosion of the reinforcement. Six decks, representing 5% of the total sample, were selected for a second more detailed survey. Three of the six decks had epoxy-coated reinforcement. This number represents 11% of the total sample of bridge decks with epoxy-coated reinforcement. Corrosion of the epoxy-coated reinforcement was discovered in areas of cracking and shallow cover on two of these three bridge decks. New construction bridge sites were also visited to perform holiday testing on epoxy-coated reinforcement upon arrival to the site, after placement, and after casting. The results from the holiday testing showed that an average of 31 holidays per meter (9 holidays per foot) were created during the concrete casting operation. Increasing the coating thickness by 101.6 μm (4 mils) and lowering the hose used when pumping concrete to the level of the top mat of reinforcement reduced the holidays created to 11.5 holidays per meter (3.5 holidays per foot) after casting. The results of 16 specimens subjected to cycles of exposure in the laboratory evaluation indicated that if the mat to mat resistance was greater than 5000 ohms, then over the 77-week period no corrosion activity was observed. No corrosion activity occurred even though the chloride levels at the reinforcement location were well above 0.79 kg/m^3 (1.3 lb/yd^3), the threshold level for initiation of corrosion. A possible way to achieve a high mat to mat resistance would be by limiting the damage to the epoxy coatings.

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Methods of Corrosion Protection and Durability of Concrete Bridge Decks Reinforced with Epoxy-Coated Bars – Phase I

Introduction

Prior to the use of de-icing salts on bridge decks, concrete was thought to be a relatively low maintenance material. The significant damage to bridge decks in the United States due to corrosion of the reinforcing steel resulting from salt application has changed this image. Epoxy-coated reinforcement was first proposed as a solution to the problem of bridge deck deterioration due to reinforcing steel corrosion in the early 1970's. Although no indication of any corrosion of epoxy-coated reinforcement in Indiana had been reported, several experimental studies and field observations suggest that premature corrosion of epoxy-coated reinforcement may occur. As other corrosion protection systems used in Indiana begin to economically compete with epoxy-coated reinforcement, a reassessment of the benefits obtained by using epoxy-coated reinforcement is justified.

The objective of this study was to determine the performance of epoxy-coated reinforcement in concrete bridge decks in regards to durability. This objective was accomplished through the completion of an inspection of one hundred twenty three bridge decks in the field and a laboratory evaluation. The field evaluation included an investigation of the damage created to the coating of epoxy-coated reinforcement during bridge deck construction operations as well as an investigation of existing concrete bridge decks. The laboratory evaluation used exposure testing to provide an accelerated corrosive environment in order to evaluate aspects of the performance not available from the field evaluation. These aspects included the effect of epoxy coating thickness, and the amount of coating damage on the corrosion performance of epoxy-coated reinforcement.

Findings

One hundred and twenty three concrete bridge decks containing epoxy-coated reinforcement, uncoated reinforcement, and various other corrosion protection systems were included in a field investigation. An initial visual inspection of these bridge decks showed that 44% had signs of distress from corrosion of the reinforcement. Six decks, representing 5% of the total sample, were selected for a second more detailed survey. Three of the six decks had epoxy-coated reinforcement. This number represents 11% of the total sample of bridge decks with epoxy-coated reinforcement. Corrosion of the epoxy-coated reinforcement was discovered in areas of cracking and shallow cover on two of these three bridge decks. New construction bridge sites were also visited to perform holiday testing on epoxy-coated reinforcement upon arrival to the site, after placement, and after casting. The results from the holiday testing showed that an average of 31 holidays per meter (9 holidays per foot) were created during the concrete casting operation. Increasing the coating thickness by 101.6 µm (4 mils) and lowering the hose used when pumping concrete to the level of the top mat of reinforcement reduced the holidays created to 11.5 holidays per meter (3.5 holidays per foot) after casting.

The results of 16 specimens subjected to cycles of exposure in the laboratory evaluation indicated that if the mat to mat resistance was greater than 5000 ohms, then over the 77-week period no corrosion activity was observed. No corrosion activity occurred even though the chloride levels at the reinforcement location were well above 0.79 kg/m^3 (1.3 lb/yd^3), the threshold level for initiation of corrosion. A possible way to achieve a high mat to mat resistance would be by limiting the damage to the epoxy coating.

Implementation

The results of the laboratory evaluation of this study indicate that corrosion of epoxy-coated reinforcement can be prevented with a high mat to mat resistance. A high mat to mat resistance can be provided by the use of epoxy-coated reinforcement with limited damage to the coating. The results of the field evaluation show that excessive damage to epoxy-coated reinforcement is being created in the bridge deck construction operations. The field evaluation showed that a thicker epoxy coating will limit the amount of damage to the coating. The laboratory phase showed that reducing the number of defects will increase the mat to mat resistance when utilizing epoxy-coated reinforcement, thus improving corrosion performance.

Based on the results of this study, it is recommended that an increase of 152.4 μm (6 mils) to the minimum coating thickness be implemented for use in bridge deck steel reinforcement. This increase implies an allowable range of 304.5 to 457.2 μm (12 to 18 mils). It is anticipated that the increase of only 152.4 μm (6 mils) in coating thickness will not adversely affect the bond performance. The use of a thicker coating will significantly decrease the damage to the epoxy coating, which will increase the effectiveness of epoxy-coated reinforcement as a corrosion protection method.

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TABLE OF CONTENTS

Page

Page

Page

APPENDICES

LIST OF TABLES

Table Page

LIST OF FIGURES

Figure Page

IMPLEMENTATION SUGGESTIONS

The results of the laboratory evaluation of this study indicate that corrosion of epoxy-coated reinforcement can be prevented with a high mat to mat resistance. A high mat to mat resistance can be provided by the use of epoxy-coated reinforcement with limited damage to the coating. The results of the field evaluation show that excessive damage to epoxy-coated reinforcement is being created in the bridge deck construction operations. The field evaluation showed that a thicker epoxy coating will limit the amount of damage to the coating. The laboratory phase showed that reducing the number of defects will increase the mat to mat resistance when utilizing epoxy-coated reinforcement, thus improving the corrosion performance.

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CHAPTER 1 – INTRODUCTION

1.1 Background

Prior to the use of de-icing salts on bridge decks, concrete was thought to be a relatively low maintenance material. The significant damage to bridge decks in the United States due to corrosion of the reinforcing steel resulting from salt application has changed this image. De-icing salts first began to be used on bridge decks in the late 1950’s, and corrosion of the reinforcing steel emerged as a problem in the 1960’s. Epoxy-coated reinforcement was first proposed as a solution to the problem of bridge deck deterioration due to reinforcing steel corrosion in the early 1970's. Since epoxy-coated bars were thought to significantly increase the service life of bridge decks, although the increase was never quantified, it was generally accepted that epoxy-coated bars were cost-effective.

Epoxy-coated reinforcement was first used in a bridge deck in Indiana in 1976. Since that time, the combination of epoxy-coated reinforcement and a minimum cover of 63.5 mm (2.5 in.) of Class C concrete has become the primary method of corrosion protection used by the Indiana Department of Transportation. Before this study in Indiana, an extensive review of the performance of bridge decks with epoxy-coated reinforcement in the field had not been performed. Although no indication of any corrosion of epoxy-coated reinforcement in Indiana had been reported, several experimental studies and field observations suggest that premature corrosion of epoxy-coated reinforcement may occur. Premature corrosion of epoxy-coated reinforcement may limit the increase in service life of bridge decks thought to be realized by the use of epoxy-coated reinforcement. As other corrosion protection systems used in Indiana begin to economically compete with epoxy-coated reinforcement, a reassessment of the benefits obtained by using epoxy-coated reinforcement is justified.

1.2 Objective

The objective of this study is to determine the performance of epoxy-coated reinforcement in concrete bridge decks in regards to durability. This objective was accomplished through the completion of an inspection of one hundred twenty three bridge decks in the field and a laboratory evaluation. The field evaluation included an investigation of the damage created to the coating of epoxy-coated reinforcement during bridge deck construction operations as well as an investigation of existing concrete bridge decks. The laboratory evaluation used exposure testing to provide an accelerated corrosive environment in order to evaluate aspects of the performance not available from the field evaluation. These aspects included the effect of epoxy coating thickness, and the amount of coating damage on the corrosion performance of epoxy-coated reinforcement.

The goal of this study is to provide information necessary for the examination of the corrosion performance of epoxy-coated reinforcement. Specifically, the effect of coating damage and thickness on the corrosion performance of epoxy-coated reinforcement is addressed. This information will be helpful in comparing the benefits of epoxy-coated reinforcement with other corrosion protection methods.

1.3 Scope

The scope of this report is indicated by the chapter title and organization. The fundamentals of corrosion of reinforcement are included in Chapter 2. Chapter 3 provides a literature review on the relevant issues concerning epoxy-coated reinforcement. This includes the coating process and specifications, corrosion performance of epoxy-coated reinforcement in bridge structures, and corrosion performance in laboratory studies. Chapter 4 contains the information regarding the field evaluation including the description and the results of the field evaluation. The laboratory evaluation including test description, results, and analysis is provided in Chapter 5. Chapter 6 contains the summary, conclusions, recommendations, implementation, and future research.

CHAPTER 2 – FUNDAMENTALS OF CORROSION OF REINFORCEMENT

The destruction or deterioration of a material after reacting with the environment is defined as corrosion[1]. Corrosion damage to reinforced concrete structures is one of the most expensive corrosion problems in the United States[2]. In 1994, USA Today estimated the cost of repairing all of the nation's deficient bridges at $78 billion[3]. However, only $5 billion a year is currently spent on bridge repair and replacement[3]. The understanding of the process of corrosion is necessary in order to evaluate and develop solutions to this costly problem.

This chapter focuses on why and how corrosion occurs. Discussed in this chapter are explanations of: the basics of corrosion theory, corrosion of uncoated reinforcement in concrete bridge decks, and corrosion of epoxy-coated reinforcement in concrete bridge decks.

2.1 Basics of Corrosion Theory

Metals such as steel are formed through extractive metallurgy. This process extracts metals using heat energy from naturally occurring chemical compounds, known as ores. This energy is stored and provides the driving force for corrosion. The energy is released as the metal corrodes and returns back to its original state. The energy required during the forming of a metal and released during corrosion of the metal varies from metal to metal. Table 2.1 lists some common metals with the energy required to convert to metal[4]. The metals that require more energy to convert from ore will be the most “eager” to return to the naturally occurring state through corrosion.

2.1.1 Electrochemical Process of Corrosion

Corrosion of metals can be classified as either dry corrosion or wet corrosion. Dry corrosion occurs without a liquid phase and the metal reacts with gases or vapors. This form of corrosion usually occurs at high temperatures. Wet corrosion involves a reaction between metal and an aqueous solution. The greatest amount of damage to materials is created by wet corrosion. Since corrosion of reinforcement is a wet corrosion process, only wet corrosion will be discussed in this chapter.

Wet corrosion is almost always electrochemical in nature[2]. A chemical reaction involving the transfer of electrons or involving oxidation or reduction reactions is defined as an electrochemical reaction. Current flow, from one area of the metal surface through a solution to another area of the metal surface, is created during corrosion. The solution the current flows through is called the electrolyte. The electrolyte is conductive due to the presence of ions, which are positively or negatively charged atoms, in the solution. The area of metal from which electrons migrate is called the anode. The cathode is defined as the area where the electrons return to the metal. The electron flow is conveniently measured as current. The circuit must be completed through the metal or by a conductive connection between two pieces of metal. Metal dissolution occurs at the anode, no metal dissolution occurs at the cathode.

The electrochemical nature of the corrosion process can be illustrated by the corrosion of zinc when exposed to hydrochloric acid[1]. Zinc reacts with hydrochloric acid to form soluble zinc chloride and hydrogen gas as shown in Equation 2.1.

$$Zn + 2HCl \rightarrow ZnCl_2 + H_2 \qquad \text{Eqn. 2.1}$$

Both hydrochloric acid and zinc chloride disassociate in water and exist in ionic form. Therefore, Equation 2.1 can be rewritten as:

$$Zn + 2H^+ + 2Cl^- \rightarrow Zn^{2+} + 2Cl^- + H_2 \qquad \text{Eqn. 2.2}$$

Equation 2.2 can be simplified by eliminating $2Cl^-$ from both sides of the reaction:

$$Zn + 2H^+ \rightarrow Zn^{2+} + H_2 \qquad \text{Eqn. 2.3}$$

Zinc reacts with the hydrogen ions of an acidic solution to form zinc ions and hydrogen gas. Zinc is oxidized to zinc ions while hydrogen ions are reduced to hydrogen gas. Equation 2.3 is known as a cell reaction and can be divided into two half-cell reactions.

$$Zn \rightarrow Zn^{2+} + 2e^- \ \text{(Anodic Reaction)} \qquad \text{Eqn. 2.4}$$

$$2H^+ + 2e^- \rightarrow H_2 \ \text{(Cathodic Reaction)} \qquad \text{Eqn. 2.5}$$

Metal dissolves releasing electrons into the metal at the anode by Equation 2.4. The electrons migrate to the cathode where they react with H^+ in solution to form H_2 by Equation 2.5. Water, the electrolyte, is required as the carrier for ions such as Zn^{2+} and H^+. In order to avoid an accumulation of charge, both of the half-cell reactions must occur simultaneously and at the same rate. Figure 2.1 provides a visual representation of the corrosion process[1].

All corrosion processes can be characterized by their half-cell reactions. The anodic reaction is the oxidation of metal into its ionic form and can be expressed by the general form:

$$M \rightarrow M^{n+} + ne^-$$ Eqn. 2.6

Several different cathodic reactions are common in corrosion of metals[1]. The most common reactions are listed below.

Hydrogen evolution (acidic solutions):

$$2H^+ + 2e^- \rightarrow H_2$$ Eqn. 2.7

Oxygen reduction (acidic solutions):

$$O_2 + 4H^+ + 4e^- \rightarrow 2H_2O$$ Eqn. 2.8

Oxygen reduction (neutral/basic solutions):

$$O_2 + 2H_2O + 4e^- \rightarrow 4OH^-$$ Eqn. 2.9

Metal ion reduction:

$$M^{3+} + e^- \rightarrow M^{2+}$$ Eqn. 2.10

Metal ion deposition:

$$M^+ + e^- \rightarrow M$$ Eqn. 2.11

2.1.2 Corrosion of Steel (Iron)

The previous half-cell reactions can be used to explain the corrosion of reinforcement. When steel is exposed to water and the atmosphere, corrosion occurs. The anodic reaction is:

$$Fe \rightarrow Fe^{2+} + 2e^-$$ Eqn. 2.12

Water and saltwater are the most common electrolytes reinforcement encounters. The pH of water or saltwater is approximately 7, providing a nearly neutral environment. Dissolved oxygen is provided to the system from the atmosphere. Therefore, corrosion of reinforcement usually involves Equation 2.9 as the cathodic reaction[4]. Deicing salts or seawater may provide sodium or chloride ions to the system; however, these ions do not participate in the electrochemical reactions. The effect of chloride ions on the corrosion of reinforcement is discussed in later sections. Combining Equation 2.9 and Equation 2.12 gives the following cell reaction:

$$2Fe + O_2 + 2H_2O \rightarrow 2Fe^{2+} + 4OH^- \qquad \text{Eqn. 2.13}$$

The combination of ferrous ions (Fe^{2+}) and hydroxyl ions (OH^-) produces ferrous hydroxide ($Fe(OH)_2$) by the following reaction:

$$2Fe^{2+} + 4OH^- \rightarrow 2Fe(OH)_2 \qquad \text{Eqn. 2.14}$$

Ferrous hydroxide is a solid and will precipitate from the solution. However, this oxide is unstable in solutions containing dissolved oxygen, and will oxidize to ferric hydroxide by the following reaction:

$$2Fe(OH)_2 + \tfrac{1}{2}O_2 + H_2O \rightarrow 2Fe(OH)_3 \qquad \text{Eqn. 2.15}$$

Ferric hydroxide will dehydrate after exposure to the atmosphere to produce ferric oxide (Fe_2O_3). Ferric oxide is the reddish brown compound commonly referred to as rust.

Occasionally, more than one reduction reaction can occur during the corrosion process. An aerated and acidic environment would allow for both Equation 2.7 and Equation 2.8 to function as cathodic reactions, and combine with iron dissolution as the anodic reaction. Since the rates of oxidation and reduction must always be equal, the addition of a second cathodic reaction will accelerate the anodic reaction. This explains why acidic solutions containing dissolved oxygen are more corrosive then deaerated acids. The opposite is also true, reducing the rate of either the cathodic or anodic reaction will slow the corrosion rate. Elimination of dissolved oxygen from the solution will stop oxygen reduction, which will halt the iron dissolution. Therefore, iron or steel will not corrode in deaerated water or saltwater.

2.1.3 Experimental Measurements

The cell potential, E, can be measured to determine the energy change in any electrochemical cell[2]. As discussed previously, a cell reaction can be divided into two half-cell reactions. If one of the half-cells is a known or reference electrode, the second can be isolated for investigation. The concepts of standard potentials, reference electrodes, and corrosion potentials allow for experimental measurements to be taken in corrosion cells.

2.1.3.1 Standard Potentials

Corrosion of a metal releases energy that can be related to the potential of the metal. When a reference point is established the potential can be measured. The standard hydrogen half-cell is the most commonly used reference point. The standard hydrogen half-cell, also known as the standard hydrogen electrode (SHE), has been fixed as the zero point on the potential scale[2]. The hydrogen half-cell is constructed by suspending a platinum foil specimen in a sulfuric acid solution. The sulfuric acid solution has unit activity H^+ and is bubbled with purified hydrogen. A diagram of the SHE is shown in Figure 2.2. Potentials of metals are measured with reference to the SHE as units of volts. Metals that have positive potentials when measured against the SHE are referred to as noble or passive metals. Active metals have a negative potential with respect to the SHE. Table 2.2 lists the position of some common metals with respect to the SHE.

2.1.3.2 Reference or Half-Cell Electrodes

Although the SHE is the standard electrode chosen as the reference point for the potential scale, it is not commonly used for potential measurements. The SHE is awkward to use in many experimental situations, and other more convenient standard half-cells have been developed. Table 2.3 provides a list of the most common half-cells and their potentials. The Copper-Copper-Sulfate Electrode (CSE) is commonly used for experimental measurements in reinforced concrete structures. Figure 2.3 shows a schematic diagram of a CSE. Measurements made with any half-cell other than the SHE can be related to the SHE through addition or subtraction of the half-cell potential.

2.1.3.3 Corrosion Potentials

Potentials of corroding metals can be measured in the field and laboratory using reference electrodes[4]. The corrosion potential is usually measured by connecting the metal to the positive terminal of a voltmeter and the reference electrode to the negative terminal. This process requires a voltmeter with a high internal resistance. The corrosion potential is measured as the voltage difference between the metal and the reference electrode. The type of reference electrode should be recorded as well as the magnitude and sign of the voltage difference. Temperature, the corrosive environment, and the type of metal can all affect the corrosion potential.

2.1.4 Forms of Corrosion

Metals may corrode in a different manner depending on the nature of the metal, the environment, stress, temperature, velocity of the electrolyte, presence of oxygen, etc[4]. Each form is visually recognizable, and is classified based on appearance. There are eight forms of corrosion, and although they are interrelated, each is unique[1,2]. The eight forms of corrosion are: uniform corrosion, galvanic corrosion, crevice corrosion, pitting corrosion, environmentally induced cracking, hydrogen damage, intergranular corrosion, dealloying, and erosion corrosion. A brief description of each of these forms of corrosion is given below. Pitting and galvanic corrosion are the two forms of corrosion most commonly encountered in corrosion of reinforcement, and will be discussed in more detail in the following section.

The most common form of corrosion is uniform corrosion. This form of corrosion occurs as uniform attack over the exposed surface of metal. For uniform corrosion to occur, the corrosive environment must be the same over the entire surface of the metal, and the metal must be metallurgically and compositionally uniform[2]. The most widespread occurrence of uniform corrosion is atmospheric corrosion.

Galvanic corrosion occurs when two dissimilar metals are joined and exposed to a corrosive environment. Electrons will flow between the connected metals as a result of the potential difference. After the metals are connected, the corrosion of the more active metal will increase and the corrosion of the noble or passive metal will decrease[2]. The noble metal becomes the cathode and the active metal becomes the anode. Figure 2.4 shows the effect of galvanic corrosion for coated steel. The steel in Figure 2.4 is coated with either tin or zinc. If the coating is damaged and the metal is exposed to a corrosive environment, a galvanic couple will develop. Tin is more noble than steel, and steel is more noble than zinc. Therefore, in the case of tin-coated steel, the steel will act as the anode and corrode. However, in the case of zinc-coated steel, the zinc is more active and will become the anode. The steel will act as the cathode in this case and will not corrode. Area and distance are two important concepts in galvanic corrosion. As the size of the cathode increases relative to the anode, the corrosion rate will accelerate. With regard to distance, the rate of galvanic corrosion is always greatest near the junction of the couple and will decrease with distance away from the junction. Table 2.4 lists the electromotive force or EMF series for pure metals and oxygen with reference to the SHE. This chart can be used to predict galvanic corrosion between two pure metals by determining which metal is more active; however, alloys are more common than pure metals in most situations. Table 2.5 provides the galvanic series in seawater for various alloys. This table can be used to predict galvanic corrosion for alloys. Table 2.4 can also be used to determine which metals will corrode in an acidic environment. All metals with half-cell potentials more negative than the hydrogen half-cell will have a tendency to corrode in acid solutions. Metals more negative than the two oxygen reduction reactions will have a tendency to corrode in any environment containing dissolved oxygen.

Corrosion that occurs within a crevice or sheltered area on a metal surface exposed to a corrosive environment is called crevice corrosion[2]. The shelter is created by contact with another material that does not have to be metal. Holes, lap joints, gasket surfaces, crevices under bolts, and other situations that trap stagnant solution promote the formation of crevice corrosion. Galvanic effects may also compound crevice corrosion if two metals are in contact.

Pitting is a form of localized corrosion. A cavity or hole with a surface diameter about the same or less than the depth of the hole or cavity can be defined as a pit[4]. Pitting can be one of the most destructive forms of corrosion; often pits are over looked due to their small size. Pitting with only a small percentage of metal loss has caused failures in equipment and other structures. Metals that are covered with a thin protective or passive surface film and exposed to stagnant corrosive conditions are very susceptible to pitting. Pits usually form at damaged or weak spots in the surface film. The process of pitting corrosion produces a unique self-perpetuating system[4]. Figure 2.5 shows a schematic diagram of metal being pitted by an aerated sodium chloride solution[1]. Oxygen reduction occurs adjacent to the pit while metal dissolution occurs inside the pit. Positively charged ions accumulate in the pit because of rapid metal dissolution. Negatively charged ions are attracted to the pit to maintain charge balance, this results in the formation of metal chlorides (MCl). Metal chlorides react with water to form a high concentration of hydrogen ions[1]. Equation 2.16 shows this reaction.

$$M^+Cl^- + H_2O = MOH + H^+ + Cl^- \qquad \text{Eqn. 2.16}$$

Both hydrogen and chloride ions accelerate metal dissolution. Due to the concentrated solution, no oxygen reduction occurs within the pit. Oxygen reduction does occur on the surfaces adjacent to the pit; and these areas act as the cathode while the pit functions as the anode.

Environmentally induced cracking includes stress corrosion cracking, corrosion fatigue cracking, and hydrogen-induced cracking[2]. This form of corrosion occurs in an environment that causes very little uniform corrosion, however, brittle fracture of otherwise ductile materials results. Alloys are susceptible to stress corrosion cracking, which occurs under constant tensile stress in a static environment. Corrosion fatigue cracking occurs in a corrosive environment when the metal is under cyclic stresses. Hydrogen induced cracking occurs when hydrogen produced from a cathodic reaction diffuses into the alloy lattice.

While hydrogen induced cracking may be reversible if the hydrogen is allowed to escape, other forms of hydrogen damage are irreversible[2]. Hydrogen may react with carbides in steel to form methane, which can cause voids, surface blisters, and decarburization. Reactive metals such as titanium, zirconium, magnesium, tantalum, and niobium may be embrittled by hydride formation.

Intergranular corrosion occurs when the grain boundaries of a metal are corroded preferentially[1]. This form of corrosion is localized and usually caused by impurities or depletion of alloying elements at the grain boundaries. Corrosion initiates at the surface of the metal and progresses along the grain boundaries. Eventually the alloy or metal will disintegrate or lose strength. Weld decay and knife-line attacks of stainless steels are two common examples of intergranular corrosion. In both of these cases, improper heat treatment promotes the corrosion process.

Dealloying is a form of corrosion that occurs when the active alloying element is preferentially corroded[2]. The leaching out of alloying elements will result in a material with poor mechanical properties. Brass and cast iron commonly suffer from this form of corrosion.

Erosion corrosion occurs with the combination of a corrosive fluid and high velocity. A fast moving corrosive fluid will remove the protective corrosion product film and expose the active metal or alloy[2]. This form of corrosion is often found in steel pipes used to carry fluids. A special case of erosion corrosion is cavitation, which occurs in very high velocity situations.

2.1.5 Concentration Cells

Many different concentration cells may be encountered in corrosion of metals. The formation of concentration cells produces a potential difference on the surface of the metal, which will promote corrosion. This potential difference will establish separate anodic and cathodic sites on the metal surface. The three main types of concentration cells encountered in the corrosion of metals are oxygen concentration cells, metal ion concentration cells, and chloride concentration cells.

2.1.5.1 Oxygen Concentration Cells

As shown in Equations 2.7 and 2.8, oxygen is needed to support a cathodic reaction. However, oxygen concentrations on the surface of metal may also promote corrosion (anodic activity)[4]. When a difference in oxygen concentration develops on the surface of metal, equilibrium forces oxygen reduction at the areas of high concentration. This reduction of oxygen is accomplished through a cathodic reaction. Areas with low levels of oxygen will become anodic and experience metal dissolution.

2.1.5.2 Metal Ion Concentration Cells

A high concentration of metal ions on the surface of a metal will promote corrosion. The metal will have a tendency to return to equilibrium. This force will cause areas of low concentration of metal ions to become anodic and increase metal dissolution.

Areas of high metal ion concentration will act as the cathode and metal deposition ("plating") will occur.

2.1.5.3 Chloride Concentration Cells

Chloride concentration cells are similar to oxygen and metal ion concentration cells. The potential difference between two different levels of chloride concentration promotes corrosion. The area of higher chloride concentration will become anodic, and the area of low chloride concentration will become cathodic[4]. Corrosion promoted by chlorides is an important issue in corrosion of reinforcement and will be discussed in detail in a later section.

2.1.6 Passivity

Certain environmental conditions can lead to the reduction or loss of chemical reactivity in some metals[4]. Under these conditions, ordinarily active metals will behave as noble metals. Common examples of these metals are iron, nickel, chromium, titanium, and alloys containing these metals. Passivation of metals remains to be somewhat of a mystery. A surface film is known to develop on passivated metal, which acts as a protective barrier. Unknown is whether the film is actually a very thin oxide layer or an adsorbed layer. Passive layers are difficult to examine due to their very thin and fragile nature. The oxide theory is generally accepted for the case of iron and steel. It has been shown that iron and steel will passivate in oxygenated basic solutions with a pH range of 11 to 13. As iron or steel passivates in a basic solution a ten-fold reduction in corrosion rate when compared with neutral or acidic solutions will occur[4]. However, at values greater than a pH of 13 the passive oxide layer on iron will dissolve and corrosion will resume.

2.2 Corrosion of Uncoated Reinforcement in Concrete Bridge Decks

Corrosion of reinforcement occurs when the passive oxide layer on steel is compromised. The alkalinity of cement paste promotes the growth of a protective, passive oxide layer providing low corrosion rates. However, carbonation and chloride contamination will destroy the passive layer. Once the layer is damaged or destroyed, corrosion will occur in the presence of oxygen and moisture. The reaction of carbon dioxide with cement components leads to carbonation. Carbonation lowers the pH of the concrete to a level at which the passive layer will be destroyed. Carbonation generally will not occur in sound, dense concrete with adequate cover, and therefore, is not a problem in concrete bridge decks[4]. Chloride contamination, however, is a significant problem in concrete bridge decks. Penetration of chloride ions does not lead to a drop in pH of the concrete; rather, the chloride ions react directly with the steel to destroy the passive layer. The loss of passivity may occur with as little as 0.025 to 0.033 percent Cl^- by concrete weight[4].

2.2.1 Corrosion Process and Effects

Corrosion of reinforcement in concrete bridge decks occurs through the same electrochemical process discussed in Section 2.1. Chloride, oxygen, and moisture are all necessary for corrosion of the reinforcement to occur. After breakdown of the passive layer, potential differences promote corrosion of the reinforcement by galvanic corrosion.

Four components are necessary for the galvanic corrosion to occur: an anode, a cathode, an electrolyte, and an external path between the anode and cathode. The four components of a galvanic corrosion cell are commonly found in concrete bridge decks. Steel ties and chairs act as a metallic path, and moist concrete acts as an electrolyte. Anodic and cathodic areas will form due to potential differences among the reinforcement. Differences in chloride ion concentration, moisture content, oxygen content, and many other factors can cause potential differences.

Generally, two forms of galvanic corrosion, macrocell corrosion and microcell corrosion can cause corrosion of reinforcement. Macrocell corrosion involves the corrosion between large anodic and cathodic areas on different reinforcing steel bars or reinforcing mats. Since the anodic and cathodic areas are located on different reinforcing steel bars or mats, an electrical connection between the separate areas is required for corrosion to occur. Macrocell corrosion often occurs on bridge decks in areas where the top mat of reinforcing steel is uniformly exposed to chlorides and the bottom mat is not. This different exposure condition creates a potential difference, which promotes the top mat of reinforcement to act as the anode and corrode while the bottom mat of reinforcement is protected. Conversely, microcell corrosion occurs over a very small, localized area. This situation occurs when only a small portion of the passive oxide layer on the reinforcing steel is damaged. The small damaged area will develop as the anode, while the adjacent undamaged areas will serve as the cathode. A large cathode area to anode area ratio will create excessively high corrosion rates and pitting may occur. Pitting may result in a significant loss of cross sectional area of the reinforcing steel.

Microcell corrosion is often observed in areas of cracking in concrete bridge decks where a small area of reinforcement may be exposed to high levels of chloride ions.

The effects of corrosion of reinforcement in concrete bridge decks are: loss of load carrying capacity of the steel due to section loss, possible loss of bond between the reinforcing steel and the concrete, and damage to the concrete from the corrosion products. Usually, the damage to the concrete determines the service life of the bridge deck. The corrosion products formed during the rusting process occupy a greater volume than the original steel. This increased volume creates tensile forces in the concrete, which eventually cause cracking. Cracks expose the steel to more chlorides, oxygen, and moisture that accelerate the corrosion process. Delaminations may occur as a result of the corrosion. Delaminations are areas in the bridge deck where cracks have caused a separation or hollow area in the concrete that may not be visually apparent. With progressive corrosion, cracks and delaminations will join and the concrete will eventually break off, referred to as “spalling”.

2.2.2 Electrochemical Process

The following equations explain the process of corrosion of reinforcement when exposed to chloride ions. Areas of reinforcement exposed to chlorides become anodic when the passive oxide layer is destroyed. Cathodic areas form in areas where the passive layer is still intact. Equation 2.12 is the anodic reaction that takes place to form positively charged ions and electrons. Chloride ions react with Fe^{2+} to produce $FeCl_2$ (ferrous chloride) by the following equation[4].

$$Fe^{2+} + 2Cl^{-} \rightarrow FeCl_2 \qquad \text{Eqn. 2.17}$$

The cathodic reaction occurring simultaneously is shown by Equation 2.18.

$$O_2 + 2H_2O + 4e^- \rightarrow 4OH^- \qquad \text{Eqn. 2.18}$$

Ferrous chloride then reacts with water and the OH^- ions formed by Equation 2.18 to form ferrous hydroxide ($Fe(OH_2)$) at the anodic sites. This corrosion product is greenish black in color and Equation 2.19 illustrates this reaction. This reaction also produces chloride ions that are then free to react with additional iron and the corrosion process continues.

$$FeCl_2 + H_2O + OH^- \rightarrow H^+ + 2Cl^- \qquad \text{Eqn. 2.19}$$

After this reaction occurs ferrous hydroxide reacts with oxygen to form ferric hydroxide which eventually forms ferric oxide (Fe_2O_3) or rust as seen in Equations 2.20 and 2.21[4].

$$2Fe(OH)_2 + ½O_2 + H_20 \rightarrow 2Fe(OH)_3 \qquad \text{Eqn. 2.20}$$

$$2Fe(OH)_3 \rightarrow Fe_2O_3 + 3H_2O \qquad \text{Eqn. 2.21}$$

2.2.3 Factors Influencing Corrosion of Reinforcement in Concrete Bridge Decks

Many factors can promote the corrosion of reinforcement in concrete bridge decks. Some important factors affecting corrosion of reinforcement are: type of exposure, cracking of concrete surface, depth of concrete cover, alkalinity of cement, permeability of concrete, and concrete resistance[4]. Type of exposure can include temperature, chloride exposure, oxygen, water, and cycles of wetting and drying. Without oxygen and moisture, corrosion cannot occur. If the cement alkalinity is high, chloride ions will be required to destroy the passive oxide layer that will form in situations with pH in the range of 11 to 13. Concrete permeability will determine how quickly oxygen, water, and chloride ions will reach the layer of steel. In cases of macrocell corrosion, a high concrete resistance will impede the corrosion current flow. Control of some of these factors can be used to limit the degree of corrosion of reinforcement in concrete bridge decks.

2.3 Corrosion of Epoxy-Coated Reinforcement in Concrete Bridge Decks

Epoxy-coated reinforcement is a commonly used corrosion protection method in concrete bridge decks. Several theories exist to explain how epoxy-coated reinforcement helps to mitigate or eliminate corrosion. The most obvious benefit of epoxy-coated reinforcement is as a barrier to protect the reinforcement from harmful reactants such as chlorides. Epoxy-coated reinforcement also provides high electrical resistance, which will inhibit corrosion current. Limited oxygen underneath the epoxy coating may also slow the cathode half-cell reaction.

Although epoxy-coated reinforcement is commonly used as a corrosion protection method, several circumstances have been observed which may limit the effectiveness of the method. Debonding of the epoxy coating has been observed in some laboratory and field applications[5]. Debonding is the loss of adhesion of the coating to the reinforcement. Water absorption, anodic activity, and cathodic activity may all play a role in debonding the coating from the reinforcement. The results of coating debonding can be serious; loss of adhesion can create a potential difference at the reinforcement surface. Corrosion may initiate and progress if chloride ions permeate the coating.

Damage to the coating can also limit the effectiveness of the epoxy-coated reinforcement. Damaged areas expose small regions to the atmosphere and to chlorides, and corrosion activity may initiate in these areas. When the bar with the damaged coating is electrically isolated from other reinforcement, the anode and cathode must both form in the small damaged area. Due to the limited exposed area, corrosion activity will be insignificant. However, if the bar with the damaged coating is electrically connected to other bars with damaged coating or uncoated reinforcement, corrosion activity can be excessive. The case of a damaged epoxy-coated bar connected to an uncoated bar will provide the worst condition. In this situation the damaged areas will become anodic and the uncoated bars will become cathodic. This provides a small anode area to large cathode area, which will accelerate corrosion activity and may lead to pitting. Figure 2.6 shows the effect of damage and electrical continuity on epoxy-coated reinforcement[6]. The potential for excessive corrosion to occur with the use of damaged epoxy-coated reinforcement underscores the need to control and investigate defects that may be created in the transportation, placement, and concrete casting operations.

Underfilm or filiform corrosion is a special case of crevice corrosion that may occur on epoxy-coated reinforcement. Protective films in warm, humid climates are most susceptible to this form of corrosion[1]. Corrosion initiates at break in the protective coating and takes the form of filaments or threads of corrosion product. Corrosion takes place at the head of these filaments and is blue-green in color. This color indicates the presence of ferrous ions. The tail of the thread is reddish brown indicating formation of Fe_2O_3. The filaments are able to grow and move in a straight line underneath the coating with available space and moisture. A humidity level of 60% - 65% is required for growth of the filaments[1]. Underfilm corrosion is self-propagating and continues to damage the coating and spread underneath. Figure 2.7 shows the process of underfilm corrosion. Since oxygen and water are required for propagation of the corrosion, a low permeability coating is desirable. Limiting the damage to the coating will also reduce the amount of water and oxygen under the coating.

2.4 Summary

Expensive corrosion damage to reinforced concrete structures has prompted research in the area of corrosion of reinforcement and solutions to mitigate or eliminate this problem. Corrosion of reinforcement in concrete bridge decks is a wet corrosion process that is electrochemical in nature. Oxygen, moisture, and chlorides are all necessary for corrosion of reinforcement to occur. Chlorides destroy the passive layer on steel reinforcement allowing corrosion to initiate. Once the passive layer is destroyed, potential differences promote microcell or macrocell galvanic corrosion. Progressive corrosion will lead to damage to the concrete in the form of delaminations and spalls. Eventually, significant loss in cross-sectional area of reinforcement may occur.

Epoxy-coated reinforcement is a commonly used corrosion protection method in concrete bridge decks. High electrical resistance and a barrier to harmful reactants are benefits with the use of epoxy-coated reinforcement. However, debonding and damage to the epoxy coating may limit its effectiveness as a corrosion protection method.

Table 2.1 Energy Required of Common Metals to Convert from Ores to Metal[4]

Potassium	Most Energy Required
Magnesium	
Beryllium	
Aluminum	
Zinc	
Chromium	
Iron	
Nickel	
Tin	
Copper	
Silver	
Platinum	
Gold	Least Energy Required

Table 2.2 Standard Potential of Common Metals with Respect to SHE[4]

Metal	Standard Potential, Volts	
Potassium	-2.922	Active
Magnesium	-2.340	
Aluminum	-1.670	
Zinc	-0.762	
Chromium	-0.710	
Iron	-0.440	
Nickel	-0.250	
Hydrogen	0.000	
Copper	0.345	
Silver	0.800	
Platinum	1.200	
Gold	1.680	Noble or Passive

Table 2.3 Half-Cells and Their Potentials Relative to the Standard Hydrogen Half-Cell[4]

Half-cell	Potential, Volts
Standard Hydrogen Electrode (SHE)	0.000
Copper-Copper Sulfate Electrode (CSE)	0.316
Silver-Silver Chloride Electrode (Ag-$AgCl_2$)	0.222
Saturated Calomel Electrode (SCE)	0.242
Normal Calomel Electrode	0.280
Tenth Normal Calomel	0.334

Table 2.4 Standard Electromotive Force Potentials (Reduction Potentials)[2]

	Reaction	Potential (Volts vs. SHE)
Noble	$Au^{3+} + 3e^- = Au$	+1.498
	$Cl_2 + 2e^- = 2Cl^-$	+1.358
	$O_2 + 4H^+ + 4e^- = 2H_2O$	+1.229
	$Pt^{2+} + 3e^- = Pt$	+1.118
	$NO_3^- + 4H^+ + 3e^- = NO + 2H_2O$	+0.957
	$Ag^+ + e^- = Ag$	+0.799
	$Hg_2^{2+} + 2e^- = 2Hg$	+0.799
	$Fe^{3+} + e^- = Fe^{2+}$	+0.771
	$O_2 + 2H_2O + 4e^- = 4OH^-$	+0.401
	$Cu^{2+} + 2e^- = Cu$	+0.342
	$Sn^{4+} + 2e^- = Sn^{2+}$	+0.150
	$2H^+ + 2e^- = H_2$	0.000
	$Pb^{2+} + 2e^- = Pb$	-0.126
	$Sn^{2+} + 2e^- = Sn$	-0.138
	$Ni^{2+} + 2e^- = Ni$	-0.250
	$Co^{2+} + 2e^- = Co$	-0.277
	$Cd^{2+} + 2e^- = Cd$	-0.403
	$Fe^{2+} + 2e^- = Fe$	-0.447
	$Cr^{3+} + 3e^- = Cr$	-0.744
	$Zn^{2+} + 2e^- = Zn$	-0.762
	$2H_20 + 2e^- = H_2 + 2OH^-$	-0.828
	$Al^{3+} + 3e^- = Al$	-1.662
	$Mg^{2+} + 2e^- = Mg$	-2.372
	$Na^+ + e^- = Na$	-2.710
Active	$K^+ + e^- = K$	-2.931

Table 2.5 Galvanic Series of Some Metals and Alloys in Seawater[1]

↑	Platinum
Noble or Cathodic End	Gold
	Graphite
	Titanium
	Silver
	Chlorimet 3 (62 Ni, 18 Cr, 18 Mo)
	Hastelloy C (62 Ni, 17 Cr, 15 Mo)
	18-8 Mo stainless steel (passive)
	18-8 stainless steel (passive)
	Chromium stainless steel 11-30% Cr (passive)
	Inconel (80 Ni, 13 Cr, 7Fe) (passive)
	Nickel (passive)
	Silver Solder
	Monel (70 Ni, 30 Cu)
	Cupronickels (60-90 Cu, 40-10 Ni)
	Bronzes (Cu-Sn)
	Copper
	Brasses (Cu-Zn)
	Chlorimet 2 (66 Ni, 32 Mo, 1 Fe)
	Hastelloy B (60 Ni, 30 Mo, 6 Fe, 1 Mn)
	Inconel (active)
	Nickel (active)
	Tin
	Lead
	Lead-Tin Solders
	18-8 Mo Stainless Steel (active)
	18-8 Stainless Steel (active)
	Ni-Resist (high Ni cast iron)
	Chromium stainless steel, 13% Cr (active)
	Cast Iron
	Steel or Iron
	2024 Aluminum (4.5 Cu, 1.5 Mg, 0.6 Mn)
	Cadmium
	Commercially Pure Aluminum (1100)
Active or Anodic End	Zinc
↓	Magnesium and Magnesium Alloys

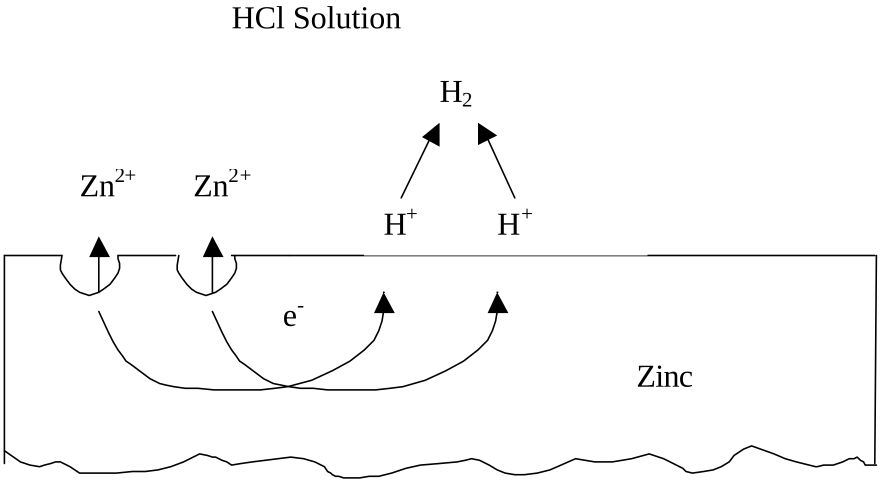


Figure 2.1 Corrosion of Zinc in Hydrochloric Acid[1]

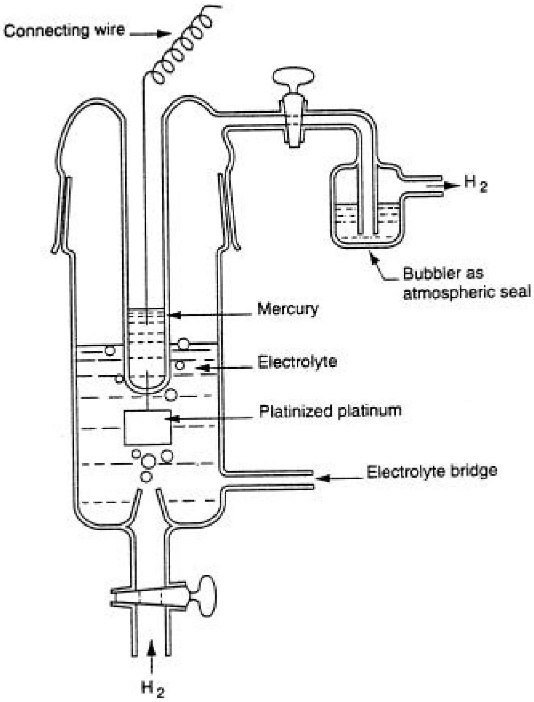


Figure 2.2 Standard Hydrogen Electrode (SHE)[2]

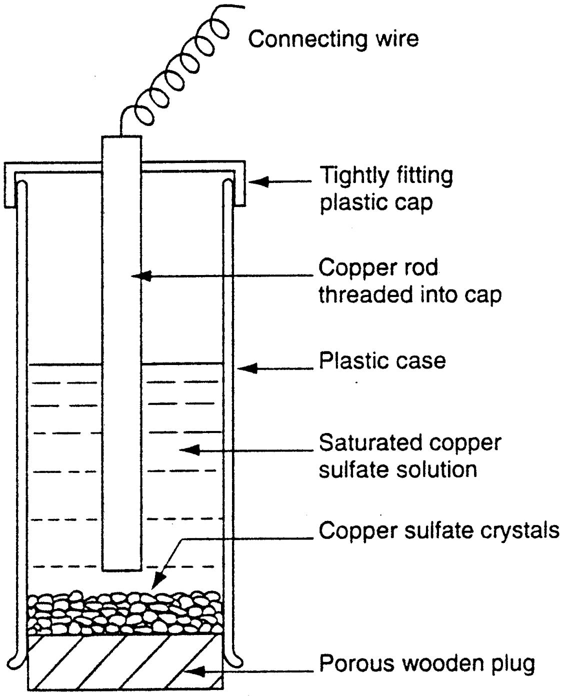


Figure 2.3 Copper-Copper Sulfate Electrode (CSE)[2]

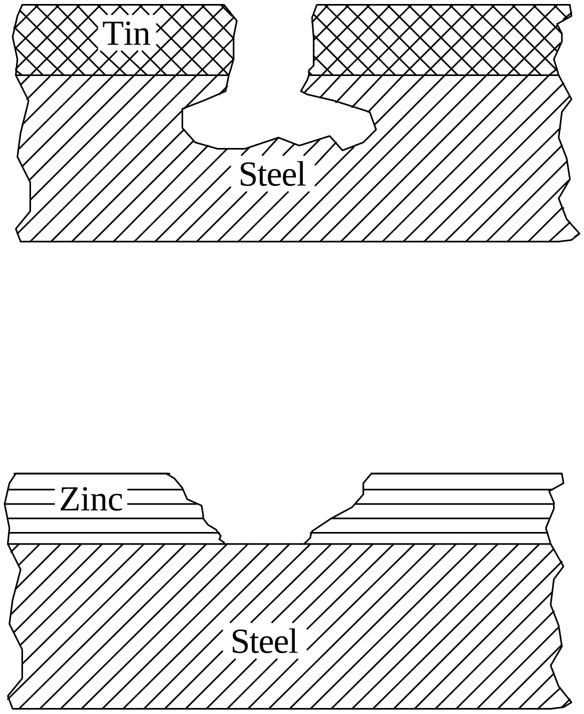


Figure 2.4 Galvanic Corrosion at Damaged Site in Tin and Zinc Coated Steel[4]

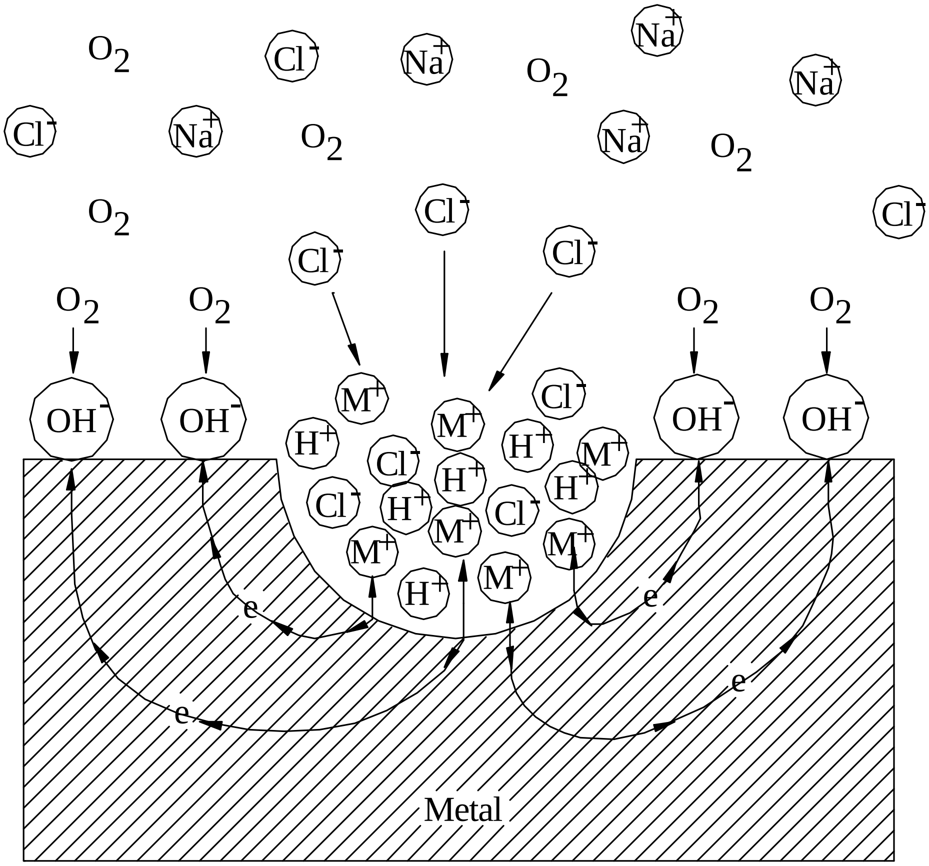
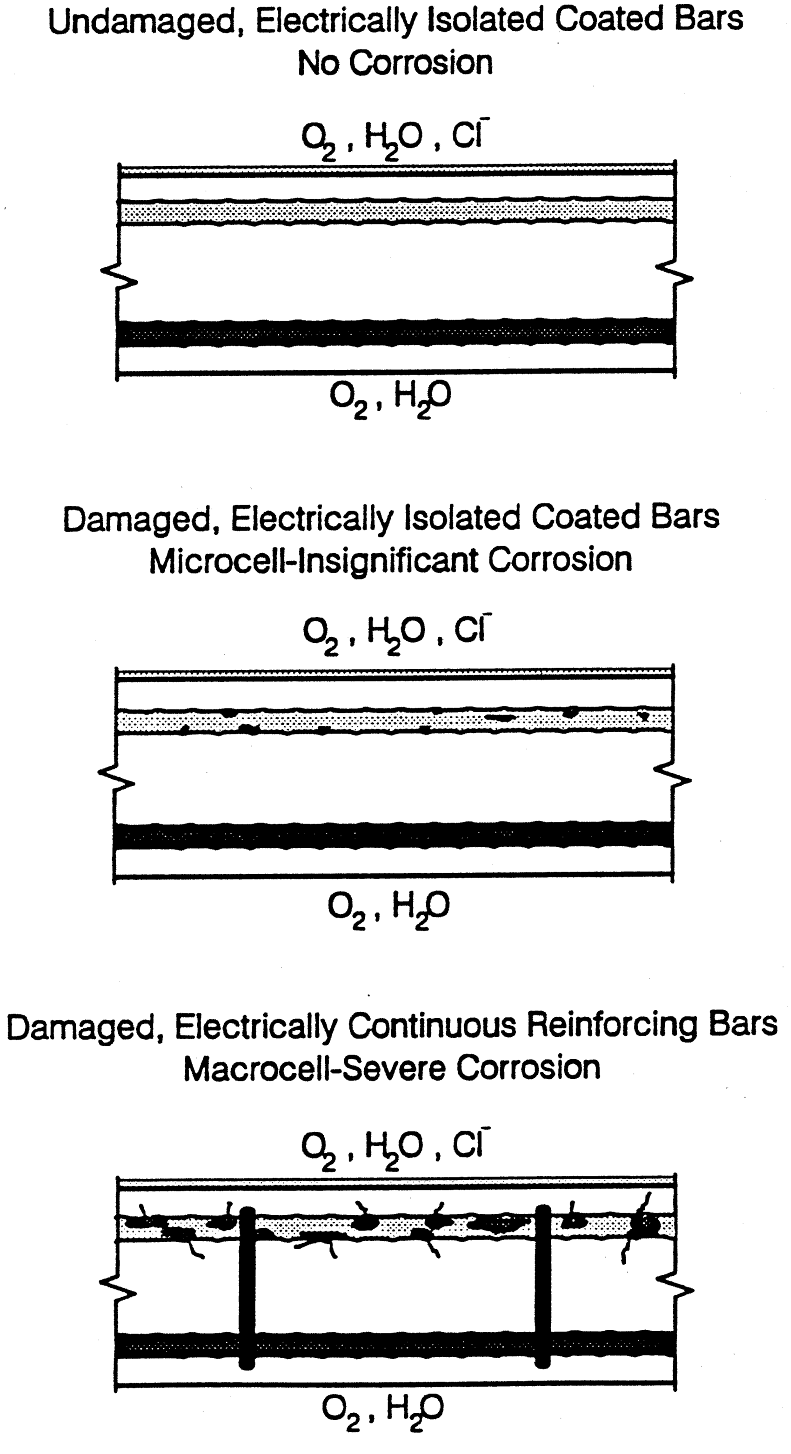


Figure 2.5 Process of Growing Pit[1]



Undamaged, Electrically Isolated Coated Bars
No Corrosion

Damaged, Electrically Isolated Coated Bars
Microcell-Insignificant Corrosion

Damaged, Electrically Continuous Reinforcing Bars
Macrocell-Severe Corrosion

Figure 2.6 Effect of Damage and Electrical Continuity on Corrosion Activity[6]

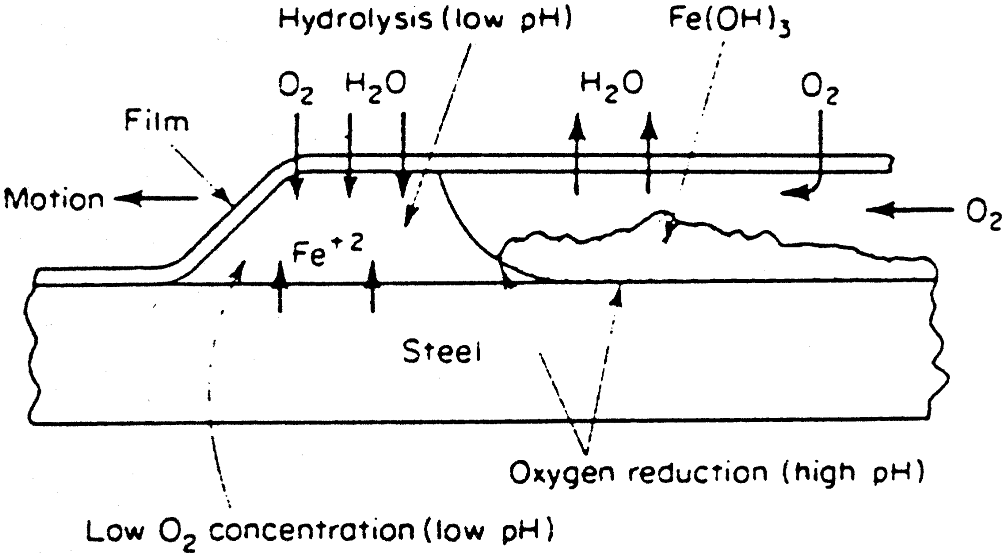


Figure 2.7 Underfilm Corrosion[1]

CHAPTER 3 – LITERATURE REVIEW

A relevant review of literature is required in order to assess the manufacturing process and the past performance of epoxy-coated reinforcement. Organic coatings for reinforcing steels were first investigated as a solution to the problem of bridge deck deterioration due to reinforcing steel corrosion in the early 1970's[7]. Epoxy coatings emerged as the best organic coating for use as a corrosion protection method. However, some recent field and laboratory studies have indicated the problem of premature corrosion of epoxy-coated reinforcement. The following literature review provides information on three areas: coating process and specifications of epoxy-coated reinforcement, corrosion performance of epoxy-coated reinforcement in bridge structures, and corrosion performance of epoxy-coated reinforcement in laboratory studies.

3.1 Coating Process and Specifications of Epoxy-Coated Reinforcement

The performance of epoxy-coated reinforcement in a corrosive environment is a function of the integrity of the coating. A coating process was developed to ensure adequate thickness of coating, proper bond to the reinforcement, and adequate coating continuity. Also, specifications were developed to limit damage to the coated reinforcement during the manufacturing, transportation, and placement of epoxy-coated reinforcement. The coating process and specifications are discussed in the following section.

3.1.1 Coating Process

The coating process involves the application of an epoxy coating to reinforcing steel. An investigation conducted by the Federal Highway Administration in the early 1970's found that epoxy coatings applied by electrostatic spray techniques can provide an adequate coating[7]. Forty-seven organic coating materials were investigated in this report. Chemical and physical durability, permeability to chlorides, protective qualities, and bond to the concrete as determined by pullout and creep tests were all evaluated. The results of the investigation concluded that only the epoxy coatings met all of these requirements, and that the powder epoxy coatings performed better than the liquid epoxies. An optimum coating thickness of 177.8 ± 50.8 μm (7 ± 2 mils) was recommended after consideration of flexibility, bond strength, creep characteristics, and minimum corrosion protection requirements. A mil is defined as one thousandth of an inch (0.001 in.), while a μm is 1×10^{-6} meters. The report concluded that the epoxy coating provided an economical solution to corrosion of reinforcement by delaying or even preventing the onset of corrosion of reinforcing bars. The process of coating reinforcing bars with epoxy is specified by ASTM A775/A775M[8]. Generally, the coating process includes the following steps[8]:

1. The surface of the steel reinforcing bars to be coated shall be cleaned by abrasive blast cleaning to near-white metal.

2. Multidirectional, high-pressure dry air knives shall be used after blasting to remove dust, grit, and other foreign matter from the steel surface. The air knives shall not deposit oil on the steel reinforcing bars.

3. It is permissible for a chemical wash or conversion of the steel reinforcing bar surface, or both, to be used to enhance coating adhesion. This pretreatment shall be applied after cleaning and before coating, in accordance with the written application instructions specified by the pretreatment manufacturer.

4. The powder coating shall be applied to the cleaned and pretreated surface as soon as possible after surface treatments have been completed, and before visible oxidation of the surface occurs. In no case should the coating be delayed more than 3 hours after cleaning.

5. The fusion-bonded epoxy powder shall be applied in accordance with the written recommendations of the material supplier for initial steel surface temperature range and post application cure requirements. During continuous operations, the temperature of the surface immediately prior to coating shall be measured using infrared guns or temperature indicating crayons, or both, at least once every 30 min.

6. The coating shall be applied by electrostatic spray or other suitable method.

3.1.2 Specifications

Requirements for organic coatings are specified by ASTM A775/A775M[8]. The following properties are required to be evaluated in accordance with ASTM A775: chemical resistance, cathodic disbondment, salt spray resistance, chloride permeability, coating flexibility, relative bond strength in concrete, abrasion resistance, and impact resistance. The tests for these properties are not discussed in ASTM A775/A775M and are outside the scope of this study. The manufacturer of the organic coating is responsible for including all test results in the report.

ASTM A775/A775M[8] also specifies coating thickness, coating continuity, and coating flexibility requirements for the coated bars. In order to be accepted, at least 90% of the recorded thickness measurements after curing must be between 175 to 350 μm (7 to 12 mils). A single reading below 125 μm (5 mils) is cause for rejection. Similarly, the Indiana Department of Transportation[9] requires epoxy-coated reinforcement to be between 150 to 350 μm (6 to 12 mils) as evaluated according to ASTM A775. An average of three individual readings between three consecutive deformations is recorded as a single measurement. Five evenly spaced measurements are required along each side of the test bar for a minimum of ten measurements per bar. Thickness measurements are only required along straight lengths of reinforcing bar, on a minimum of two bars of each size every four production hours. If the thickness requirement is not satisfactory, two retests on random samples are allowed. If both of the retests are satisfactory, the samples are accepted.

To ensure adequate coating continuity, holiday checks are required before shipment from the coating applicator's plant. A holiday is defined as a discontinuity in the coating not detectable with normal or corrected vision. Holiday detection by a 67.5 V, 80,000 Ω, wet-sponge, direct current detector or equivalent is required. Only an average of three holidays per meter (one per linear foot) is allowed. Random continuity checks are required.

The coating flexibility is to be evaluated by a bend test specified by ASTM A775/A775M[8]. Any cracking or disbondment visible to the unaided eye on the outside radius of the bent bar is cause for rejection. Also, any fracture or partial failure of the reinforcing bar is considered a failure of the flexibility test. Bend tests are required every four production hours on at least one bar of each size. If the coating flexibility test fails, two retests on random samples are required. If both of the retests meet the requirements, the coated bars may be accepted.

After the coating application, improper handling of the epoxy coating may damage the coating. ASTM A775 also requires that padded contact areas should be used when handling coated bars. Padded bundling bands are required and sagging of the bars during lifting and storing should be avoided. Identification of the coated bars is required throughout the coating process to shipment.

ASTM D3963/D3963M[10] provides standard specifications for job-site practices with the use of epoxy-coated reinforcement. These guidelines suggest that epoxy-coated reinforcement should be stored separate from uncoated steel, and prolonged storage of epoxy-coated reinforcement at the job site is discouraged. Epoxy-coated reinforcement should be stored above the ground on wooden or padded supports. If long term storage (more than two months) of the coated bars is required they should be covered with a protective material with adequate ventilation. Coated ties and bar supports should be used in placing the coated reinforcement. Vibrators with rubber heads should be used to avoid damage to the epoxy coating during consolidation. All damage created in the transportation, storage, or placement of coated bars should be repaired with an approved patching compound. For any 0.3 m (1 ft) section of epoxy-coated reinforcement, the total surface damage before patching should not be greater than 2% of the bar surface area. After patching, the patched areas should not exceed 5% of the bar surface area. The preceding percentages do not include the areas at sheared ends of the reinforcement.

3.2 Performance of Epoxy-Coated Reinforcement in Bridge Structures

In 1986, signs of corrosion of epoxy-coated reinforcement were first discovered in the substructure of the Long Key Bridge in the Florida Keys after only six years in service[11]. After this discovery, an investigation of five structures with lengths greater than 610 m (2,000 ft) in the Florida Keys was performed. The investigation revealed significant corrosion in four of the five substructures. The corrosion damage was concentrated in the area immediately above the high water mark (splash zone). Typical delaminations on the piers of two of the substructures are shown in Figures 3.1 and 3.2. This incident raised concerns that premature corrosion of epoxy-coated reinforcement may occur and limit the increase in service life of bridge decks utilizing epoxy-coated reinforcement. The following literature survey summarizes several investigations of the performance of epoxy-coated reinforcement in bridge structures.

3.2.1 Performance in Bridge Substructures

After the initial corrosion was found in the substructure of the five bridges in the Florida Keys, the Florida Department of Transportation performed a survey of all bridges containing epoxy-coated reinforcement located in a marine environment[11]. The survey included taking cores from the bridge decks to determine if any corrosion of the epoxy-coated reinforcement was present. The results of the study found no corrosion of the epoxy-coated reinforcement; however, gross disbondment of the epoxy-coated reinforcement was observed in nearly every case. It was suggested that no corrosion had occurred in the reinforcement because the chloride levels were not high enough to initiate corrosion. However, corrosion is expected in the future when the chloride levels begin to rise. As a result of this study, epoxy-coated reinforcement is no longer used in highway construction in Florida.

3.2.2 Performance in Bridge Decks

The summaries of the following reports on field performance of epoxy-coated reinforcement in bridge decks are listed in chronological order by the date the reports were published.

3.2.2.1 Pennsylvania, 1984

An investigation of twenty-two bridge decks in Pennsylvania was conducted in 1984 to assess any concrete damage caused by corrosion of the reinforcement[12]. The bridge decks were constructed in the years of 1974 to 1978. Eleven of the bridge decks were constructed with bare reinforcement and eleven were constructed with epoxy-coated reinforcement. The bridge decks containing epoxy-coated reinforcement were chosen first and then duplicate bridge decks containing bare reinforcement were chosen. The results of the visual inspection revealed forty percent of the bridge decks containing bare reinforcement had some deterioration due to corrosion of the reinforcement. The bridge decks containing epoxy-coated reinforcement showed no signs of deterioration. A follow-up, in-depth investigation was performed and more extensive deterioration of the bridge decks containing bare steel was discovered while no deterioration was discovered in the bridge decks containing epoxy-coated reinforcement.

3.2.2.2 Virginia, Wisconsin, Pennsylvania, New York, and Ohio, 1990

Thirteen bridge decks containing epoxy-coated reinforcement in the states of Virginia, Wisconsin, Pennsylvania, New York and Ohio were evaluated in 1990[13]. The bridge decks were constructed in the years from 1974 to 1981. Visual and delamination surveys were performed along with cores taken in sound, unsound, and cracked concrete. The survey found that in half of the bridge decks the chloride content had not reached the threshold level for corrosion in uncracked areas. However, the chloride content in the other half of the bridge decks and in cracked areas was well in excess of the level required for initiation of corrosion. No corrosion activity was found in the epoxy-coated reinforcement away from cracked areas. In cracked areas the part of the epoxy-coated bar directly exposed by the cracks often showed some corrosion but no significant section loss. The reinforcement not directly exposed in the cracked region occasionally showed some signs of corrosion. Overall, a total of 85 cores containing 107 epoxy-coated reinforcing bars were removed for examination. Only 13% of the top mat reinforcement showed any signs of corrosion, and this was limited to areas of cracking. Of the thirteen decks surveyed only two had any delaminations, and the area affected was less then 1% of the total deck area. The epoxy-coated reinforcement removed from the cores was generally in good condition and the steel surface bright and shiny underneath the coating, except at areas of cracking extending to the depth of the reinforcement. The epoxy-coated reinforcement removed from all the cores had significant holidays, mashed areas, and small bare steel areas. These defects did not seem to affect the performance of the reinforcement away from areas of cracking. The coating thickness measured with a nondestructive thickness gage varied from 129.5 μm to 363.3 μm (5.1 mils to 14.3 mils).

As a whole, the epoxy-coated reinforcement contained in these decks was performing well with only isolated problems at cracked locations.

3.2.2.3 NCHRP Report 370

NCHRP Report 370 summarizes field investigations indicating both good and poor performance of epoxy-coated reinforcement in bridge decks[14]. Evaluations of bridge decks conducted in Maryland, Minnesota, Virginia, and Pennsylvania all showed good performance of the epoxy-coated reinforcement. However, some of the bridge decks included in these investigations were relatively early in age and had chloride levels below the level required for initiation of corrosion of the reinforcement. An investigation of two bridge decks in Virginia, which were known to contain poor quality epoxy-coated reinforcement, found no corrosion induced damage to the concrete. However, the bridge decks had only been in service for seven years and chloride levels were below the threshold level except in areas of transverse cracking. No delaminations or distress were detected even in the cracked areas with high chloride levels. An investigation by the Pennsylvania Department of Transportation to evaluate all protective systems in bridge decks found epoxy-coated reinforcement to be in excellent condition. The study involved visual inspections of 32 bridge decks and detailed examinations of four others. The chloride level in the bridge decks was in excess of the level required for initiation of corrosion. The average coating thickness was found to be 233.7 μm (9.2 mils), and the average age for the bridge decks was 7.7 years. The age of the four bridge decks in the detailed investigation was 10 to 12 years. In the detailed investigation corrosion was found in only one epoxy-coated bar.

Several instances of poor performance of epoxy-coated reinforcement are also reported in NCHRP Report 370[14]. In addition to the substructure members of the bridges in the Florida Keys, the following instances of corrosion in epoxy-coated reinforcement were reported:

1. A New York bridge deck constructed in the mid-1970s with epoxy-coated reinforcing steel (Flintflex 6080), a recognized poor quality coating, was reported in 1990 to be exhibiting widespread delamination, spalling and severe corrosion of the epoxy-coated reinforcing steel.

2. The epoxy-coated reinforcing steel near the expansion dams on two bridge decks in Ontario was recently examined during replacement of the dams. This area of the decks was not waterproofed and severe corrosion of the epoxy-coated reinforcing steel was noted in both instances. The Ontario Ministry of Transportation is presently evaluating these structures and bars.

3. Another New York bridge deck located in the Albany area and constructed in 1981 was found to contain badly corroded epoxy-coated bars and to exhibit horizontal cracking that is typical of corrosion induced delamination. Concrete cover varied from 50.8 to 61 mm (2.0 to 2.4 in). Four of the seven cores taken from this structure in 1990 after nine years of service were found to be broken into three or four pieces with the epoxy-coated bars at the center.

4. Evaluation of existing epoxy-coated reinforcing steel was performed on a nine-year-old coastal bridge deck in Georgia. This revealed locations of complete loss of coating bond in spite of the fact that the cover was 127 to 177.8 mm (5 to 7 in). Although no cracking or spalling distress was apparent, the steel recovered from two of the six cores that were obtained exhibited corrosion. It was recommended that a phasing out of the use of epoxy-coated reinforcement on bridge decks and marine environment concrete be considered.

3.2.2.4 Federal Highway Administration

A recent Federal Highway Administration report summarizes field investigations of 92 bridge decks reinforced with epoxy-coated bars[15]. Reports from the states of

California, Indiana, Kansas, Michigan, Minnesota, New York, Ohio, Pennsylvania, Virginia, West Virginia, and Wisconsin are included. Also, the performance of bridge decks and barrier walls from the Canadian provinces of Alberta, Nova Scotia, and Ontario were included. The inspections included a visual examination for spalls, cracking, and patches. Areas of delamination were located by chain dragging. Cover readings, chloride content, half-cell potentials, resistivity readings, rate of corrosion, and rebar extractions through cores were all performed. No significant distress was found on any of the bridge decks surveyed. The maximum area of delamination discovered accounted for less then 1% of the total deck area. Cracking was present on some of the bridge decks but was not thought to be corrosion related. The chloride level for the majority of the bridge decks was above the threshold required for initiation of corrosion.

Approximately 202 epoxy-coated bar segments were removed from the bridge decks, 81% of these showed no corrosion activity. In cracked areas of the decks, the epoxy-coated reinforcement did not appear to be performing as well as in uncracked areas. Corrosion of epoxy-coated reinforcement was discovered in areas of heavy cracking and shallow cover. However, in uncracked areas no corrosion was found even when the level of chloride concentration was as high as 7.6 kg/m^3 (12.8 $lb.yd^3$). Coating disbondment and softening were found as a result of extensive exposure to a moist environment. It appeared that the number of defects in the coating and the amount of disbondment did influence the performance of the epoxy-coated reinforcement. Overall, the bridge decks evaluated showed that epoxy-coated reinforcement has provided adequate corrosion protection for up to 20 years of service life. There was no evidence of significant concrete distress due to the corrosion of the epoxy-coated reinforcement, and little or no maintenance work had been performed on the bridge decks.

3.2.2.5 Virginia, 1996

An investigation of the corrosion performance of epoxy-coated reinforcement was conducted on three bridge decks in Virginia in 1996[16]. All three of the bridge decks were constructed with epoxy-coated reinforcement in the top mat only and were 17 years of age when the inspections were performed. Twelve cores were taken from each bridge deck to extract epoxy-coated bar segments. The cores from one of the bridge decks showed no corrosion of the epoxy-coated reinforcement. Minor corrosion was found on four of the cores taken from the second bridge deck. On the third bridge deck, seven of the cores taken showed some corrosion, and two showed severe corrosion of the epoxy-coated reinforcement. The study also reported widespread coating disbondment in the reinforcement on all of the bridge decks. The study concluded from the investigations on these three bridge decks that the epoxy coating on reinforcing bars in Virginia will debond in about 15 years, and that epoxy-coated reinforcement may not extend the service life of bridge decks.

3.3 Performance of Epoxy-Coated Reinforcement in Laboratory Studies

Various laboratory studies have been performed to assess the performance of epoxy-coated reinforcement. Typically, the studies are designed to determine the behavior of the coated reinforcement when exposed to corrosive environments. The most common corrosive environment for epoxy-coated reinforcement used in bridge structures is chloride exposure, whether from deicing salts or marine exposure. The following literature survey summarizes several laboratory studies involving epoxy-coated reinforcement.

3.3.1 University of New Brunswick

A study conducted at the University of New Brunswick reports the results of a study on corrosion of epoxy-coated reinforcement in concrete[17]. The objective of the study was to determine the rate of corrosion of the rebar in concrete. A simulated marine environment was used to induce corrosion of uncoated and epoxy-coated reinforcement. Twelve slabs were constructed with one U-shaped reinforcing bar. The concrete cover was approximately 20 mm (0.8 in.) in all directions. A stainless steel rod acting as a counter electrode was located in the center of each slab for corrosion rate measurements. Four of the slab specimens were cast with uncoated reinforcement, the remaining eight specimens contained epoxy-coated reinforcement with varying amounts of damage to the coating. Four of the specimens contained epoxy-coated reinforcement with no damage to the coating; two specimens contained epoxy-coated reinforcement with 1% damage; and two of the specimens contained epoxy-coated reinforcement with 2% damage. The damage to the coating was created by removing 6 x 6 mm (0.24 x 0.24 in) patches of epoxy. A cycle of a two-hour wetting period with simulated seawater, followed by a four-hour drying period was repeated four times each 24-hour period. This exposure testing was continued for a period of two years. Open circuit potentials, instantaneous corrosion rate using linear polarization methods, and AC impedance measurements were performed over the two-year study. At the end of one and two years, some of the reinforcing bars were removed from the specimens. This allowed for a visual and microscopic examination to be performed.

The results from the study found that the corrosion rate of the specimens containing epoxy-coated reinforcement was negligible when compared to the corrosion rate of specimens containing uncoated reinforcement despite the amount of damage to the coating. The specimens with no damage to the epoxy coating showed no corrosion activity over the two-year exposure period. The specimens containing epoxy-coated reinforcement with 1% and 2% damage to the coating had corrosion current densities of 0.02 $\mu A/cm^2$ and 0.03 $\mu A/cm^2$, respectively, after two years. Specimens with uncoated reinforcement had corrosion current densities of 10.1 $\mu A/cm^2$ after two years. Corrosion current density is the measured corrosion current divided by the surface area of the reinforcement. A visual inspection of the epoxy-coated reinforcement removed from the slabs also showed no visible signs of corrosion of the reinforcement or cracking of the concrete surrounding the reinforcement.

3.3.2 United Kingdom

Concrete specimens reinforced with epoxy-coated reinforcement were subjected to exposure testing in a study performed in the United Kingdom by McKenzie[18]. The study evaluated uncoated reinforcement as well as epoxy-coated reinforcement with uncoated ends, repaired ends, damage to the coating, and bent bars. The goal of the study was to determine the effect of defects on the durability of epoxy-coated reinforcement. Both salt ponded specimens and specimens with salt added to the concrete mix were used for the exposure testing. Two different concrete test specimens were used, beams and slabs. The beam specimens were used to monitor the performance visually, while the slab specimens were monitored by electrochemical measurements, which included half-cell potentials and galvanic currents.

The dimensions of the beam specimens were 400 x 100 x 100 mm (15.75 x 3.93 x 3.93 in.) and contained four reinforcing bars. Figure 3.3 shows the cross section of a beam specimen. Specimens contained either uncoated or epoxy-coated reinforcement. One bar at each of the cover depths of each specimen containing epoxy-coated reinforcement was damaged at four locations. The damage consisted of a hole created with a paint borer of approximately 1 mm (0.04 in.) in diameter. Also, one of the cut ends of the epoxy-coated bar was not patched. The slab specimens were 500 x 300 x 60 mm (19.69 x 11.81 x 2.36 in.) and contained coated and uncoated bent and straight bars. The configuration of these specimens is shown in Figure 3.4. The slab specimens were designed so only half of the slab would be ponded. An external resistor was connected to allow for measurement of current flow. The specimens were placed outside and the specimens not containing admixed salt were ponded with a 3% salt solution weekly over a two-year period.

The study found that concrete cracking and corrosion of the reinforcement were reduced for the specimens containing epoxy-coated reinforcement compared with specimens containing uncoated reinforcement. Corrosion was found spreading under the coating from defects, beneath the coating on bent bars, and under patched areas. However, the damage was limited to light rusting and did not result in any section loss of the bar. In contrast, significant loss in bar section of uncoated bars was observed. Half-cell potentials on the epoxy-coated reinforcement were more variable then the measurements on the uncoated reinforcement. Corrosion current of the epoxy-coated reinforcement was negligible when compared with the uncoated reinforcement.

3.3.3 Yeomans

In a study conducted by Yeomans[19], the corrosion resistance of black steel, galvanized steel, and epoxy-coated steel in concrete was investigated. The goal of the study was to compare the corrosion performance of epoxy-coated reinforcement and galvanized reinforcement in concrete. Specifically, the effect of depth of cover and influence of damage to the coatings was addressed. Reinforced concrete cylinders were cast with each type of reinforcement in various arrangements. Only one type of reinforcement was included in each cylinder. Specimens were exposed to either salt water wetting and drying cycles or a continuous salt fog. During the wetting and drying cycles the specimens were completely immersed in 3.5% sodium chloride solution at 40°C (104°F) for three days and then oven dried at 60°C (140°F) for four days. The continuous salt fog was a solution of 3.5% sodium chloride at 40°C (104°F) and 100% relative humidity.

Half-Cell potentials, chloride analysis, and metal loss were measured. The study found that the cycles of wetting and drying produced a corrosion rate twenty times higher than the salt fog exposure. Chloride penetration was also faster for the wetting and drying cycles. Epoxy-coated reinforcement generally provided excellent corrosion resistance throughout the test period, even in high chloride concentration situations. However, at cut ends and damaged areas, the epoxy-coated reinforcement performed similar to black steel. In several cases, corrosion proceeded under the coating and resulted in coating debondment. The study also found that holidays and minor damage to the coating were responsible for the large negative half-cell measurements. The magnitude of the half-cell measurements would suggest corrosion at these areas, but little evidence of corrosion was found. This result suggests that half-cell measurements may be unreliable for indicating corrosion of epoxy-coated reinforcement.

3.3.4 Federal Highway Administration

Most recently epoxy-coated reinforcement has been evaluated in a five year Federal Highway Administration (FHWA) research project on various corrosion resistant reinforcing bars. This project involved screening tests for the first two years to select the most promising corrosion resistant reinforcing bars for in-concrete testing[20]. The bars were chosen based on current use by various agencies, economy, and their performance in the screening tests. After screening, eleven bars were chosen for further testing. Six of the eleven bars chosen were epoxy-coated, and four involve surface treatments prior to coating. The following list shows the differences and designation given to the epoxy-coated bars:

- Epoxy-coated bars coated with 3M Scotchkote 213 (Epoxy-A)
- Two bendable epoxy-coated bar types (Epoxy-B, Epoxy-C)
- Two nonbendable epoxy-coated types (Epoxy-D, Epoxy-E)
- One post-baked nonbendable epoxy coating (Epoxy-F)

Steel surface treatments were used prior to coating with Epoxies B, C, E, and F. The testing period on concrete slabs consisted of a twelve week cycle of ponding a 15% NaCl solution for four days followed by a three day drying period at 38°C (100°F). Then twelve weeks of continuous ponding under 15% NaCl solution at 16 to 27° C (60° to 80°F). This 24-week cycle was repeated four times for a total exposure period of 96 weeks. Preliminary results were available after 48 weeks[21]. The concrete slabs contained two layers of 16 mm (5/8 in.) reinforcing bars and measured 300 x 300 x 175 mm (12 x 12 x 7 in.). The bottom mat contained straight bars and the top contained either straight or bent bars. Parallel cracks were formed directly over a reinforcing bar in some of the configurations. Clear cover of 25.4 mm (1 in.) was used in all specimen configurations. External resistors were connected between the top and bottom mat of reinforcement to allow for corrosion current measurements. Figure 3.5 shows the test setup for the three configurations. Table 3.1 provides the specimen configurations. Four duplicates of each configuration were fabricated. The conclusions of the study with regard to epoxy-coated reinforcement at 48 weeks are:

1. The use of a black bar cathode had a moderate to significant negative influence on the corrosion current performance of five different epoxy-coated bars when the coating damage was 0.5 percent. The straight and bent epoxy-coated bars with coatings A, B, C, E, and F with 0.5 percent damage in initially uncracked concrete generally had poor corrosion current performance, averaging only 2.3 times less than the black bar control specimen. Under these conditions, the time-to-cracking for these 15 different specimen types was about 1 to 5 years. Several of the Epoxy E and Epoxy F specimens exhibited cracks after 1.4 years of testing. Only coating D produced consistent good corrosion performance with straight and prebent bars in uncracked concrete and a black bar cathode, averaging 90 times less current than the black bar controls with an estimated time-to-cracking of about 20 years.

2. The use of precracked specimens generally produced large increases in corrosion currents in all bar types. Only coatings A and D, with 0.004 percent coating damage, provided excellent corrosion performance in precracked concrete, averaging 115 times less total corrosion than the black bar control specimens, with an estimated time-to-cracking of

about 25 years. These data clearly indicate that cracks should be minimized or repaired to allow for maximum service life.

3. The use of epoxy coatings A, B, C, D, and E in both mats consistently produced excellent corrosion performance when the coating damage was either 0.5 or 0.004 percent on straight bars in uncracked concrete. Under these ten conditions, these five different epoxy-coated bars had, on-average, about 300 times less corrosion than the controls, with an average estimated time-to-cracking of about 60 years. The same five epoxy-coated straight bars with 0.004 percent damage, when used with a black bar cathode in uncracked concrete also had excellent performance, averaging 170 times less corrosion, with an average time-to-cracking of about 40 years.

4. With the 0.5 percent damage, only coating D on prebent bars had moderate corrosion performance with 80 times less corrosion than the control. With the 0.004 percent damage, coatings A, B, and D had excellent performance with an average of 200 times less corrosion. Bent bars with coatings E and F and a black bar cathode exhibited cracking after only 1.4 years of testing. Only coating D with 0.5 or 0.004 percent coating damage on prebent bars had consistently excellent corrosion performance, averaging 100 times less corrosion than the control.

5. The use of steel surface pretreatment did not appear to increase the corrosion performance of the epoxy-coated bars. At-hole adhesion does not always correlate to corrosion performance.

6. Special efforts should be made during construction to avoid electrical continuity between all epoxy-coated bars when used in both mats, and other miscellaneous black steel in the structure, and construction should not mix epoxy-coated bars and conventional black bars that can accidentally become electrically connected.

3.3.5 University of Texas at Austin

At the University of Texas at Austin a three part experimental program was conducted to study the corrosion performance of epoxy-coated reinforcement[6]. The goal of the study was to determine the effect of coating damage, exposure conditions, and cracking due to loading on the corrosion performance of epoxy-coated reinforcement.

The first part of the experimental program consisted of an immersion test. Uncoated and epoxy-coated reinforcement was bent into U-shapes and immersed in a 3.5% sodium chloride solution. The amount of damage created to the coating of the epoxy-coated reinforcement by bending depended on the deformation type and bar size. Some of the test specimens were patched with an epoxy-patching compound to evaluate the effectiveness of patching. The reinforcement was subjected to a cycle of three days submersion in the sodium chloride solution and three days drying over a period of two years. The results of the immersion test found corrosion was initiated at all damaged areas regardless of the size. The long-term exposure to the corrosive environment resulted in a breakdown of the protective qualities of the coating. Underfilm corrosion and debonding of the coating was observed. However, the epoxy-coated reinforcement did perform better than the uncoated reinforcement, which experienced severe and uniform corrosion resulting in loss of bar cross-section.

The second part of the experimental program consisted of companion bars bent in the same manner as the specimens in the immersion test. These bent bars were cast into concrete prisms for macrocell corrosion study. The bent bars were cast as the top bar combined with two or three straight uncoated rebars at the bottom. The reinforcement was electrically connected with a resistor to allow for measurement of macrocell corrosion current. The specimens were subjected to a four-day cycle that consisted of two days ponding of a 3.5% sodium chloride solution, followed by two days of drying. The specimens were tested for a total of two years. The results of the study showed that corrosion of the epoxy-coated reinforcement was initiated at twice the level required to initiate corrosion in uncoated reinforcement. The epoxy-coated reinforcement was found to limit the severity of the corrosion of the reinforcement even when the coating was damaged. However, the amount of damage did influence the amount of corrosion resistance provided. The size and frequency of damage were important to the corrosion performance of the epoxy-coated reinforcement. Patching the damaged areas reduced the corrosion but did not fully protect the bare areas. Also discovered in this study, the performance of heavily damaged (greater than 2% surface area exposed) epoxy-coated reinforcement appeared to be deteriorating with longer periods of exposure.

The third part of the experimental program consisted of a beam exposure study. Straight, bent, and spliced epoxy-coated bars were cast in beams with different arrangements. The beams were then cracked under structural loading. Various levels of coating damage were tested. Longitudinal bars in "as received" condition, longitudinal bars severely damaged (3% of greater exposed surface area) with or without patching, stirrups in "as received" condition with or without patching, stirrups severely damaged with patching, and splice bars with patched ends were all included in the specimens. The specimens were exposed to 3.5% sodium chloride solution for three days while a continuous loading was applied to produce cracking. The load was removed and the specimens were allowed to dry for eleven days during which one additional loading cycle was included. The wetting and drying cycle was performed five times. The results of the beam exposure study found that the severity of corrosion in the epoxy-coated bars was related to loading condition and damage level. The specimens containing the "as received" bars performed best, while the bars with damaged coating showed the worst performance. Patching was only somewhat effective in preventing corrosion at damaged areas. Cracking led to a buildup of chlorides at the bar level and accelerated corrosion regardless of the crack width. Pitting was observed on stressed epoxy-coated reinforcement with 3% surface area damage in areas of cracking with 0.33 mm (0.013 in.) crack width.

Overall, the three part experimental study found that epoxy-coated reinforcement performed better than uncoated reinforcement in all cases. The severity of corrosion was less in all cases and the initiation of corrosion was delayed by the coating. Damage to coated bars was found to be an important factor affecting the corrosion performance of epoxy-coated reinforcement. Patching damaged areas reduced but did not eliminate corrosion activity.

3.4 Literature Review Summary

In general, the literature review of both field and laboratory investigations suggest that epoxy-coated reinforcement is effective as a corrosion protection method. With the exception of the bridge substructure members in Florida, the survey of field investigations found only limited occurrences of corrosion of epoxy-coated reinforcement. However, the field investigations indicate that cracks, shallow cover, and damage to the epoxy coating can lead to poor corrosion performance of the epoxy-coated reinforcement. Several of the field investigations discovered corrosion and debonding of the epoxy-coated reinforcement in areas of shallow cover and cracking. Occasional corrosion of the reinforcement was discovered at damaged areas of the epoxy coating. The laboratory studies overall found that the corrosion performance of epoxy-coated reinforcement was consistently better than uncoated reinforcement even in cases of severe damage to the coating. The studies also found that concrete cracking and damage to the coating will decrease the effectiveness of epoxy-coated reinforcement as a corrosion protection method. The majority of the studies indicated that the amount of damage to the epoxy coating influenced the degree of corrosion activity.

Table 3.1 Specimen Configurations[21]

Top Mat (Anode)	Bottom Mat (Cathode)	PreCracked?	Percent Damage to Coating
Straight	Black	No	0.5
Straight	Epoxy-Coated	No	0.5
Straight	Black	No	0.004
Straight	Epoxy-Coated	No	0.004
Bent	Black	No	0.5
Bent	Black	Yes	0.004
Straight	Black	Yes	0.5
Straight	Black	Yes	0.004

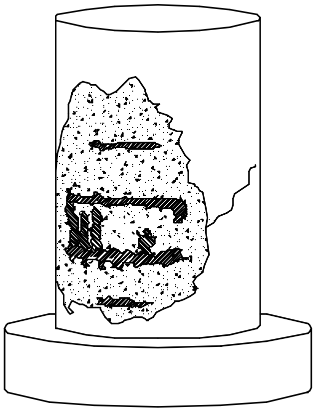


Figure 3.1 Typical Delamination Found on the Seven Mile Bridge in 1987[11]

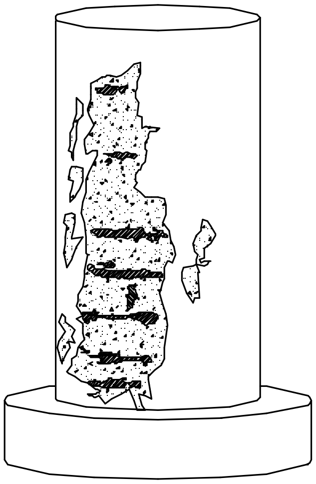


Figure 3.2 Typical Delamination Found on the Nile Channel Bridge in 1987[11]

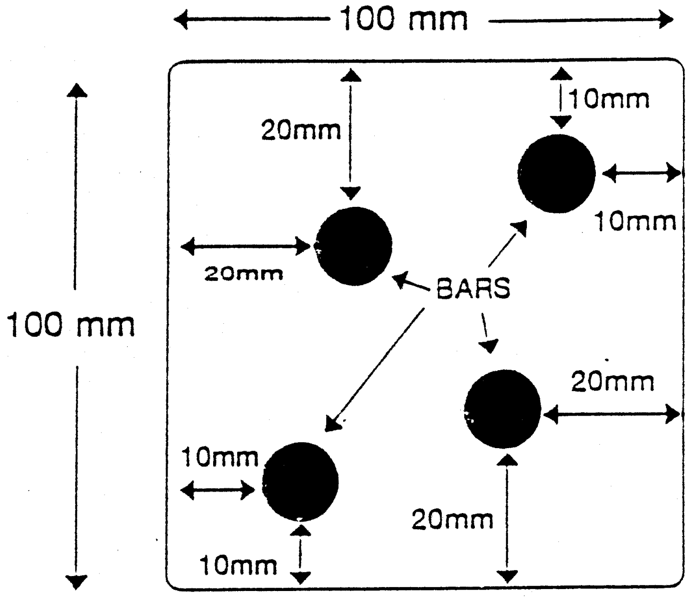


Figure 3.3 Cross-Section of Beam Specimens[18]

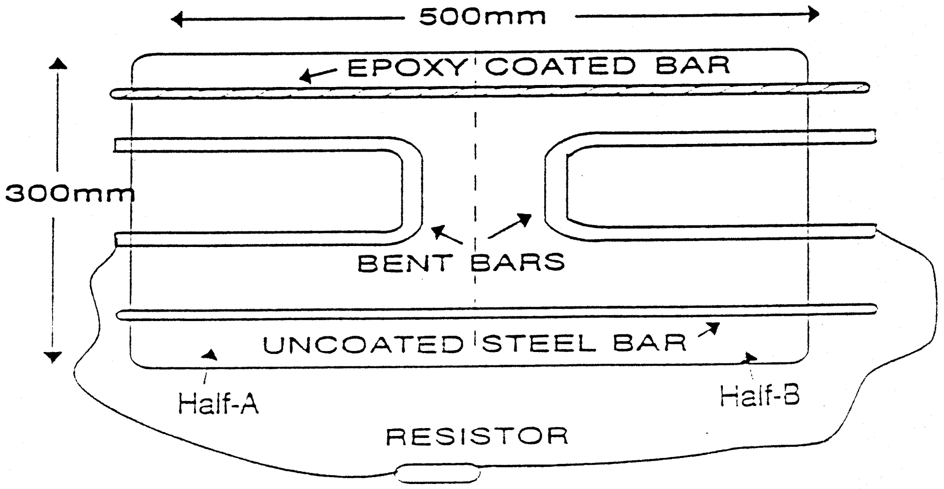


Figure 3.4 Plan View of Slab Specimens[18]

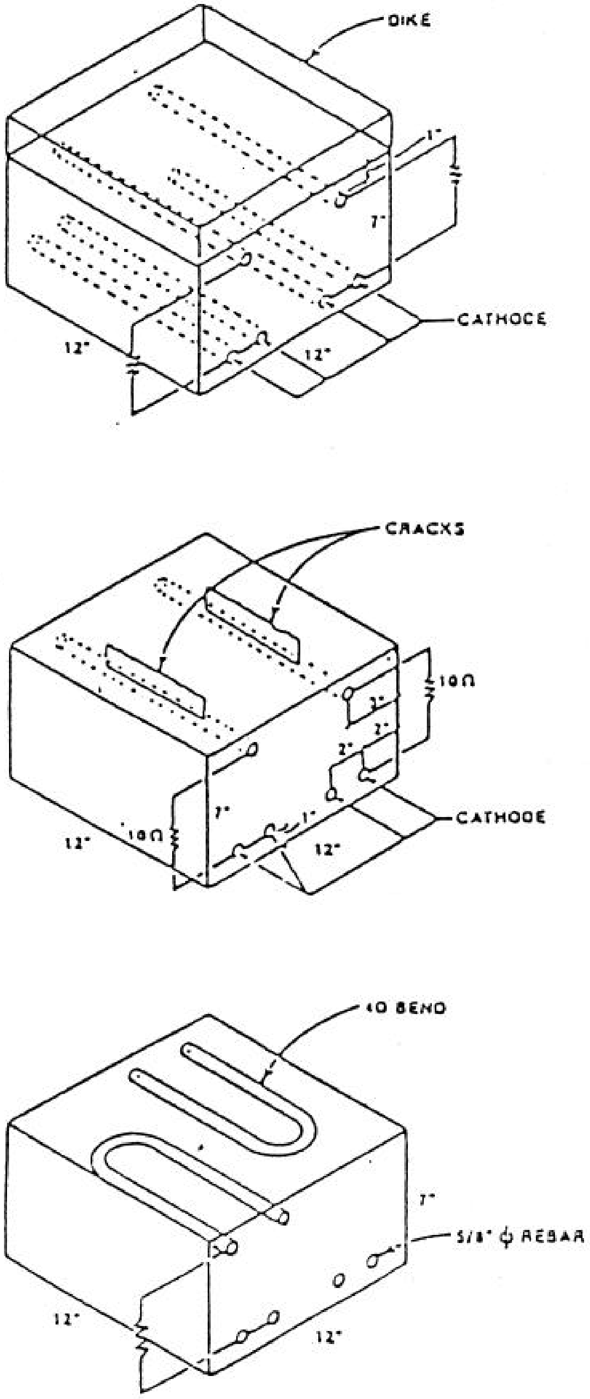


Figure 3.5 Diagram of Concrete Test Slabs[21]

CHAPTER 4 – FIELD EVALUATION

In an attempt to evaluate the field performance of epoxy-coated reinforcement in bridge decks, four major steps were taken. First, a survey of corrosion protection methods used by Indiana and other states was conducted. Next, holiday testing was performed on epoxy-coated reinforcement at bridge deck construction sites. The third step consisted of a series of initial visual bridge inspections on one hundred and twenty three concrete bridge decks in the state of Indiana. The final stage of the field evaluation consisted of a second more detailed inspection on six concrete bridge decks in Indiana selected based on the results of the initial inspections. The results from the field performance evaluation of epoxy-coated reinforcement in concrete bridge decks and slabs are presented in this chapter. An additional objective of the field evaluation was to obtain information necessary to refine the laboratory phase of this study. The specific information required was the number and location of defects created in bridge deck construction operations.

4.1 State Survey

A questionnaire was created and sent out to all other state Departments of Transportation for collecting information on their practices concerning methods of corrosion protection. Fifteen states responded to the survey and a summary of the methods of corrosion protection these states currently use is provided in Table 4.1. An "X" is used to mark the corrosion protection methods each state replied were in use. A blank survey form and complete summary of the all the responses from each state are included in Appendix A.

The results of the survey show that epoxy-coated reinforcement is the most common method of corrosion protection among the fifteen states as thirteen states responded positively. Modified concrete overlays and increased depth of cover were the second most common methods as eleven respondents indicated they used them. The earliest reported usage of epoxy-coated rebar in bridge decks was 1976. All states responding to the survey indicated that the bridge decks containing epoxy-coated bars are performing satisfactorily to date. The widespread usage of epoxy-coated bars emphasizes the importance of evaluating the effectiveness of epoxy-coated bars.

4.2 Field Investigations of New Construction Concrete Bridge Decks

New construction bridge sites were visited to perform holiday testing on epoxy-coated reinforcement. Holiday and defect testing at the construction sites was conducted to obtain data on the number of holidays typically found on epoxy-coated reinforcement from the transportation, placement, and concrete casting operations. Testing for holidays was performed at three different stages: after the epoxy-coated reinforcement arrived on site, after the epoxy-coated reinforcement was placed and tied, and after the concrete casting.

4.2.1 Test Procedure

Epoxy-coated reinforcement was tested for holidays and defects using the KTA-TATER model M/1 Wet Sponge Holiday Detector. This equipment is a sensitive device manufactured to detect holidays and defects in thin film protective coatings. The equipment consists of a battery powered electronic instrument with a ground cable and cable connection to a handle with clamps for holding a wet sponge. The ground cable was connected to the reinforcement ensuring a connection to the bare steel. The sponge was saturated with a conductive wetting solution and attached to the handle with metal clamps. The detector produces a current flow of 700 mA and an audible signal when the circuit is completed. The testing was performed by connecting the ground wire to the reinforcement and moving the handle with the wet sponge over the reinforcement. When a break in the coating was detected, the signal sounds and the defect or holiday was recorded.

4.2.2 Defects and Holidays Created during Transportation to Bridge Site

Nine bridge deck construction sites were visited immediately after the epoxy-coated reinforcement had arrived on site. Holiday and defect testing was performed after the reinforcement arrived at the site on randomly chosen reinforcing bars at the job site. One bar of each bar size used in the bridge deck was tested. The length of the majority of the bars tested was approximately 1.5 m (5 feet). Figure 4.1 shows an epoxy-coated bar being tested after transportation to the bridge site but prior to placement. Table 4.2 provides the results from the testing of the reinforcement after arriving on site. The average number of defects and holidays created during the transportation process was 7.4 per meter of bar (2.3 per foot of bar).

4.2.3 Defects and Holidays Created during Placement of Reinforcement

Defects created during the placement of epoxy-coated reinforcement were measured at six bridge deck construction sites. The measurements were taken immediately before the concrete was cast on the reinforcement that was tied and placed in the bridge deck. With the exception of one bridge site, ten reinforcing bars were tested from each bridge deck. Due to time constraints, only four reinforcing bars could be tested at one of the sites. The reinforcing bars were chosen at random and lengths of 2.9 m (9.5 ft.) or 4.4 m (14.5 ft.) were tested. Defect testing of epoxy-coated reinforcement after being tied and placed is illustrated in Figure 4.2. The results from the defect testing after the reinforcement was tied and placed are provided in Table 4.3. These values are cumulative and include not only the defects created in the placement process, but also the defects created in the transportation process. The average number of defects and holidays found after the placement process was 8.1 per meter (2.5 per foot). This value is only slightly higher than the average number of defects and holidays created in the transportation process. Thus, very few additional holidays were created on the epoxy-coated reinforcement in the placement procedure.

4.2.4 Defects and Holidays Created during Casting of Concrete

The six bridge deck construction sites visited to test the reinforcement after placement were the same six visited to evaluate defects created in the concrete casting operation. To determine if additional holidays were incurred in the concrete casting process, a test bar was tied onto the top mat of reinforcement at the chosen bridge sites. The holidays and defects preexisting on the test bar were identified and counted. Concrete was cast over the test bars in the same manner as the rest of the bridge deck as seen in Figure 4.3. The test bars were then removed from the fresh concrete, washed, and the additional holidays created by the concrete casting operation determined. The results of the defect and holiday testing are provided in Table 4.4. These results show that an excessive amount of defects were created during the concrete casting operation. Also apparent was the influence of the method used to place the concrete. Concrete placed using a pump system where the concrete was pumped vertically up over a crane and dropped onto the reinforcement created the highest number of defects. In one instance, the defects created by a direct hit from the pump created an additional 104 defects per meter (31 defects per foot) of bar. The average number of defects and holidays created by all the methods observed was an additional 31 per meter (9.4 per foot) of bar. All defects were observed on the top surface of the reinforcement confirming that the damage were caused by the concrete impacting the upper portion of the bar as the concrete was placed.

In addition to the results shown in Table 4.4, tests at two supplementary bridge casts were added to determine whether the number of additional holidays, caused by the concrete casting operation using the pump method, could be reduced. The test bars included samples with varying coating thickness to determine the effect of a thicker coating of epoxy in reducing the number of defects. Also, the pump used to deliver the concrete was lowered over several test bars to reduce the vertical drop of the concrete.

Table 4.5 provides the results from these two additional test sites. The additional testing showed that increasing the thickness of the epoxy-coated reinforcement dramatically reduced the number of additional holidays created in the pouring process. Test bars 2 and 4 showed a 71% and 75% reduction in number of defects, respectively, with an increase in coating thickness of 4 mils. Test bar 6 showed a 50% reduction in number of defects with an increase in coating thickness of 2 mils. Lowering the pump to reduce the vertical drop of the concrete also reduced the number of holidays created by an average of 50%.

4.2.5 Summary of Field Investigations of New Construction Concrete Bridge Decks

A summary of the findings from the defect and holiday testing is provided in Table 4.6. The results of the original holiday testing showed that after delivery to the site, placement, and the casting operation, an average top mat epoxy-coated rebar contained approximately 40 holidays per meter of bar (12 holidays per foot of bar). The majority of the holidays were created during the casting operation. The investigation found that the most defects were created using the pump method of concrete placement. This field evaluation provides results indicating that an increase on average of 0.102 mm (4 mils) in the thickness of the epoxy coating will dramatically reduce the number of defects incurred in the concrete casting operation when using the pump method by an average of 73%.

4.3 Initial Field Investigations of Existing Concrete Bridge Decks

In the initial field investigation, 131 bridge decks were originally chosen for visual inspections. The Indiana Department of Transportation provided the bridge structures chosen for the field evaluation. All of the bridge decks, with the exception of six experimental bridge decks, were constructed during the years of 1972 to 1980. Five of the experimental bridge decks were constructed in 1987 and one in 1989. A requirement in the selection of the bridge decks was that the original bridge deck had not been rehabilitated other than by patching. The initial visual inspection entailed taking photographs and videos of the bridge deck, obtaining concrete cover measurements, recording crack patterns and rust stains, and measuring areas of spalling, scaling, and delamination/debonding. Cover measurements were taken in a 12 by 4.5 m (40 by 15 ft.) grid configuration, with readings taken every 1.5 m (5 ft.). The equipment used to measure concrete cover was a James Instruments Rebar Datascan as seen in Figure 4.4. Delaminated areas were determined by chain dragging and marked as shown in Figure 4.5. Areas of delamination, spalling, and patching were combined to give a total distress area. The total distress area was divided by the total area surveyed to give the percent distress area. The bridge decks can be categorized by the corrosion protection method utilized in its construction. Table 4.7 shows the cement content and the water to cement ratio for the different concrete mix designs included in the bridge survey[9].

- 47 bridge decks constructed with black rebar and Class C concrete with 38 mm (1.5 in.) latex modified concrete overlay
- 35 bridge decks constructed with black rebar and Class A concrete
- 28 bridge decks constructed with epoxy-coated rebar and Class C concrete
- 8 bridge decks constructed with black rebar and Class C concrete
- 4 bridge decks constructed with galvanized steel and Class C concrete
- 8 bridge decks constructed with various experimental methods

- 1 bridge deck constructed with Class AA concrete (Kentucky specs.)

The eight experimental methods included the following: three bridge decks constructed with an epoxy overlay, two bridge decks with shrinkage compensating concrete, two bridge decks with DCI corrosion inhibitor, and one bridge deck with high early strength latex modified concrete. Uncoated reinforcement was used in all eight bridge decks.

4.3.1 Results of Initial Visual Inspections

One lane and one shoulder was surveyed on all bridges. Eight of the bridge decks were replaced before the visual inspection could be conducted. Analysis of the remaining 123 bridge decks surveyed showed that 44% had signs of distress. Signs of distress include, spalling, areas of delamination/debonding, and rust stains. Cover measurements were taken on 113 of the surveyed bridge decks. The average cover reading on 35% of these decks was below the specified design cover. Table 4.8 shows a breakdown of the bridges by the five main types of corrosion protection method and the percent showing: any signs of distress, percent with significant distress (greater than 6% of surveyed area showing distress) and percent with severe distress (greater than 20% of surveyed area showing distress). Figure 4.6 shows a spall with reinforcement exposed and Figure 4.7 shows rust staining around a crack; both observed in the initial field investigation. Only one of the eight bridge decks using the experimental corrosion methods listed previously showed any deterioration. The epoxy overlay on this bridge deck was deteriorating and debonding in many areas; however, no corrosion related distress was observed. Corrosion distress of the experimental bridge decks was not expected due to the early age of the bridge decks. The long term performance of the experimental bridge decks is being evaluated, as part of the NEEP 12 study[+] and more information on these decks will be available from that study. A complete listing of the results from the initial bridge inspections is provided in Appendix B. The list of bridges is presented by corrosion protection method and percent area of distress. The Indiana Department of Transportation bridge deck rating number is also included for each bridge deck. The bridges are rated on a scale of 0 to 9, with 9 representing excellent condition. A rating of 6 would imply satisfactory condition with minor deterioration. Information regarding the amount of deicing salt applied per lane or mile in each district would have been helpful in comparing bridge decks across the state of Indiana; however, this information was not readily available from the Indiana Department of Transportation.

4.3.2 Summary of Initial Visual Inspections

Initial field investigations of 123 bridge decks in Indiana indicated corrosion related distress in 44% of the bridge decks surveyed. The initial field investigations also found that 35% of the bridge decks surveyed had an average cover reading below the specified design value. Epoxy-coated reinforcement combined with Class C concrete provided the most successful corrosion protection method as only 11% of the bridge decks in this category showed distress. This percentage was the lowest of all categories of corrosion protection methods. Uncoated reinforcement with a design cover of 25.4 mm (1.0 in.) Class C concrete and 38.1 mm (1.5 in.) latex modified overlay was the second least successful corrosion protection method. In this category, 52% of the bridge

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[+] Private Communication with Tommy Nantung, Section Engineer, Indiana Department of Transportation Research Division, P.O. Box 2279, West Lafayette, IN 47906, Phone #: 765-463-1521 decks showed signs of distress. The range of age of construction for this method is comparable to that of epoxy-coated bridges. The least successful corrosion protection method was uncoated reinforcement with a design cover of 50.8 mm (2 in.) of Class A concrete. This method is no longer used in Indiana.

4.4 Detailed Field Investigations of Existing Concrete Bridge Decks

One bridge deck from each of the main corrosion protection categories was chosen for a second more detailed survey. Table 4.9 shows the specifics of the bridge structures chosen for the detailed survey. The following items were included in the second detailed inspections:

- Three 102 mm (4 in.) cores were taken to extract rebar and examine concrete in good and bad areas.
- Ten 25 mm (1 in.) or 38 mm (1.5 in.) cores were taken to be ground for chloride analysis.
- Half-Cell Potentials were recorded on bridge decks with uncoated reinforcement
- Cover Readings, Delaminations, Crack Patterns, were recorded as needed

The goal of the second detailed inspection was to gain information about the condition of the bridge decks that the initial visual inspection could not provide. Specifically, powder samples were analyzed to determine the level of chlorides present, cores were examined to assess concrete quality and rebar deterioration, and a mapping of the half-cell potentials, delaminations, and cracking patterns was performed to identify deteriorated portions of the deck. The three 102 mm (4 in.) cores, taken to examine the rebar quality, were removed from three different areas of the bridge deck. When possible, one core was removed from an area of sound concrete, delaminated concrete, and cracked concrete. Figure 4.8 illustrates the process of removing a core from a bridge deck. The Indiana Department of Transportation supplied the core rig and operators. The smaller cores removed for chloride analysis were spaced evenly over the surface of the bridge deck. An example of a 38 mm (1.5 in.) diameter core removed for chloride analysis is shown in Figure 4.9. The half-cell potential measurements were taken according to ASTM procedures. A direct connection to the top mat of reinforcement was established and connected to the positive terminal of a high internal resistance multimeter. A copper copper-sulfate electrode (CSE) was connected to the common terminal of the multimeter. The concrete was sprayed with water to moisten before each half-cell reading was recorded. The readings (voltage) were spaced at 1.5 m (5 ft.) intervals in a grid over the entire surveyed area. According to ASTM the readings can be interpreted as follows:

- Half-Cell Potentials > -0.20 volts, indicate 90% probability of no corrosion activity
- -0.35 volts < Half-Cell Potentials < -0.20 volts, corrosion activity unsure
- Half-Cell Potentials < -0.35 volts, indicate 90% probability of corrosion activity

Mappings of the half-cell potentials across the surface of the deck can be used to indicate areas where the top mat of reinforcement is corroding. Areas with half-cell potential voltage readings less than –0.35 volts are anodic and areas with voltage readings greater than –0.20 volts are cathodic. All mappings of the bridge decks are drawn to scale.

4.4.1 Results of Detailed Bridge Deck Survey

One lane and one shoulder of each of the six bridge decks listed in Table 4.9 was investigated in the detailed survey. The bridges chosen for the detailed survey were selected based on the level of distress observed in the initial visual survey. For each of the five main corrosion protection categories, the bridge deck with the highest level of distress was chosen for the detailed survey. However, no signs of distress were found on two of the selected bridge decks. None of the bridges surveyed with epoxy-coated reinforcement with Class C concrete and uncoated bottom mat of reinforcement showed any signs of distress. The bridge structure chosen for this category was selected based on the close proximity to other structures selected for the detailed survey. Also, it was originally thought that one bridge structure with epoxy-coated reinforcement and Class C concrete had 38 mm (1.5 in.) of latex modified concrete overlay. This bridge showed no signs of distress but was chosen for the detailed survey as a six method of corrosion protection. During the detailed survey it was discovered through the inspection of the cores removed from the bridge deck that no overlay was present. Therefore, this bridge deck provides a duplicate in the category of epoxy-coated reinforcement with Class C concrete. Table 4.10 provides the main results from the detailed survey. Figure 4.10 shows the results from the chloride analysis. A discussion of each bridge deck follows.

4.4.1.1 Structure #37-47-5980

Structure #37-47-5980 is located at State Road 37 Northbound over US Highway 50 in the southwest region of Indiana. The bridge structure is a four span continuous composite steel beam with a total length of approximately 91 m (300 ft.) built in 1973.

The average daily traffic for this bridge is 5100 cars. A side view of the bridge structure is provided by Figure 4.11. Extensive cracking on the underside of the bridge deck is shown in Figure 4.12. The bridge deck has uncoated reinforcement and a design cover of 50.8 mm (2.0 in.) of Class A concrete as a corrosion protection method. However, as presented in Table 4.10, the cover survey found an average cover of only 44.45 mm (1.75 in.) with a standard deviation of 12.4 mm (0.49 in.). As shown in Table 4.7, Class A concrete is a more permeable concrete mix that was used in bridge deck construction in Indiana until 1976 at which time the less permeable Class C concrete was specified for use in bridge decks. The chloride concentration profiles in Figure 4.10 show that the average chloride level at the depth of the top mat of reinforcement was in excess of the estimated level required for initiation of corrosion. However, the level of chloride is less than the chloride level in four of the other bridge decks all constructed in later years. The location of this bridge structure explains this difference. The southwest region of Indiana is milder in climate and the bridge deck would be expected to receive fewer deicing applications, thus explaining the lower chloride concentrations. The field investigations found 32% of the surveyed area of the bridge deck exhibiting signs of corrosion distress. A mapping of the delaminated, spalled, and patched areas is provided by Figure 4.13. The majority of the distress area on this bridge deck was found along the shoulder of the roadway. Figure 4.14 provides the location of the cores taken for rebar examination and chloride analysis. A contour plot of the half-cell potential readings is provided in Figure 4.15. This plot also shows that the shoulder area of the bridge deck had the most half-cell readings indicating corrosion. Two of the three 102 mm (4 in.) cores taken through the reinforcement confirmed corrosion of the reinforcement. Figure 4.16 shows the removal of a core from a cracked and delaminated region. Rust staining was observed in the concrete area exposed by the core. Figures 4.17 and 4.18 illustrate the contrast between the condition of the reinforcement in an area of sound area concrete and the condition of the reinforcement in an area of delamination.

4.4.1.2 Structure #32-18-2182

Structure #32-18-2182 is located at Tillitson Avenue over State Road 32. The structure is a five span continuous prestressed concrete I-Beam bridge with a total length of approximately 96 m (315 ft) built in 1975. The average daily traffic for this bridge is 13,200 cars. Figure 4.19 provides a side view of the structure. The bridge deck was constructed with uncoated reinforcement and a design cover of 63.5 mm (2.5 in.) of Class C concrete as a corrosion protection method. The concrete cover survey found an average cover of 67.56 mm (2.66 in.) with a standard deviation of 15.5 mm (0.61 in.), which exceeds the design requirements. Class C concrete is the concrete mix currently specified for bridge deck construction in Indiana. The chloride concentration profiles in Figure 4.10 show that the average chloride level at the depth of the top mat of reinforcement was the highest of all six bridges surveyed and in excess of the estimated level required for initiation of corrosion. The field investigations found 71% of the surveyed bridge deck area to be exhibiting signs of corrosion distress. The majority of the area of distress was found in the first two spans as shown in Figure 4.20. Figure 4.21 shows the location of the cores taken from the bridge deck. The bridge deck also had extensive cracking over the majority of the deck surface as pictured in Figure 4.22. The half-cell contour plot provided in Figure 4.23 shows corrosion activity in two bands.

Delaminations were also found in these areas. Half-cell potential readings could not be taken in areas of patching due to the interference of the asphalt patching material. Two of the three 102 mm (4 in.) cores taken through the reinforcement confirmed corrosion of both the transverse and longitudinal reinforcement. Figure 4.24 shows the core taken in an area of delamination. The delamination was severe in this area of the bridge deck and the hollow area underneath is evident in Figure 4.24. Corrosion damage was observed on the prestressed concrete I-beams of the substructure as shown in Figures 4.25 and 4.26.

4.4.1.3 Structure #6-50-5187

Structure #6-50-5187 is located at US Highway 6 over the Yellow River. The structure is a three span continuous prestressed concrete I–Beam bridge with a total length of approximately 42.67 m (140 ft) built in 1980. The average daily traffic for this bridge is 7700 cars. A profile of the bridge is provided in Figure 4.27. The bridge deck has uncoated reinforcement and a design cover of 25.4 mm (1.0 in.) of Class C concrete and 38.1 mm (1.5 in.) of latex modified overlay as a corrosion protection method. The concrete cover survey found an average cover of 65.53 mm (2.58 in.) with a standard deviation of 6.6 mm (0.26 in.) which exceeds the design requirements. The field investigations found 31% of the bridge deck area that was surveyed exhibiting signs of corrosion distress, which was concentrated in one main area as shown in Figure 4.28. Figure 4.29 provides the location of the cores taken from the bridge deck. Corrosion activity was indicated by Half-Cell Potential measurements in the same distress areas as found by the visual inspection and chain drag, as shown in Figure 4.30. One of the three 102 mm (4 in.) cores taken through the reinforcement confirmed corrosion of the reinforcement. Figure 4.31 shows the core that was taken in the cracked area. The chloride concentration profiles in Figure 4.10 show that the average chloride level at the depth of the top mat of reinforcement was the lowest of all six bridges surveyed and slightly below the estimated level required for initiation of corrosion. The reason for the high level of corrosion distress with a low chloride concentration is unknown. Figure 4.32 shows the extensive areas of patching present on the bridge deck. Figures 4.33 and 4.34 show the corrosion damage present on the outer edges of the underside of the bridge deck. This corrosion damage is likely to have been caused by deicing chemicals washing over the edge of the bridge deck. The open guardrail design used in this bridge deck is no longer used in Indiana to prevent this type of corrosion damage.

4.4.1.4 Structure #6-50-6624

Structure #6-50-6624 is located at US Highway 6 over State Road 331 and is pictured in Figure 4.35. The bridge structure type is a three span hinged composite steel beam with a total length of approximately 49 m (160 ft) built in 1980. The average daily traffic for this bridge is 8200 cars. The bridge deck has epoxy-coated reinforcement and a design cover of 63.5 mm (2.5 in.) of Class C concrete. The concrete cover survey found an average cover of 54.61 mm (2.15 in.) with a standard deviation of 8.9 mm (0.35 in.), which is lower than the design requirements. The chloride concentration profiles in Figure 4.10 show that the average chloride level at the depth of the top mat of reinforcement exceeds the estimated threshold level for initiation of corrosion. The field investigations found only 0.7% of the bridge deck exhibiting signs of corrosion distress. This area of distress was located in one region and is shown in Figure 4.36 along with the location of cores taken. A close up of the area of distress is shown in Figure 4.37. The rest of the bridge deck surface showed no signs of corrosion distress. The reinforcement was exposed in this area as seen in Figure 4.38. The epoxy coating was removed from the bar in this area and extensive corrosion was evident. The field investigation found this region of distress to have less then one inch of cover in the localized area surrounding the spalls and delamination. The delamination can be clearly seen around the level of reinforcement in Figure 4.39. The 102 mm (4 in.) core taken through the reinforcement confirmed corrosion of the reinforcement in this region and is pictured in Figure 4.40. The bar closest to the surface in this area showed extensive underfilm corrosion as seen in Figure 4.41. Figure 4.41 also shows the rebar located in the perpendicular direction under the top bar. This bar showed little signs of corrosion, however the coating was debonded and easy to remove. Figure 4.42 shows rebars removed from a sound area of concrete. The larger bar was discolored and the coating was debonded and easy to remove as shown in Figure 4.43. The average epoxy coating thickness found on the bars removed from the cores was 197.6 μm (7.78 mils) with a standard deviation of 20.1 μm (0.79 mils).

4.4.1.5 Structure #331-50-6608

Structure #331-50-6608 is located at State Road 331 over US 30. The bridge structure type is a two span continuous welded girder with a total length of approximately 69 m (225 ft) built in 1976. The average daily traffic for this bridge is 2500 cars. The structure is shown in Figure 4.44. The bridge deck has epoxy-coated reinforcement and a design cover of 63.5 mm (2.5 in.) of Class C concrete. The concrete cover survey found an average cover of 55.63 mm (2.19 in.) with a standard deviation of 7.4 mm (0.29 in.), which is lower than the design requirements. The chloride concentration profiles in Figure 4.10 show that the average chloride level at the depth of the top mat of reinforcement exceeds the estimated threshold level for initiation of corrosion. The field investigations found no evidence of corrosion distress as shown in Figure 4.45, which provides the core locations. However, one of the 102 mm (4 in.) cores taken through the reinforcement at a cracked location showed signs of corrosion. Rust stains were evident at the rib locations of the reinforcement as seen in Figures 4.46 and 4.47. Figure 4.48 shows the underside of the reinforcement removed from the cracked region. The coating was cracked with evidence of corrosion. Also, the coating was easy to remove and underfilm corrosion was found as shown in Figure 4.49. No other signs of corrosion were found on any other removed bars. Figure 4.50 shows the reinforcement removed from a sound area of the deck with no corrosion present. The coating of these bars was well adhered and difficult to remove. The average epoxy coating thickness found on the bars removed from the cores was 204.2 μm (8.04 mils) with a standard deviation of 15.5 μm (0.61 mils).

4.4.1.6 Structure #6-50-6577

Structure #6-50-6577 is located at US Highway 6 over Stock Ditch as pictured in Figure 4.51. The bridge structure type is a reinforced concrete slab with a total length of approximately 27 m (90 ft) built in 1980. The average daily traffic for this bridge is 9500 cars. The bridge deck has epoxy-coated top mat reinforcement and an uncoated bottom mat of reinforcement with a design cover of 63.5 mm (2.5 in.) of Class C concrete. The concrete cover survey found an average cover of 74.42 mm (2.93 in.) with a standard deviation of 13.5 mm (0.53 in.), which is greater than the design requirements. The chloride concentration profiles in Figure 4.10 show that the average chloride level at the depth of the top mat of reinforcement exceeds the estimated threshold level for initiation of corrosion. The field investigations found no evidence of corrosion distress as shown in Figure 4.52, which provides the core locations. None of the 102 mm (4 in.) cores taken through the reinforcement showed any signs of corrosion. Corrosion, however, was found along the edge of the underside of the bridge deck as shown in Figure 4.53. Again, the open guardrail design promotes this damage. The average epoxy coating thickness found on the bars removed from the cores was 205.2 μm (8.08 mils) with a standard deviation of 21.8 μm (0.86 mils).

4.4.2 Summary of Detailed Bridge Deck Survey

The detailed field investigation found corrosion of epoxy-coated reinforcement in areas of cracking and insufficient concrete cover. In the area of cracking in bridge structure #331-50-6608 where corrosion of the epoxy-coated reinforcement was found, no delaminations were present to indicate any signs of distress. The detailed field investigation also discovered the lowest level of chloride concentrations in bridge structure #6-50-5187, which contained a latex modified overlay. However, the level of distress, 31% of the surveyed area, is much greater then that found in the three bridges with epoxy-coated reinforcement of the same vintage or older. No data was collected during the bridge deck survey that explains this anomaly. Plausible explanations for the high level of corrosion distress with a low chloride concentration may include moisture content or amount of deck cracking. Further research is warranted to investigate this occurrence. The latex modified bridge and the epoxy-coated bridges were located within the same county indicating similar exposure conditions.

4.5 Information for Laboratory Evaluation

Results from the field investigation of new construction concrete bridge decks were used in developing the laboratory-testing program. The field evaluation provided results which showed that after delivery to the site, placement, and the casting operation was conducted, an average top mat epoxy-coated bar would contain approximately 40 holidays per meter of bar (12 holidays per foot of bar). This information was used in the laboratory evaluation to determine the maximum number of defects to be created on the epoxy-coated reinforcement.

Table 4.1 State Survey Results

	Method of Corrosion Protection						
State	Cathodic Protection	Corrosion Inhibiting Admixture	Modified Concrete Overlays	Membranes / Epoxy Overlays	Type K Cement	Epoxy Coated Rebar	Increased Depth of Cover
Arkansas						X	
Florida							
Indiana	X	X	X	X	X	X	X (2.5”)
Iowa	X	X	X			X	X (2.5”)
Kansas			X			X	X (3”)
Maine		X	X				
Michigan	X		X	X	X	X	X (3”)
Missouri	X		X	X		X	X (3”)
Montana	X		X	X		X	X (2-3/8”)
New York	X	X	X	X	X	X	X (3.25”)
South Carolina		X				X	X
Texas	X	X	X			X	
Vermont				X		X	X
West Virginia	X	X	X	X		X	X
Wyoming		X	X	X		X	X
Total	8/15	8/15	11/15	8/15	3/15	13/15	11/15

Table 4.2 Defects of Epoxy-Coated Steel after Arriving on Site

Structure #	Location	Bar Size mm (US)	Length Sampled	# of Holidays	# of Holidays per meter of Bar (# of Holidays per Foot of Bar)
I70-77-5643B	I-70 WBL over Belmont St.	12.7 (#4)	1.5 m (5 ft.)	12	8 (2.4)
		15.9 (#5)	1.5 m (5 ft.)	10	6.7 (2)
		19.1 (#6)	1.5 m (5 ft.)	3	2 (0.6)
I70-77-2385A	I-70 WBL over Harding St.	12.7 (#4)	1.5 m (5 ft.)	14	9.3 (2.8)
		15.9 (#5)	1.5 m (5 ft.)	15	10 (3)
		19.1 (#6)	1.5 m (5 ft.)	8	5.3 (1.6)
46-11-7754	SR 46 over Big Slough Creek	12.7 (#4)*	1.8 m (6 ft.)	6	3.3 (1)
		15.9 (#5)*	2.1 m (7 ft.)	49	23.3 (7)
		19.1 (#6)	2 m (6.5 ft.)	16	8 (2.5)
		19.1 (#6)*	1.8 m (5.8 ft.)	9	5 (1.5)
		22.2 (#7)	1.8 m (6 ft.)	30	16.7 (5)
35-09-7741	US 35 over Crooked Creek	12.7 (#4)	1.5 m (5 ft.)	13	8.7 (2.6)
		15.9 (#5)	1.5 m (5 ft.)	15	10 (3)
		19.1 (#6)	1.5 m (5 ft.)	13	8.7 (2.6)
		22.2 (#7)	1.5 m (5 ft.)	30	20 (6)
334-06-7680	SR 334 over I-65	12.7 (#4)	1.5 m (5 ft.)	0	0 (0)
		15.9 (#5)	1.5 m (5 ft.)	5	3.3 (1)
		19.1 (#6)	1.5 m (5 ft.)	3	2 (0.6)

Table 4.2 (concluded)

Structure #	Location	Bar Size mm (US)	Length Sampled	# of Holidays	# of Holidays per meter of Bar (# of Holidays per Foot of Bar)
31A-36-4655	SR 11 over I-65	12.7 (#4)	1.2 m (4 ft.)	3	2.5 (0.75)
		15.9 (#5)	1.5 m (5 ft.)	10	6.7 (2)
		19.1 (#6)	1.4 m (4.5 ft.)	4	2.9 (0.89)
		22.2 (#7)	1.5 m (5 ft.)	5	0.7 (1)
I74-170-4685	I-74 over Johnson Fork	12.7 (#4)	1.5 m (5 ft.)	3	2 (0.6)
		15.9 (#5)	1.5 m (5 ft.)	2	1.3 (0.4)
		19.1 (#6)	1.5 m (5 ft.)	7	4.7 (1.4)
41-56-7636	US 41 over Kankakee River	12.7 (#4)*	1.5 m (5 ft.)	8	5.3 (1.6)
		15.9 (#5)*	1.5 m (5 ft.)	13	8.7 (2.6)
		22.2 (#7)*	1.5 m (5 ft.)	21	14 (4.2)
67-55-7753	SR 67 over White Lick Cr.	12.7 (#4)	1.5 m (5 ft.)	14	9.3 (2.8)
		15.9 (#5)	0.9 m (3 ft.)	13	14.4 (4.3)

*Note: All reinforcing bars had the spiral deformation pattern except those noted with a *. Those noted with a * had the diamond deformation pattern.*

Table 4.3 Defects of Epoxy-Coated Steel after Placement in Bridge Deck

Structure #	Location	Bar Size mm (US)	Length Sampled	# of Holidays	# of Holidays per meter of Bar (# of Holidays per Foot of Bar)
31A-36-4655	SR 11 over I-65	15.9 (#5)	4.4 m (14.5 ft.)	35	8 (2.4)
		15.9 (#5)	4.4 m (14.5 ft.)	18	4.1 (1.2)
		15.9 (#5)	4.4 m (14.5 ft.)	26	5.9 (1.8)
		15.9 (#5)	4.4 m (14.5 ft.)	25	5.7 (1.7)
		15.9 (#5)	4.4 m (14.5 ft.)	19	4.3 (1.3)
		15.9 (#5)	4.4 m (14.5 ft.)	28	6.4 (1.9)
		15.9 (#5)	4.4 m (14.5 ft.)	43	9.8 (3.0)
		15.9 (#5)	4.4 m (14.5 ft.)	47	10.7 (3.2)
		15.9 (#5)	4.4 m (14.5 ft.)	57	13 (3.9)
		15.9 (#5)	4.4 m (14.5 ft.)	84	19 (5.8)
I70-77-5643B	I-70 WBL over Belmont St.	15.9 (#5)	4.4 m (14.5 ft.)	34	7.7 (2.3)
		15.9 (#5)	4.4 m (14.5 ft.)	43	1 (3.0)
		15.9 (#5)	4.4 m (14.5 ft.)	23	5.2 (1.6)
		15.9 (#5)	4.4 m (14.5 ft.)	24	5.5 (1.7)
		15.9 (#5)	4.4 m (14.5 ft.)	39	8.9 (2.7)
		15.9 (#5)	4.4 m (14.5 ft.)	37	8.4 (2.6)
		15.9 (#5)	4.4 m (14.5 ft.)	21	4.8 (1.5)
		15.9 (#5)	4.4 m (14.5 ft.)	44	10 (3.0)
		15.9 (#5)	4.4 m (14.5 ft.)	16	3.6 (1.1)
		15.9 (#5)	4.4 m (14.5 ft.)	30	6.8 (2.1)

Table 4.3 (continued)

Structure #	Location	Bar Size mm (US)	Length Sampled	# of Holidays	# of Holidays per meter of Bar (# of Holidays per Foot of Bar)
35-09-7741	US 35 over Crooked Creek	15.9 (#5)	2.9 m (9.5 ft.)	9	3.1 (1.0)
		15.9 (#5)	2.9 m (9.5 ft.)	16	5.5 (1.7)
		15.9 (#5)	2.9 m (9.5 ft.)	14	4.8 (1.5)
		15.9 (#5)	2.9 m (9.5 ft.)	16	5.5 (1.7)
		15.9 (#5)	2.9 m (9.5 ft.)	8	2.8 (0.8)
		15.9 (#5)	2.9 m (9.5 ft.)	19	6.6 (2.0)
		15.9 (#5)	2.9 m (9.5 ft.)	16	5.5 (1.7)
		15.9 (#5)	2.9 m (9.5 ft.)	17	5.9 (1.8)
		15.9 (#5)	2.9 m (9.5 ft.)	20	6.9 (2.1)
		15.9 (#5)	2.9 m (9.5 ft.)	19	6.6 (2.0)
46-11-7754	SR 46 over Big Slough Creek	15.9 (#5)	2.9 m (9.5 ft.)	34	11.7 (3.6)
		15.9 (#5)	2.9 m (9.5 ft.)	45	15.6 (4.7)
		15.9 (#5)	2.9 m (9.5 ft.)	29	10 (3.1)
		15.9 (#5)	2.9 m (9.5 ft.)	36	12.4 (3.8)
67-55-7753	SR 67 over White Lick Cr.	15.9 (#5)	4.4 m (14.5 ft.)	30	6.8 (2.1)
		15.9 (#5)	4.4 m (14.5 ft.)	41	9.3 (2.8)
		15.9 (#5)	4.4 m (14.5 ft.)	22	5 (1.5)
		15.9 (#5)	4.4 m (14.5 ft.)	40	9.1 (2.8)
		15.9 (#5)	4.4 m (14.5 ft.)	44	10 (3.0)
		15.9 (#5)	4.4 m (14.5 ft.)	34	7.7 (2.3)
		15.9 (#5)	4.4 m (14.5 ft.)	26	5.9 (1.8)
		15.9 (#5)	4.4 m (14.5 ft.)	45	10.2 (3.1)
		15.9 (#5)	4.4 m (14.5 ft.)	27	6.1 (1.9)
		15.9 (#5)	4.4 m (14.5 ft.)	29	6.6 (2.0)

Table 4.3 (concluded)

Structure #	Location	Bar Size mm (US)	Length Sampled	# of Holidays	# of Holidays per meter of Bar (# of Holidays per Foot of Bar)
334-06-7680	SR 334 over I-65	15.9 (#5)	4.4 m (14.5 ft.)	61	13.9 (4.2)
		15.9 (#5)	4.4 m (14.5 ft.)	34	7.7 (2.3)
		15.9 (#5)	4.4 m (14.5 ft.)	46	10.5 (3.2)
		15.9 (#5)	4.4 m (14.5 ft.)	54	12.3 (3.7)
		15.9 (#5)	4.4 m (14.5 ft.)	51	11.6 (3.5)
		15.9 (#5)	4.4 m (14.5 ft.)	49	11.1 (3.4)
		15.9 (#5)	4.4 m (14.5 ft.)	53	12.0 (3.7)
		15.9 (#5)	4.4 m (14.5 ft.)	38	8.6 (2.6)
		15.9 (#5)	4.4 m (14.5 ft.)	28	6.4 (1.9)
		15.9 (#5)	4.4 m (14.5 ft.)	46	10.5 (3.2)

Table 4.4 Defects of Epoxy-Coated Steel after Concrete Cast

Structure #	Location	Bar Size	Length Sampled	# of Holidays	# of Holidays per meter of Bar (# of Holidays per Foot of Bar)	Notes
31A-36-4655	SR 11 over I-65	15.9 (# 5)	0.9 m (3 ft.)	8 Add.	8.9 Add. (2.7)	No direct hit from pump or vibrator.
I70-77-2385A	I-70 WBL over Harding St.	15.9 (# 5)	0.9 m (3 ft.)	94 Add.	104 Add. (31.3)	No direct hit by vibrator. Direct hit by pump. (Continuous Flow)
35-09-7741	US 35 over Crooked Creek	15.9 (# 5)	0.6 m (2 ft.)	3 Add.	5 Add. (1.5)	Concrete poured over test rebar directly from concrete truck.
46-11-7754	SR 46 over Big Slough Creek	15.9 (# 5)	0.6 m (2 ft.)	10 Add.	16.7 Add. (5)	Concrete poured over test rebar directly from bucket.
67-55-7753	SR 67 over White Lick Cr.	15.9 (# 5)	0.6 m (2 ft.)	25 Add.	41.7 Add. (12.5)	Direct hit from pump.
334-06-7680	SR 334 over I-65	15.9 (# 5)	0.8 m (2.5 ft.)	11 Add.	13.8 Add. (4.4)	No direct hit from pump or vibrator.

Table 4.5 Holiday and Defect Results from Additional Bridge Casts

Test Bar #	Avg. Thickness mm (mils)	Additional Defects per meter of bar (per foot of bar)	Notes
1	0.17 (6.8)	83.7 (25.5)	Bar 1 and 2 tied in to reinforcing mat side by side, poured from usual pump height.
2	0.27 (10.5)	24.6 (7.5)	
3	0.18 (7.1)	45.9 (14)	Bar 3 and 4 tied in to reinforcing mat side by side, pump lowered down to level of reinforcement.
4	0.29 (11.3)	11.5 (3.5)	
5	0.22 (8.8)	34.4 (10.5)	Bar 4 and 6 tied in to reinforcing mat side by side, used pump reducer to make flow more uniform.
6	0.26 (10.4)	18.0 (5.5)	

Table 4.6 Holiday and Defect Testing Results

After Arriving on Site - Nine Bridge Sites	
Average: 7.4 (2.3)	Holidays / Meter of Bar (Holidays / Foot of Bar)
Maximum: 23.3 (7)	Holidays / Meter of Bar (Holidays / Foot of Bar)
Minimum: 0	Holidays / Meter of Bar (Holidays / Foot of Bar)
After Tied and Placed - Six Bridge Sites	
Average: 8.1 (2.5)	Holidays / Meter of Bar (Holidays / Foot of Bar)
Maximum: 19.0 (5.8)	Holidays / Meter of Bar (Holidays / Foot of Bar)
Minimum: 2.8 (0.8)	Holidays / Meter of Bar (Holidays / Foot of Bar)
After Cast - Six Bridge Sites	
Average: 30.8 (9.4)	Holidays / Meter of Bar (Holidays / Foot of Bar)
Maximum: 104 (31.3)	Holidays / Meter of Bar (Holidays / Foot of Bar)
Minimum: 4.9 (1.5)	Holidays / Meter of Bar (Holidays / Foot of Bar)
Total Average Bar = 39 Holidays / Meter of Bar (12 Holidays / Foot of Bar)	

Table 4.7 Concrete Specifications[9]

Concrete	Cement Content kg/m^3 (lb/yd^3)	Maximum Water to Cement Ratio
Class A	335 (564)	0.532
Class C	390 (658)	0.443
Class AA	368 (620)	0.444

Table 4.8 Results of Initial Visual Inspections

Corrosion Protection Method	% of Bridges Surveyed Showing Any Sign of Distress	% of Bridges with > 6 % Area of Distress (Significant)	% of Bridges with > 20 % Area of Distress (Severe)	Years of Construction
Black rebar and Class A concrete	71% (20 of 28)	29% (8 of 28)	14% (4 of 28)	1972 -1976
Black rebar and Class C concrete with 38 mm (1.5 in.) LMC overlay	52% (24 of 46)	15% (7 of 46)	2% (1 of 46)	1974 - 1980
Galvanized rebar with Class C concrete	50% (2 of 4)	25 % (1 of 4)	0% (0 of 4)	1976
Black rebar with Class C concrete	38% (3 of 8)	12.5% (1 of 8)	12.5% (1 of 8)	1973 - 1980
Epoxy Coated rebar with Class C concrete	11% (3 of 28)	0% (0 of 28)	0% (0 of 28)	1976 - 1980

Table 4.9 Bridge Structures Chosen for Detailed Survey

Bridge Structure #	Corrosion Protection Method	Type of Structure	Region of Indiana	Year Built	Design Cover
37-47-5980	Black Rebar with Class A Concrete	Continuous Composite Steel Beam	Southwest	1973	50.8 mm (2.0 in.)
32-18-2182	Black Rebar with Class C Concrete	Continuous Prestressed Concrete I - Beam	East Central	1975	63.5 mm (2.5 in.)
6-50-5187	Black Rebar with Class C Concrete with 38 mm (1.5 in.) Latex Modified Overlay	Continuous Prestressed Concrete I - Beam	North Central	1980	63.5 mm (2.5 in.)
6-50-6624	Epoxy-Coated Rebar with Class C Concrete	Hinged Composite Steel Girder and Beam	North Central	1980	63.5 mm (2.5 in.)
331-50-6608	Epoxy-Coated Rebar with Class C Concrete	Continuous Welded Girder	North Central	1976	63.5 mm (2.5 in.)
6-50-6577	Epoxy-Coated Rebar with Class C Concrete and Uncoated Bottom Mat of Reinforcement	Reinforced Concrete Slab	North Central	1980	63.5 mm (2.5 in.)

Table 4.10 Results of Detailed Survey

Bridge Structure #	Corrosion Protection Method	Year Built	% Distress of the Surveyed Area	Average Cover	Number of 102 mm (4 in.) cores showing corrosion
37-47-5980	Black Rebar with Class A Concrete	1973	32	44.45 mm (1.75 in.)	2
32-18-2182	Black Rebar with Class C Concrete	1975	71	67.56 mm (2.66 in.)	2
6-50-5187	Black Rebar with Class C Concrete with 38 mm (1.5 in.) Latex Modified Overlay	1980	31	65.53 mm (2.58 in.)	1
6-50-6624	Epoxy-Coated Rebar with Class C Concrete	1980	0.68	54.61 mm (2.15 in.)	1
331-50-6608	Epoxy-Coated Rebar with Class C Concrete	1976	0	55.63 mm (2.19 in.)	1
6-50-6577	Epoxy-Coated Rebar with Class C Concrete and Uncoated Bottom Mat of Reinforcement	1980	0	74.42 mm (2.93 in.)	0

Figure 4.1 Holiday and Defect Testing after Arriving on Site

Figure 4.2 Holiday and Defect Testing after Placement in Bridge Deck

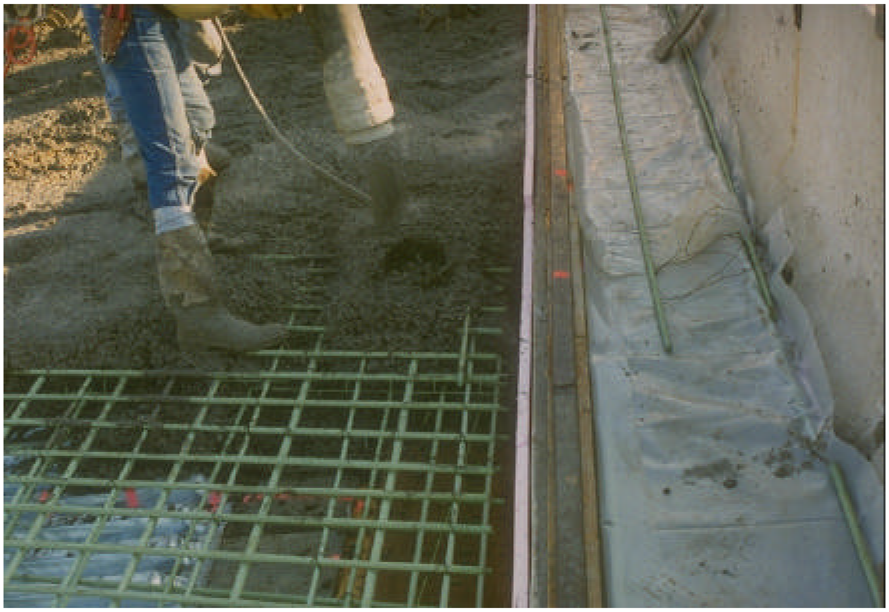


Figure 4.3 Concrete Pumping Operation over Test Bar

Figure 4.4 Cover Measurements Using James Instruments Rebar Datascan

Figure 4.5 Delaminations Detected and Marked after Chain Drag

Figure 4.6 Spall with Reinforcement Exposed

Figure 4.7 Rust Stain in Area of Cracking

Figure 4.8 Coring Machine used to Extract Cores from Bridge Deck

Figure 4.9 Core Taken for Chloride Analysis

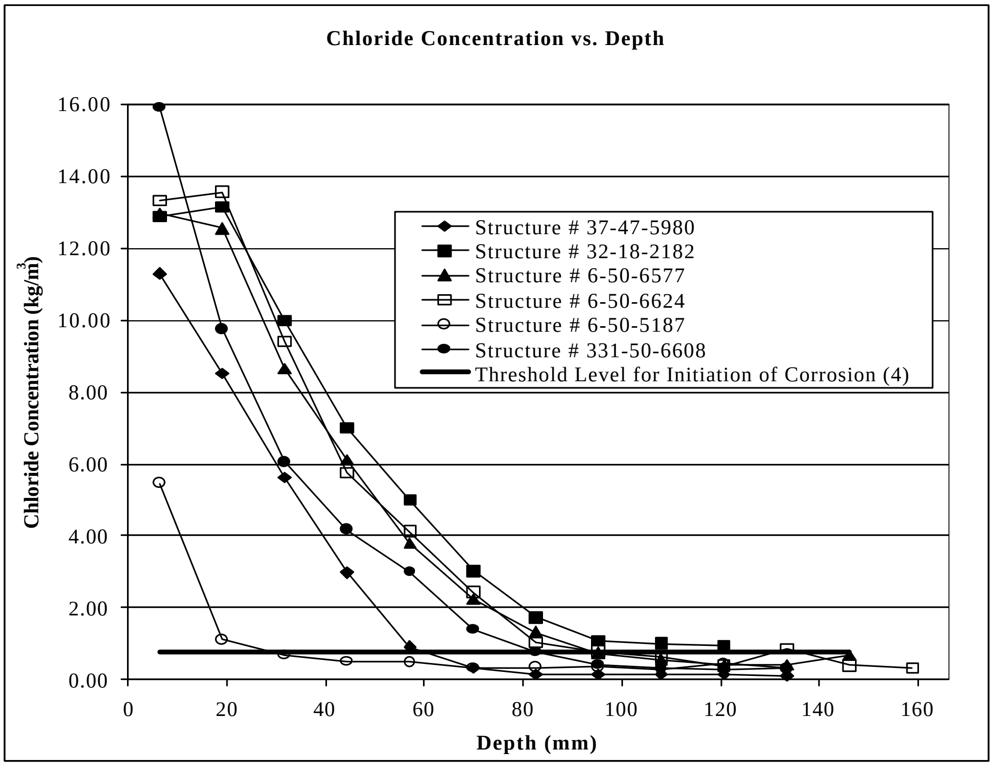


Figure 4.10 Chloride Concentration Profiles

Figure 4.11 Structure #37-47-5980

Figure 4.12 Cracking seen on Underside of Bridge Deck (Structure #37-47-5980)

Structure #37-47-5980 Northbound Lane
Black Rebar with Class A Concrete
Lawrence County, Vincennes District
Built 1973

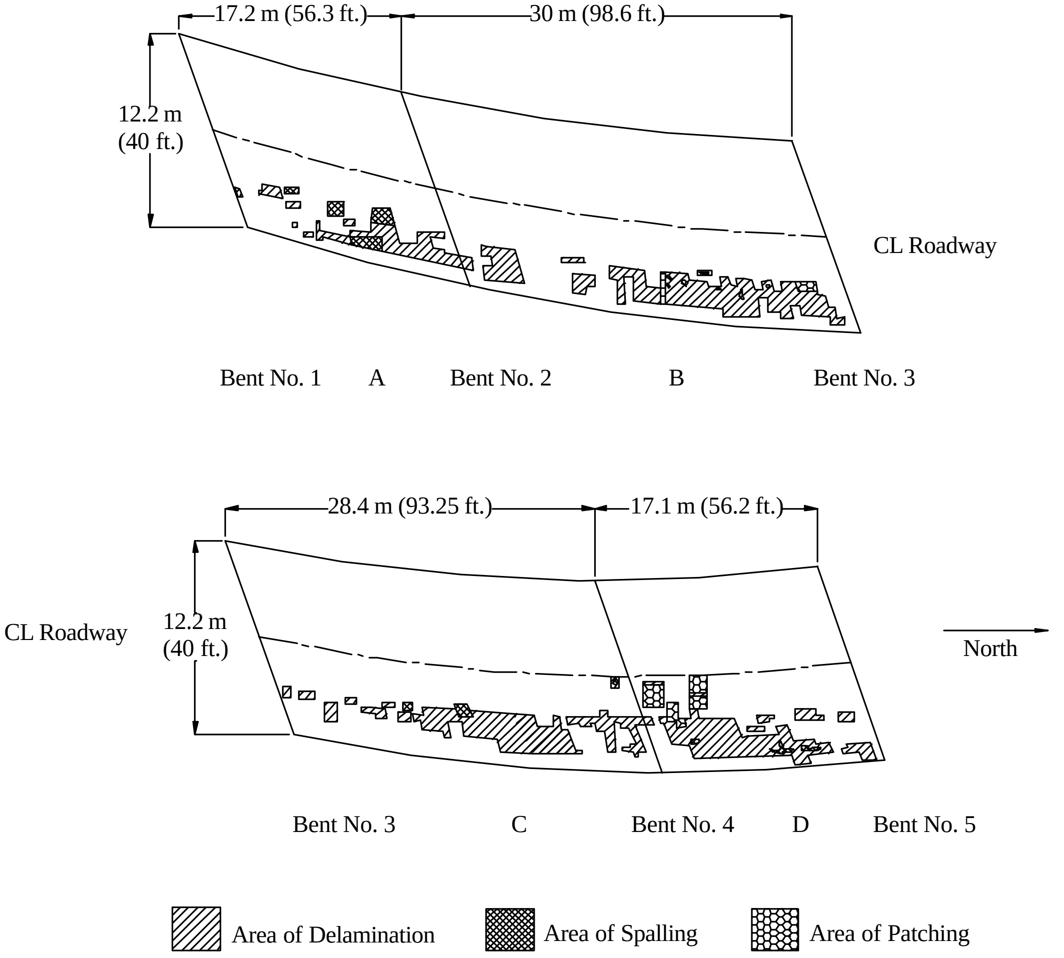


Figure 4.13 Delaminations, Spalls, and Patching Found on Structure #37-47-5980

Structure #37-47-5980 Northbound Lane
Black Rebar with Class A Concrete
Lawrence County, Vincennes District
Built 1973

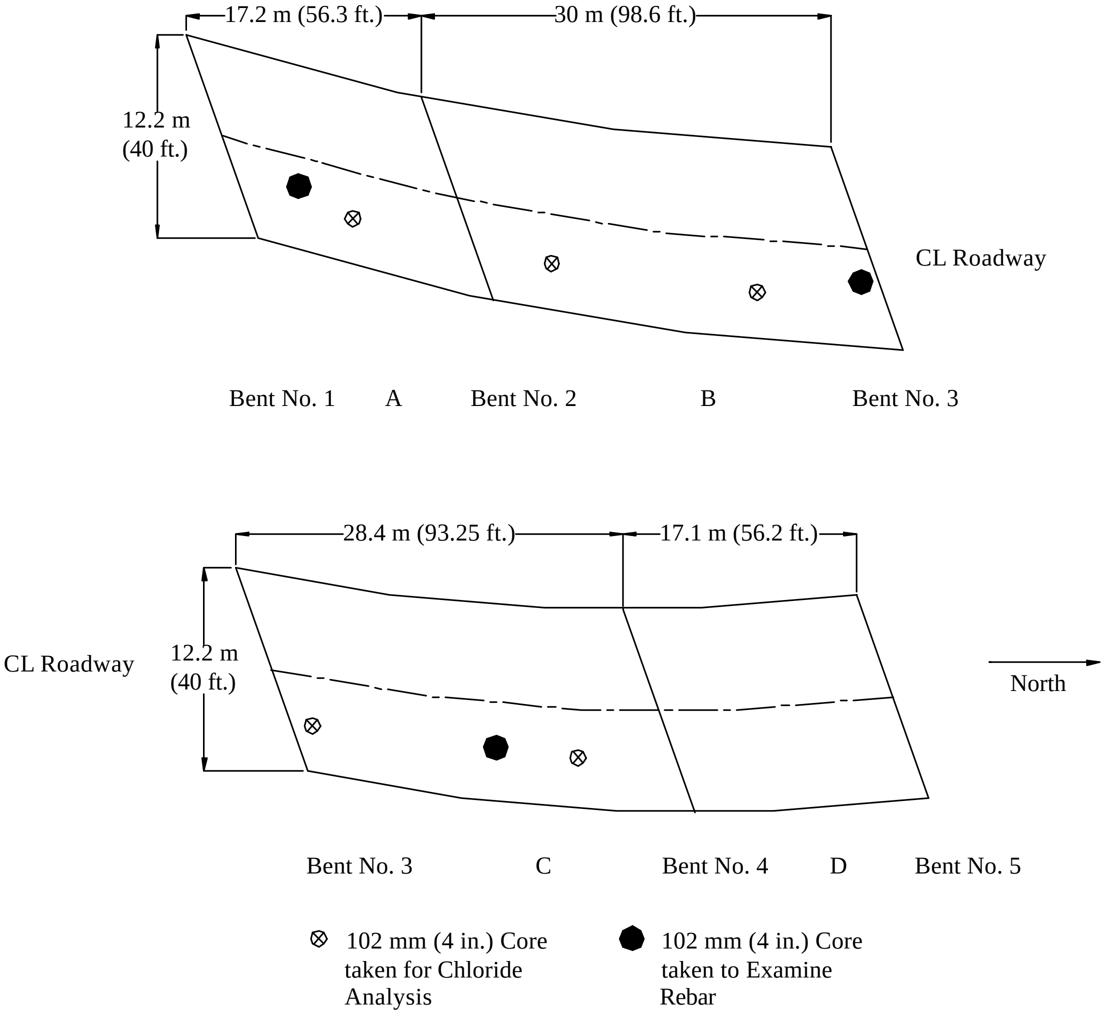


Figure 4.14 Location of Cores Taken From Structure #37-47-5980

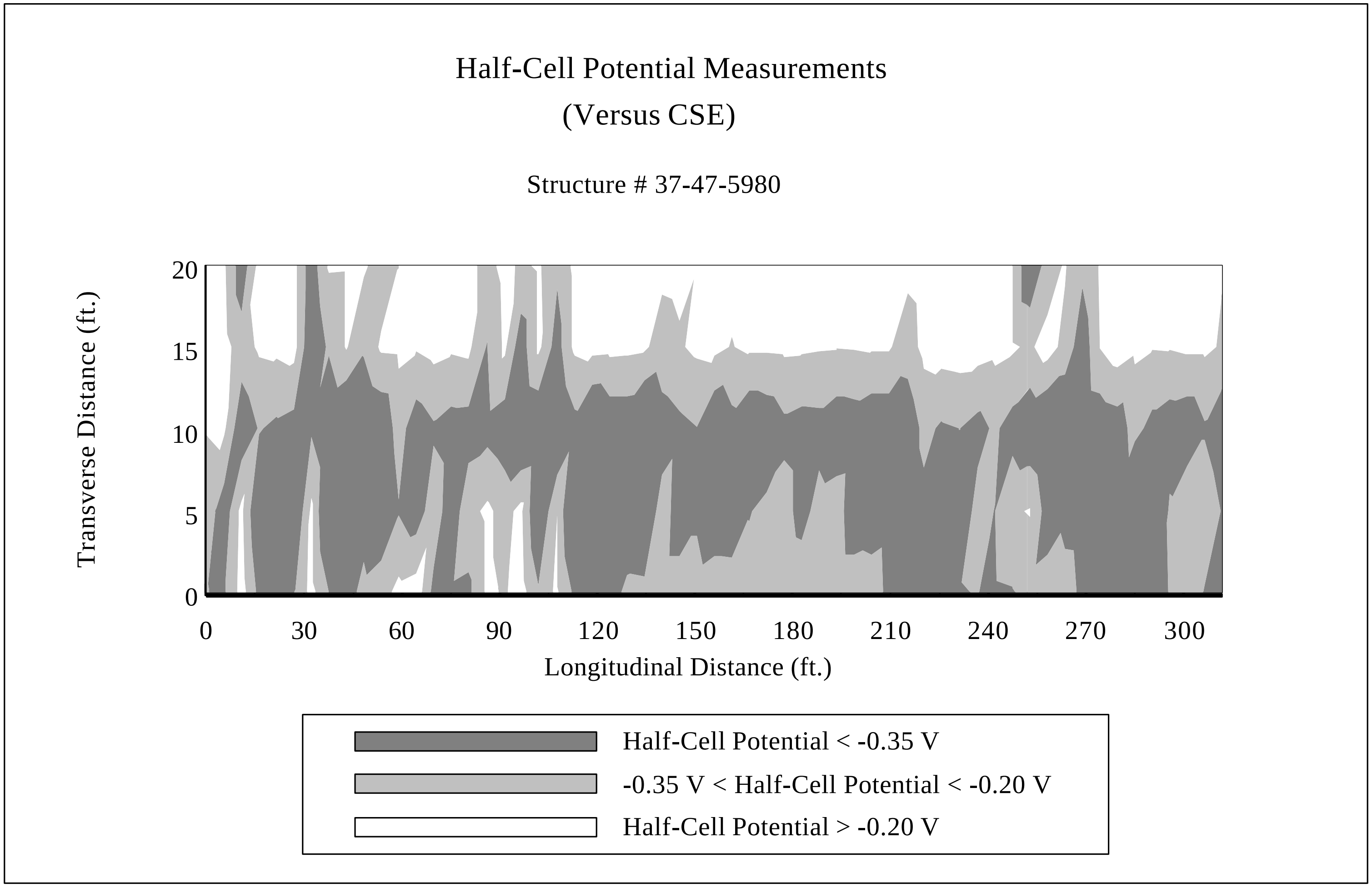


Figure 4.15 Half-Cell Measurements for Structure #37-47-5980

Figure 4.16 Core Taken in Cracked Region (Structure #37-47-5980)

Figure 4.17 Reinforcement from Core Taken in Sound Area of Deck (Str. #37-47-5980)

Figure 4.18 Reinforcement Removed in Delaminated Area of Deck (Str. #37-47-5980)

Figure 4.19 Structure #32-18-2182

Structure #32-18-2182 Northbound Lane
Black Rebar with Class C Concrete
Delaware County, Greenfield District
Built 1975

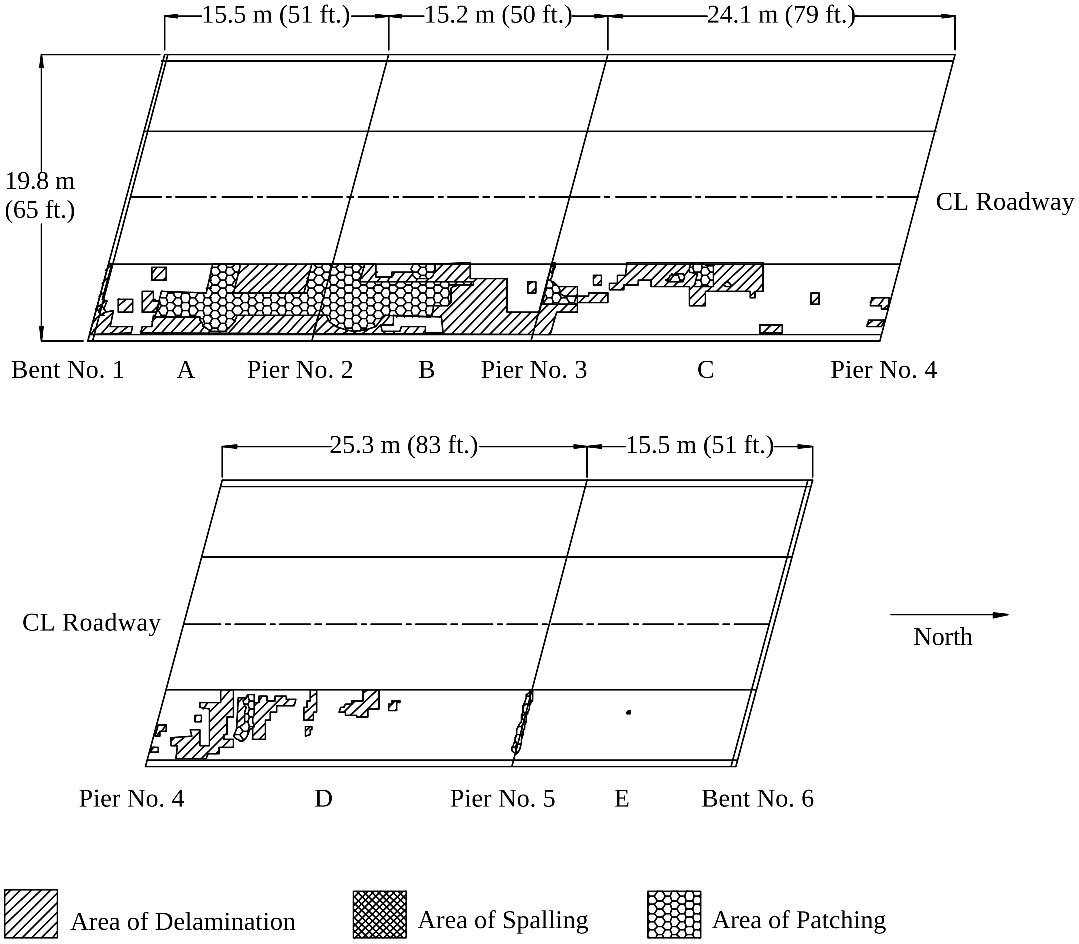


Figure 4.20 Delaminations, Spalls, and Patching Found on Structure #32-18-2182

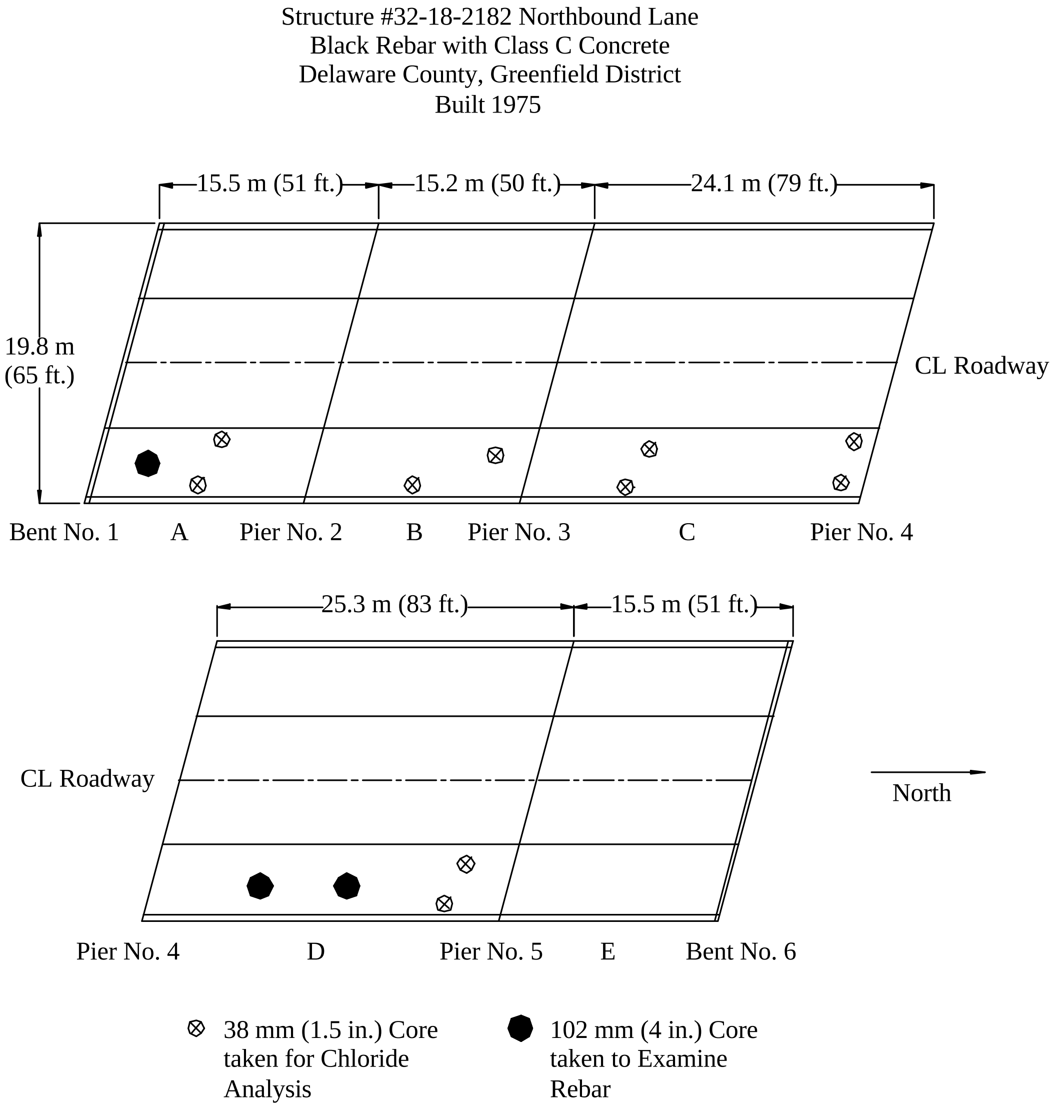


Figure 4.21 Location of Cores Taken From Structure #32-18-2182

Figure 4.22 Extensive Patching on Bridge Deck (Structure #32-18-2182)

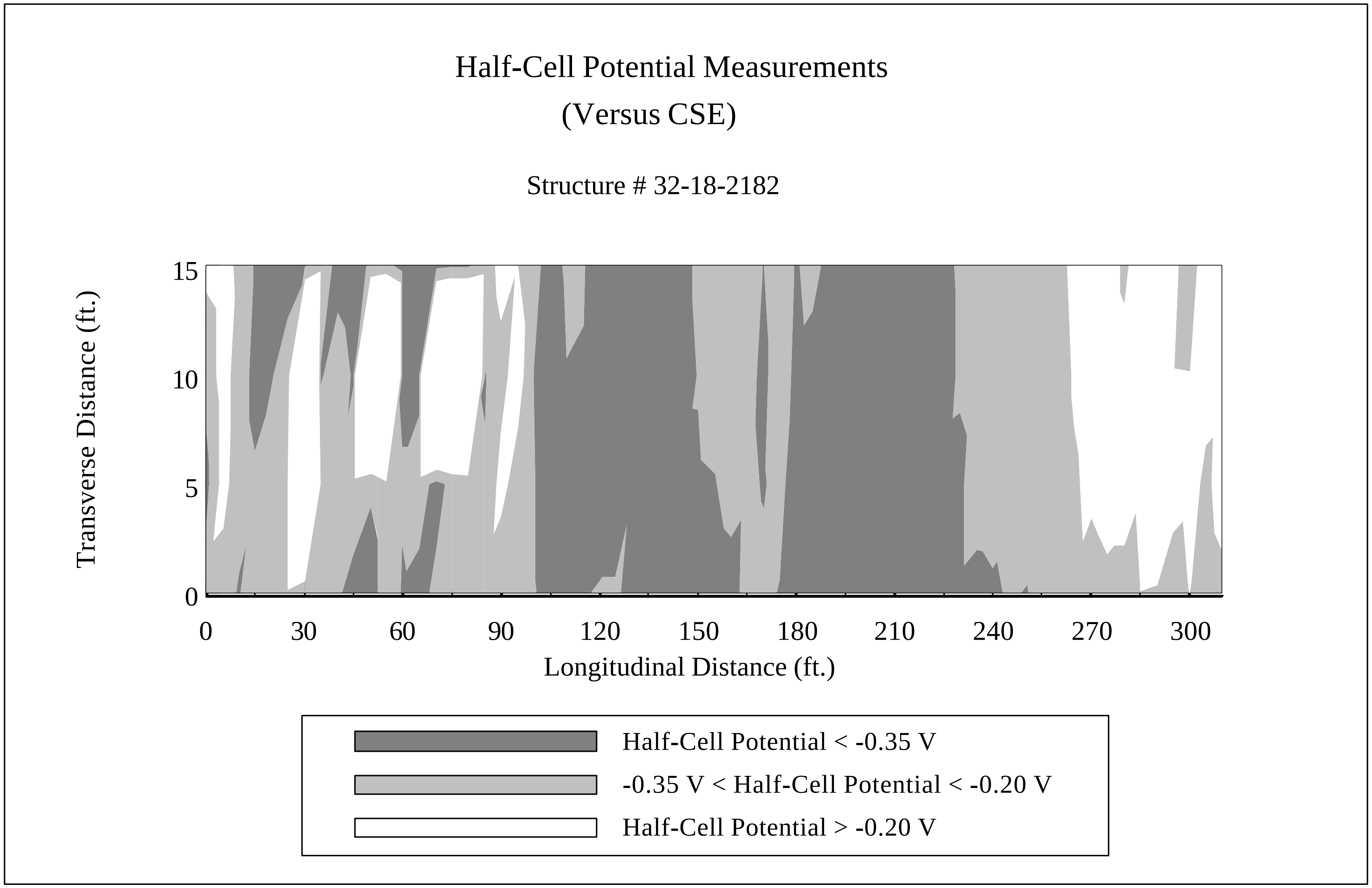


Figure 4.23 Half-Cell Measurements for Structure #32-18-2182

Figure 4.24 Delamination Present in the Bridge Deck (Structure #32-18-2182)

Figure 4.25 Corrosion Damage to Interior Prestressed Member (Structure #32-18-2182)

Figure 4.26 Damage to Exterior Prestressed I-Beam (Structure #32-18-2182)

Figure 4.27 Structure #6-50-5187

Structure #6-50-5187 Eastbound Lane
Black Rebar with Class C Concrete and 38 mm (1.5 in.) LMO
Marshall County, Laporte District
Built 1980

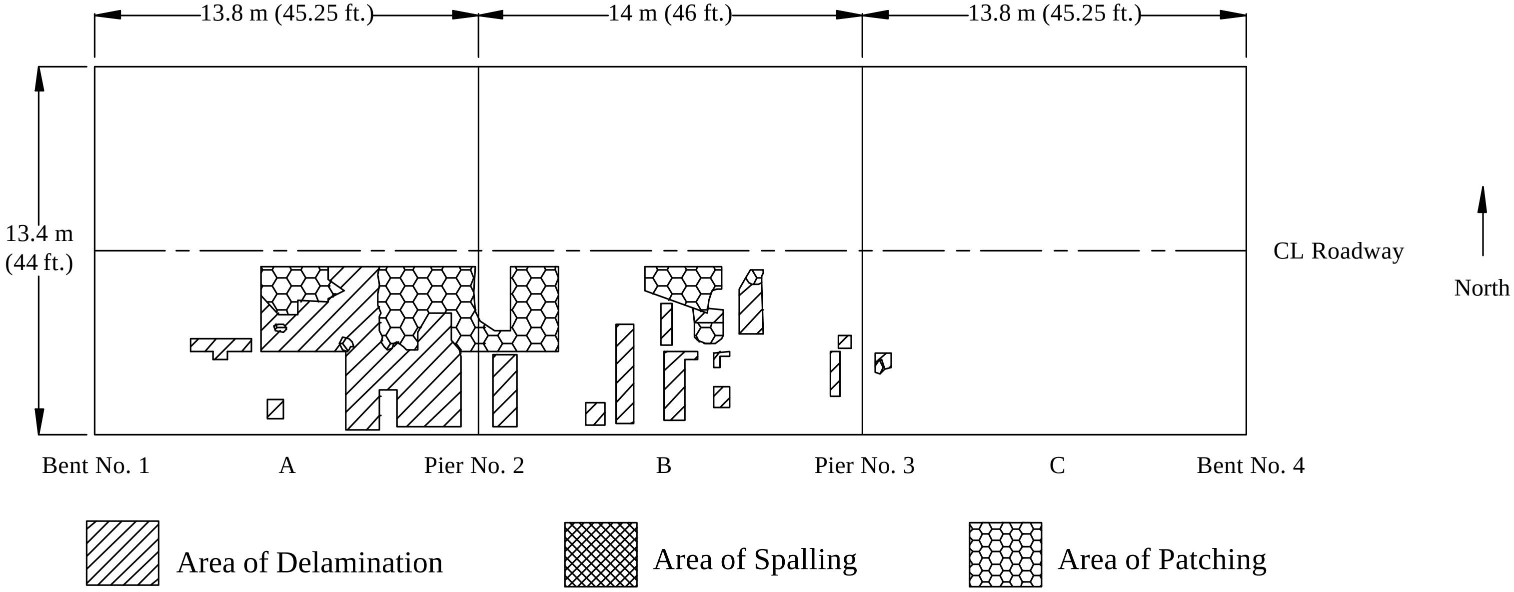


Figure 4.28 Delaminations, Spalls, and Patching Found on Structure #6-50-5187

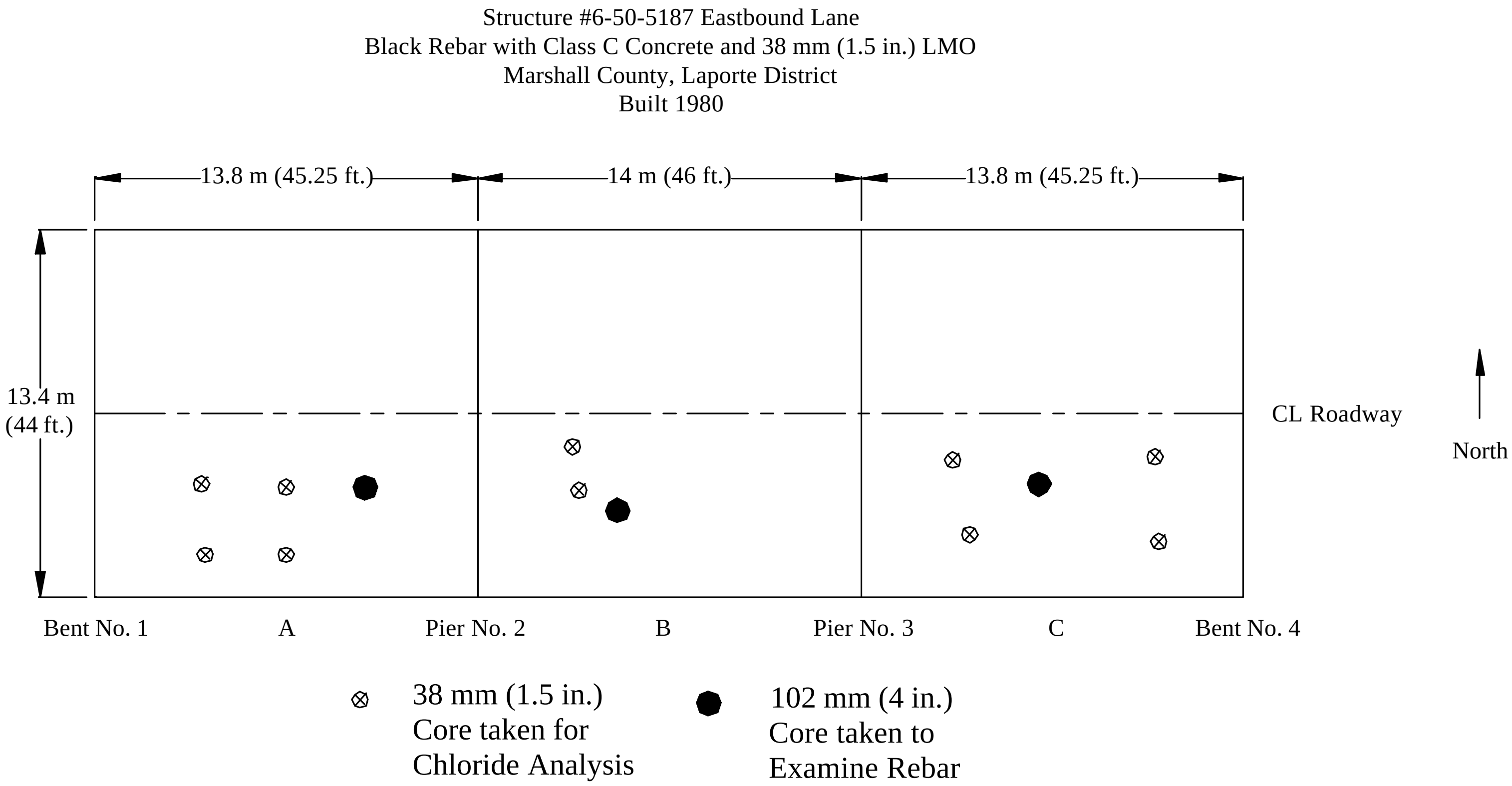


Figure 4.29 Location of Cores Taken From Structure #6-50-5187

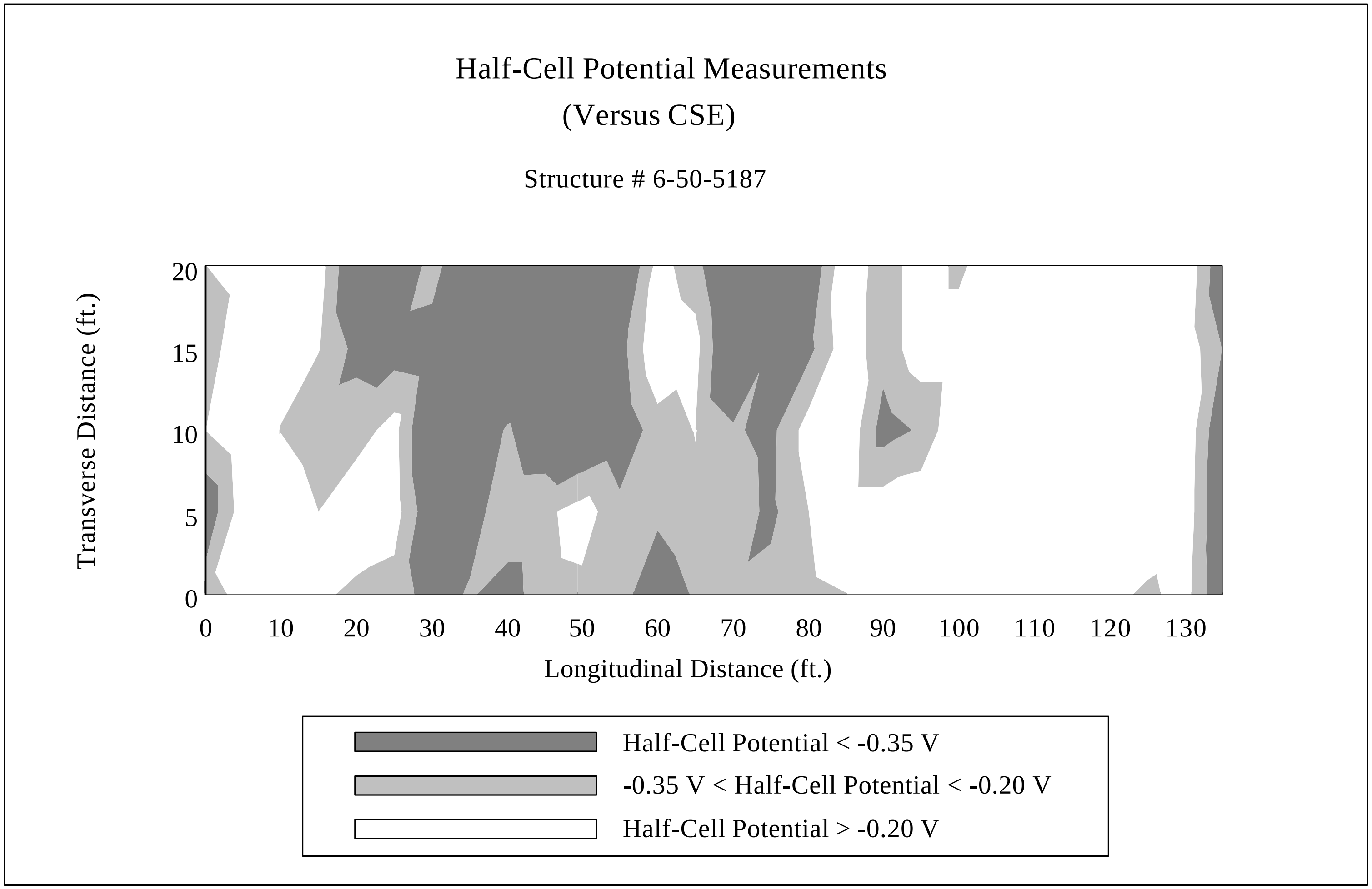


Figure 4.30 Half-Cell Measurements for Structure #6-50-5187

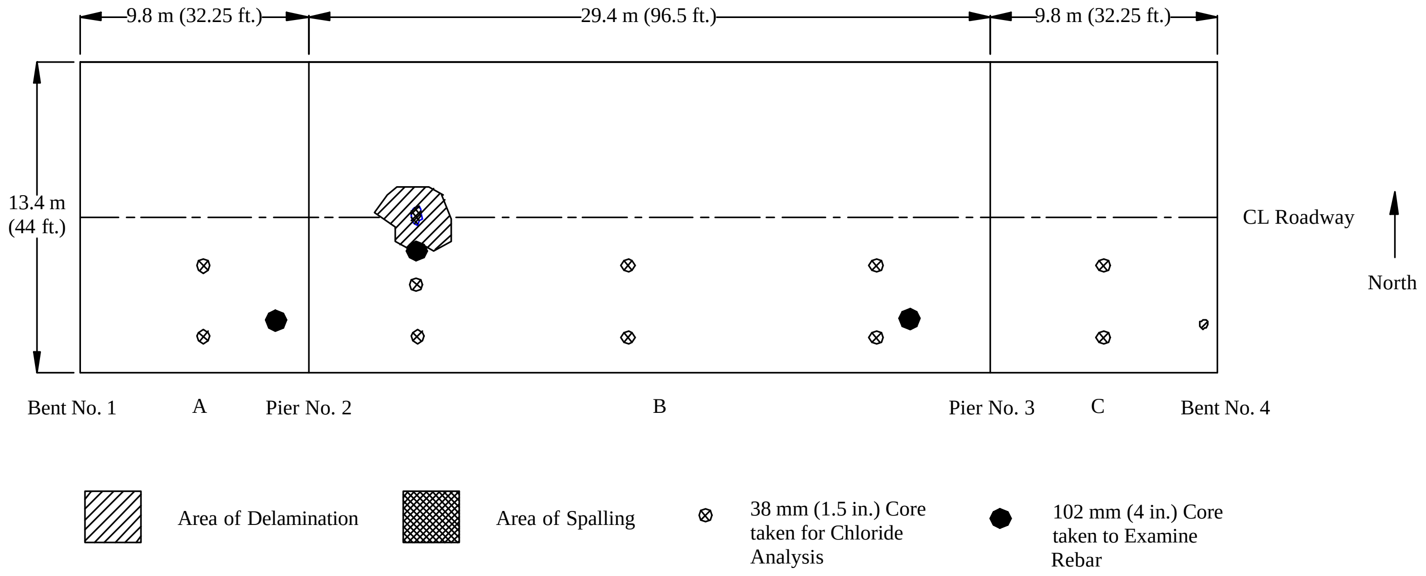
Figure 4.31 Core Taken in Cracked Area (Structure #6-50-5187)

Figure 4.32 Extensive Patching of Bridge Deck (Structure #6-50-5187)

Figure 4.33 Corrosion Damage to Underside of Deck (Structure #6-50-5187)

Figure 4.34 Concrete Spalling from Underside of Deck (Structure #6-50-5187)

Figure 4.35 Structure #6-50-6624



Structure #6-50-6624 Eastbound Lane
Epoxy-Coated Rebar with Class C Concrete
Marshall County, Laporte District
Built 1980

Figure 4.36 Delaminations, Spalls, and Location of Cores for Structure #6-50-6624

Figure 4.37 Delaminated Area with Spalls and Exposed Reinforcement
(Structure # 6-50-6624)

Figure 4.38 Exposed Rebar with Epoxy Coating Removed (Structure #6-50-6624)

Figure 4.39 Delamination from Corrosion of Rebar (Structure #6-50-6624)

Figure 4.40 Core Removed from Delaminated Area of Deck (Structure #6-50-6624)

Figure 4.41 Rebars Removed from Core Taken in Delaminated Area
(Structure #6-50-6624)

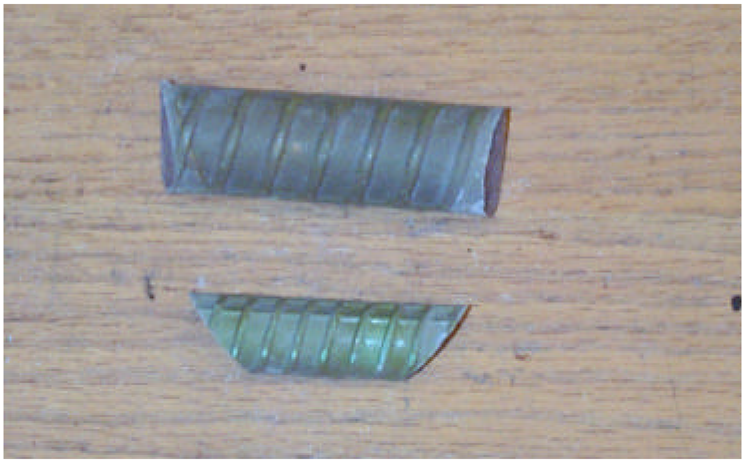


Figure 4.42 Discoloration of Rebar Removed from Sound Area of Deck
(Structure #6-50-6624)

Figure 4.43 Rebar with Coating Easily Removed with Knife (Structure #6-50-6624)

Figure 4.44 Structure #331-50-6608

Structure #331-50-6608 Northbound Lane
Epoxy-Coated Rebar with Class C Concrete
Marshall County, Laporte District
Built 1976

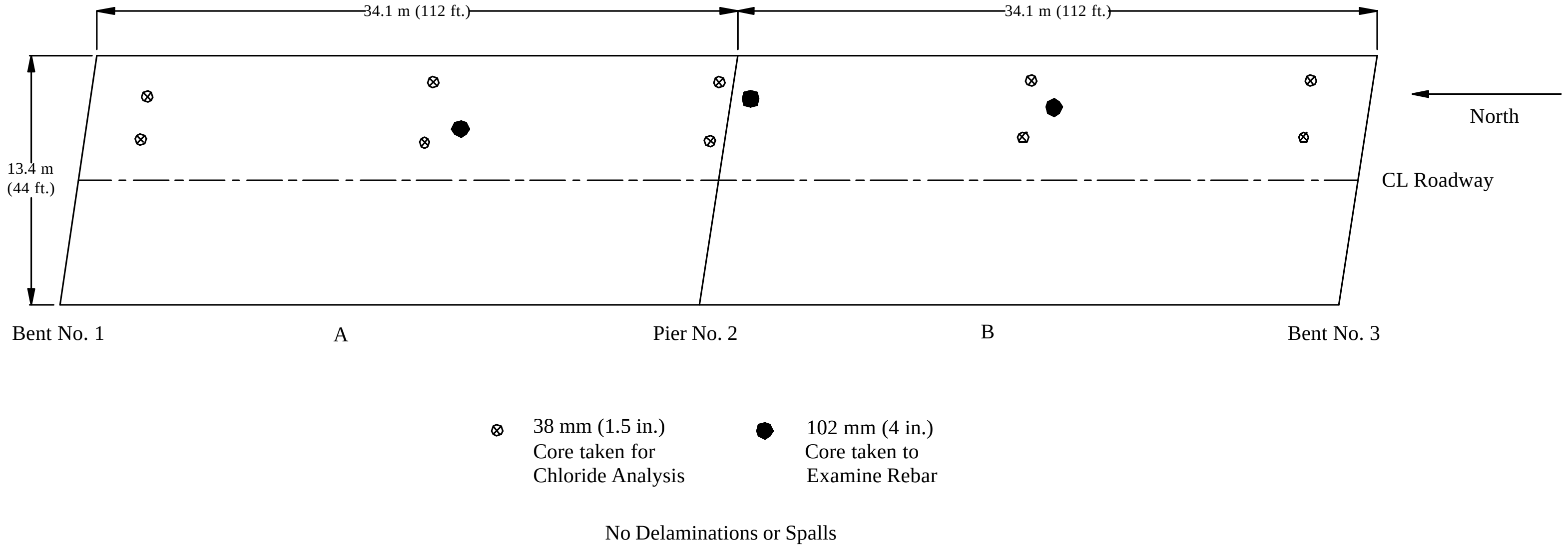


Figure 4.45 Location of Cores for Structure #331-50-6608

Figure 4.46 Core Taken in Cracked Location (Structure #331-50-6608)

Figure 4.47 Rebar Removed from Cracked Location in Deck (Structure #331-50-6608)

Figure 4.48 Alternate Side of Rebar Removed from Cracked Area
(Structure #331-50-6608)

Figure 4.49 Underfilm Corrosion Found after Removing Coating
(Structure #331-50-6608)

Figure 4.50 Rebars Removed from Sound Area of Deck (Structure #331-50-6608)

Figure 4.51 Structure #6-50-6577

Structure #6-50-6577 Eastbound Lane
Epoxy-Coated Rebar with Uncoated Bottom Mat and Class C Concrete
Marshall County, Laporte District
Built 1980

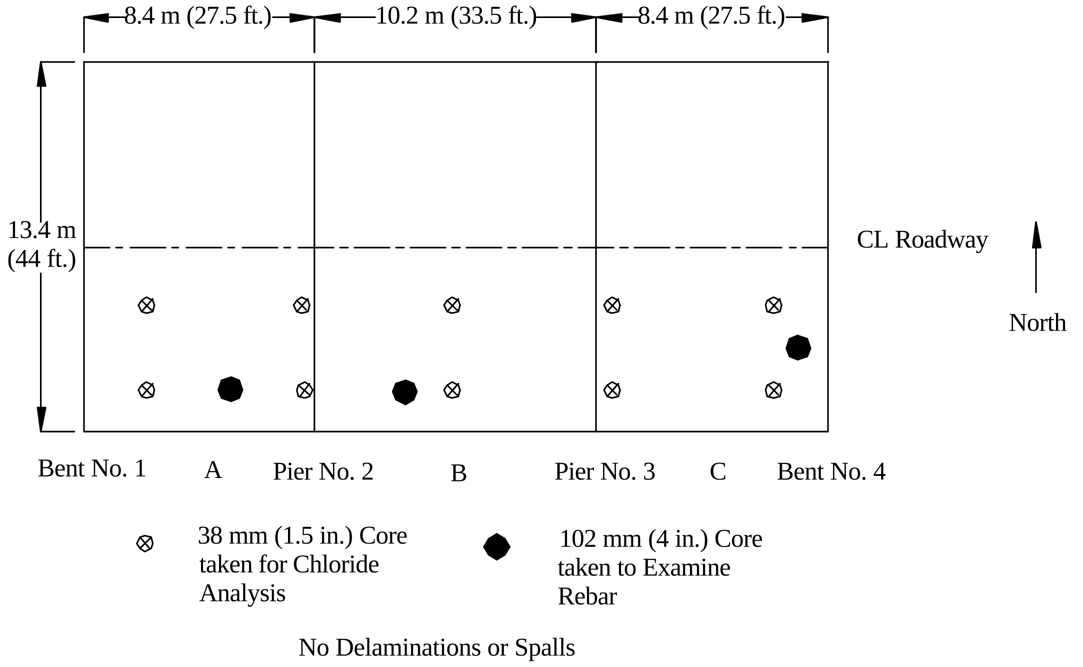


Figure 4.52 Location of Cores for Structure #6-50-6577

Figure 4.53 Concrete Spalling on Underside of Deck (Structure #6-50-6577)

CHAPTER 5 – LABORATORY EVALUATION

A laboratory phase was conducted to evaluate epoxy-coated reinforcement as a corrosion protection method. Concrete slab specimens reinforced with epoxy-coated reinforcement and companion specimens reinforced with uncoated reinforcement were subjected to exposure testing. The specific variables addressed in the laboratory phase were concrete cover, epoxy coating thickness, and defects in the epoxy coating. The objective of the laboratory evaluation was to determine the effect these test parameters have on the corrosion performance of epoxy-coated reinforcement. The expected outcome of the laboratory evaluation was to find that thicker concrete cover delays the onset of corrosion, and that a thicker epoxy coating with fewer defects provides superior corrosion performance. Results from the 77 weeks of exposure testing are provided in this chapter. Information gathered in the field evaluation concerning the amount of damage to epoxy-coated reinforcement in bridge construction operations was used in the design of the specimens. Specifically, the average maximum amount of damage to the epoxy-coated reinforcement was used as the maximum damage amount in the laboratory evaluation. This chapter discusses the test specimens, test procedure, results, and data analysis in detail. The results and analysis of the data included in this chapter are unique to the 77 week exposure period. No extrapolation to the performance of the specimens after 77 weeks is suggested or implied.

5.1 Test Specimens

The laboratory phase consists of exposure testing of concrete slab specimens reinforced entirely with epoxy-coated steel and companion specimens reinforced entirely with uncoated steel. Sixteen specimens were tested to evaluate the durability of epoxy-coated bars under conditions typically observed for Indiana bridge decks. The specific variables are:

- Concrete cover of 25.4 and 63.5 mm (1 and 2.5 inches)
- Lower and upper bounds of coating thickness specifications
- 19.7 and 39.4 defects per meter (6 and 12 defects per foot) of epoxy-coated upper mat steel

A summary of the specimens is provided in Table 5.1. The following notation is used to identify each specimen: “U” indicates uncoated reinforcement, while “E” indicates epoxy-coated reinforcement, the next number represents the thickness of concrete cover in inches, “6m or 12m” is used to identify the thickness of the epoxy coating in mils if any, “6d or 12d” is used to indicate either 6 defects per foot of top bar or 12 defects per foot of top bar, and (a) and (b) indicate duplicate specimens. All of the sixteen specimens contain 15.9 mm diameter (#5) reinforcing bars and class C concrete mix. The same concrete was used in all of the specimens to eliminate it as a variable in the program. The specimens are 355.6 mm (14 in.) in width, 203.2 mm (8 in.) in depth, and 1.07 m (3.5 ft.) in length as shown in Figure 5.1. Information obtained in the field evaluation of new construction bridge decks was used to determine the amount of damage to be created on the epoxy-coated reinforcement. The four bottom bars of the specimens reinforced with epoxy-coated steel have 9.8 defects per meter (3 defects per foot of bar) to represent the amount of damage incurred in the transportation and placement operations. An equal number of defects were placed on the top and the bottom surfaces of the reinforcement. The two top bars were prepared as the bottom bars but with all additional defects placed on the top surface of the reinforcement to simulate damage created during the concrete casting operation. The upper mat of steel consists of two bars spaced at 152.4 mm (6 in.) center to center, which is a typical spacing for bridge decks.

5.1.1 Concrete

Class C concrete mix was used for all sixteen specimens and the specimens were cast from the same batch of concrete. A local ready mix plant supplied the concrete and the mix design for the concrete is provided in Table 5.2. The concrete was cast indoors in the laboratory. The concrete was placed directly from the ready mix truck and mechanically vibrated. Cylinders and flexure beams were cast at the same time as the test specimens. The concrete had a slump of 127 mm (5 in.) and an air content of 5%. Covering with wet burlap and plastic provided moist curing of the specimens for seven days. The 28-day compressive strength of the concrete as determined from testing of the concrete cylinders was 41.4 MPa (6600 psi). One day after the cast, plastic settlement cracks were observed over the top reinforcement in all specimens with 25.4 mm (1 in.) of concrete cover, as visible in Figure 5.2.

5.1.2 Reinforcement

Midwest Pipe Coating, Inc. supplied the uncoated and epoxy-coated reinforcement. All reinforcement had a nominal yield strength of 413.7 MPa (60 ksi) and a spiral deformation pattern. The thickness of the epoxy coating requested was both 152.4 μm (6 mils) and 304.8 μm (12 mils). The reinforcement was ordered in lengths of 2.7 m (9 ft.) and cut to lengths of 1.2 m (4 ft.) in the laboratory. Screw holes were drilled and tapped into the reinforcement to allow for an electrical connection. The epoxy-coated reinforcement was tested for thickness, defects and holidays after being cut to size. Precautions were taken to prevent damage to the epoxy-coated during the fabrication process. All cut ends were patched with an epoxy-patching compound.

The thickness of the epoxy coating was measured with a Nordsen dry film gage as shown in Figure 5.3. Six measurements were taken along the bars on the top and bottom surfaces for a total of twelve measurements. Each measurement was the average of three individual readings taken in adjacent areas between the deformations. The coating thickness measurements are included in Table 5.3. For the 48 bars requested with a thickness of 152.4 μm (6 mils), the average thickness was found to be 193 μm (7.6 mils) with a standard deviation of 25.4 μm (1 mil). For the 24 bars requested with a thickness of 193 μm (12 mils), the average thickness was found to be 251.5 μm (9.9 mils) with a standard deviation of 15.2 μm (0.6 mil).

Holiday and defect testing of the bars after fabrication was performed as described in Section 4.2.1. Each bar was tested and the number of defects and holidays on each surface recorded. Table 5.4 provides the number of defects and holidays found for each bar along with the thickness of the epoxy coating. The findings of the holiday and defect testing of these bars agree with the findings from Chapter 4. As shown in Table 5.5, an increase in the thickness of the epoxy coating decreased the number of defects incurred. The two separate batches of thinner and thicker bars were transported and handled in exactly the same manner; however, the bars coated with an average thickness of 251.5 μm (9.9 mils) incurred many less defects then the bars coated with an average coating thickness of 193 μm (7.6 mils). This data shows that an increase of approximately 50.8 μm (2 mils) in coating thickness decreased the defects incurred in the transportation and fabrication process by 85%.

5.1.3 Fabrication and Instrumentation

The reinforcement for the specimens was cut to 1.2 m (4 ft.) in length. The reinforcement extended beyond the forms on both ends 76.2 mm (3 in.) to allow for direct electrical connection. All defects on the epoxy-coated reinforcement were placed on the inner 0.9 m (3 ft.) section of reinforcement. The defects already present on the reinforcement were not patched and were included in the total number of defects. The largest naturally occurring defect on the reinforcement was approximately 1 mm by 5 mm (0.04 by 0.20 in.) in size. Some defects were created to provide the required number of defects for each bar. All defects created on the bars were fabricated using a small utility knife. The defects were circular in shape and approximately 1 mm (0.04 in.) in diameter. The location of each defect was recorded and marked on the reinforcement.

The sides of the concrete specimens were coated with an epoxy paint to simulate one dimensional moisture flow. The reinforcement extending from the specimens was epoxy-coated with an epoxy-patching compound to protect from any exterior corrosion.

An area for ponding salt water was constructed on the top surface of each specimen. Figure 5.4 shows one of the durability specimens. Each bar within every mat of reinforcement was electrically connected to provide electrical continuity between the bars, as shown in Figure 5.5. For each specimen a wire was connected to each mat of reinforcement. The wire from the top mat was then connected to a red plug, while the wire from the bottom mat was connected to a black plug. Separate wires from each of these plugs were connected to a switch and a resistor. The switch allows the circuit to be completed with the resistor. With the switch in the "on" position the resistor is connected, with the switch "off" the resistor is disconnected and the circuit is broken. The resistor and connections are enclosed within boxes for protection, as shown in Figure 5.6. The resistor chosen for each specimen depended on the AC mat to mat resistance of the specimen. A resistor less than 1/10 of the initial mat to mat resistance was chosen to prevent interference with the corrosion current. Table 5.6 provides the initial mat to mat resistance of the specimens and the actual resistor reading for each specimen.

The set up for the specimens also included halogen lights for the drying cycle. Eight wooden frames were constructed to hang the lamps over the concrete specimens to provide adequate heat. Two 500 watt lamps for each specimen provided heat to warm the surface of the concrete specimens to 48.9 °C (120 °F). The switch boxes containing the resistor and connections were also attached to the wooden frame. The specimens were placed in pans and on blocks to elevate the specimens to ensure the bottom surface was exposed to air. The frame setup with lamps is shown in Figure 5.7.

5.2 Test Procedure

For 77 weeks the specimens were exposed to a “Southern Exposure” cycle[22] in which a 15% sodium chloride solution was used for ponding. A weekly test cycle was performed as follows:

- Pond saltwater on surface of the specimens to a depth of about 25.4 mm (1 in.).
- After four days, remove saltwater that remains. Rinse the surface with fresh water.
- Leave the specimens moist for one day.
- Dry the surface of the specimen using a heat source for the two remaining days of the weekly cycle.

Corrosion current, AC electrical resistance between the top and bottom mats of reinforcement, and half-cell potentials were measured weekly. The weekly testing was performed at the end of the drying phase before saltwater was applied. Powder samples from the specimens were extracted to detect chloride concentrations at 12 weeks, 24 weeks, 36 weeks, and 61 weeks. The following sections will describe each test and its procedure.

5.2.1 Corrosion Current

Electrical connections to both the top and bottom mat of reinforcement allowed for the determination of the corrosion current between the mats of reinforcement. The specimens simulated the “worst case” in which a concrete bridge deck has electrical continuity between the top and bottom mat of reinforcement. Electrical continuity is always present in bridge decks containing uncoated reinforcement and some studies have shown that it is not uncommon to find continuity in bridge decks containing epoxy-coated reinforcement. The top and bottom mats of reinforcement were connected in a circuit with a resistor for the entire weekly cycle. This allowed for the corrosion current flowing between the mats of reinforcement to stabilize. The circuit was only disconnected for the AC resistance and half-cell potential measurements. With the resistor connected, a multimeter was used to measure the voltage drop across the resistor as pictured in Figure 5.8. The measured voltage was converted to current by Ohm's Law, V = IR. The corrosion current determined by this test is the macrocell corrosion occurring between the top and bottom mats of reinforcement. Any microcell activity occurring within the same mat of reinforcement cannot be determined by this method.

5.2.2 AC Mat to Mat Resistance

The AC mat to mat resistance was measured using a Nilsson soil resistance meter as pictured in Figure 5.9. The weekly testing was performed with the circuit disconnected. The resistance between the top and bottom mat of reinforcement is a function not only of the concrete but also the reinforcement. Permeability, moisture content, and chloride ion concentration of the concrete all effect the mat to mat resistance of the concrete. Epoxy coating on the reinforcement also increases the mat to mat resistance by electrically insulating the reinforcement. Greater distance between the mats of reinforcement will provide a higher resistance. A high mat to mat resistance will impede corrosion current flow between the top and bottom mats of reinforcement.

5.2.3 Half-Cell Potentials

Half-cell potentials for the specimens containing uncoated reinforcement were measured in the laboratory using a similar procedure to the field evaluation. Measurements were taken with the circuit disconnected and the top mat of reinforcement connected to the multimeter. Three readings were taken per specimen, spaced evenly over the length of the specimen. Saturated sponges were placed on each area to be measured to moisten the concrete. A high internal resistance multimeter was used to measure the potential readings. The setup for taking half-cell readings is pictured in Figure 5.10. Differences in potential along the length of the specimen can indicate microcell activity. In the case of uncoated reinforcement, areas with voltage readings less than –0.35 volts are anodic and areas with voltage readings greater than –0.20 volts are cathodic.

5.2.4 Powder Samples

Concrete powder samples were taken at 12, 24, 36, and 61 weeks to determine chloride contamination. Samples from flexure beams not exposed to chlorides, but cast from the same batch of concrete, were also taken to determine the baseline chloride concentration in the concrete. Rotary hammer drills were necessary for this procedure. Two specimens were sampled at each time period. The procedure involved drilling 12.7 mm (half-inch) increments and extracting the powder for analysis. Precautions were taken to minimize the contamination of each specimen with powder from previous levels. The Materials and Tests Division of the Indiana Department of Transportation analyzed the powder samples using the AASHTO Standard Test T260 method. This method determines the total chloride ion concentration of the concrete powder sample. The specimens were patched after drilling with “Set 45” concrete patching compound.

5.3 Results of the Laboratory Evaluation

Results of corrosion current, AC mat to mat resistance, half-cell potentials, and powder samples for the 77 week exposure testing period are presented in this section. Visual observations of the specimens during this period and a forensic examination of the specimens are also included. Half of the sixteen specimens were autopsied at week 64 for a forensic examination. Exposure testing of the remaining eight specimens was discontinued at week 77 and a forensic examination of the specimens was conducted.

5.3.1 Corrosion Current Results

Results from the measurement of corrosion current are presented in Figures 5.11 and 5.12. More negative current indicates more corrosion activity, leading to the consumption of more metal. Figure 5.11 provides the results for all of the durability specimens. All four of the specimens containing uncoated reinforcement show active corrosion. Corrosion current flow for the specimens containing uncoated reinforcement with 25.4 mm (1.0 in.) cover began immediately. Corrosion of the specimens containing uncoated specimens with 63.5 mm (2.5 in.) cover did not initiate until approximately the 25 week of exposure. The magnitude of current flow is also different for the specimens with different concrete cover. The specimens with only 25.4 mm (1 in.) of cover had a dramatic jump in current flow within the first 5 weeks of exposure. After 5 weeks, the corrosion rate slowed and the current leveled off around 5000 μA with a gradual increase following. However, the specimens with 63.5 mm (2.5 in.) of cover began a gradual increase in corrosion current around the 25th week and are still showing a gradual increase.

Figure 5.11 also illustrates the drastic difference in performance of the uncoated reinforcement when compared with the epoxy-coated reinforcement. None of the specimens containing epoxy-coated reinforcement showed corrosion activity close to the magnitude of current observed in the specimens with uncoated reinforcement. Figure 5.12 shows the corrosion current results for only the specimens containing epoxy-coated reinforcement. As shown, active corrosion has occurred in four of the specimens. However, the largest current is still approximately fifty times lower then the current observed in the specimens with uncoated reinforcement. The corrosion activity for these four specimens began between the 20th to 25th week of exposure. Three of the four specimens with corrosion current activity have reinforcement with 39.4 defects per meter (12 defects per foot), and 152.4 μm (6 mils) of coating thickness. The fourth specimen contains reinforcement with 19.7 defects per meter (6 defects per foot), and 152.4 μm (6 mils) of coating thickness. Three additional specimens containing epoxy-coated reinforcement also showed minor corrosion activity towards the end of the exposure period.

Both of these figures show a spike in the corrosion current at week 38. This deviation resulted from a variation in the weekly cycle. During week 38, the heat lamps were not turned on during the drying cycle to investigate the effect of temperature and drying on the corrosion current measurements. The surface temperature at which all the readings were taken for week 38 was 22.2 °C (72 °F). This variation in the weekly cycle lowered the temperature and allowed more moisture to remain in the concrete. The lower temperature decreased the corrosion current considerably in specimens containing both uncoated and epoxy-coated reinforcement. The difference in moisture content apparently did not affect the current significantly, as an increase in current would have been expected with an increase in moisture content.

5.3.2 AC Mat to Mat Resistance Results

Results from the measurements of AC mat to mat resistance are provided in Figures 5.13 through 5.15. Table 5.6 provides the initial AC mat to mat resistance of all the specimens before the exposure testing began. The specimens containing uncoated reinforcement had resistance readings two orders of magnitude lower than the specimens containing epoxy-coated reinforcement. The specimens with 25.4 mm (1 in.) of cover had resistance readings higher than the specimens with 63.5 mm (2.5 in.) of concrete cover. The additional cover with the same specimen depth resulted in the two mats of reinforcement being closer together, which accounts for the lower resistance. Also, the specimens with more damage to the coating had lower resistance readings. There was no difference in resistance readings due to thickness of coating. This suggests that the thinner coatings provide adequate electrical insulation.

Figure 5.13 provides the results from the specimens containing uncoated reinforcement over the 77-week exposure period. The graph shows the resistance readings initially increased slightly at the beginning of the exposure testing period and then decreased towards the end of the exposure period.. The specimens with 63.5 mm (2.5 in.) of concrete cover have resistance readings only slightly greater than the specimens with 25.4 mm (1.0 in.) of cover. Figure 5.14 provides the resistance readings for the specimens with epoxy-coated reinforcement and 63.5 mm (2.5 in.) of concrete cover. Specimens E2.5-6m-6d(a), E2.5-6m-12d(a), and E2.5-6m-12d(b) all have resistance readings that decreased with time. The remaining specimen, E2.5-6m-6d(b) showed a slight increase in resistance over the exposure period. Figure 5.15 provides the test results from the specimens containing epoxy-coated reinforcement and 25.4 mm (1 in.) of concrete cover. Most of these specimens showed resistance readings that either increased or remained constant throughout the exposure period. Specimens E1.0-6m-12d(a), E1.0-6m-12d(b), and E1.0-6m-6d(b) had a slight decrease in resistance over the exposure period. Specimen E1.0-6m-6d(a) had erratic readings after week 40 of unknown cause and Specimen E1.0-12m-6d(b) had a sharp increase followed by a sharp decrease in resistance readings during the 62nd to 65th week, also of unknown cause. All specimens shown in these three figures had an increase in resistance at week 38 when the readings were taken at a decreased temperature of 22.2 °C (72 °F).

5.3.3 Half-Cell Potential Results

Results for the half-cell potential measurements are provided in Figure 5.16. The average of the three reading taken for each specimen is presented in the graphs. No significant difference in half-cell potential over the length of the specimens was observed, indicating little if any microcell activity in the top mat of reinforcement. Figure 5.16 provides the test results for the specimens containing uncoated reinforcement. The half-cell measurements for these specimens correlate very well with the corrosion current measurements. The graph indicates that corrosion activity occurred immediately for the specimens with only 25.4 mm (1 in.) of concrete cover. However, the onset of corrosion for the specimens with 63.5 mm (2.5 in.) of cover is delayed until the 25th week. Both of these results agree with the corrosion current results presented previously.

5.3.4 Powder Sample Results

Results from the powder samples for chloride analysis are provided in Tables 5.7 through 5.12. The baseline chloride concentration obtained from concrete not exposed to chlorides is provided in Table 5.7. Both test holes drilled for analysis are presented in the table. Surprisingly, the majority of the chloride levels found are slightly greater than the threshold level for initiation of corrosion, 0.79 kg/m^3 (1.3 lb/yd^3). The individual results from the test holes at weeks 12, 24, 36, and 61 are provided in Tables 5.8 through 5.11. A summary of all the average chloride concentration found at each depth for each sampling period is provided in Table 5.12. These results are shown graphically in Figure 5.17. The graph shows that by week 12, the chloride concentration was well in excess of the threshold level for initiation of corrosion at the depth of 25.4 mm (1 in.). By 24 weeks the chloride concentration was well in excess of the threshold level at the depth of 63.5 mm (2.5 in.).

5.3.5 Visual and Forensic Inspection Results

Rust stains appeared on the top of the uncoated specimens with 25.4 mm (1 in.) of cover at week 10. The staining was localized around the longitudinal settlement cracks and is shown in Figure 5.18. These pre-existing longitudinal cracks widened as the corrosion progressed. Rust stains and cracking also appeared on the top of the uncoated specimens with 63.5 mm (2.5 in.) of cover at week 55. Rust staining on the surface of the concrete was not observed on any of the other specimens over the exposure period. Light scaling occurred on some of the specimens, most likely from the process of removing the water from the specimens with a vacuum. Delaminations were not detected on any of the specimens.

At the end of 64 weeks of exposure testing a forensic investigation of the eight duplicate specimens was performed. The reinforcement from the following eight specimens was removed for examination: U2.5(b), U1.0(b), E2.5-6m-6d(a), E1.0-6m-6d(a), E1.0-12m-6d(a), E1.0-6m-12d(a), E2.5-6m-12d(a), and E1.0-12m-12d(a). Figure 5.19 shows the removal of the reinforcement from the specimens. The inspection of the reinforcement found corrosion only on the bars from the top mat of reinforcement. No corrosion was observed on any of the bars from the bottom mats of reinforcement. This finding along with the absence of pitting in any of the specimens confirms that macrocell corrosion was the primary form of corrosion occurring in the specimens.

The specimens containing uncoated reinforcement showed uniform corrosion of the top bars. Figure 5.20 shows the top bars from specimen U1.0(b). Uniform corrosion over the entire length of the bar with minor loss of cross-sectional area had occurred. As shown in Figure 5.21 none of the bottom bars of the specimens had any corrosion. The top bars from specimen U2.5(b) are shown in Figure 5.22. The uniform corrosion of these bars was not as extensive as the bars extracted from specimen U1.0(b) and the corrosion was limited to the inner two feet of the length of the bar. No loss of cross-sectional area was observed in these bars.

Inspection of the top bars removed from the specimens containing epoxy-coated reinforcement found only minor corrosion with no loss in cross-sectional area. In all cases the epoxy coating was well adhered and difficult to remove in areas away from rusting. No corrosion was found under the coating in areas away from the rusted areas. Figures 5.23 through 5.28 show the condition of the bars removed from the specimens containing epoxy-coated bars. The findings from each specimen are presented below.

- Specimen E2.5-6m-6d(a) – Minor rusting around two defects was found on one of the top bars. Light rusting along a 152.4 mm (6 in.) length of bar was found on the second top bar.
- Specimen E1.0-6m-6d(a) – No corrosion was found on one of the top bars, light rusting along a 203.2 mm (8 in.) length of bar was found on the second bar.
- Specimen E1.0-12m-6d(a) – Very minor rusting found around one defect on one of the top bars, and one small area of rusting along a 76.2 mm (3 in.) length of bar was found on the second bar.
- Specimen E1.0-6m-12d(a) – Both of the top bars were moderately rusted along a 203.2 mm (8 in.) length of bar.
- Specimen E2.5-6m-12d(b) – No evidence of corrosion was found on one of the top bars. The second top bar had three separate areas of rusting along a 76.2 mm (3 in.) length of bar.
- Specimen 1.0-12m-12d(a) – Both bars had light rusting along a 50.8 mm (2 in.) length of bar.

At the end of the 77-week exposure period a forensic investigation of the remaining eight specimens was performed. The reinforcement from the following eight specimens was removed for examination: U2.5(a), U1.0(a), E2.5-6m-6d(b), E1.0-6m-6d(b), E1.0-12m-6d(b), E1.0-6m-12d(b), E2.5-6m-12d(b), and E1.0-12m-12d(b). As was found at week 64, the inspection of the reinforcement found corrosion only on the bars from the top mat of reinforcement. No corrosion was observed on any of the bars from the bottom mats of reinforcement.

The specimens containing uncoated reinforcement showed uniform corrosion of the top bars. Figure 5.29 shows the top bars from specimen U1.0(a). Uniform corrosion over the entire length of the bar with small loss of cross-sectional area was observed. The top bars from specimen U2.5(a) are shown in Figure 5.30. Light corrosion along the length of one of the top bars was observed. The second bar had minor rusting along a 152.4 mm (6 in.) length of bar. No loss of cross-sectional area was observed in these bars.

Inspection of the top bars removed from the specimens containing epoxy-coated reinforcement found only minor corrosion with no loss in cross-sectional area. In all cases the epoxy coating was well adhered and difficult to remove in areas away from rusting. No corrosion was found under the coating in areas away from the rusted areas. Figures 5.31 through 5.36 show the condition of the bars removed from the specimens containing epoxy-coated bars. The findings from each specimen are presented below.

- Specimen E2.5-6m-6d(b) – No evidence of corrosion on one of the top bars. Moderate rusting of the second bar was observed along a 304.8 mm (12 in.) length.

- Specimen E1.0-6m-6d(b) – Moderate to light rusting along most of the length of one bar was observed. Only light rusting along a 50.8 mm (2 in.) length was found on the second bar.
- Specimen E1.0-12m-6d(b) – Light rusting around three defects of one bar and minor rusting around one defect of the second bar was found.
- Specimen E1.0-6m-12d(b) – Moderate rusting along a 304.8 mm (12 in.) length of bar was found for one of the bars. The second bar had minor rusting around three defects.
- Specimen E2.5-6m-12d(a) – Very minor rusting around three defects of one bar was observed. Light rusting of the second bar along a 508 mm (20 in.) length was found.
- Specimen 1.0-12m-12d(b) – Light rusting around six defects of one bar was found. Minor rusting along a 76.2 mm (3 in.) length and around one defect of the second bar was observed.

5.4 Analysis of Data

Data from the results of the corrosion current measurements and the AC mat to mat resistance measurements warranted further study. A combined analysis of both of these results was performed in order to investigate the effect of the test variables on corrosion activity.

5.4.1 Relationship Between Corrosion Current and Resistance

As discussed previously, a high resistance between the two mats of reinforcement will impede corrosion current. The data from the corrosion current measurements and the AC mat to mat resistance illustrate this principle. Figure 5.37 provides a graph showing corrosion current plotted versus resistance. This figure illustrates two findings. First, very low mat to mat resistance values will provide no opposition to corrosion current, as illustrated in Figure 5.37 by the resistance readings close to zero. These readings correspond to those from the specimens containing uncoated reinforcement. The corrosion current for these specimens depended only on the exposure period and showed no variation with resistance. Second, very high mat to mat resistance values will provide almost infinite resistance to corrosion current. Figure 5.38 shows no corrosion activity for resistance values greater than approximately 5000 ohms. The region between these two extremes shows a varying relationship between corrosion current and resistance. Figure 5.38 provides a graph showing this region in detail. The graph shows increasing current with decreasing resistance values. To prevent macrocell corrosion a practical approach would be to increase the resistance between the mats of reinforcement.

5.4.2 Factors Affecting Mat to Mat Resistance

The relationship between corrosion current and resistance has been previously established. This section will evaluate which test variables had the most affect on the resistance between the mats of reinforcement. To evaluate the affect of the test variables on the mat to mat resistance a statistical analysis was performed on data from the six of the seven specimens containing epoxy-coated reinforcement that showed corrosion activity. Data from specimens E2.5-6m-6d(a), E2.5-6m-12d(a), E2.5-6m-12d(b), E1.0-6m-12d(a), E1.0-12m-6d(a), E1.0-6m-6d(b) was considered. Only data from week 25 and later was included to ensure the chloride concentration at the reinforcement level was great enough to initiate corrosion. Although the seventh specimen, E1.0-6m-12d(b), showed some corrosion activity towards the end of the exposure period, active corrosion had not occurred over a significant length of time. Data from this specimen was not included in the statistical analysis due to the short time period this specimen showed active corrosion. The variables considered in the analysis included: weeks of exposure, average coating thickness of top mat reinforcement, defects on the top mat reinforcement, distance between the top and bottom mat of reinforcement, and surface temperature. The statistical program SAS version 6.11 by the SAS Institute, Inc. was used to analyze the data. A multiple linear regression model was used in the analysis. Appendix C contains the SAS program created for the analysis and a sample input-output run.

Table 5.13 provides the SAS results for all possible combinations of the five variables. The table includes three criteria for evaluation of the models: R^2, Adjusted R^2, and C_p. R^2 indicates what percentage of the total variation the model explains. For example, the regression model containing all five variables in Table 5.12 has a R^2 value of 0.7743. This indicates that this model accounts for 77.43% of the variation, with 22.57% of the variation unaccounted for by the five variables. R^2 will always be at a maximum with all the variables in the model included. However, when the addition of new variables provides only a marginal increase in R^2, the new variable is not justified for inclusion in the model. Therefore, the criteria when considering R^2 is to choose the model with the highest R^2 with the fewest justified variables. The Adjusted R^2 term (R_a^2)

is a criteria which is adjusted for the number of variables within the model. The criterion when considering R_a^2 is to choose the model with the largest R_a^2. The last term, C_p, also evaluates the effectiveness of the model. The criterion is to choose the model with the C_p closest to the number of variables in the model.

Following these criteria, the model with all five variables included accounts for the most variation in the model. The following relationship was found for AC mat to mat resistance:

$$\text{Resistance} = -82.12(\text{Week}) - 469.53(\text{Thickness}) - 390.01(\text{Defects}) + 773.21(\text{Dist}) - 117.39(\text{Temp}) + 24466$$

Where:

Week = # of weeks of exposure testing

Thickness = thickness of epoxy coating in mils

Defects = # of defects per foot of top bar

Dist = distance between the two mats of reinforcement in inches

Temp = surface temperature in °F

This relationship accounts for 77.43% of the variation in the mat to mat resistance. While this relationship is not particularly useful in practice, the information provided by the statistical analysis about each variable is useful.

According to the results provided in Table 5.13, the variables with the most influence on the resistance values were temperature, weeks of exposure, and the number of defects on the epoxy-coated reinforcement. If the data from week 38, when measurements were taken at 22.2 °C (72 °F), is eliminated from the database used for the statistical analysis the weeks of exposure and number of defects become the most critical variables affecting the mat to mat resistance. The thickness of the epoxy-coated reinforcement and the distance between mats of reinforcement provided the least impact on the resistance readings. Addition of the thickness variable to the regression model only increased the R^2 term slightly from 0.7650 to 0.7743 and this variable could have been excluded from the regression model.

The result from the statistical analysis regarding thickness of coating can be misleading. From the data provided by the exposure testing of the specimens it would appear that the number of defects and thickness of coating are two independent variables. This is accurate for the laboratory simulation because the number of defects on the reinforcement was controlled and created if necessary. However, as seen from the field evaluation and the initial testing of the defects on the reinforcement for the laboratory evaluation, thickness and damage to the coating are related. In all instances, thinner epoxy coatings received more damage in the transportation, placement, and concrete casting operations.

5.5 Summary of Laboratory Evaluation

- Plastic settlement cracks in the specimens with 25.4 mm (1.0 in.) of cover allowed the chlorides to reach the reinforcement as soon as the sodium chloride solution was applied. Cracking may have allowed more oxygen to reach the reinforcement, which would accelerate the corrosion process in the specimens with uncoated reinforcement.
- The extra cover and the absence of cracks in the specimens with uncoated reinforcement and 63.5 mm (2.5 in.) of cover delayed the initiation of corrosion in these specimens.

- The difference in the current levels between the specimens with two different concrete covers with uncoated reinforcement is most probably due to the level of oxygen available. The thicker concrete cover with no cracking limits the availability of oxygen. This lack of oxygen slowed the cathodic reaction, which in turn limited the rate of the anodic (corrosion of the steel) reaction.
- Epoxy-coated reinforcement had little if any corrosion activity. Only four of the specimens developed active corrosion current over the 77-week exposure period. The epoxy coating also limits the oxygen provided for the cathodic reaction, but more importantly, the coating increases the AC mat to mat resistance dramatically.
- The half-cell potential readings for the uncoated reinforcement correlated well with the corrosion current readings.
- The forensic investigation of half of the specimens found uniform corrosion on the uncoated reinforcement removed from the specimens. Some cross-section loss of the uncoated reinforcement removed from the specimens with 25.4 mm (1 in.) of cover was observed. Only minor rusting of the epoxy-coated reinforcement removed from the specimens had occurred. No cross-section loss of the epoxy-coated reinforcement was observed.
- Analysis of corrosion current measurements and AC mat to mat resistance measurements from specimens indicated a correlation. Once the chlorides penetrate to the level of reinforcement, the corrosion current is dependent on the resistance between the two mats of reinforcement.
- The use of uncoated reinforcement provided a low mat to mat resistance value, which did not impede corrosion current.

- Values of mat to mat resistance greater than 5000 ohms provided excellent resistance to corrosion current.
- High values of resistance were provided by epoxy-coated reinforcement with fewer defects.
- A statistical analysis was performed that showed the number of defects in the epoxy coating was a critical variable affecting the mat to mat resistance of epoxy-coated reinforcement.

Table 5.1 Specimen Summary

Specimen	Rebar Type	Bar Size	Cover mm (in.)	Comment
U2.5(a)	Uncoated	15.9 mm (# 5)	63.5 (2.5)	Control Specimen
U1.0(a)	Uncoated	15.9 mm (# 5)	25.4 (1.0)	Control Specimen
U2.5(b)	Uncoated	15.9 mm (# 5)	63.5 (2.5)	Control Specimen (Duplicate)
U1.0(b)	Uncoated	15.9 mm (# 5)	25.4 (1.0)	Control Specimen (Duplicate)
E2.5-6m-6d(a)	Epoxy-Coated	15.9 mm (# 5)	63.5 (2.5)	6 Defects per foot of bar, Low end of coating thickness - 6 mils
E1.0-6m-6d(a)	Epoxy-Coated	15.9 mm (# 5)	25.4 (1.0)	6 Defects per foot of bar, Low end of coating thickness - 6 mils
E2.5-6m-6d(b)	Epoxy-Coated	15.9 mm (# 5)	63.5 (2.5)	6 Defects per foot of bar, Low end of coating thickness - 6 mils (Duplicate)
E1.0-6m-6d(b)	Epoxy-Coated	15.9 mm (# 5)	25.4 (1.0)	6 Defects per foot of bar, Low end of coating thickness - 6 mils (Duplicate)
E1.0-12m-6d(a)	Epoxy-Coated	15.9 mm (# 5)	25.4 (1.0)	6 Defects per foot of bar, High end of coating thickness - 12 mils

Table 5.1 (concluded)

Specimen	Rebar Type	Bar Size	Cover (in.)	Comment
E1.0-12m-6d(b)	Epoxy-Coated	15.9 mm (# 5)	25.4 (1.0)	6 Defects per foot of bar, High end of coating thickness - 12 mils (Duplicate)
E2.5-6m-12d(a)	Epoxy-Coated	15.9 mm (# 5)	63.5 (2.5)	12 Defects per foot of bar, Low end of coating thickness - 6 mils
E1.0-6m-12d(a)	Epoxy-Coated	15.9 mm (# 5)	25.4 (1.0)	12 Defects per foot of bar, Low end of coating thickness - 6 mils
E2.5-6m-12d(b)	Epoxy-Coated	15.9 mm (# 5)	63.5 (2.5)	12 Defects per foot of bar, Low end of coating thickness - 6 mils (Duplicate)
E1.0-6m-12d(b)	Epoxy-Coated	15.9 mm (# 5)	25.4 (1.0)	12 Defects per foot of bar, Low end of coating thickness - 6 mils (Duplicate)
E1.0-12m-12d(a)	Epoxy-Coated	15.9 mm (# 5)	25.4 (1.0)	12 Defects per foot of bar, High end of coating thickness - 12 mils
E1.0-12m-12d(b)	Epoxy-Coated	15.9 mm (# 5)	25.4 (1.0)	12 Defects per foot of bar, High end of coating thickness - 12 mil (Duplicate)

Table 5.2 Concrete Mix Design

Component	Batch Weights per m^3 (per yd^3)
Type I Cement	390 kg (658 lb)
Water	139 kg (235 lb)
Crushed Limestone	1067 kg (1800 lb)
Sand	783 kg (1320 lb)
Air Entraining Agent	387 mL (10 oz.)
Water Reducer	657 mL (13 oz.)

Table 5.3 Coating Thickness Measurements

Bar Number	Surface	Thickness Measurements (mils)						Average µm (mils)
1	Top	7	7	5	6	6	5	154.43
	Bottom	6	7	5	6	7	6	(6.08)
2	Top	13	9	8	7	8	6	222.25
	Bottom	11	10	10	9	7	7	(8.75)
3	Top	11	9	9	9	8	5	209.55
	Bottom	10	9	8	9	7	5	(8.25)
4	Top	8	7	6	5	7	4	154.43
	Bottom	8	7	6	6	6	3	(6.08)
5	Top	9	7	7	8	5	4	154.43
	Bottom	8	5	5	5	5	5	(6.08)
6	Top	6	7	8	7	7	8	175.77
	Bottom	6	5	6	6	7	10	(6.92)
7	Top	10	7	8	9	8	7	192.53
	Bottom	8	7	7	8	7	5	(7.58)
8	Top	6	9	9	7	8	10	211.58
	Bottom	6	8	9	8	9	11	(8.33)
9	Top	7	11	8	10	8	8	222.25
	Bottom	10	9	10	9	8	7	(8.75)
10	Top	7	8	6	6	5	4	163.07
	Bottom	9	8	7	6	6	5	(6.42)
11	Top	7	7	7	7	7	8	177.80
	Bottom	5	6	8	7	7	8	(7.0)
12	Top	6	5	6	5	5	7	148.08
	Bottom	4	6	6	6	6	8	(5.83)
13	Top	10	9	6	8	7	6	190.50
	Bottom	10	8	6	7	7	6	(7.5)
14	Top	8	9	6	3	5	4	154.43
	Bottom	8	8	6	6	5	5	(6.08)
15	Top	4	7	8	9	9	9	198.88
	Bottom	5	6	9	8	9	11	(7.83)
16	Top	7	8	10	9	9	10	224.28
	Bottom	7	7	9	8	10	12	(8.83)
17	Top	9	8	8	8	8	10	220.22
	Bottom	9	8	9	9	8	10	(8.67)
18	Top	10	9	9	7	7	6	211.58
	Bottom	13	9	8	7	8	7	(8.33)
19	Top	4	6	7	6	7	8	148.08
	Bottom	4	6	5	5	6	6	(5.83)

Table 5.3 (continued)

Bar Number	Surface	Thickness Measurements (mils)						Average µm (mils)
20	Top	12	9	9	10	11	12	249.68
	Bottom	13	10	9	8	8	7	(9.83)
21	Top	7	7	7	7	6	6	165.10
	Bottom	7	7	6	7	6	5	(6.50)
22	Top	6	8	8	8	8	9	198.88
	Bottom	5	8	8	8	9	9	(7.83)
23	Top	9	9	8	7	6	5	194.82
	Bottom	9	9	9	7	8	6	(7.67)
24	Top	8	8	7	8	9	6	205.23
	Bottom	10	9	8	8	10	6	(8.08)
25	Top	6	6	8	8	8	9	188.47
	Bottom	6	7	7	7	9	8	(7.42)
26	Top	10	10	9	8	8	7	213.87
	Bottom	9	10	8	8	7	7	(8.42)
27	Top	9	8	8	6	5	4	184.15
	Bottom	10	9	10	7	6	5	(7.25)
28	Top	10	8	8	8	9	8	220.22
	Bottom	11	9	9	8	9	7	(8.67)
29	Top	5	6	7	7	8	9	179.83
	Bottom	5	7	7	7	7	10	(7.08)
30	Top	8	9	11	9	10	11	234.95
	Bottom	7	9	10	8	9	10	(9.25)
31	Top	10	10	9	9	7	6	209.55
	Bottom	10	8	9	7	7	7	(8.25)
32	Top	8	8	6	6	7	7	167.13
	Bottom	6	6	7	6	7	5	(6.58)
33	Top	10	10	9	9	9	7	224.28
	Bottom	10	10	8	8	9	7	(8.83)
34	Top	6	7	8	8	9	8	203.20
	Bottom	11	5	9	9	8	8	(8.00)
35	Top	9	8	9	8	8	7	192.53
	Bottom	9	7	7	7	7	5	(7.58)
36	Top	6	7	7	7	6	7	173.48
	Bottom	6	7	7	7	7	8	(6.83)
37	Top	9	9	9	8	8	9	213.87
	Bottom	7	8	9	8	9	8	(8.42)
38	Top	11	9	6	9	8	7	201.17
	Bottom	10	7	7	7	7	7	(7.92)

Table 5.3 (continued)

Bar Number	Surface	Thickness Measurements (mils)						Average µm (mils)
39	Top	5	5	6	6	7	8	165.10
	Bottom	6	7	6	6	8	8	(6.50)
40	Top	10	6	6	6	6	6	156.72
	Bottom	6	5	6	6	6	5	(6.17)
41	Top	8	8	8	9	9	11	228.60
	Bottom	8	8	8	9	10	12	(9.00)
42	Top	7	9	6	7	8	10	198.88
	Bottom	7	8	7	8	8	9	(7.83)
43	Top	7	7	7	6	9	9	186.18
	Bottom	6	6	7	8	8	8	(7.33)
44	Top	9	8	6	5	8	7	184.15
	Bottom	9	8	5	7	7	8	(7.25)
45	Top	8	7	9	8	7	8	201.17
	Bottom	6	7	9	9	9	8	(7.92)
46	Top	7	6	6	8	7	8	175.77
	Bottom	6	6	6	7	8	8	(6.92)
47	Top	9	8	6	7	7	7	179.83
	Bottom	9	8	6	7	6	5	(7.08)
48	Top	9	9	7	7	9	7	215.90
	Bottom	9	10	9	9	10	7	(8.50)
49	Top	8	9	9	9	10	12	256.03
	Bottom	9	10	9	10	12	14	(10.08)
50	Top	11	9	9	9	8	7	241.30
	Bottom	12	10	9	11	10	9	(9.50)
51	Top	6	8	11	9	10	11	228.60
	Bottom	7	8	9	8	9	12	(9.00)
52	Top	8	8	8	8	8	12	224.28
	Bottom	9	7	9	9	9	11	(8.83)
53	Top	7	10	9	10	11	13	234.95
	Bottom	7	8	8	8	8	12	(9.25)
54	Top	7	10	11	11	11	11	241.30
	Bottom	8	9	9	9	9	9	(9.50)
55	Top	12	10	10	13	11	7	258.32
	Bottom	12	10	9	11	10	7	(10.17)
56	Top	12	11	10	10	9	8	245.62
	Bottom	12	9	9	10	9	7	(9.67)
57	Top	8	8	10	9	9	13	254.00
	Bottom	8	9	11	10	12	13	(10.00)

Table 5.3 (concluded)

Bar Number	Surface	Thickness Measurements (mils)						Average µm (mils)
58	Top	8	9	9	9	8	9	251.97
	Bottom	9	11	11	12	11	13	(9.92)
59	Top	13	10	10	9	9	7	258.32
	Bottom	13	11	11	12	9	8	(10.17)
60	Top	9	9	9	9	11	12	260.35
	Bottom	10	11	10	9	12	12	(10.25)
61	Top	12	10	9	9	8	8	239.27
	Bottom	11	11	9	10	8	8	(9.42)
62	Top	8	9	10	12	10	13	287.78
	Bottom	9	12	15	14	11	13	(11.33)
63	Top	15	11	10	9	11	9	251.97
	Bottom	12	9	9	8	9	7	(9.92)
64	Top	12	12	10	9	8	7	224.28
	Bottom	9	8	7	8	9	7	(8.83)
65	Top	12	11	10	10	10	12	256.03
	Bottom	11	10	9	9	9	8	(10.08)
66	Top	10	10	9	11	9	10	247.65
	Bottom	12	10	10	11	7	8	(9.75)
67	Top	9	8	10	9	9	10	256.03
	Bottom	10	9	10	11	12	14	(10.08)
68	Top	8	11	11	11	11	11	264.67
	Bottom	9	10	10	11	11	11	(10.42)
69	Top	7	10	11	12	11	11	262.38
	Bottom	9	10	10	10	11	12	(10.33)
70	Top	8	11	10	11	13	12	268.73
	Bottom	9	10	11	10	11	11	(10.58)
71	Top	9	9	10	11	12	13	271.02
	Bottom	8	10	11	10	12	13	(10.67)
72	Top	10	10	9	8	8	9	230.63
	Bottom	13	10	8	8	9	7	(9.08)

Table 5.4 Defects and Holidays after Transportation and Fabrication

Bar Number	Average Coating Thickness μm (mils)	Surface	Number of Defects and Holidays	Total Defects and Holidays
1	154.43 (6.08)	Top Bottom	15 20	35
2	222.25 (8.75)	Top Bottom	5 4	9
3	209.55 (8.25)	Top Bottom	8 3	11
4	154.43 (6.08)	Top Bottom	17 25	42
5	154.43 (6.08)	Top Bottom	32 39	71
6	175.77 (6.92)	Top Bottom	3 6	9
7	192.53 (7.58)	Top Bottom	2 5	7
8	211.58 (8.33)	Top Bottom	1 2	3
9	222.25 (8.75)	Top Bottom	11 2	13
10	163.07 (6.42)	Top Bottom	2 20	22
11	177.80 (7.0)	Top Bottom	31 23	54
12	148.08 (5.83)	Top Bottom	32 50	82
13	190.50 (7.5)	Top Bottom	8 7	15
14	154.43 (6.08)	Top Bottom	40 32	72
15	198.88 (7.83)	Top Bottom	16 54	70
16	224.28 (8.83)	Top Bottom	2 9	11
17	220.22 (8.67)	Top Bottom	0 8	8
18	211.58 (8.33)	Top Bottom	1 6	7
19	148.08 (5.83)	Top Bottom	33 22	55

Table 5.4 (continued)

Bar Number	Average Coating Thickness µm (mils)	Surface	Number of Defects and Holidays	Total Defects and Holidays
20	249.68 (9.83)	Top Bottom	3 0	3
21	165.10 (6.50)	Top Bottom	11 14	25
22	198.88 (7.83)	Top Bottom	2 0	2
23	194.82 (7.67)	Top Bottom	1 9	10
24	205.23 (8.08)	Top Bottom	5 0	5
25	188.47 (7.42)	Top Bottom	7 28	35
26	213.87 (8.42)	Top Bottom	0 7	7
27	184.15 (7.25)	Top Bottom	5 35	40
28	220.22 (8.67)	Top Bottom	0 6	6
29	179.83 (7.08)	Top Bottom	3 7	10
30	234.95 (9.25)	Top Bottom	1 3	4
31	209.55 (8.25)	Top Bottom	3 10	13
32	167.13 (6.58)	Top Bottom	30 35	65
33	224.28 (8.83)	Top Bottom	18 15	33
34	203.20 (8.00)	Top Bottom	13 3	16
35	192.53 (7.58)	Top Bottom	5 9	14
36	173.48 (6.83)	Top Bottom	1 15	16
37	213.87 (8.42)	Top Bottom	0 14	14
38	201.17 (7.92)	Top Bottom	5 3	8

Table 5.4 (continued)

Bar Number	Average Coating Thickness µm (mils)	Surface	Number of Defects and Holidays	Total Defects and Holidays
39	165.10 (6.50)	Top Bottom	12 31	43
40	156.72 (6.17)	Top Bottom	25 39	64
41	228.60 (9.00)	Top Bottom	0 12	12
42	198.88 (7.83)	Top Bottom	11 7	18
43	186.18 (7.33)	Top Bottom	23 2	25
44	184.15 (7.25)	Top Bottom	29 1	30
45	201.17 (7.92)	Top Bottom	2 1	3
46	175.77 (6.92)	Top Bottom	2 21	23
47	179.83 (7.08)	Top Bottom	2 2	4
48	215.90 (8.50)	Top Bottom	1 4	5
49	256.03 (10.08)	Top Bottom	2 0	2
50	241.30 (9.50)	Top Bottom	6 0	6
51	228.60 (9.00)	Top Bottom	3 0	3
52	224.28 (8.83)	Top Bottom	5 0	5
53	234.95 (9.25)	Top Bottom	0 9	9
54	241.30 (9.50)	Top Bottom	0 2	2
55	258.32 (10.17)	Top Bottom	3 0	3
56	245.62 (9.67)	Top Bottom	2 1	3
57	254.00 (10.00)	Top Bottom	1 1	2

Table 5.4 (concluded)

Bar Number	Average Coating Thickness μm (mils)	Surface	Number of Defects and Holidays	Total Defects and Holidays
58	251.97 (9.92)	Top Bottom	1 1	2
59	258.32 (10.17)	Top Bottom	1 1	2
60	260.35 (10.25)	Top Bottom	1 3	4
61	239.27 (9.42)	Top Bottom	1 4	5
62	287.78 (11.33)	Top Bottom	0 1	1
63	251.97 (9.92)	Top Bottom	2 5	7
64	224.28 (8.83)	Top Bottom	2 0	2
65	256.03 (10.08)	Top Bottom	3 0	3
66	247.65 (9.75)	Top Bottom	4 0	4
67	256.03 (10.08)	Top Bottom	9 1	10
68	264.67 (10.42)	Top Bottom	2 3	5
69	262.38 (10.33)	Top Bottom	1 1	2
70	268.73 (10.58)	Top Bottom	0 0	0
71	271.02 (10.67)	Top Bottom	4 1	5
72	230.63 (9.08)	Top Bottom	2 0	2

Table 5.5 Defect and Holiday Summary

Statistics for Number of Defects and Holidays	Thinner Epoxy Coating	Thicker Epoxy Coating
Mean	23.9	3.7
Standard Deviation	22.4	2.4
Minimum	2	0
Maximum	82	10

Table 5.6 Resistor and Initial AC Mat to Mat Resistance

Specimen	Initial AC Mat to Mat Resistance (Ohms)	Resistor (Ohms)
U2.5(a)	20	1.27
U1.0(a)	27	1.19
U2.5(b)	18	1.40
U1.0(b)	25	1.38
E2.5-6m-6d(a)	4000	100.14
E1.0-6m-6d(a)	6900	100.12
E2.5-6m-6d(b)	4300	100.10
E1.0-6m-6d(b)	5300	100.01
E1.0-12m-6d(a)	5200	100.09
E1.0-12m-6d(b)	6300	100.20
E2.5-6m-12d(a)	2500	100.07
E1.0-6m-12d(a)	4200	99.83
E2.5-6m-12d(b)	3000	100.18
E1.0-6m-12d(b)	4100	100.09
E1.0-12m-12d(a)	3800	100.11
E1.0-12m-12d(b)	4700	100.03

Table 5.7 Baseline Powder Sample Results

Depth	Chloride Concentration (kg/m^3)	
	Test Hole #1 – B1	Test Hole #2 – B2
0 – 12.7 mm (0 – 0.5 in.)	0.65	0.58
12.7 – 25.4 mm (0.5 – 1 in.)	1.28	0.46
25.4 – 38.1 mm (1 – 1.5 in.)	0.93	1.02
38.1 – 50.8 mm (1.5 - 2 in.)	1.09	1.17
50.8 – 63.5 mm (2 – 2.5 in.)	1.01	0.85
63.5 – 76.2 mm (2.5 – 3 in.)	0.63	0.94
76.2 – 88.9 mm (3 – 3.5 in.)	0.58	0.79
88.9 – 101.6 mm (3.5 – 4 in.)	0.79	0.79

Table 5.8 Powder Sample Results at 12 Weeks

Depth	Chloride Concentration (kg/m^3)	
	Test Hole #1 – Y1	Test Hole #2 – Z1
0 – 12.7 mm (0 – 0.5 in.)	33.9	37.33
12.7 – 25.4 mm (0.5 – 1 in.)	21.93	23.08
25.4 – 38.1 mm (1 – 1.5 in.)	12.07	12.87
38.1 – 50.8 mm (1.5 - 2 in.)	6.44	5.86
50.8 – 63.5 mm (2 – 2.5 in.)	1.12	1.30
63.5 – 76.2 mm (2.5 – 3 in.)	0.95	0.99
76.2 – 88.9 mm (3 – 3.5 in.)	0.74	0.95
88.9 – 101.6 mm (3.5 – 4 in.)	0.84	0.87

Table 5.9 Powder Sample Results at 24 Weeks

Depth	Chloride Concentration (kg/m^3)	
	Test Hole #1 – W1	Test Hole #2 – X1
0 – 12.7 mm (0 – 0.5 in.)	Insufficient Sample	36.28
12.7 – 25.4 mm (0.5 – 1 in.)	30.81	31.24
25.4 – 38.1 mm (1 – 1.5 in.)	24.66	28.18
38.1 – 50.8 mm (1.5 - 2 in.)	16.90	22.29
50.8 – 63.5 mm (2 – 2.5 in.)	9.19	13.29
63.5 – 76.2 mm (2.5 – 3 in.)	2.22	9.50
76.2 – 88.9 mm (3 – 3.5 in.)	0.67	3.23
88.9 – 101.6 mm (3.5 – 4 in.)	0.64	1.19
101.6 – 114.3 mm (4 – 4.5 in.)	0.67	1.00
114.3 – 127 mm (4.5 – 5 in.)	0.60	0.95

Table 5.10 Powder Sample Results at 36 Weeks

Depth	Chloride Concentration (kg/m^3)	
	Test Hole #1 – U1	Test Hole #2 – V1
0 – 12.7 mm (0 – 0.5 in.)	33.00	35.65
12.7 – 25.4 mm (0.5 – 1 in.)	33.48	39.41
25.4 – 38.1 mm (1 – 1.5 in.)	37.13	36.31
38.1 – 50.8 mm (1.5 - 2 in.)	31.33	34.50
50.8 – 63.5 mm (2 – 2.5 in.)	23.58	20.75
63.5 – 76.2 mm (2.5 – 3 in.)	14.41	15.08
76.2 – 88.9 mm (3 – 3.5 in.)	14.37	9.53
88.9 – 101.6 mm (3.5 – 4 in.)	12.08	7.56
101.6 – 114.3 mm (4 – 4.5 in.)	6.85	2.64
114.3 – 127 mm (4.5 – 5 in.)	3.14	0.90

Table 5.11 Powder Sample Results at 61 Weeks

Depth	Chloride Concentration (kg/m^3)	
	Test Hole #1 – A1	Test Hole #2 – A2
0 – 12.7 mm (0 – 0.5 in.)	37.51	36.71
12.7 – 25.4 mm (0.5 – 1 in.)	28.81	31.44
25.4 – 38.1 mm (1 – 1.5 in.)	29.79	25.46
38.1 – 50.8 mm (1.5 - 2 in.)	24.13	19.54
50.8 – 63.5 mm (2 – 2.5 in.)	17.18	16.45
63.5 – 76.2 mm (2.5 – 3 in.)	17.94	16.38
76.2 – 88.9 mm (3 – 3.5 in.)	15.12	14.39
88.9 – 101.6 mm (3.5 – 4 in.)	11.38	12.42
101.6 – 114.3 mm (4 – 4.5 in.)	8.84	7.31
114.3 – 127 mm (4.5 – 5 in.)	5.18	4.93
127 – 139.7 mm (5 – 5.5 in.)	2.31	1.36
139.7 – 152.4 mm (5.5 – 6 in.)	0.92	0.95

Table 5.12 Summary of Powder Sample Results

Depth	Chloride Concentration (kg/m^3)				
	0 weeks	12 weeks	24 weeks	36 weeks	61 weeks
0 – 12.7 mm (0 – 0.5 in.)	0.62	35.62	36.28	34.33	37.11
12.7 – 25.4 mm (0.5 – 1 in.)	0.87	22.51	31.03	36.45	30.13
25.4 – 38.1 mm (1 – 1.5 in.)	0.98	12.47	26.42	36.72	27.63
38.1 – 50.8 mm (1.5 - 2 in.)	1.13	6.15	19.60	32.92	21.84
50.8 – 63.5 mm (2 – 2.5 in.)	0.93	1.21	11.24	22.17	16.82
63.5 – 76.2 mm (2.5 – 3 in.)	0.79	0.97	5.86	14.75	17.16
76.2 – 88.9 mm (3 – 3.5 in.)	0.69	0.85	1.95	11.95	14.76
88.9 – 101.6 mm (3.5 – 4 in.)	0.79	0.89	0.92	9.82	11.9
101.6 – 114.3 mm (4 – 4.5 in.)			0.84	4.75	8.08
114.3 – 127 mm (4.5 – 5 in.)			0.78	2.02	5.06
127 – 139.7 mm (5 – 5.5 in.)					1.84
139.7 – 152.4 mm (5.5 – 6 in.)					0.94

Table 5.13 SAS Output

# in Model	R-square	Adjusted R-square	C(p)	Variables in Model
1	0.27121230	0.26815017	519.66452	TEMP
1	0.23710257	0.23389712	555.03217	WEEK
1	0.23505317	0.23183911	557.15715	DEFECTS
1	0.10510608	0.10134602	691.89654	THICK
1	0.05584112	0.05187407	742.97832	DIST
2	0.55462757	0.55086915	227.79724	WEEK DEFECTS
2	0.49760527	0.49336565	286.92245	DEFECTS TEMP
2	0.44890564	0.44425506	337.41806	WEEK TEMP
2	0.37774317	0.37249206	411.20492	WEEK THICK
2	0.36677353	0.36142984	422.57911	THICK TEMP
2	0.33456147	0.32894595	455.97911	WEEK DIST
2	0.32285681	0.31714252	468.11542	DIST TEMP
2	0.24195194	0.23555491	552.00396	DEFECTS DIST
2	0.23807195	0.23164218	556.02704	THICK DEFECTS
2	0.10777414	0.10024481	691.13008	THICK DIST
3	0.74748680	0.74427689	29.82558	WEEK DEFECTS TEMP
3	0.57551858	0.57012263	208.13579	WEEK DEFECTS DIST
3	0.57528889	0.56989002	208.37395	WEEK THICK TEMP
3	0.55889329	0.55328600	225.37419	WEEK THICK DEFECTS
3	0.53589244	0.52999277	249.22329	WEEK DIST TEMP
3	0.50348584	0.49717422	282.82500	DEFECTS DIST TEMP
3	0.49945043	0.49308752	287.00923	THICK DEFECTS TEMP
3	0.37782810	0.36991913	413.11686	WEEK THICK DIST
3	0.36887602	0.36085326	422.39907	THICK DIST TEMP
3	0.24246689	0.23283723	553.47003	THICK DEFECTS DIST
4	0.76504000	0.76104068	13.62505	WEEK DEFECTS DIST TEMP
4	0.75038501	0.74613624	28.82049	WEEK THICK DEFECTS TEMP
4	0.58428629	0.57721031	201.04473	WEEK THICK DEFECTS DIST
4	0.57534021	0.56811196	210.32073	WEEK THICK DIST TEMP
4	0.50467449	0.49624342	283.59252	THICK DEFECTS DIST TEMP
5	0.77432271	0.76950054	6.00000	WEEK THICK DEFECTS DIST TEMP

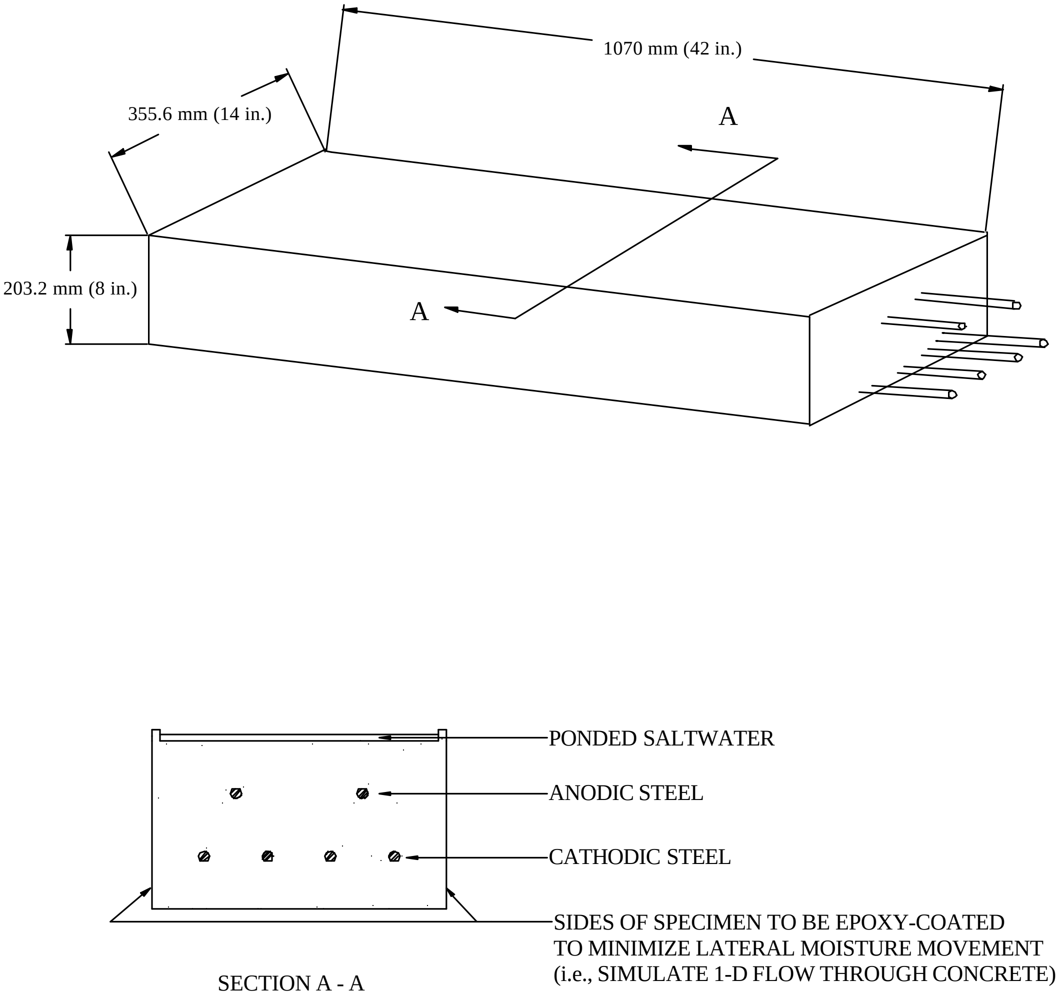


Figure 5.1 Durability Specimens

Figure 5.2 Cracking over Reinforcement

Figure 5.3 Coating Thickness Measurements

Figure 5.4 Test Specimen after Setup

Figure 5.5 Electrical Connections to Reinforcement

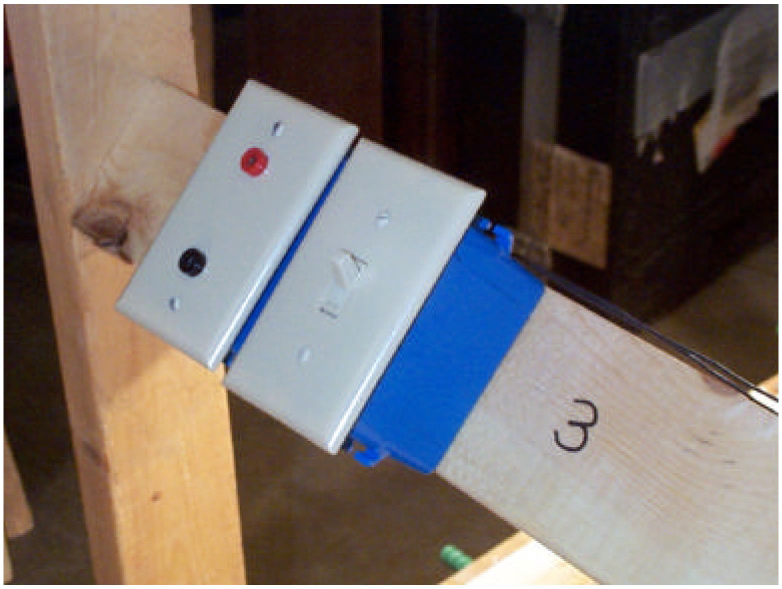


Figure 5.6 Boxes for Switch Connection and Resistor

Figure 5.7 Frame Setup

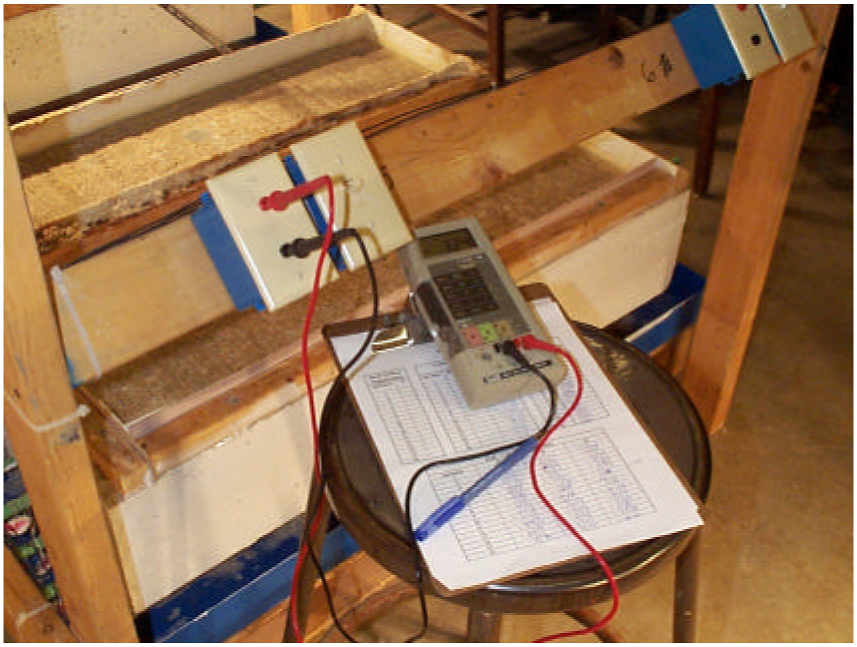


Figure 5.8 Corrosion Current Measurements

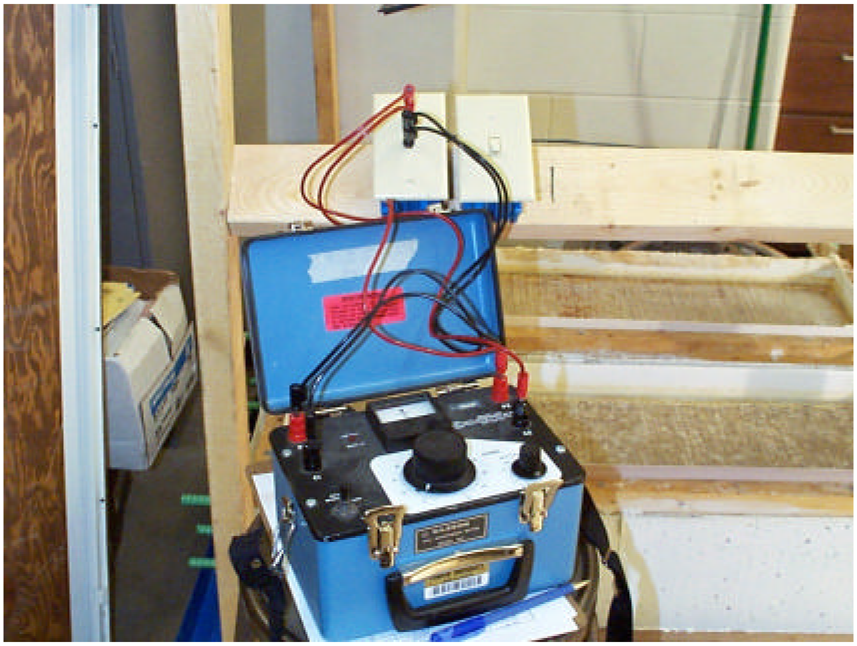


Figure 5.9 AC Mat to Mat Resistance Measurements

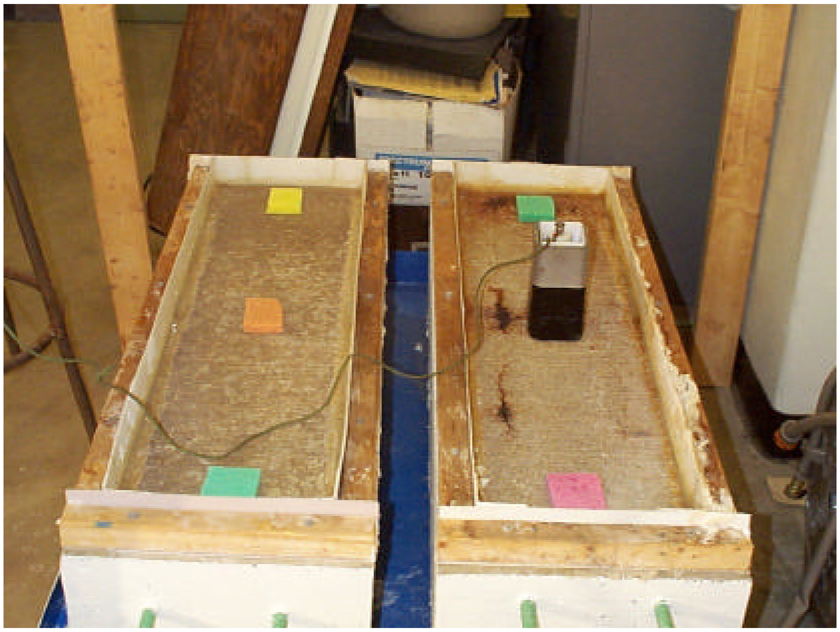


Figure 5.10 Half-Cell Potential Measurements

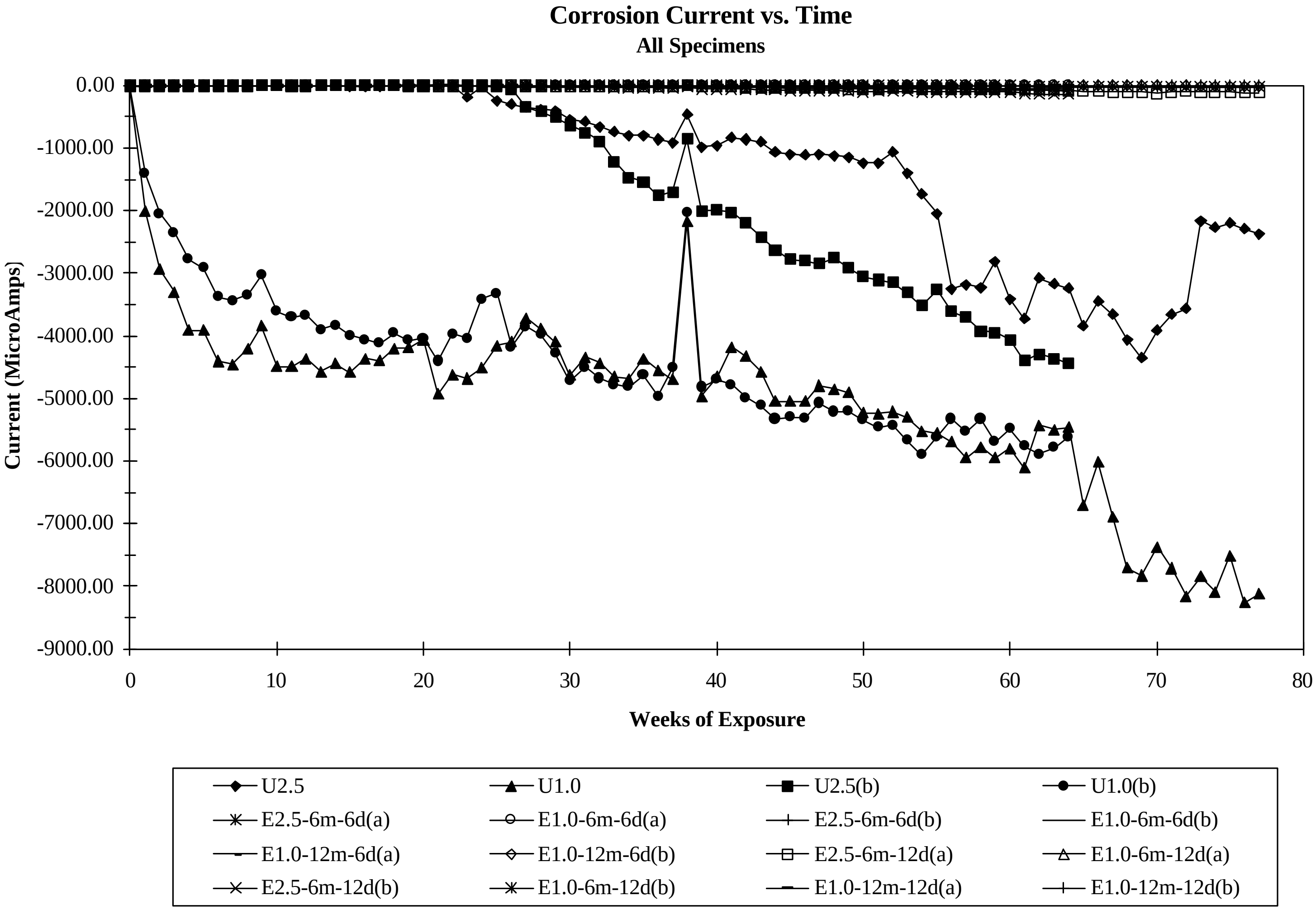


Figure 5.11 Corrosion Current Measurements for all Specimens

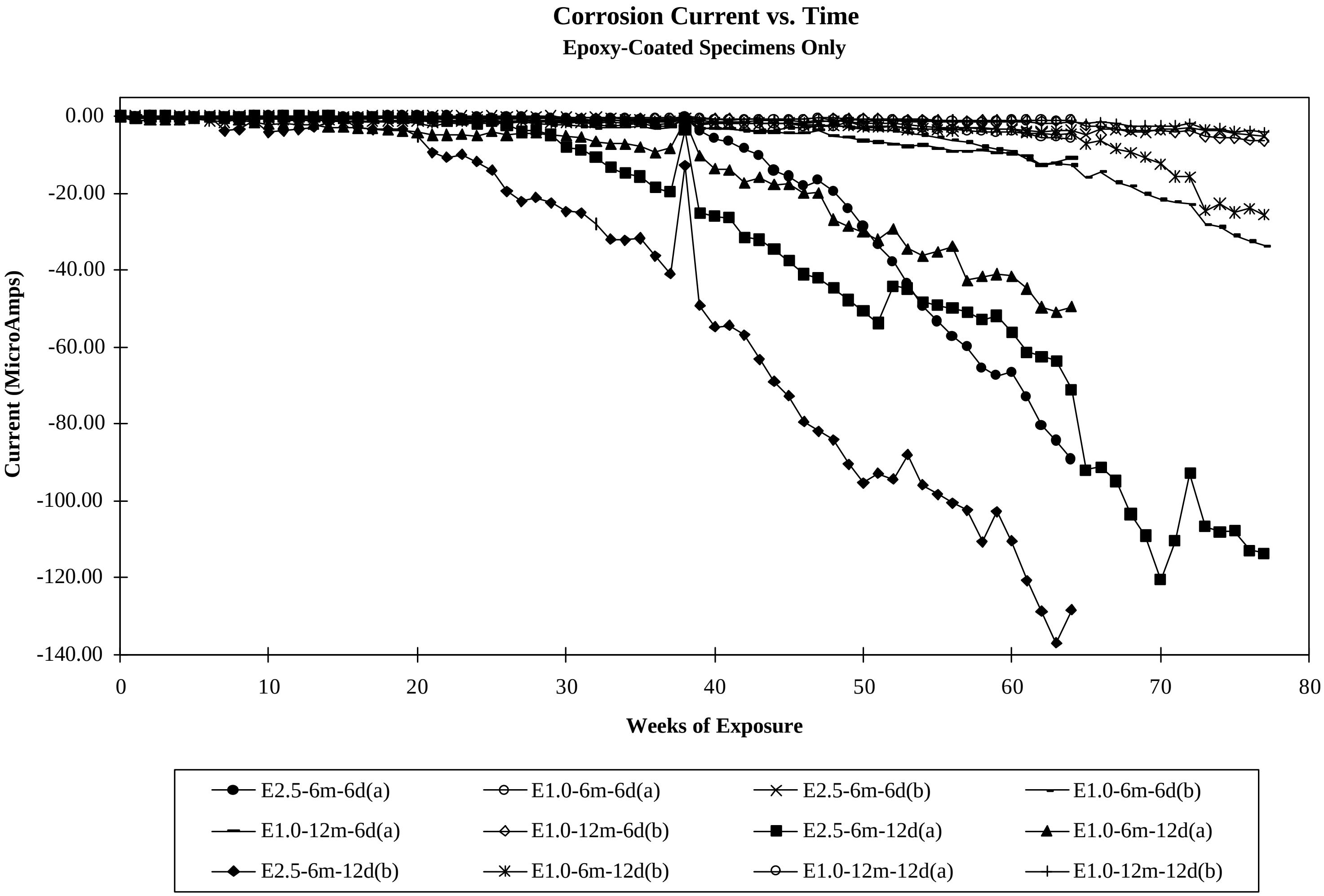


Figure 5.12 Corrosion Current Measurements for Epoxy-Coated Specimens Only

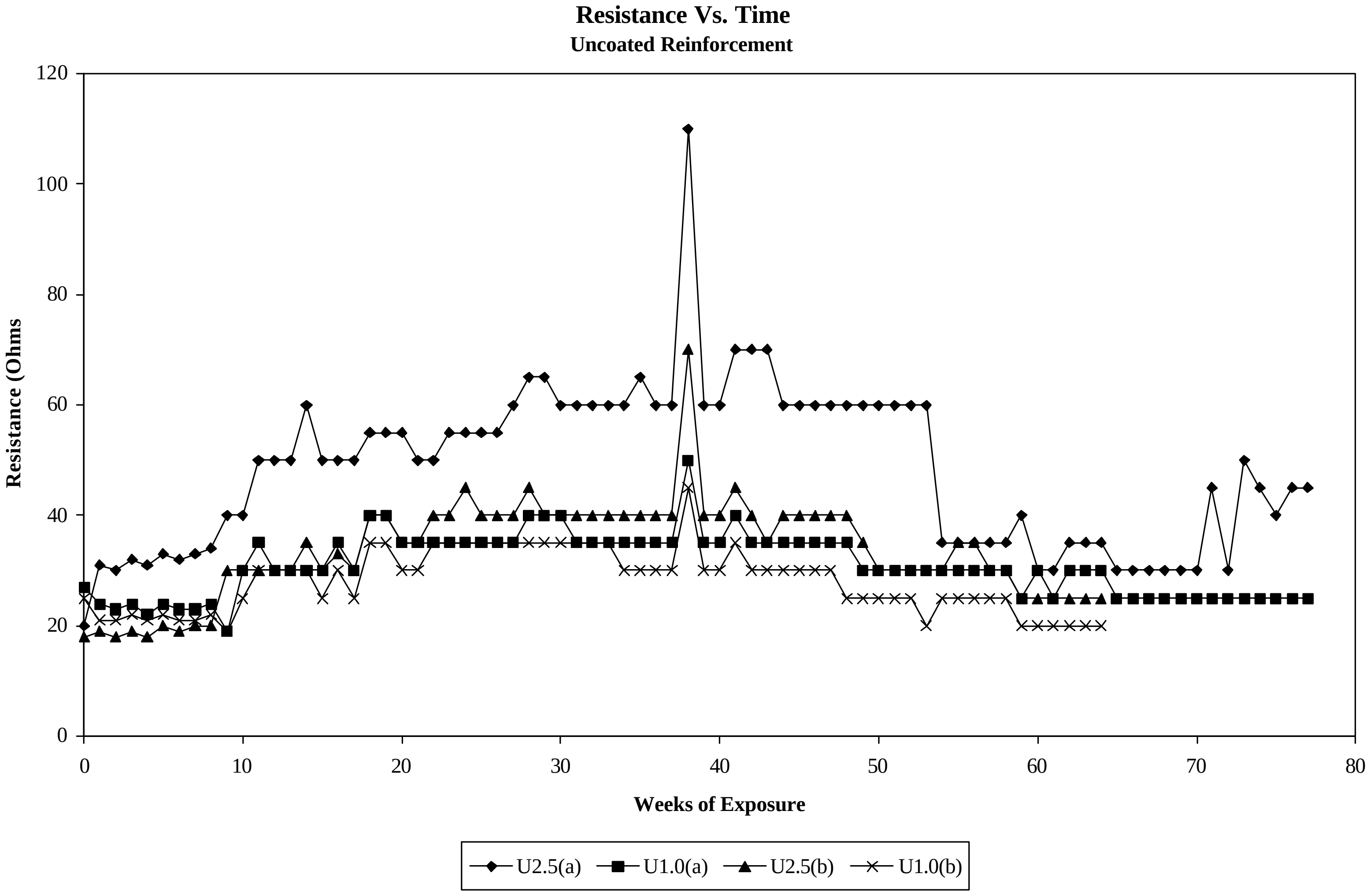


Figure 5.13 Resistance Measurements for Specimens Containing Uncoated Reinforcement

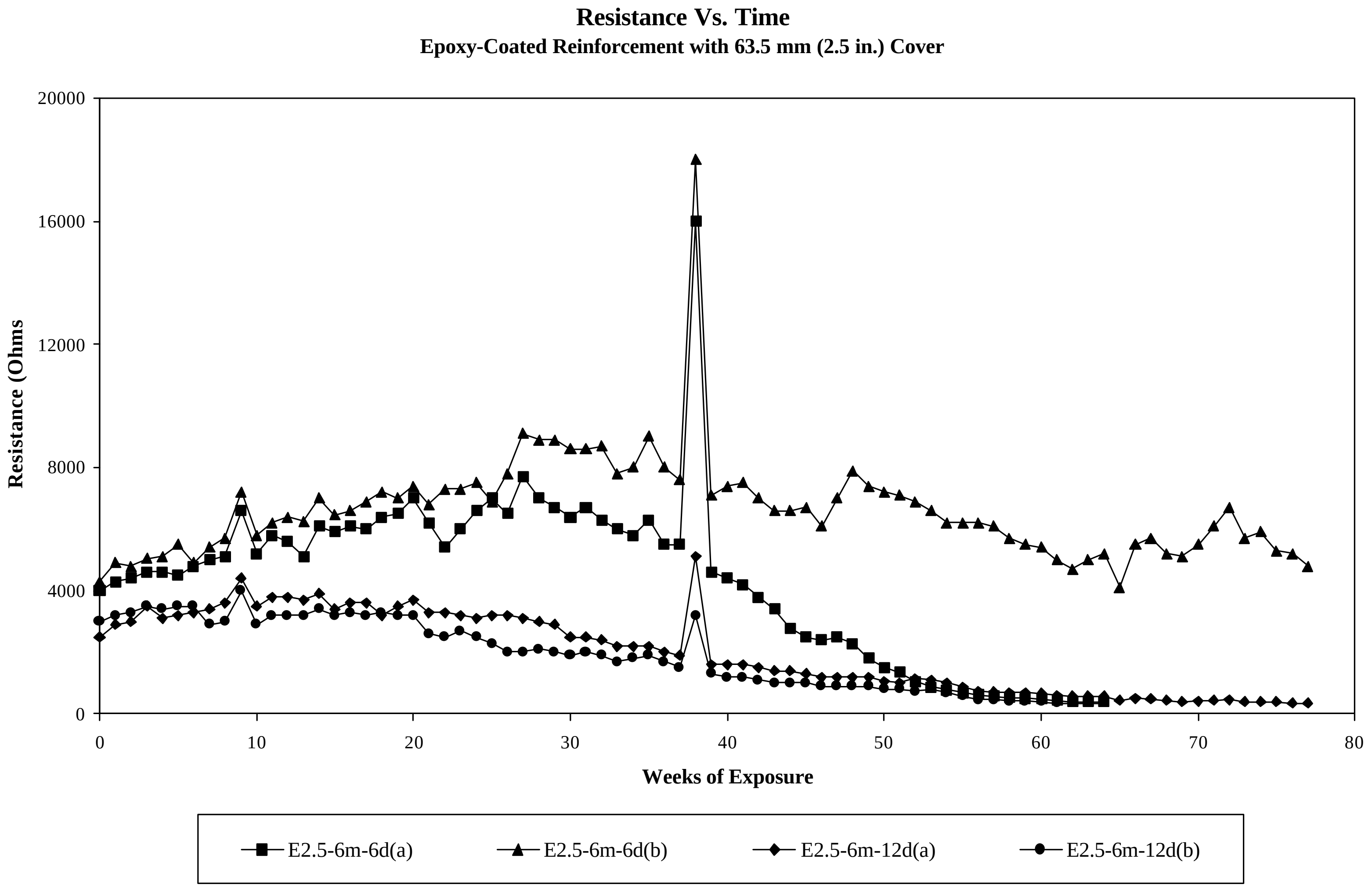


Figure 5.14 Resistance Measurements for Specimens with Epoxy-Coated Reinforcement and 63.5 mm (2.5 in.) Cover

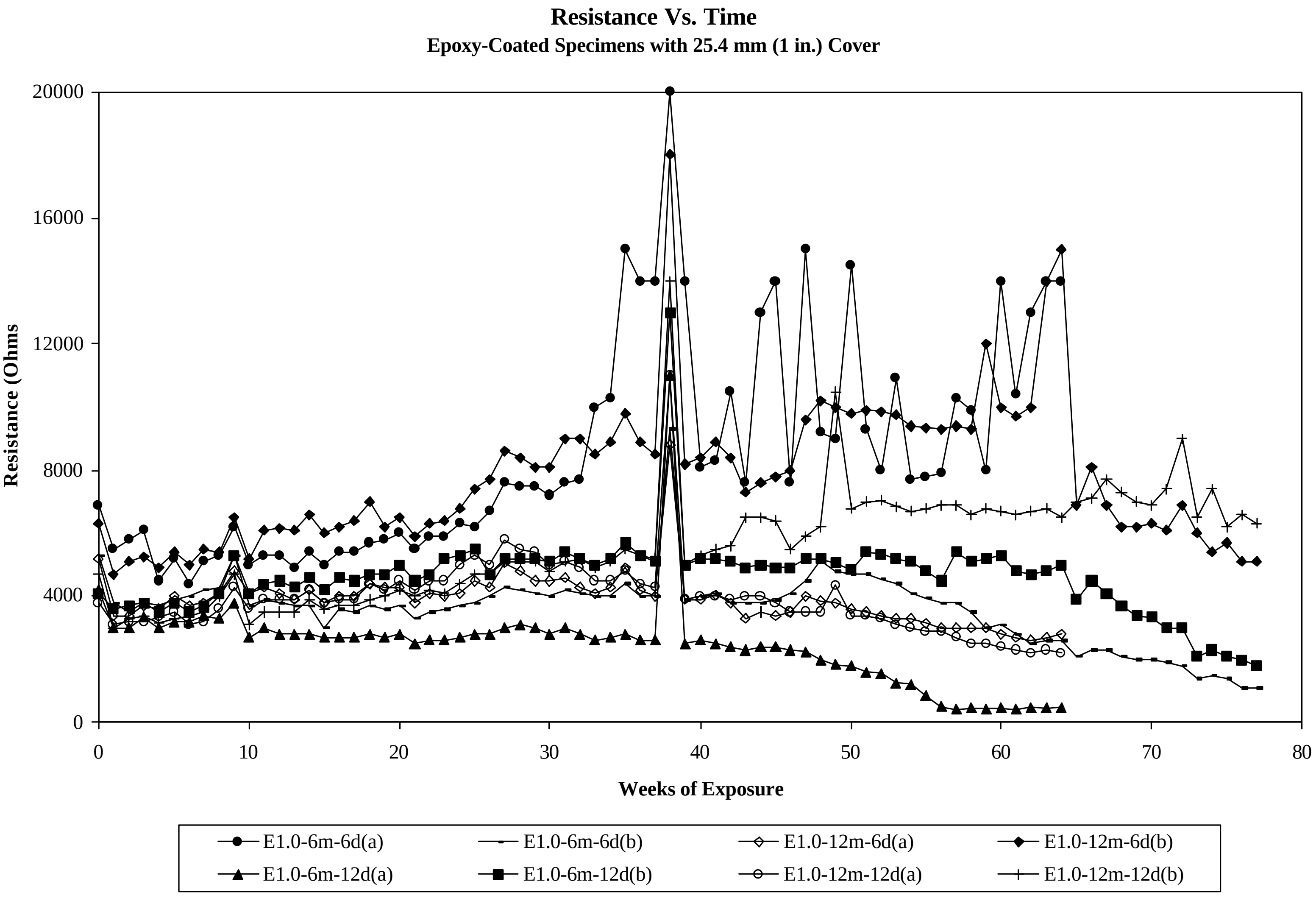


Figure 5.15 Resistance Measurements for Specimens with Epoxy-Coated Reinforcement and 25.4 mm (1.0 in.) Cover

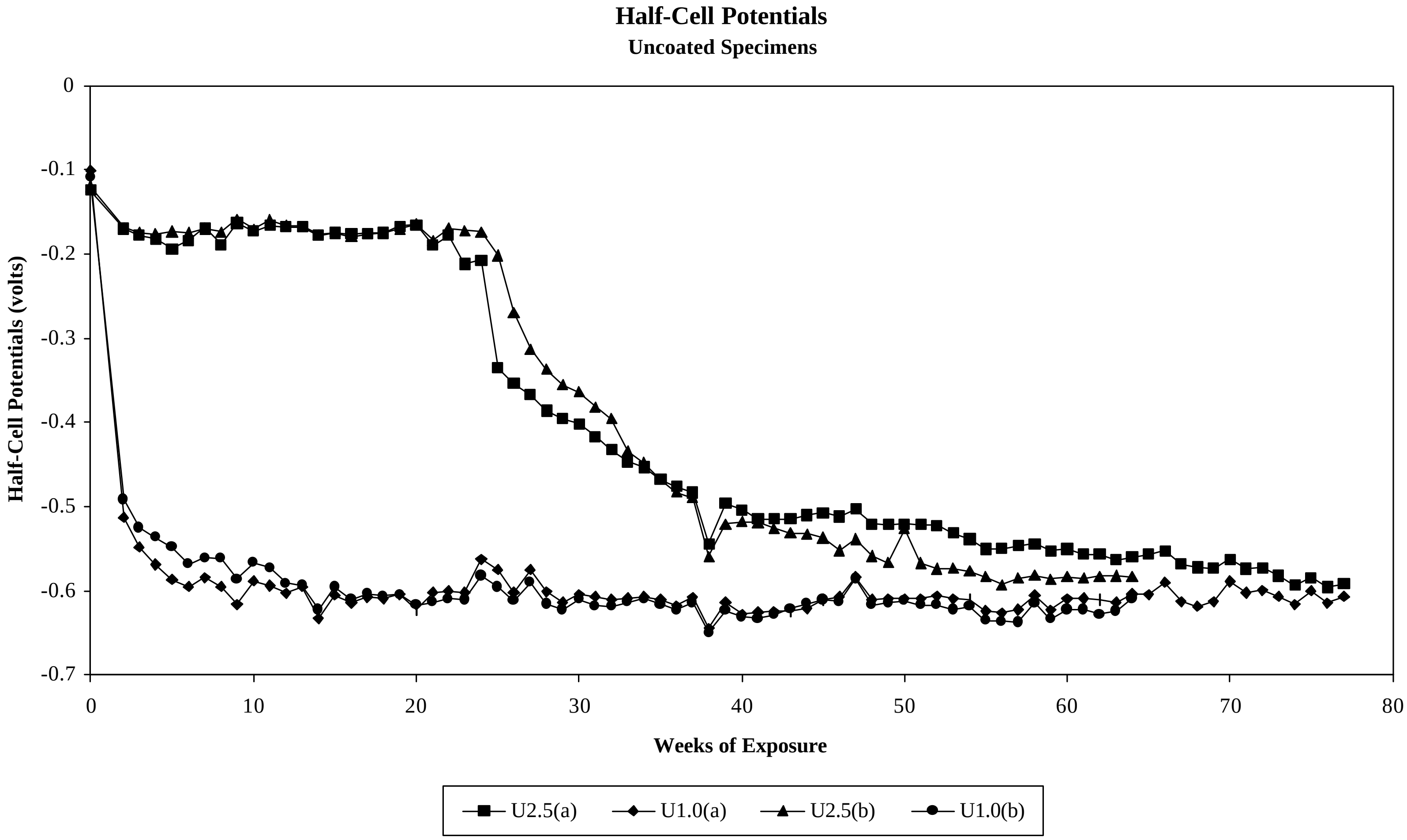


Figure 5.16 Half-Cell Potential Measurements for Specimens Containing Uncoated Reinforcement

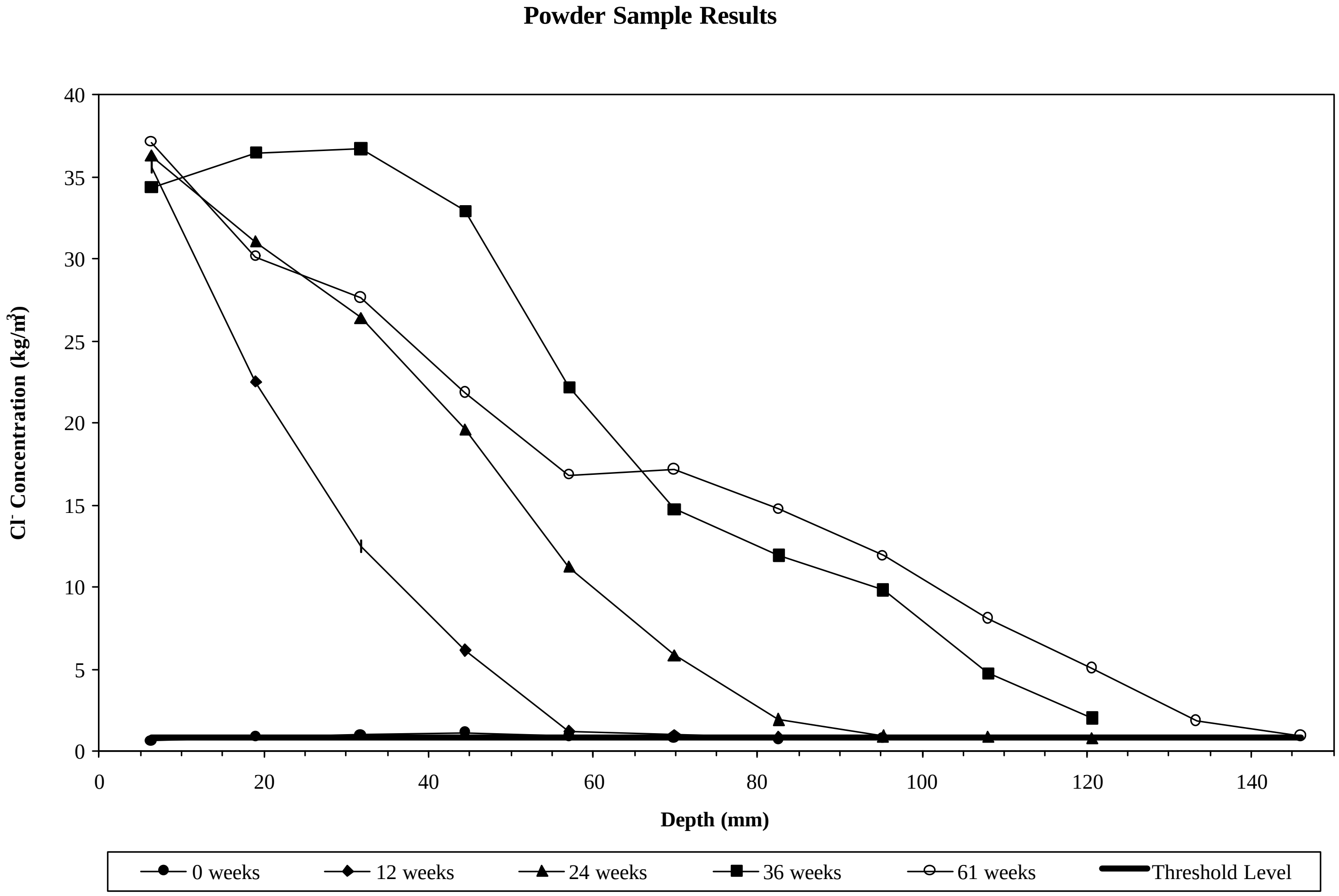


Figure 5.17 Powder Sample Results

Figure 5.18 Rust Stains at Week 10

Figure 5.19 Removal of Reinforcement from Specimens

Figure 5.20 Top Bars from Specimen U1.0(b) at Week 64

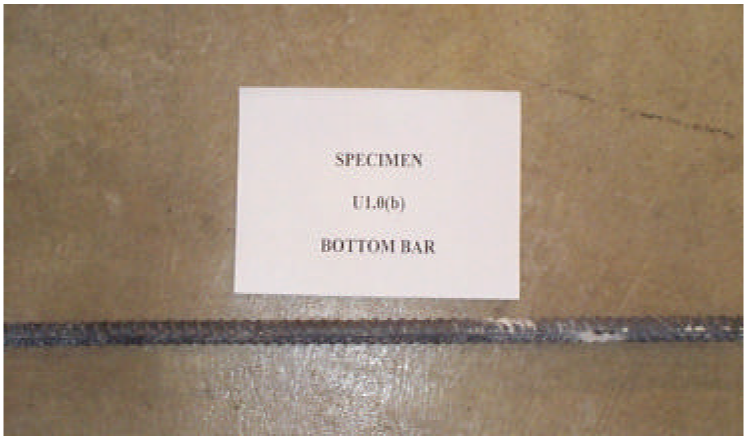


Figure 5.21 Bottom Bar from Specimen U1.0(b) at Week 64

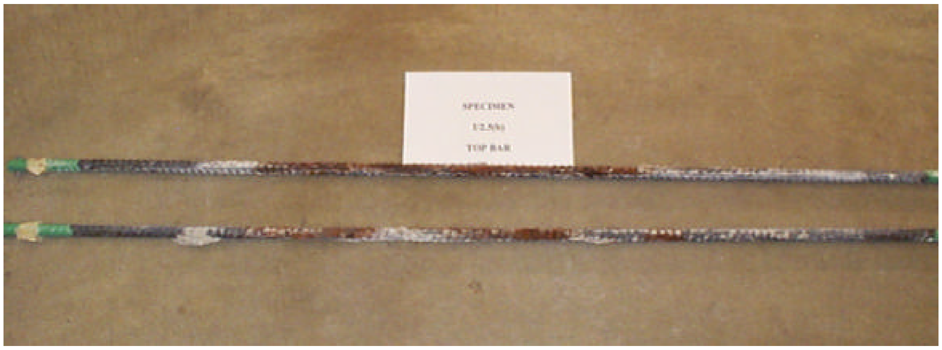


Figure 5.22 Top Bars from Specimen U2.5(b) at Week 64

Figure 5.23 Top Bars from Specimen E2.5-6m-6d(a) at Week 64

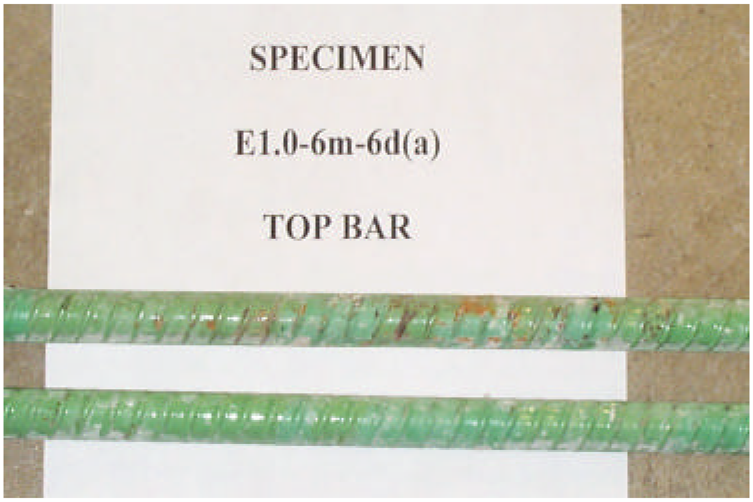


Figure 5.24 Top Bars from Specimen E1.0-6m-6d(a) at Week 64

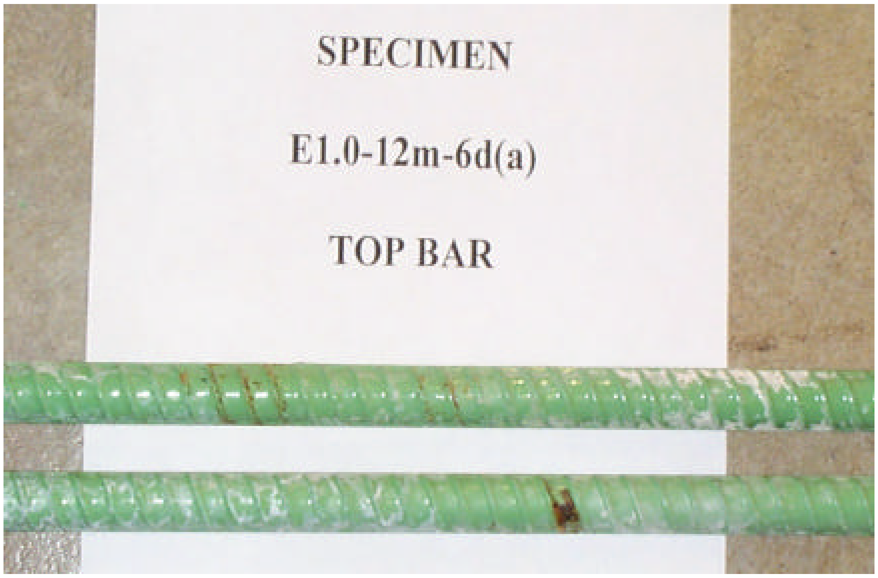


Figure 5.25 Top Bars from Specimen E1.0-12m-6d(a) at Week 64

Figure 5.26 Top Bars from Specimen E1.0-6m-12d(a) at Week 64

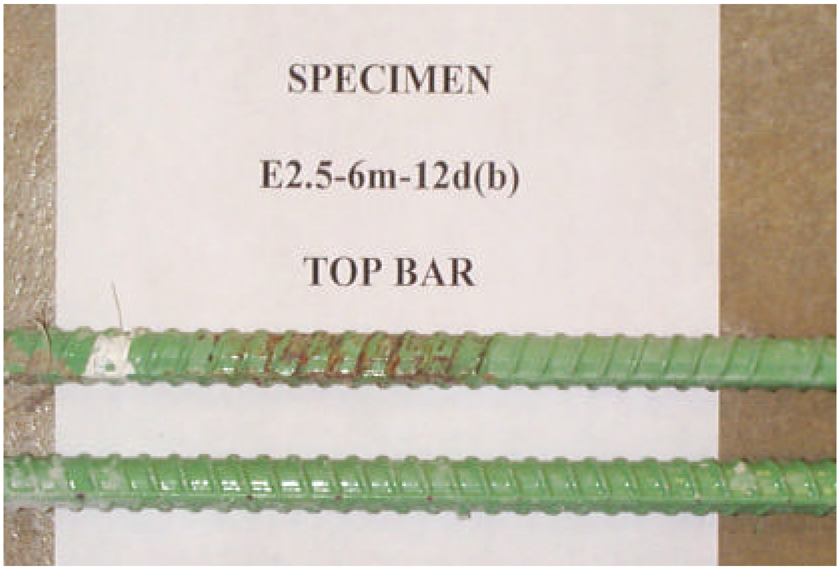


Figure 5.27 Top Bars from Specimen E2.5-6m-12d(b) at Week 64

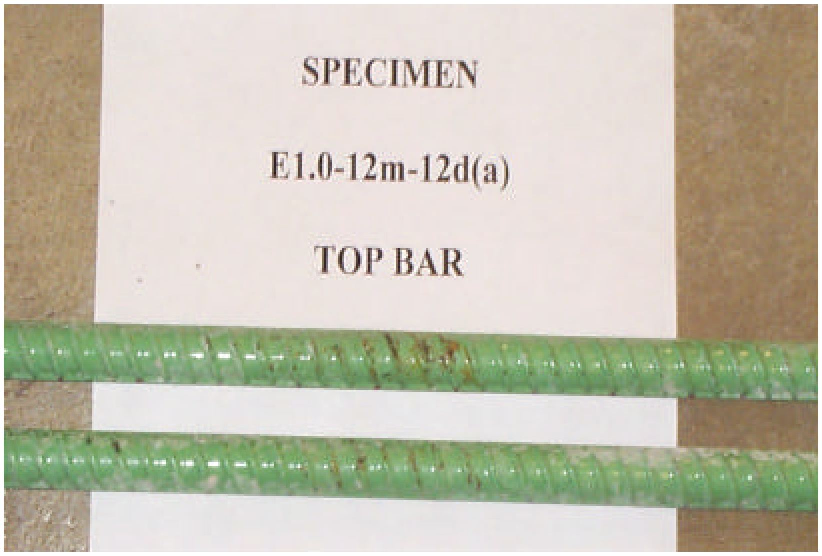


Figure 5.28 Top Bars from Specimen E1.0-12m-12d(a) at Week 64

Figure 5.29 Top Bars from Specimen U1.0(a) at Week 77

Figure 5.30 Top Bars from Specimen U2.5(a) at Week 77

Figure 5.31 Top Bars from Specimen E2.5-6m-6d(b) at Week 77

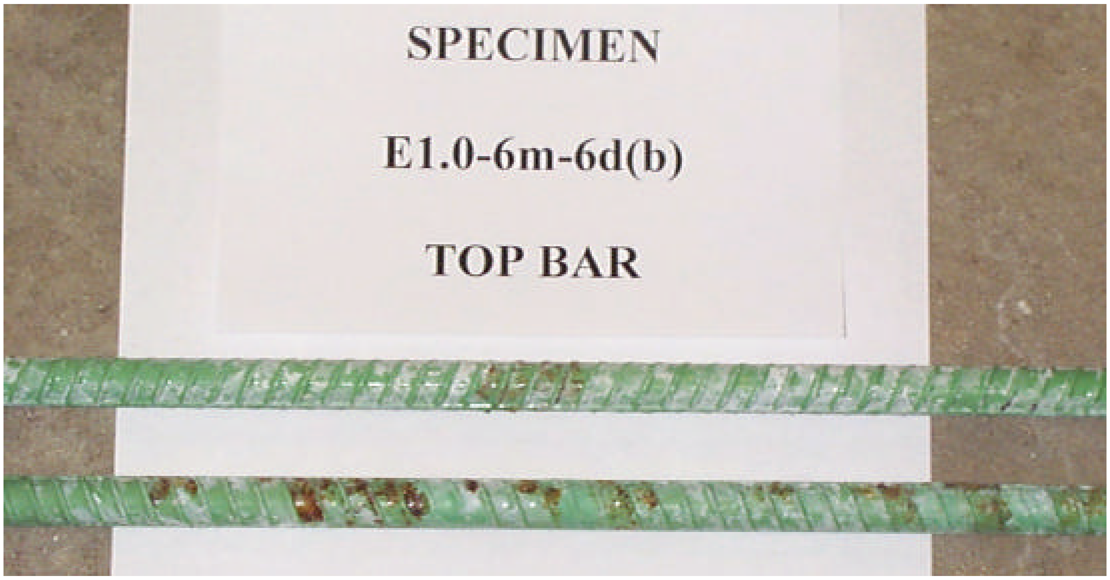


Figure 5.32 Top Bars from Specimen E1.0-6m-6d(b) at Week 77

Figure 5.33 Top Bars from Specimen E1.0-12m-6d(b) at Week 77

Figure 5.34 Top Bars from Specimen E1.0-6m-12d(b) at Week 77

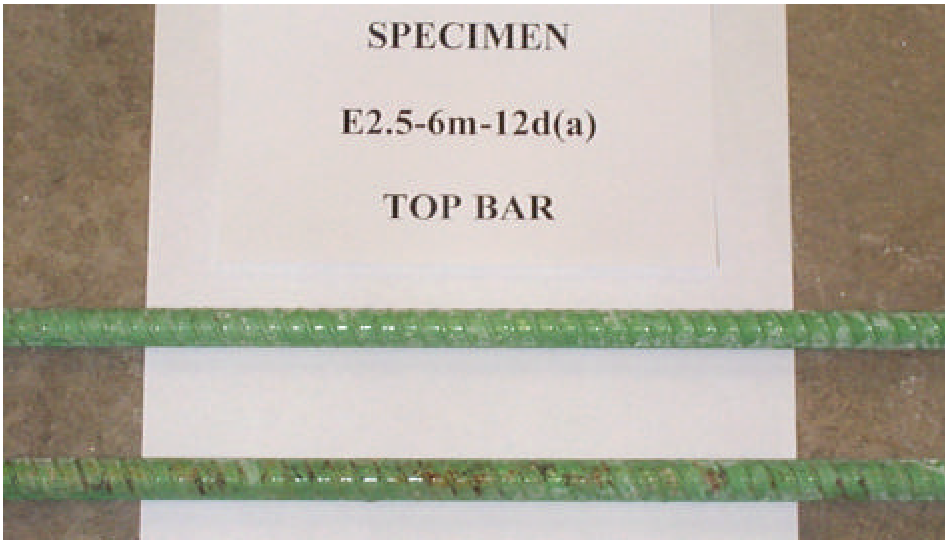


Figure 5.35 Top Bars from Specimen E2.5-6m-12d(a) at Week 77

Figure 5.36 Top Bars from Specimen E1.0-12m-12d(b) at Week 77

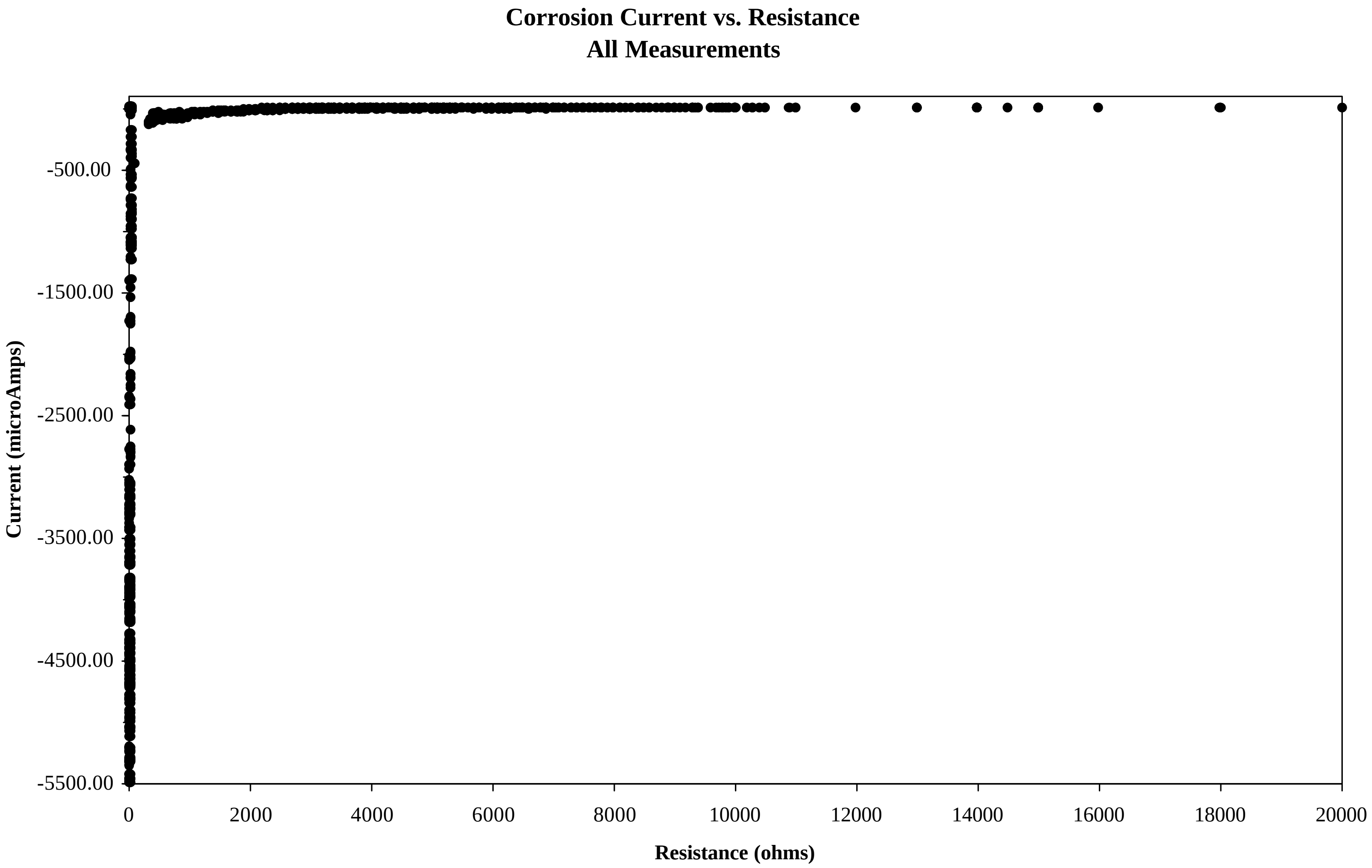


Figure 5.37 Corrosion Current versus Resistance, All Measurements

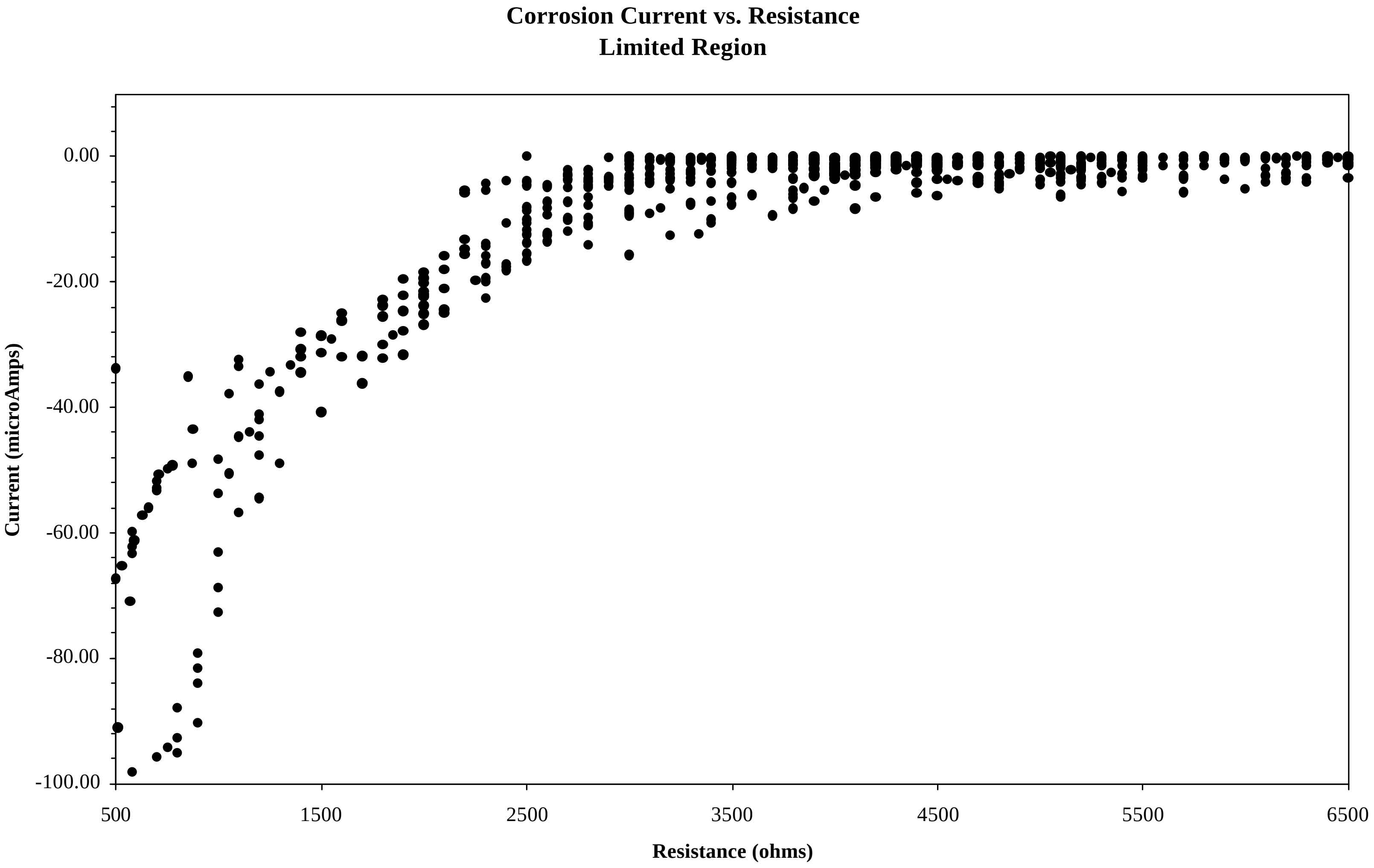


Figure 5.38 Corrosion Current versus Resistance, Limited Region

CHAPTER 6 – SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

6.1 Summary

The primary objective of this study was to evaluate the performance of epoxy-coated reinforcement in Indiana with respect to corrosion. In order to complete this task, a field evaluation and a series of laboratory tests were performed. The field evaluation consisted of four major steps: a mail survey of corrosion protection methods used by Indiana and other states, holiday testing at bridge construction sites, initial visual inspections of one hundred twenty three bridge decks in Indiana, and a second detailed inspection of six bridge decks in Indiana. The laboratory evaluation consisted of exposure testing of concrete specimens reinforced with uncoated and epoxy-coated reinforcement over a period of 77 weeks.

6.1.1 Field Evaluation

- The fifteen respondents to the state department of transportation survey show that epoxy-coated reinforcement is the most common method of corrosion protection as thirteen states responded positively. The use of modified concrete overlays and increased depth of cover were the second most common methods as eleven respondents indicated they used these methods. The earliest reported application of epoxy-coated rebar in bridge decks was 1976 in Indiana. All states responding to the survey indicated that the bridge decks containing epoxy-coated bars have performed satisfactorily to date.

- The results of the holiday testing conducted in this study showed that after delivery to the site, placement, and the casting operation was conducted, an average top mat epoxy-coated rebar would contain approximately 40 holidays per meter of bar (12 holidays per foot of bar). The majority of the holidays were created during the casting operation due to the pump method of concrete placement. In this casting procedure, the concrete was allowed to drop vertically at high speeds on the top mat of reinforcement. A follow-up field evaluation showed that an increase on average of 0.102 mm (4 mils) in the thickness of the epoxy coating reduced the number of defects by an average of 73%. Lowering the pump to reduce the vertical drop of the concrete also reduced the number of holidays created by an average of 50%.
- Initial inspections of 123 bridge decks in Indiana indicated corrosion related distress in 44% of the bridge decks surveyed. The initial field investigations also found that 35% of the bridge decks surveyed had an average cover reading below the specified design value. However, an average cover reading below the specified design cover did not always correspond to corrosion related distress of the bridge deck. Some of the bridge decks containing epoxy-coated reinforcement had an average cover reading below the specified design cover without observed corrosion related distress. However, in bridge decks containing uncoated reinforcement, corrosion related distress was commonly observed on bridge decks with an average cover reading below the specified design cover. Epoxy-coated reinforcement combined with Class C concrete provided the most successful corrosion protection method as only 11% of

the bridge decks in this category showed distress. This percentage was the lowest of all categories of corrosion protection methods. Uncoated reinforcement and a design cover of 25.4 mm (1.0 in.) of Class C concrete with 38.1 mm (1.5 in.) of latex modified overlay was the second least successful corrosion protection method. In this category, 52% of the bridge decks showed signs of distress. The range of age of construction for this method is comparable to that of epoxy-coated bridges. The worst corrosion protection method was uncoated reinforcement with 50.8 mm (2 in.) of Class A concrete with 71% of the bridge decks showing signs of distress. Class A concrete is no longer used in bridge deck construction in Indiana.

- The detailed field investigation of six bridge decks in Indiana found corrosion of epoxy-coated reinforcement in areas of cracking and insufficient concrete cover. In the area of cracking in bridge structure #331-50-6608 where corrosion of the epoxy-coated reinforcement was found, no delaminations were present to indicate any sign of distress. A total of nine cores were removed from bridge decks containing epoxy-coated reinforcement and corrosion was discovered in two of these cores. The coating on the reinforcement from these two cores was debonded, easy to remove, and underfilm corrosion was observed. The coating on the reinforcement was also debonded in one additional core, but no corrosion of the reinforcement was observed. The detailed field investigation also discovered the lowest level of chloride concentrations in bridge structure #6-50-5187, which contained a latex modified overlay. The average chloride concentration at the level of reinforcement was 0.5 kg/m^3 (0.8 lb/yd^3), which is below the threshold level for initiation of corrosion. However, the level of distress, 31% of the surveyed area, is much greater then that found in the three bridges with epoxy-coated reinforcement of the same vintage or older, located within the same county.

6.1.2 Laboratory Evaluation

- Corrosion of the uncoated reinforcement in the specimens with 25.4 mm (1.0 in.) of cover began within the first week of exposure. The plastic settlement cracks in the specimens with 25.4 mm (1.0 in.) of cover allowed the chlorides to reach the reinforcement as soon as the sodium chloride solution was applied.
- The additional cover and the absence of cracks in the specimens with uncoated reinforcement and 63.5 mm (2.5 in.) of cover delayed the initiation of corrosion in these specimens until the 25th week of exposure.
- A difference in the corrosion current between the two different concrete cover dimensions with uncoated reinforcement was observed. The specimens with only 25.4 mm (1 in.) of cover had corrosion currents that stabilized at around 5000 μA initially and dropped to 8000 μA towards the end of the exposure period. However, the specimens with 63.5 mm (2.5 in.) of cover had corrosion currents of a maximum of 4000 μA.
- Epoxy-coated reinforcement had little if any corrosion activity. Only four of the specimens developed active corrosion current over the exposure period. The level of corrosion current was approximately fifty times lower than the current of the specimens with uncoated reinforcement. Three of the four specimens with active corrosion current activity have reinforcement with 39.4 defects per meter (12 defects

per foot), and 152.4 μm (6 mils) of coating thickness. The fourth specimen contains reinforcement with 19.7 defects per meter (6 defects per foot) and 152.4 μm (6 mils) of coating thickness.

- The specimens with uncoated reinforcement had resistance readings two orders of magnitude lower than the specimens containing epoxy-coated reinforcement. The specimens with 25.4 mm (1 in.) of cover had resistance readings higher than the specimens with 63.5 mm (2.5 in.) of concrete cover. Specimens with more damage to the epoxy coating had lower resistance readings. No difference in resistance readings due to thickness of epoxy coating was observed.
- The half-cell potential readings for the uncoated reinforcement correlated well with the corrosion current readings.
- The forensic investigation of the specimens found uniform corrosion on the uncoated reinforcement removed from the specimens. Some cross-section loss of the uncoated reinforcement removed from the specimens with 25.4 mm (1 in.) of cover was observed. Only minor rusting of the epoxy-coated reinforcement removed from the specimens had occurred. No cross-section loss of the epoxy-coated reinforcement was observed.
- Analysis of corrosion current measurements and AC mat to mat resistance indicated that once the chlorides penetrate to the level of reinforcement, the corrosion current is dependent on the resistance between the two mats of reinforcement.
- The use of uncoated reinforcement provided a low mat to mat resistance, which did not impede corrosion current.

- Values of mat to mat resistance greater than 5000 ohms provided excellent resistance to corrosion current.
- High values of resistance were provided by epoxy-coated reinforcement with few defects.
- A statistical analysis was performed that indicated the number of defects in the epoxy coating was one of the most critical variables affecting the mat to mat resistance of epoxy-coated reinforcement.

6.2 Conclusions

6.2.1 Field Evaluation

- Increasing the thickness of epoxy-coated reinforcement will dramatically decrease the damage created to the bars during the bridge deck casting operation. The field evaluation found that an increase on average of 0.102 mm (4 mils) in the thickness of the epoxy coating reduced the number of defects incurred in the concrete casting operation when using the pump method by an average of 73%. Lowering the pump to reduce the vertical drop of the concrete also reduced the number of holidays created by an average of 50%.
- Epoxy-coated reinforcement combined with Class C concrete provided the most successful corrosion protection method as only 11% of the bridge decks inspected in this category during the initial bridge inspections showed signs of corrosion distress.
- Uncoated reinforcement and a design cover of 25.4 mm (1.0 in.) of Class C concrete and 38.1 mm (1.5 in.) of latex modified overlay was not an effective corrosion

protection method as 52% of the bridge decks inspected in this category during the initial bridge inspections showed signs of corrosion distress.

- Cracking and insufficient concrete cover may decrease the effectiveness of epoxy-coated reinforcement as a corrosion protection method. Corrosion of the epoxy-coated reinforcement was discovered during the detailed bridge inspection in areas of cracking and shallow cover.

6.2.2 Laboratory Evaluation

- Increasing the thickness of the epoxy coating will decrease the damage created to the coating in the transportation and fabrication process. Data from the laboratory evaluation showed that an increase of approximately 50.8 μm (2 mils) in coating thickness decreased the defects incurred during the transportation and fabrication process by 85%.
- Cracking allows chlorides to penetrate to the level of the reinforcement immediately upon saltwater application and permits more oxygen to reach the reinforcement, which accelerated the corrosion process in the specimens with uncoated reinforcement.
- When compared to uncoated reinforcement epoxy-coated reinforcement provided excellent corrosion protection even in cracked concrete with a high level of damage to the coating.
- The difference in the corrosion current levels between the two different concrete cover dimensions with uncoated reinforcement is most probably due to the level of oxygen available. The thicker concrete cover with no cracking limits the availability

of oxygen. This lack of oxygen slowed the cathodic reaction, which in turn limited the rate of the anodic (corrosion of the steel) reaction.

- The epoxy coating also limits the oxygen provided for the cathodic reaction, but more importantly, the coating increases the AC mat to mat resistance dramatically.
- To prevent macrocell corrosion a practical approach is to increase the resistance between the mats of reinforcement. A high mat to mat resistance will impede corrosion current. Epoxy-coated reinforcement with fewer defects provides a high mat to mat resistance.

6.2.3 General

The results of the laboratory evaluation indicate that corrosion of epoxy-coated reinforcement can be prevented with a high mat to mat resistance. A high mat to mat resistance can be provided by the use of epoxy-coated reinforcement with limited damage to the coating. The results of the field evaluation show that excessive damage to epoxy-coated reinforcement is being created in the bridge deck construction operations. Both the field and the laboratory evaluations show that a thicker epoxy coating will limit the amount of damage to the coating, which will increase the mat to mat resistance when utilizing epoxy-coated reinforcement.

6.3 Recommendations

Based on the results of this study, it is recommended that an increase of 152.4 μm (6 mils) to the minimum coating thickness be implemented for use in bridge deck steel reinforcement. This increase implies an allowable range of 304.5 to 457.2 μm (12 to 18 mils). It is anticipated that the increase of only 152.4 μm (6 mils) in coating thickness will not adversely affect the bond performance. The use of a thicker coating will significantly decrease the damage to the epoxy coating, which will increase the effectiveness of epoxy-coated reinforcement as a corrosion protection method.

A second recommendation is to reevaluate the field performance of bridge decks with epoxy-coated reinforcement at a future time. The future performance of these bridge decks cannot be predicted by this study and a second field investigation will provide more information concerning the additional service life achieved through the use of epoxy-coated reinforcement.

6.4 Additional Research

This study illustrated that epoxy coatings currently manufactured and used can be easily damaged. Future research into alternate, more durable coatings is justified. Also, the use of thicker epoxy coatings, which is recommended in this study, and their effect on the behavior of concrete bridge decks warrants further research. This research should focus on the bond performance of bars with coating thickness up to 18 mils. Although the recommended range of allowable coating thickness will likely lead to the production of coatings at the lower limit of the range for economic reasons, the upper thickness limit should also be investigated. Additionally, the effect of the thicker coating on both cracking and deflections of concrete members is required.

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LIST OF REFERENCES

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APPENDICES

APPENDIX A

APPENDIX A

A.1 STATE SURVEY FORM

METHODS OF CORROSION PROTECTION QUESTIONNAIRE

State: ________________

Questionnaire Completed by:

Name: ________________
Title: ________________
Organization: ________________
Department: ________________
Address: ________________
____
Phone: ________________
Fax: ________________

1. What methods of corrosion protection are currently used in bridge decks in your region? Please check the methods that apply. In the space provided beside each method, please specify whether the method is used only in new construction or rehabilitation of existing bridge decks or both.

a. ☐ Cathodic Protection ________________
b. ☐ Corrosion Inhibiting Admixtures ________________
c. ☐ Modified Concrete Overlays (Please Specify Type) ________________
d. ☐ Membranes/Epoxy Overlays ________________
e. ☐ Type K Cement ________________
f. ☐ Epoxy Coated Rebar ________________
g. ☐ Increased Depth of Concrete Cover ________________
h. ☐ Others (Please specify, include combination of the above also) ________________
____

2. Which of the methods checked above are currently in your specifications or standard drawings?

____
____

3. Why were these methods chosen?

____
____

____
____
____

4. What criteria have you used in your choice of all corrosion protection methods? Please check the criteria that apply, and make any comments in the space provided.

a. ☐ Economics (unit cost?) _________________________
b. ☐ Performance in the lab _________________________
c. ☐ Previous Research / Finding _________________________
d. ☐ Manufacturer's information _________________________
e. ☐ Site Selection _________________________
f. ☐ State Experience _________________________
g. ☐ Purpose of Rehab _________________________
h. ☐ Others (Please Specify) _________________________
____

5. How well are all the corrosion protection methods performing and how long have they been in place?

____
____
____
____
____
____
____

6. What criteria is used to evaluate the performance of all the corrosion protection systems?

____
____
____
____
____
____
____

7. What is the frequency of the bridge deck inspection?

____
____

A.2 STATE SURVEY RESULTS

1. What methods of corrosion protection are currently used in bridge decks in your region?

Indicate new or rehab. construction.

a. Cathodic Protection:

Arkansas:	Don't Use
Florida:	Don't Use
Indiana:	Rehab. of existing decks
Iowa:	Three experimental bridges
Kansas:	Don't Use
Maine:	Don't Use
Michigan:	Three experimental bridges
Missouri:	Rehab. only
Montana:	Two mid 80's installations not monitored
New York:	Experimental only
South Carolina:	Don't Use
Texas:	Rehab of one structure, experimental
Vermont:	Don't Use
West Virginia:	Rehab.
Wyoming:	Don't Use

b. Corrosion Inhibiting Admixture:

Arkansas:	Don't Use
Florida:	Don't Use
Indiana:	New Construction, experimental
Iowa:	A few selected bridges
Kansas:	Don't Use
Maine:	New Construction, precast slabs
Michigan:	Don't Use
Missouri:	Don't Use
Montana:	Don't Use
New York:	Experimental only
South Carolina:	New Construction
Texas:	New Construction, experimental
Vermont:	Don't Use
West Virginia:	Experimental only
Wyoming:	Rehab.

c. Modified Concrete Overlays:

Arkansas:	Don't Use
Florida:	Don't Use
Indiana:	New construction and rehab, latex,

	modified, silica fume
Iowa:	Rehab with dense concrete overlays
Kansas:	New construction and rehab, 1.5" silica fume overlay
Maine:	New and rehab., silica fume wearing surfaces
Michigan:	Latex, silica fume
Missouri:	Latex modified, low slump, silica fume, thin fiber reinforced, Gemcrete
Montana:	New construction and rehab, latex modified, high density
New York:	New construction and rehab, microsilica, low slump, latex
South Carolina:	Don't Use
Texas:	Rehab, Dense concrete overlays
Vermont:	Don't Use
West Virginia:	New and Rehab, latex modified concrete
Wyoming:	Rehab., latex and silica fume

d. Membranes / Epoxy Overlays:

Arkansas:	Don't Use
Florida:	Don't Use
Indiana:	New construction, experimental
Iowa:	Don't Use
Kansas:	Don't Use
Maine:	Don't Use
Michigan:	Very few rehab., membrane with asphalt
Missouri:	Some membranes in place don't use anymore. Epoxy concrete overlays.
Montana:	Epoxy overlays, MMA overlays
New York:	Rehab.
South Carolina:	Don't Use
Texas:	Don't Use
Vermont:	New construction and rehab.
West Virginia:	Membranes 20 years ago
Wyoming:	Rehab., Sheet membrane with asphalt overlay

e. Type K Cement:

Arkansas:	Don't Use
Florida:	Don't Use
Indiana:	New construction, experimental
Iowa:	Don't Use
Kansas:	Don't Use

Maine:	Don't Use
Michigan:	Some new construction
Missouri:	Don't Use
Montana:	Don't Use
New York:	NYSDOT - No, Sister State Agency NYS Thruway Authority - has
South Carolina:	Don't Use
Texas:	Don't Use
Vermont:	Don't Use
West Virginia:	Don't Use
Wyoming:	Don't Use

f. Epoxy Coated Rebar

Arkansas:	New construction
Florida:	Don't Use
Indiana:	New construction
Iowa:	New construction
Kansas:	New construction and rehab.
Maine:	Don't Use
Michigan:	New construction and Top layer rehab.
Missouri:	All superstructure steel and caps under open joints
Montana:	Both top and bottom mats
New York:	New construction and rehab.
South Carolina:	New construction
Texas:	New construction and rehab.
Vermont:	New construction
West Virginia:	All construction
Wyoming:	New construction and rehab.

g. Increased Depth of Concrete Cover:

Arkansas:	Don't Use
Florida:	Don't Use
Indiana:	63.5 mm (2.5 in.)
Iowa:	New construction requires 63.5 mm (2.5 in.)
Kansas:	New construction – 76.2 mm (3 in.)
Maine:	Don't Use
Michigan:	New and rehab., 76.2 mm (3 in.)
Missouri:	76.2 mm (3 in.) minimum top mat
Montana:	60.3 mm (2.375 in.) minimum cover
New York:	New construction – 82.6 mm (3.25 in.)

South Carolina:	New construction
Texas:	Don't Use
Vermont:	New construction
West Virginia:	New construction
Wyoming:	New construction

h. Others - Please specify, include combination of the above also

Arkansas:	None
Florida:	None
Indiana:	All new: (a) epoxy-coated rebar w/ 63.5 mm (2.5 in.) (b) Super plasticized concrete with epoxy-coated rebar (HRWR) (c) Transverse post-tensioning in the deck (d) Silica fume modified concrete
Iowa:	None
Kansas:	c, f, and g used together
Maine:	New construction – membrane waterproofing system for many deck slabs
Michigan:	None
Missouri:	All CP bridges have either AC or PC overlays
Montana:	Have also tried HMWM's and silanes
New York:	f and g used together
South Carolina:	None
Texas:	Rehab. ACP with seal coat membrane. Deck sealers - silane, siloxane, linseed oil
Vermont:	None
West Virginia:	Microsilica, microlite, thorotop overlays
Wyoming:	Rehab. methylmethacrylate overlay

2. Which of the methods checked above are currently in your specifications or standard drawings

Arkansas:	f
Florida:	None
Indiana:	Can be found in standards specifications or contract special provisions
Iowa:	f and g
Kansas:	c, f, and g

Maine:	b, c, and h
Michigan:	c, f, and g
Missouri:	a, c, 4, f, and g
Montana:	c, d, and f
New York:	c, f, g, and h
South Carolina:	b and f
Texas:	c, f, and h
Vermont:	d, f, and g
West Virginia:	b, c, and f
Wyoming:	c, d, f, and g

3. Why were these methods chosen?

Arkansas:	
Florida:	
Indiana:	a, c, f, g chosen based on other states' experiences and our own. b, d, e chosen based on experimental basis
Iowa:	Most effective and economical
Kansas:	Dense overlay inhibits water from reaching steel, extra clearance buys time, when water does reach steel, epoxy coating protects it
Maine:	We use DCI-S by W.R. Grace for corrosion inhibitor. We hold the opinion that it adds protection from high chloride intrusion. Silica fume and membrane systems have proven successful.
Michigan:	Durability, Cost, and Ease of Application
Missouri:	Proven reliability over several years. Design considerations - thin fiber reinforced concrete and epoxy concrete to keep from reducing barrier height.
Montana:	They seem to be the best of current commercially available products. We have enough experience with them to put them in as standard procedures.
New York:	Ease of construction, seven reports of problems with epoxy coatings
South Carolina:	
Texas:	Corrosion protection for decks is necessary in northern portion of state where deicing salts are used.
Vermont:	Membranes have been proven effective under bituminous overlays, other methods

	have proven performance
West Virginia:	Results of experimentation
Wyoming:	

4. What criteria have you used in your choice of all corrosion protection methods?

Arkansas:	
Florida:	
Indiana:	a, d, e, f, and g
Iowa:	a, c, and f (Testing has shown epoxy bars to be effective)
Kansas:	f (25 + years experience with low slump)
Maine:	c, d, and f
Michigan:	a, c, f, and g
Missouri:	a. (Epoxy concrete, quick cheap way to prolong deck life) b. (Most materials tested in lab first, epoxy concretes for example) c. (Used findings from other state DOT's , especially silica fume) d. (Used in evaluation but also try to verify in our lab, or check other states) e. (Special bridges like orthotropic steel deck needed special system - Transpo.) f. (Most methods of CP started off as experimental projects and after experience, put in specifications, others (membranes) taken out.
Montana:	a, c, e, and g
New York:	a, c, d, f, and g (Desire for a concrete riding surface - avoids rutting and joint shove
South Carolina:	d and f
Texas:	a. (Cathodic protection is too expensive, Texas can replace decks for ~ $7 / s.f.) c. (Epoxy coated rebar) d. (Corrosion inhibitors) f. (Linseed Oil) g. (Deck study showing - Lower water cement ratio, increased clear cover, increased deck thickness, deck sealer, increase curing requirements, limit loading of structure until 21 day cure)
Vermont:	a, c, and f
West Virginia:	b, c, f, and g

Wyoming:	c, d, e, f, and g

5. How well are all the corrosion protection methods performing and how long have they been in place?

Arkansas:	
Florida:	
Indiana:	
Iowa:	Epoxy coated bars have been standard since 1976
Kansas:	Epoxy rebar used in decks since early 80's. Silica fume overlay used since 1994. Not a lot of history on the silica fume overlay, but it appears to be performing well.
Maine:	Silica Fume wearing surfaces were started in 1986. DCI was first used in 1980. Membrane systems - 25+ years
Michigan:	Overlays - some cracking but expect 15 year life. Since 1975 +/- Epoxy Coated Reinf. - since 1980 +/- - Our investigation shows epoxy performing satisfactorily to date 76.2 mm (3 in.) cover - some cracking from much cement and epoxy reinf. Night pours seem to work best. - Since 1975 +/-
Missouri:	(a) 16 years, 120 bridges, only one documented failure, some anode problems with carbon based anodes but no failures of overlay system on top. (c) latex modified, low slump and silica fume, 4-19 years experience, some cracking and debonding of latex and low slump after 10 years. (d) Membranes - 20 years plus, kept chlorides out but shoved and caused AC overlay failures. (e) Epoxy concrete - 5 years, some debonding caused by poor construction practices, some loss of friction properties. (f) top mat since 1978, both mats since 1985, have tested several bridges and found no failure (g) made 76.2 mm (3 in.) cover since 1978

	when epoxy steel first used.
Montana:	Latex and High density - since 1979 or 1980 - seems to work well. We don't really know what the cathodic systems are doing. Epoxy coated rebar – extensive use since 1984, no bad reports yet. Silane sealer - a couple of projects in late 80's and a few more later - no bad experiences but we think HMWM is better. All HMWM's since 91 or 92. MMA overlays - nothing over a couple years old.
New York:	We have used modified concrete overlays since 1975. Our first overlays were modeled after the work in Iowa. We have since revised removal criteria and cold joint preparation. Performance has been good, especially when total top mat exposure is used. Epoxy coated bars have been standard since 1977, no performance problems. We are planning a statistically valid sampling and evaluation of in service epoxy rebars specifically to look for coating debonding predicted by Ken Clear.
South Carolina:	Do not know, 3 to 5 years
Texas:	Epoxy-coated steel has been used since the early 80's. Linseed Oil has been used since the late '60's. Silane and siloxanes have been used only for several years. Corr. Inhibitors have been used for substructure and deck construction recently (experimental at this time) ACP and seal coat have been used to extend life of bridge decks since the 1960's.
Vermont:	All are somewhat effective and have been in place 10 to 20 years.
West Virginia:	Very good - 20 years
Wyoming:	b - First installation this summer c - Latex, 15-20 years, good Silica fume, 5 yrs, fair d - Some hot applied membranes, 20+ years, good. Newer cold applied membranes, 5+ years, good f - 14 years, good so far g - 15 years, good so far h - 3 to 5 years, good so far

6. What criteria is used to evaluate the performance of all the corrosion protection systems?

Arkansas:
Florida:
Indiana: 1. Chloride sampling 2. half cell potential readings
Iowa: Biannual visual inspections for signs of deterioration
Kansas: Deck Reports - delaminations, potentials, % chlorides, crack sketches
Maine: Chloride values taken during construction and again after 5 years. All condition of the structure as time passes.
Michigan: Amount of Spalls and Cracks
Missouri: 1. Yearly 4 hour depolarization test and deck performance.
2. Membrane waterproof and resistance.
3. Latex, low slump, silica fume, thin fiber salt scale, freeze-thaw, chloride permeability
4. Epoxy-salt scale and penetration
5. Epoxy rebars - holidays, mill thickness and effect of electromagnetic testing on mill thickness.
Montana: Do they work? A simple yes or no. The problem is we have to wait 20 or 30 years to find out. Even a nominal extension of the deck life is worth an investment at the beginning of a project.
New York: Electrical half-cell potentials and the lack of spalling and delaminations.
S. Carolina: No written criteria
Texas: Site inspections, research projects Texas is currently investigating performance of epoxy coated rebar and deck sealers through sponsored research.
Vermont: Visual inspection, half cell potential readings, and during construction, soundings
W. Virginia: Absence of deck deterioration
Wyoming: Fairly subjective observation, maintenance history

7. What is the frequency of the bridge inspection?

Arkansas:
Florida: Every two years
Indiana: Minimum of once every two years by district forces
Iowa: Every two years

Kansas: Every 4 years by bridge staff
Maine: Two year minimum, more often if special need
Michigan: Varies 1 to 2 years, up to 5 years
Missouri: Every year, special inspections more often
Montana: 2 or 4 year intervals
New York: Two year maximum interval
South Carolina: National Bridge Inspection Guidelines
Texas: Every two years
Vermont: Every two years
West Virginia: Every two years
Wyoming: Every two years, maintenance on ongoing basis

APPENDIX B

APPENDIX B

Table B.1 Results from Initial Bridge Inspections

#	Structure #	District	Date Inspected	Method of Corrosion Protection	Type of Structure	Skew	Spans In Meters	Lane Width m (ft.)	Adt X 100	Year Built (Deck)	% Area of Spalls in Survey Area	% Area of Delamination/ Debonding in Survey Area	% Area of Patches in Survey Area	% Total Distress Area	Rust Stains in Survey Area	Cracking in Survey Area	Design Cover mm (in.)	Cover Avg. mm (in.)	INDOT Deck Rating	Notes
8	037-047-5980	Vincennes	22-Aug-96	Black Rebar with Class A Concrete	Cont Comp. St. Beam	VA	2@17.2, 30, 28.4	12.2 (40)	51	1973	0.21	21.91	9.51	31.6315		Trans. cracks every .9 – 1.2 m (3 – 4 ft), Long. crack length of bridge	50.8 (2)	44.45 (1.75)	N/A	
17	I65-110-5693	Greenfield	1-Jul-96	Black Rebar with Class A Concrete	Cont. Steel Beam	3	10.8, 13.8, 10.8	21.9 (71.8)	297	1973	0.2	25		25.22	Stains around cracks	Transverse cracks every 1.5 – 1.8 m (5 – 6 ft)	50.8 (2)	42.16 (1.66)	7	
6	023-71-5116	Laporte	30-May-96	Black Rebar with Class A Concrete	Cont. Reinf. Conc. Slab	10	8.2, 11, 8.2	13.41 (44)	48	1973		21	0.076	21.145			50.8 (2)		5	Rebar exposed on underside edge of deck
30	446-47-5819	Vincennes	25-Jun-96	Black Rebar with Class A Concrete	Cont. Steel Beam	30	2@13.7, 16.5	13.9 (45.7)	10	1974	0.038	20.77		20.838	Stains around cracks	Long. cracks in delaminated areas	50.8 (2)	37.85 (1.49)	7	
11	037-53-2440	Seymour	23-Jul-96	Black Rebar with Class A Concrete	Comp. Steel Beam	30	2@10.7, 13.7	12.6 (41.3)	62	1973	0.631	17.76		18.98	Stains in area of delam.	Occasional trans. cracking, Long. cracking in every area of delam.	50.8 (2)	40.13 (1.58)	6	
7	031-50-2451	Laporte	24-May-96	Black Rebar with Class A Concrete	Cont. Steel Beam		13.1, 15.7, 13.1	12.2 (40)	61	1973	0.255	15.7	0.364	16.392	Many stains along cracks	Many small transverse and long. cracks	50.8 (2)	43.43 (1.71)	N/A	Three cores removed from bridge
22	035-09-4706	Laporte	22-May-96	Black Rebar with Class A Concrete	Comp Cont. St. Beam	26	2@27.3	13.41 (44)	30	1974		10.9		10.9	Small Stains	Long. Crack in every Delam. Area, small trans. Cracks every 10'	50.8 (2)	43.43 (1.71)	7	
15	I65-110-5622	Greenfield	1-Jul-96	Black Rebar with Class A Concrete	Comp Cont. Steel Beam	20	12.5, 3@21, 15.1	15.7 (51.5)	297	1973	0.21	5.57		6.32	Stains around cracks	Transverse Cracks every .9 – 1.2 m (3 – 4 ft)	50.8 (2)	53.09 (2.09)	7	
16	I65-110-5692	Greenfield	1-Jul-96	Black Rebar with Class A Concrete	Cont. Comp. Steel Beam	18	10.4, 19, 10.4	15.9 (52.3)	297	1973	0.13	5.45		5.58	Some	Transverse Cracks every .9 – 1.2 m (3 – 4 ft)	50.8 (2)	60.45 (2.38)	7	
29	446-47-5818	Vincennes	25-Jun-96	Black Rebar with Class A Concrete	Cont. Steel Beam	10	15.8, 19.2, 11.5	13.41 (44)	9	1974	0.134	4.9		5.079		Occasional Transverse Cracks	50.8 (2)	46.48 (1.83)	7	
18	I65-110-2422	Greenfield	1-Jul-96	Black Rebar with Class A Concrete	Cont. Steel Beam		16.2, 25.1, 16.2	15.8 (52)	364	1973	0.07	4.6		4.74	Stains around cracks	Transverse cracks every 1.2 – 1.5 m (4 – 5 ft)	50.8 (2)	65.79 (2.59)	6	
26	045-07-6144	Seymour	27-Jun-96	Black Rebar with Class A Concrete	Cont. Reinf. Conc. Slab	30	13.7, 18.3, 13.7	13.5 (44.2)	17	1974	0.604	4.13		4.736		One long. crack length of bridge, Transverse Cracks over Supports	50.8 (2)	52.58 (2.07)	6	
1	I265-02-2404	Seymour	15-Jul-96	Black Rebar with Class A Concrete	Cont. Steel Beam	35	15.3, 20.9, 15.3	14.35 (47.1)	127	1972	0.843	3.3		4.143			50.8 (2)	43.43 (1.71)	7	
3	I265-5-5642	Seymour	15-Jul-96	Black Rebar with Class A Concrete	Cont. Steel Beam	24	15.8, 18.6, 15.8	12.2 (40)	101	1972		2.61		2.701		Few Transverse cracks over supports	50.8 (2)	54.86 (2.16)	7	
10	037-53-2439	Seymour	23-Jul-96	Black Rebar with Class A Concrete	Comp Cont. Steel Beam	11	21.3, 3@26.8, 17.4	12.2 (40)	62	1973	0.169	1.02		1.574	Stains in area of delam.	Trans. cracking over supports, Long. cracking in every area of delam.	50.8 (2)	39.88 (1.57)	6	
14	I65-109-5691	Greenfield	1-Jul-96	Black Rebar with Class A Concrete	Comp Cont. Steel Beam	19	2@23.3	15.7 (51.5)	297	1973		0.53		0.53		Trans. cracks over supports	50.8 (2)	48.51 (1.91)	7	

Table B.1 (continued)

#	Structure #	District	Date Inspected	Method of Corrosion Protection	Type of Structure	Skew	Spans In Meters	Lane Width m (ft.)	Adt X 100	Year Built (Deck)	% Area of Spalls in Survey Area	% Area of Delamination/ Debonding in Survey Area	% Area of Patches in Survey Area	% Total Distress Area	Rust Stains in Survey Area	Cracking in Survey Area	Design Cover mm (in.)	Cover Avg. mm (in.)	INDOT Deck Rating	Notes
25	044-81-6077	Greenfield	11-Jun-96	Black Rebar with Class A Concrete	Cont. Prestr. I - Beam		3@24.9, 2@24.7	13.43 (44.1)	33	1974	0.022	0.34		0.409	Few Small Stains	Occasional Transverse Cracks	50.8 (2)	67.31 (2.65)	7	
4	(19)21-52-6001	Fort Wayne	14-Jun-96	Black Rebar with Class A Concrete	Cont. Prestr. Box-Beam		1.7, 16.9, 16.7	13.41 (44)	14	1973	0.04	0.25		0.29		Occasional Transverse Cracks	50.8 (2)	49.53 (1.95)	6	
28	135-31-5657	Seymour	15-Jul-96	Black Rebar with Class A Concrete	Cont. Steel Girder	4	23.8, 2@27.4, 19.1	2@12.8 (2@42)	171	1974	0.015	0.137		0.167		Transverse cracks over supports	50.8 (2)	49.02 (1.93)	7	
12	145-62-5635	Vincennes	25-Jun-96	Black Rebar with Class A Concrete	Cont.Comp. Steel Girder	4	2@32.6	13.41 (44)	8	1973		0.157		0.157		Transverse cracks every 1.5 – 1.8 m (5 – 6 ft)	50.8 (2)	51.82 (2.04)	7	
2	I265-04-5518	Seymour	15-Jul-96	Black Rebar with Class A Concrete	Cont.Comp. Plt Girder	19	2@37.2	10.36 (34)	245	1972				0			50.8 (2)	47.50 (1.87)	7	
5	023-71-5328	Laporte	28-May-96	Black Rebar with Class A Concrete	Cont. Prestr. I – Beam	10	9.4, 9.6, 9.4	13.41 (44)	46	1973				N/A			50.8 (2)		7	Deck Replacement
9	037-047-5934	Vincennes	22-Aug-96	Black Rebar with Class A Concrete	Cont.Comp. Steel Beam	15	2@13.4, 2@23.9, 29.3	12.2 (40)	81	1973				N/A			50.8 (2)		N/A	Deck Replacement
13	258-36-6078	Seymour	27-Jun-96	Black Rebar with Class A Concrete	Cont. Reinf. Conc. Slab	30	8.2, 11. 8.2	13.41 (44)	12	1973				0			50.8 (2)	57.91 (2.28)	7	
20	016-91-6043	Laporte	22-May-96	Black Rebar with Class A Concrete	Cont. Steel Beam		16.5, 19.8, 16.5	12.2 (40)	14	1974				N/A			50.8 (2)		N/A	Deck Replacement
21	016-91-6044	Laporte	22-May-96	Black Rebar with Class A Concrete	Cont. Steel Beam		21.3, 26.5, 21.3	12.2 (40)	14	1974				N/A			50.8 (2)		N/A	Deck Replacement
23	037-41-3976J	Seymour	23-Jul-96	Black Rebar with Class A Concrete	Cont. Reinf. Conc. Slab	10	9.1, 12.8, 9.1	12.24 (40.2)	125	1974				0		Trans. Cracks every .4m (1.5 ft) over supp, Long. Crack length of bridge	50.8 (2)	82.04 (3.23)	6	
24	041-045-2073A	Laporte	21-Nov-95	Black Rebar with Class A Concrete	Cont.Comp. Steel Beam	27	27.3, 3@ 30.5, 24.7, 2@26.8, 30.5, 24.4	2@7.92 (2@26)	177	1974				0			50.8 (2)	64.77 (2.55)	6	
27	054-028-6005	Vincennes	22-Aug-96	Black Rebar with Class A Concrete	Cont. Plate Girder	45	2@25.5, 30.6	13.41 (44)	45	1974				N/A			50.8 (2)		N/A	Deck Replacement
31	I64-100-5688	Seymour	15-Jul-96	Black Rebar with Class A Concrete	Cont. Steel Girder		2@36.3, 45.1	12.17 (39.9)	52	1974				N/A			50.8 (2)		N/A	Deck Replacement
32	I64-103-5654	Seymour	15-Jul-96	Black Rebar with Class A Concrete	Prestr Conc. I – Beam	7	2@11.6, 11.9	11.99 (39.3)	52	1974				0			50.8 (2)	78.49 (3.09)	7	
33	I64-105-5656	Seymour	15-Jul-96	Black Rebar with Class A Concrete	Cont.Comp. Steel Beam		2@32	10.39 (34.1)	105	1974				0			50.8 (2)	35.31 (1.39)	7	
34	I64-108-5658	Seymour	15-Jul-96	Black Rebar with Class A Concrete	Reinf. Conc. Girder and Steel Pl.	24	11, 2@ 31.4, 39.6	12.04 (39.5)	88	1974				N/A			50.8 (2)		N/A	Deck Replacement
35	I64-108-5660	Seymour	15-Jul-96	Black Rebar with Class A Concrete	Cont.Comp. Steel Beam	21	2@28, 15.5, 8.5	9.75 (32)	176	1974				0			50.8 (2)	59.94 (2.36)	7	
63	066-074-5801	Vincennes	27-Aug-96	Black Rebar with Class A Concrete	Cont. Prestr. I – Beam	30	2@16.7, 3@16.9	12.2 (40)	42	1976				0		Occasional Transverse Cracks	50.8 (2)	58.42 (2.3)	7	
48	I275-000-5639	Seymour	10-Sep-96	Black Rebar with Class AA Concrete	Cont. Steel Girder		3@34.1, 4@33.8, 34.7	2@9.14 (2@30)	149	1974		2		2		Trans. Cracking over Supports, Occasional Long. Cracks	50.8 (2)	61.47 (2.42)	7	Northern most exp. joint open very wide 101.6 mm (4 in.)

Table B.1 (continued)

#	Structure #	District	Date Inspected	Method of Corrosion Protection	Type of Structure	Skew	Spans In Meters	Lane Width m (ft.)	Adt X 100	Year Built (Deck)	% Area of Spalls in Survey Area	% Area of Delamination/ Debonding in Survey Area	% Area of Patches in Survey Area	% Total Distress Area	Rust Stains in Survey Area	Cracking in Survey Area	Design Cover mm (in.)	Cover Avg. mm (in.)	INDOT Deck Rating	Notes
49	032-18-2182	Greenfield	11-Jun-96	Black Rebar with Class C Concrete	Cont. Prestr. I – Beam	15	2@15.2, 24.1, 25.3, 15.2	2@8.8 (2@29)	132	1975	23.59	46.45		70.821		Occasional Transverse Cracks	50.8 (2)	67.56 (2.66)	7	
87	031-71-6792	Laporte	30-May-96	Black Rebar with Class C Concrete	Cont. Prestr. I – Beam		2@14.1, 2@28.2	7.92 (26)	127	1979		3.55		3.55		Trans. cracks ev. 1.2m, (4 ft) w occ. long. cracks	50.8 (2)		7	
19	I65-111-5714	Greenfield	3-Jul-96	Black Rebar with Class C Concrete	Steel Girder	21	2@25.3	13.41 (44)	277	1973		0.4		0.4		Occasional Transverse Cracks	50.8 (2)	62.99 (2.48)	7	
38	I65-112-5728	Greenfield	2-Jul-96	Black Rebar with Class C Concrete	Cont. Prestr. I - Beam	VA	2@14.3, 26.5	19.8,16.8 ,9.4(65, 55, 31)	898	1974				0			50.8 (2)	56.64 (2.23)	8	
88	043-53-6226	Seymour	23-Jul-96	Black Rebar with Class C Concrete	Cont. Prestr. I - Beam		2@10.7, 12.2	13.41 (44)	21	1979				0			63.5 (2.5)	51.31 (2.02)	8	
93	258-36-6407	Seymour	11-Jul-96	Black Rebar with Class C Concrete	Cont. Reinf. Conc. Slab	45	2@9.1, 12.2	13.41 (44)	21	1979				0			76.2 (3)	112.27 (4.42)	7	
96	003-70-6632	Greenfield	10-Jun-96	Black Rebar with Class C Concrete	Cont. Conc. I-Beam	18	2@16.8, 17	13.41 (44)	35	1980				0			63.5 (2.5)	68.07 (2.68)	7	Small patched area near joint
118	252-41-6404	Seymour	12-Jul-96	Black Rebar with Class C Concrete	Cont. Steel Beam		2@22.9, 3@26	10.97 (36)	45	1980				0		Trans. Cracks every .9 - 1.2 m (3–4 ft) over supp.	63.5 (2.5)	95.25 (3.75)	7	
125	049-064-2562	Laporte	25-Jul-96	Black Rebar with Class C and Flexogrid	Cont.Comp. Steel Beam	22	2@16, 20	12.2 (40)	28	1987				0			63.5 (2.5)	70.61 (2.78)	8	
126	049-064-2562	Laporte	25-Jul-96	Black Rebar with Class C and Flexolith	Cont.Comp. Steel Beam	22	2@16, 20	12.2 (40)	28	1987				0			63.5 (2.5)	50.55 (1.99)	8	
124	049-064-2564	Laporte	25-Jul-96	Black Rebar w/ClassC & Wabo Impervadeck	Cont.Comp. Steel Beam	5	2@14, 17.7	12.2 (40)	32	1987				0			63.5 (2.5)	86.87 (3.42)	8	Surface not smooth, spot where membrane visible
100	006-50-5187	Laporte	28-May-96	Black Rebar w/ Class C Conc. & 1.5" LMO	Cont. Prestr. I - Beam		2@13.8, 14	13.41 (44)	77	1980		17.12	11.66	30.59		Occasional Transverse Cracks	63.5 (2.5)	65.53 (2.58)	5	Spalling underside of deck, rebar exposed both edges
65	(231)43-54-5995	Crawfords-ville	10-Oct-95	Black Rebar w/ Class C Conc. & 1.5" LMO	Cont. Plate Girder		2@39.4, 47.3	2@12.3 (2@41)	120	1976		13.4		13.4			63.5 (2.5)	57.40 (2.26)	7	
73	120-76-6606	Fort Wayne	17-Jun-96	Black Rebar w/ Class C Conc. & 1.5" LMO	Cont. Prestr. Box-Beam		2@9.4, 11	13.5 (44.4)	30	1977	0.28	11.4		11.772	Stains around cracks	Transverse cracks every 2.7–3 m (9 – 10 ft)	50.8 (2)	49.02 (1.93)	7	
54	037-29-6047	Greenfield	10-Jun-96	Black Rebar w/ Class C Conc. & 1.5" LMO	Cont. Steel Beam		2@23.5, 3@28.2	13.41 (44)	76	1976	0.087	7.9	1.07	9.127	Small stains near cracks	Transverse cracks every 1.5 – 1.8 m (5 – 6 ft)	63.5 (2.5)	53.59 (2.11)	6	
47	(I65)I70-79-5737	Greenfield	2-Jul-96	Black Rebar w/ Class C Conc. & 1.5" LMO	Cont.Comp. Steel Beam	1	2@22.1	19.9,12.6 (65, 41)	383	1974	0.87	6.4		7.404		Transverse Cracks every .9 – 1.2 m (3 – 4 ft)	63.5 (2.5)	70.10 (2.76)	6	
41	I65-112-5731	Greenfield	2-Jul-96	Black Rebar w/ Class C Conc. & 1.5" LMO	Cont.Comp. Steel Beam		7.6, 19.5, 8.4	19.9,16.8 ,12.6(65. 3,55,41)	898	1974	0.007	7.37		7.381		Occasional Transverse Cracks	63.5 (2.5)	58.42 (2.30)	6	
39	I65-112-2431	Greenfield	2-Jul-96	Black Rebar w/ Class C Conc. & 1.5" LMO	Cont.Comp. Steel Girder	32	18, 38.7, 31.4	19.8,16.8 ,9.4(65, 55, 31)	898	1974	0.38	3.12	1.75	6.77		Transverse cracks over supports	63.5 (2.5)	57.15 (2.25)	6	
66	(231)52-79-5784	Crawfords-ville	21-Sep-95	Black Rebar w/ Class C Conc. & 1.5" LMO	Cont.Comp Steel Beam	64	2@24.2, 48	8.34 (27.5)	67	1976	0.01	5		5.01	Small areas near cracks	Transverse cracks every 3 m (10 ft.)	63.5 (2.5)	52.07 (2.05)	7	
75	912-045-5086	Laporte	21-Nov-95	Black Rebar w/ Class C Conc. & 1.5" LMO	Cont. Steel Beam	VA	19.8, 2@ 28.3,23.2, 25.6, 15.8	7.62, (25)	344	1977		3.2		3.2			63.5 (2.5)	52.32 (2.06)	6	

Table B.1 (continued)

#	Structure #	District	Date Inspected	Method of Corrosion Protection	Type of Structure	Skew	Spans In Meters	Lane Width m (ft.)	Adt X 100	Year Built (Deck)	% Area of Spalls in Survey Area	% Area of Delamination/ Debonding in Survey Area	% Area of Patches in Survey Area	% Total Distress Area	Rust Stains in Survey Area	Cracking in Survey Area	Design Cover mm (in.)	Cover Avg. mm (in.)	INDOT Deck Rating	Notes
37	I65-112-2430	Greenfield	3-Jul-96	Black Rebar w/ Class C Conc. & 1.5" LMO	Cont.Comp. Steel Beam	VA	17.7, 22.6, 15.5	7.32 (24)	151	1974	0.02	2.9		2.92		Transverse cracks every 1.5 – 1.8 m (5 – 6 ft)	63.5 (2.5)	77.22 (3.04)	6	
52	022-34-5998	Greenfield	10-Jun-96	Black Rebar w/ Class C Conc. & 1.5" LMO	Cont. Steel Beam	45	2@17.8, 2@21.9, 22.3	13.41 (44)	29	1976	0.001	2.5		2.61		Transverse cracks every 1.5 – 1.8 m (5 – 6 ft), over supports	63.5 (2.5)	73.66 (2.9)	6	
36	I65-112-5725	Greenfield	1-Jul-96	Black Rebar w/ Class C Conc. & 1.5" LMO	Cont.Comp. Steel Beam	3	8.3, 34, 11.7	19.8, 17 (65, 56)	977	1974	0.11	2		2.14		Occasional Transverse Cracks	63.5 (2.5)	52.32 (2.06)	6	
60	063-086-5979	Crawfords-ville	28-Sep-95	Black Rebar w/ Class C Conc. & 1.5" LMO	Cont. Prestr. I - Beam		3@21.3	12.2 (40)	27	1976		2		2		Transverse Cracks every .9 m (3 ft) over supports	63.5 (2.5)	63.75 (2.51)	7	
62	066-87-5793	Vincennes	27-Aug-96	Black Rebar w/ Class C Conc. & 1.5" LMO	Cont. Steel Girder	20	2@15.8, 2@29.7, 36.6	12.2 (40)	41	1976		0.814		0.814		Trans. cracks following skew in delam. areas	63.5 (2.5)	81.28 (3.20)	7	
53	036-061-6038	Crawfords-ville	5-Oct-95	Black Rebar w/ Class C Conc. & 1.5" LMO	Cont. Prestr. I - Beam	30	2@17.2, 17.4	13.43 (44.1)	32	1976		0.586		0.586		Two Transverse cracks, One long. crack	63.5 (2.5)	51.31 (2.02)	7	
44	I65-112-5734	Greenfield	2-Jul-96	Black Rebar w/ Class C Conc. & 1.5" LMO	Cont.Comp. Steel Beam	VA	2@13.7, 25.1	19.35 (63.5)	462	1974	0.52			0.52		Few Transverse Cracks	63.5 (2.5)	61.47 (2.42)	7	
71	035-046-2160	Laporte	17-Sep-96	Black Rebar w/ Class C Conc. & 1.5" LMO	Cont. Plate Girder		2@32.7, 6@44.9	2@8.5 (2@28)	207	1977	0.08	0.422		0.502	Small stain around crack	Trans cracks every 1.5–1.8 m (5–6 ft), over supp	63.5 (2.5)	57.15 (2.25)	6	
43	I65-112-5733	Greenfield	3-Jul-96	Black Rebar w/ Class C Conc. & 1.5" LMO	Cont.Comp. Steel Beam		10.1, 19.5, 10.1	16.2 (53.2)	449	1974		0.38		0.394		Occasional Transverse Cracks	63.5 (2.5)	84.33 (3.32)	6	
42	I65-112-5732	Greenfield	2-Jul-96	Black Rebar w/ Class C Conc. & 1.5" LMO	Cont.Comp. Steel Beam	VA	10.1, 19.6, 10.1	19.35 (63.5)	449	1974		0.3		0.3			63.5 (2.5)	66.29 (2.61)	6	
58	063-086-2096	Crawfords-ville	28-Sep-95	Black Rebar w/ Class C Conc. & 1.5" LMO	Cont. Steel Beam	9	2@13.4, 16.8	12.2 (40)	27	1976		0.122		0.122		Few Transverse Cracks	63.5 (2.5)	59.44 (2.34)	7	
40	I65-112-5730	Greenfield	2-Jul-96	Black Rebar w/ Class C Conc. & 1.5" LMO	Cont.Comp. Steel Beam		9.4, 24.1, 7.9	20,16.7(65,55)	898	1974	0.005	0.11		0.115		Occasional Transverse Cracks	63.5 (2.5)	47.75 (1.88)	6	
119	257-63-6583	Vincennes	25-Jun-96	Black Rebar w/ Class C Conc. & 1.5" LMO	Cont.Comp. Steel Girder		2@24.7, 3@30.8	13.5 (44.2)	18	1980			0.088	0.088			63.5 (2.5)	57.66 (2.27)	7	
72	047-061-6570	Crawfords-ville	5-Oct-95	Black Rebar w/ Class C Conc. & 1.5" LMO	Precast Seg. Box Beam		2@48.3	13.43 (44.1)	12	1977	0.05			0.05		One long. crack, some transverse cracks	63.5 (2.5)	77.98 (3.07)	7	
61	063-086-2454	Crawfords-ville	28-Sep-95	Black Rebar w/ Class C Conc. & 1.5" LMO	Cont. Steel Beam	20	2@16.2, 18.6	12.2 (40)	27	1976		0.015		0.015		Few Transverse Cracks	63.5 (2.5)	69.09 (2.72)	7	
45	I65-112-5735	Greenfield	2-Jul-96	Black Rebar w/ Class C Conc. & 1.5" LMO	Cont.Comp. Steel Beam	13	2@10.4, 19.8	19,16.8(63,55)	462	1974				0			63.5 (2.5)	73.66 (2.90)	6	
46	I65-112-5736	Greenfield	2-Jul-96	Black Rebar w/ Class C Conc. & 1.5" LMO	Cont.Comp. Steel Beam	20	2@11.3, 20.7	18,15.5(59,52)	462	1974				0		Few Transverse Cracks	63.5 (2.5)	87.38 (3.44)	7	
57	056-39-2469	Seymour	16-Jul-96	Black Rebar w/ Class C Conc. & 1.5" LMO	Cont.Comp. Steel Beam	17	12.2, 19.5, 14	12.22 (40.08)	68	1976				0		Few Transverse cracks over supports	63.5 (2.5)	121.92 (4.80)	6	
59	063-086-5978	Crawfords-ville	28-Sep-95	Black Rebar w/ Class C Conc. & 1.5" LMO	Cont. Prestr. I - Beam		22.9, 4@19.1	12.2 (40)	27	1976				0		Few Transverse Cracks	63.5 (2.5)	66.80 (2.63)	8	
64	129-78-6542	Seymour	27-Jun-96	Black Rebar w/ Class C Conc. & 1.5" LMO	Cont. Reinf. Conc. Slab	30	2@12.5, 17.5	10.4 (34)	9	1976				0		Transverse Cracks every .9 m (3 ft) over supports	63.5 (2.5)	76.71 (3.02)	7	
67	262-58-6070	Seymour	17-Jul-96	Black Rebar w/ Class C Conc. & 1.5" LMO	Prestressed Conc. I-Beam	15	2@19.3, 19.5	10.4 (34)	3	1976				0		Occasional Transverse Cracks	63.5 (2.5)	112.52 (4.43)	7	
70	007-39-4917	Seymour	11-Jul-96	Black Rebar w/ Class C Conc. & 1.5" LMO	Cont.Comp. Steel Beam	15	2@20.7, 25.6	13.5 (44.2)	30	1977				0		Occasional Transverse Cracks	63.5 (2.5)	68.33 (2.69)	7	

Table B.1 (continued)

#	Structure #	District	Date Inspected	Method of Corrosion Protection	Type of Structure	Skew	Spans In Meters	Lane Width m (ft.)	Adt X 100	Year Built (Deck)	% Area of Spalls in Survey Area	% Area of Delamination/ Debonding in Survey Area	% Area of Patches in Survey Area	% Total Distress Area	Rust Stains in Survey Area	Cracking in Survey Area	Design Cover mm (in.)	Cover Avg. mm (in.)	INDOT Deck Rating	Notes
74	912-045-5087	Laporte	21-Nov-95	Black Rebar w/ Class C Conc. & 1.5" LMO	Cont. Steel Beam, Steel Beam	18	12.8, 28.5, 21.3, 14.6	7.62 (25)	26	1977				0			63.5 (2.5)	80.77 (3.18)	7	
76	I465-117-6463	Greenfield	3-Jul-96	Black Rebar w/ Class C Conc. & 1.5" LMO	Precast Seg. Box Girder		2@35.4, 3@54.6	12.27 (40.3)	713	1977				0			63.5 (2.5)		7	
77	001-24-6069	Seymour	17-Jul-96	Black Rebar w/ Class C Conc. & 1.5" LMO	Cont.Comp. Steel Beam		2@22.6, 5@28	12.88 (42.25)	7	1978				0			63.5 (2.5)	65.79 (2.59)	7	
79	013-27-6598	Greenfield	10-Jun-96	Black Rebar w/ Class C Conc. & 1.5" LMO	Cont. Reinf. Conc. Slab	45	2@9.1, 12.2	13.41 (44)	21	1978				0		One long. crack, Diag. cracks following skew	63.5 (2.5)	84.07 (3.31)	6	
82	031-71-5808	Laporte	30-May-96	Black Rebar w/ Class C Conc. & 1.5" LMO	Cont. Steel Plate Girder		2@34.6	19.9 (42.4)	117	1978				0		Few Transverse Cracks	63.5 (2.5)		8	
83	031-71-5809	Laporte	30-May-96	Black Rebar w/ Class C Conc. & 1.5" LMO	Cont. Steel Plate Girder	18	2@38.7	10.97 (36)	98	1978				0		Few Transverse Cracks	63.5 (2.5)		8	
90	066-62-6030	Vincennes	25-Jun-96	Black Rebar w/ Class C Conc. & 1.5" LMO	Cont. Prestr. I - Beam		2@22.2, 22.4	10.4 (34)	2	1979				0			63.5 (2.5)	82.04 (3.23)	8	
92	136-086-5939	Crawfords-ville	28-Sep-95	Black Rebar w/ Class C Conc. & 1.5" LMO	Cont.Comp. Steel Beam		2@32	13.41 (44)	59	1979				0		Transverse cracks every 1.5 - 2.4 m (5 – 8 ft), over supports	63.5 (2.5)	82.80 (3.26)	6	
94	I265-03-6489	Seymour	15-Jul-96	Black Rebar w/ Class C Conc. & 1.5" LMO	Comp. Steel Beam	18	33.5, 37.8	13.5 (44.2)	254	1979				0			63.5 (2.5)	60.96 (2.40)	7	
98	006-50-6578	Laporte	28-May-96	Black Rebar w/ Class C Conc. & 1.5" LMO	Cont. Prestr. I - Beam	43		13.41 (44)	77	1980				0			63.5 (2.5)	53.09 (2.09)	6	
105	037-029-3982	Greenfield	31-Oct-96	Black Rebar w/ Class C Conc. & 1.5" LMO	Cont. Steel Beam	30	2@18.3, 21.9	9.14 (30)	101	1980				0			63.5 (2.5)	85.85 (3.38)	7	
110	057-014-5166	Vincennes	29-Aug-96	Black Rebar w/ Class C Conc. & 1.5" LMO	Cont. Reinf. Conc. Slab	12	2@8.6, 2@11.3, 11.7	13.41 (44)	49	1980				N/A			63.5 (2.5)		N/A	Deck Replacement
113	124-52-6539	Fort Wayne	14-Jun-96	Black Rebar w/ Class C Conc. & 1.5" LMO	Cont.Comp. Steel Girder	13	2@25.3, 2@31.9	12.8 (42)	4	1980				0		Occasional Transverse Cracks	63.5 (2.5)	69.85 (2.75)	7	
121	912-45-2552A	Laporte	21-Nov-95	Black Rebar w/ Class C Conc. & 1.5" LMO	Cont. Steel Beam		29 Spans	2@16 (2@53)	154	1980				0			63.5 (2.5)	73.66 (2.90)	8	
122	912-45-6611	Laporte	21-Nov-95	Black Rebar w/ Class C Conc. & 1.5" LMO	Cont. Steel Beam	24, 23	11 Spans		154	1980				0			63.5 (2.5)	50.67 (2.00)	8	
123	912-045-6599	Laporte	21-Nov-95	Black Rebar w/ Class C Conc. & 1.5" LMO	Cont. Steel Beam	14	2@12.2, 19.7	2@16 (2@53)	366	1980				0			63.5 (2.5)	62.74 (2.47)	7	
128	049-064-6679	Laporte	25-Jul-96	Black Rebar with Class K Concrete	Cont.Comp. St. Beam and Girder		11, 26.2, 9.8	12.2 (40)	32	1987				0			76.2 (3)	130.81 (5.15)	8	
129	049-064-6678	Laporte	25-Jul-96	Black Rebar with Class K Concrete	Cont.Comp. St. Beam and Girder	18	2@11.3, 40.5	12.2 (40)	32	1987				0			76.2 (3)	90.42 (3.56)	8	
127	I70-109-5133A	Greenfield	31-Oct-96	Black Rebar with High Early Strg. LMC	Cont. Reinf. Conc. Slab	30	2@7.2, 9.1	12.04 (39.5)	127	R1989				0			38.1 (1.5)	82.55 (3.25)	7	* Check to see if deck replaced
130	003-72-3236A	Seymour	16-Jul-96	DCI	Steel Pony Truss		25.6	7.4 (24.2)	16	R1979		1.5		1.5					7	
131	003-72-3235A	Seymour	16-Jul-96	DCI	Steel Pony Truss		25.6	7.4 (24.2)	16	R1979				0					7	

Table B.1 (continued)

#	Structure #	District	Date Inspected	Method of Corrosion Protection	Type of Structure	Skew	Spans In Meters	Lane Width m (ft.)	Adt X 100	Year Built (Deck)	% Area of Spalls in Survey Area	% Area of Delamination/ Debonding in Survey Area	% Area of Patches in Survey Area	% Total Distress Area	Rust Stains in Survey Area	Cracking in Survey Area	Design Cover mm (in.)	Cover Avg. mm (in.)	INDOT Deck Rating	Notes
101	006-50-6624	Laporte	28-May-96	Epoxy Coated Rebar with Class C Concrete	H.Comp. St. Girder & Beam		2@9.8, 29.4	13.41 (44)	82	1980	0.23		0.45	0.68	Stains in patched Area	Occasional Transverse Cracks	63.5 (2.5)	54.61 (2.15)	6	Cover Readings < 25.4 mm (1 in.) around patched area
84	063-083-1497	Crawfordsville	5-Oct-95	Epoxy Coated Rebar with Class C Concrete	Cont. Reinf. Conc. Slab	40	2@10.7, 13.4	12.2 (40)	43	1978		0.01		0.01	Some small Stains	One long. crack, some transverse cracks	63.5 (2.5)		6	
80	014-02-6561	Fort Wayne	17-Jun-96	Epoxy Coated Rebar with Class C Concrete	Cont. Reinf. Conc Slab	20	2@6.8, 9	13.41 (44)	40	1978	0.003			0.003		Few Tran Cracks, Long. cracks every 1.5 m (5ft)	50.8 (2)	62.74 (2.47)	6	
68	331-50-6608	Laporte	28-May-96	Epoxy Coated Rebar with Class C Concrete	Cont. Welded Girder	9	2@34.1	13.41 (44)	25	1976				0		Occasional Transverse Cracks	63.5 (2.5)	55.63 (2.19)	7	Numerous Popouts
50	007-40-6527	Seymour	11-Jul-96	Epoxy Coated Rebar with Class C Concrete	Cont. Prestr. I - Beam	10	2@22.3, 22.5	13.41 (44)	29	1976				0		Occasional Transverse Cracks	63.5 (2.5)	77.98 (3.07)	6	
78	001-38-6576	Greenfield	11-Jun-96	Epoxy Coated Rebar with Class C Concrete	Cont. Reinf. Conc. Slab		2@7, 9	13.5 (44.3)	13	1978				0		One long. crack, Occasional Transverse Cracks	63.5 (2.5)		7	
81	025-054-6166	Crawfordsville	10-Oct-95	Epoxy Coated Rebar with Class C Concrete	Cont. Reinf. Conc. Slab		2@6.1, 8.1	10.4 (34)	14	1978				0		Some long. and transverse cracks	50.8 (2)	60.20 (2.37)	N/A	
85	064-026-6591	Vincennes	29-Aug-96	Epoxy Coated Rebar with Class C Concrete	Prestr Conc. I - Beam	15	24.1	13.41 (44)	54	1978				0			63.5 (2.5)	93.22 (3.67)	8	
86	003-33-4609J	Greenfield	11-Jun-96	Epoxy Coated Rebar with Class C Concrete	Cont. Reinf. Conc. Slab	30	2@7.6, 10.4	12.2 (40)	38	1979				0		Few Transverse Cracks	63.5 (2.5)	130.81 (5.15)	7	
89	050-36-6170	Seymour	11-Jul-96	Epoxy Coated Rebar with Class C Concrete	Cont. Prestr. I - Beam	40	2@19.6, 19.8	13.5 (44.2)	32	1979				0			63.5 (2.5)	66.29 (2.61)	7	
91	114-37-6622	Laporte	22-May-96	Epoxy Coated Rebar with Class C Concrete	Cont. Steel Beam		2@9.8, 12.2	13.41 (44)	17	1979				0		Small Tran Cracks every 2.7–3 m (9 – 10 ft), One long. crack full length	63.5 (2.5)	74.42 (2.93)	7	
95	003-72-6795	Seymour	27-Jun-96	Epoxy Coated Rebar with Class C Concrete	Cont. Prestr. I – Beam	30	2@13.2, 13.4	13.5 (44.2)	14	1980				0			63.5 (2.5)	70.10 (2.76)	8	
97	006-50-6577	Laporte	28-May-96	Epoxy Coated Rebar with Class C Concrete	Reinf. Conc. Slab		2@8.2, 10.2	13.41 (44)	95	1980				0		Few Transverse Cracks, Two long. cracks	63.5 (2.5)	74.42 (2.93)	5	
99	006-50-2544	Laporte	28-May-96	Epoxy Coated Rebar with Class C Concrete	Cont. Prestr. I – Beam	30	2@17.2, 17.4	13.41 (44)	77	1980				0		Few Tran Cracks, Long. Cracks every 1.5m (5 ft)	63.5 (2.5)	53.34 (2.10)	7	
102	018-08-6246	Laporte	22-May-96	Epoxy Coated Rebar with Class C Concrete	Cont. Curved Steel Beam		2@20.1, 2@25.6, 28.3	13.41 (44)	19	1980				0		Small Transverse Cracks every 2.7–3 m (9 – 10 ft)	63.5 (2.5)	91.69 (3.61)	8	
103	031-36-6567	Seymour	12-Jul-96	Epoxy Coated Rebar with Class C Concrete	Cont. Steel Beam	15	2@19.2, 3@23.8	13.43 (44.1)	34	1980				0		Few Transverse cracks over supports	63.5 (2.5)	97.54 (3.84)	7	
104	031-50-2540	Laporte	30-May-96	Epoxy Coated Rebar with Class C Concrete	Cont. Welded Girder	6	2@14.9, 18.9	15.8 (52)	172	1980				0		Transverse cracks every 1.5 – 1.8 m (5 – 6 ft)	63.5 (2.5)		7	
106	052-24-6875	Seymour	17-Jul-96	Epoxy Coated Rebar with Class C Concrete	Cont. Reinf. Conc. Slab	30	2@7.2, 9.6	13.41 (44)	38	1980				0		Two long. crack length of bridge, Occasional Transverse Cracks	63.5 (2.5)	69.60 (2.74)	7	
107	055-56-6256	Laporte	24-May-96	Epoxy Coated Rebar with Class C Concrete	Cont. Reinf. Conc. Slab		2@7.6, 10.1	10.4 (34)	5	1980				0	Some	One long. crack full length, Occasional Transverse Cracking	63.5 (2.5)	76.71 (3.02)	6	Spalling underside of deck, rebar exposed both sides

Table B.1 (concluded)

#	Structure #	District	Date Inspected	Method of Corrosion Protection	Type of Structure	Skew	Spans In Meters	Lane Width m (ft.)	Adt X 100	Year Built (Deck)	% Area of Spalls in Survey Area	% Area of Delamination/ Debonding in Survey Area	% Area of Patches in Survey Area	% Total Distress Area	Rust Stains in Survey Area	Cracking in Survey Area	Design Cover mm (in.)	Cover Avg. mm (in.)	INDOT Deck Rating	Notes
108	057-082-6646	Vincennes	27-Aug-96	Epoxy Coated Rebar with Class C Concrete	Cont. Reinf. Conc. Slab	15	2@8.2, 11	13.41 (44)	67	1980				0			63.5 (2.5)	82.30 (3.24)	6	
109	057-014-6636	Vincennes	29-Aug-96	Epoxy Coated Rebar with Class C Concrete	Cont. Reinf. Conc. Slab	22	2@6.9, 9.3	13.41 (44)	48	1980				0			63.5 (2.5)	89.41 (3.52)	7	
111	062-087-5428	Vincennes	27-Aug-96	Epoxy Coated Rebar with Class C Concrete	Cont. Prestr. Box Beam	30	5@12.8	13.5 (44.2)	29	1980				0			63.5 (2.5)	88.39 (3.48)	8	
112	101-02-6638	Fort Wayne	17-Jun-96	Epoxy Coated Rebar with Class C Concrete	Cont. Reinf. Conc. Slab	20	2@5.5, 7.3	13.41 (44)	10	1980				0		Few Transverse Cracks	63.5 (2.5)	76.45 (3.01)	6	Spalling along underside edges
114	135-36-6557	Seymour	11-Jul-96	Epoxy Coated Rebar with Class C Concrete	Cont. Reinf. Conc. Slab	15	16.2, 2@12	13.41 (44)	36	1980				0			63.5 (2.5)	87.63 (3.45)	7	
115	135-36-6325	Seymour	27-Jun-96	Epoxy Coated Rebar with Class C Concrete	Cont. Reinf. Conc. Slab	30	2@6.4, 8.5	10.4 (34)	6	1980				0			63.5 (2.5)	54.61 (2.15)	7	
116	135-07-6467	Seymour	27-Jun-96	Epoxy Coated Rebar with Class C Concrete	Cont. Reinf. Conc. Slab	25	2@8.4, 11.3	10.4 (34)	4	1980				0			63.5 (2.5)	68.33 (2.69)	7	
117	145-13-6874	Vincennes	25-Jun-96	Epoxy Coated Rebar with Class C Concrete	Cont. Prestr. I - Beam		2@14.4, 14.6	13.41 (44)	10	1980				0			63.5 (2.5)	44.70 (1.76)	7	
120	327-76-6877	Fort Wayne	17-Jun-96	Epoxy Coated Rebar with Class C Concrete	Cont. Reinf. Conc. Slab		2@8.8, 11.7	12.8 (42)	12	1980				0			63.5 (2.5)	66.80 (2.63)	7	
51	019-52-4599	Fort Wayne	14-Jun-96	Galv. Rebar with Class C Concrete	Cont. Prestr. Concrete Box-Beam	10	2@16.3, 2@22.6	13.41 (44)	13	1976	0.37	8.75		9.77	Stains in Spalled Area	One long. crack along center of delam. area, occasional trans. cracks	50.8 (2)	54.10 (2.13)	6	
56	050-047-6168	Vincennes	22-Aug-96	Galv. Rebar with Class C Concrete	Cont. Prestr. I - Beam		2@21, 21.2	13.5 (44.2)	32	1976				0.002			63.5 (2.5)	49.28 (1.94)	7	
55	041-026-6541	Vincennes	29-Aug-96	Galv. Rebar with Class C Concrete	Cont. Reinf. Conc. Slab	9	2@10.1, 13.4	12.2 (40)	93	1976				0		Transverse cracks over supports	63.5 (2.5)	92.20 (3.63)	8	
69	450-51-5802	Vincennes	25-Jun-96	Galv. Rebar with Class C Concrete	Cont. Steel Beam	40	2@20.5, 3@25.6	11.6 (38)	6	1976				0		Occasional Transverse Cracks	69.85 (2.75)	74.17 (2.92)	7	

APPENDIX C

Appendix C – SAS PROGRAM AND OUTPUT

SAS Program

```
options ls=72;
title1 'Corrosion Analysis';
data corr;
  infile 'rep.dat';
  input week thick defects dist temp res;

proc print data=corr;

proc reg data=corr;
model res=week thick defects dist temp;

proc reg data=corr;
model res=week thick defects dist temp/adjrsq cp selection=rsquare;
```

SAS Output

```
OBS   WEEK   THICK   DEFECTS    DIST    TEMP    RES

  1    25     7.29       6     3.375     120    7000
  2    25     6.58      12     3.375     120    3200
  3    25     7.93      12     4.875     120    2800
  4    25     7.32      12     3.375     120    2300
  5    26     7.29       6     3.375     120    6500
  6    26     6.58      12     3.375     120    3200
  7    26     7.93      12     4.875     120    2800
  8    26     7.32      12     3.375     120    2000
  9    27     7.29       6     3.375     120    7700
 10    27     6.58      12     3.375     120    3100
 11    27     7.93      12     4.875     120    3000
 12    27     7.32      12     3.375     120    2000
 13    28     7.29       6     3.375     120    7000
 14    28     6.58      12     3.375     120    3000
 15    28     7.93      12     4.875     120    3100
 16    28     7.32      12     3.375     120    2100
 17    29     7.29       6     3.375     120    6700
 18    29     6.58      12     3.375     120    2900
 19    29     7.93      12     4.875     120    3000
 20    29     7.32      12     3.375     120    2000
 21    30     7.29       6     3.375     120    6400
 22    30     6.58      12     3.375     120    2500
```

23	30	7.93	12	4.875	120	2800
24	30	7.32	12	3.375	120	1900
25	31	7.29	6	3.375	120	6700
26	31	6.58	12	3.375	120	2500
27	31	7.93	12	4.875	120	3000
28	31	7.32	12	3.375	120	2000
29	32	7.29	6	3.375	120	6300
30	32	6.58	12	3.375	120	2400
31	32	7.93	12	4.875	120	2800
32	32	7.32	12	3.375	120	1900
33	33	7.29	6	3.375	120	6000
34	33	6.58	12	3.375	120	2200
35	33	7.93	12	4.875	120	2600
36	33	7.32	12	3.375	120	1700
37	34	7.29	6	3.375	120	5800
38	34	6.58	12	3.375	120	2200
39	34	7.93	12	4.875	120	2700
40	34	7.32	12	3.375	120	1800
41	35	7.29	6	3.375	120	6300
42	35	6.58	12	3.375	120	2200
43	35	7.93	12	4.875	120	2800
44	35	7.32	12	3.375	120	1900
45	36	7.29	6	3.375	120	5500
46	36	6.58	12	3.375	120	2000
47	36	7.93	12	4.875	120	2600
48	36	7.32	12	3.375	120	1700
49	37	7.29	6	3.375	120	5500
50	37	6.58	12	3.375	120	1900
51	37	7.93	12	4.875	120	2600
52	37	7.32	12	3.375	120	1500
53	38	7.29	6	3.375	72	16000
54	38	6.58	12	3.375	72	5100
55	38	7.93	12	4.875	72	11000
56	38	7.32	12	3.375	72	3200
57	39	7.29	6	3.375	120	4600
58	39	6.58	12	3.375	120	1600
59	39	7.93	12	4.875	120	2500
60	39	7.32	12	3.375	120	1300
61	40	7.29	6	3.375	120	4400
62	40	6.58	12	3.375	120	1600
63	40	7.93	12	4.875	120	2600
64	40	7.32	12	3.375	120	1200
65	41	7.29	6	3.375	120	4200
66	41	6.58	12	3.375	120	1600
67	41	7.93	12	4.875	120	2500

68	41	7.32	12	3.375	120	1200
69	42	7.29	6	3.375	120	3800
70	42	6.58	12	3.375	120	1500
71	42	7.93	12	4.875	120	2400
72	42	7.32	12	3.375	120	1100
73	43	7.29	6	3.375	120	3400
74	43	6.58	12	3.375	120	1400
75	43	7.93	12	4.875	120	2300
76	43	7.32	12	3.375	120	1000
77	44	7.29	6	3.375	120	2800
78	44	6.58	12	3.375	120	1400
79	44	7.93	12	4.875	120	2400
80	44	7.32	12	3.375	120	1000
81	45	7.29	6	3.375	120	2500
82	45	6.58	12	3.375	120	1300
83	45	7.93	12	4.875	120	2400
84	45	7.32	12	3.375	120	1000
85	46	7.29	6	3.375	120	2400
86	46	6.58	12	3.375	120	1200
87	46	7.93	12	4.875	120	2300
88	46	7.32	12	3.375	120	900
89	47	7.29	6	3.375	120	2500
90	47	6.58	12	3.375	120	1200
91	47	7.93	12	4.875	120	2250
92	47	7.32	12	3.375	120	900
93	48	7.29	6	3.375	120	2300
94	48	6.58	12	3.375	120	1200
95	48	7.93	12	4.875	120	2000
96	48	7.32	12	3.375	120	900
97	49	7.29	6	3.375	120	1800
98	49	6.58	12	3.375	120	1200
99	49	7.93	12	4.875	120	1850
100	49	7.32	12	3.375	120	900
101	50	7.29	6	3.375	120	1500
102	50	6.58	12	3.375	120	1050
103	50	7.93	12	4.875	120	1800
104	50	7.32	12	3.375	120	800
105	51	7.29	6	3.375	120	1350
106	51	6.58	12	3.375	120	1000
107	51	7.93	12	4.875	120	1600
108	51	7.32	12	3.375	120	800
109	52	7.29	6	3.375	120	1050
110	52	6.58	12	3.375	120	1150
111	52	7.93	12	4.875	120	1550
112	52	7.32	12	3.375	120	750

113	53	7.29	6	3.375	120	875
114	53	6.58	12	3.375	120	1100
115	53	7.93	12	4.875	120	1250
116	53	7.32	12	3.375	120	800
117	54	7.29	6	3.375	120	775
118	54	6.58	12	3.375	120	1000
119	54	7.93	12	4.875	120	1200
120	54	7.32	12	3.375	120	700
121	55	7.29	6	3.375	120	700
122	55	6.58	12	3.375	120	870
123	55	7.93	12	4.875	120	850
124	55	7.32	12	3.375	120	580
125	56	7.29	6	3.375	120	630
126	56	6.58	12	3.375	120	750
127	56	7.93	12	4.875	120	500
128	56	7.32	12	3.375	120	470
129	57	7.29	6	3.375	120	580
130	57	6.58	12	3.375	120	710
131	57	7.93	12	4.875	120	420
132	57	7.32	12	3.375	120	450
133	58	7.29	6	3.375	120	530
134	58	6.58	12	3.375	120	700
135	58	7.93	12	4.875	120	460
136	58	7.32	12	3.375	120	420
137	59	7.29	6	3.375	120	500
138	59	6.58	12	3.375	120	700
139	59	7.93	12	4.875	120	440
140	59	7.32	12	3.375	120	420
141	60	7.29	6	3.375	120	480
142	60	6.58	12	3.375	120	660
143	60	7.93	12	4.875	120	460
144	60	7.32	12	3.375	120	390
145	61	7.29	6	3.375	120	440
146	61	6.58	12	3.375	120	590
147	61	7.93	12	4.875	120	430
148	61	7.32	12	3.375	120	360
149	62	7.29	6	3.375	120	410
150	62	6.58	12	3.375	120	580
151	62	7.93	12	4.875	120	480
152	62	7.32	12	3.375	120	350
153	63	7.29	6	3.375	120	400
154	63	6.58	12	3.375	120	580
155	63	7.93	12	4.875	120	450
156	63	7.32	12	3.375	120	340
157	64	7.29	6	3.375	120	380

158	64	6.58	12	3.375	120	570
159	64	7.93	12	4.875	120	480
160	64	7.32	12	3.375	120	350
161	48	9.54	6	4.875	120	3850
162	49	9.54	6	4.875	120	3800
163	50	9.54	6	4.875	120	3600
164	51	9.54	6	4.875	120	3500
165	52	9.54	6	4.875	120	3400
166	53	9.54	6	4.875	120	3300
167	54	9.54	6	4.875	120	3300
168	55	9.54	6	4.875	120	3150
169	56	9.54	6	4.875	120	3000
170	57	9.54	6	4.875	120	3000
171	58	9.54	6	4.875	120	3000
172	59	9.54	6	4.875	120	3000
173	60	9.54	6	4.875	120	2800
174	61	9.54	6	4.875	120	2700
175	62	9.54	6	4.875	120	2600
176	63	9.54	6	4.875	120	2700
177	64	9.54	6	4.875	120	2800
178	56	8.28	6	4.875	120	3800
179	57	8.28	6	4.875	120	3800
180	58	8.28	6	4.875	120	3500
181	59	8.28	6	4.875	120	3000
182	60	8.28	6	4.875	120	3100
183	61	8.28	6	4.875	120	2800
184	62	8.28	6	4.875	120	2500
185	63	8.28	6	4.875	120	2600
186	64	8.28	6	4.875	120	2600
187	40	8.28	6	4.875	120	4000
188	40	9.54	6	4.875	120	3900
189	41	8.28	6	4.875	120	4100
190	41	9.54	6	4.875	120	4050
191	42	8.28	6	4.875	120	3800
192	42	9.54	6	4.875	120	3800
193	43	8.28	6	4.875	120	3800
194	43	9.54	6	4.875	120	3300
195	44	8.28	6	4.875	120	3800
196	44	9.54	6	4.875	120	3500
197	45	8.28	6	4.875	120	3900
198	45	9.54	6	4.875	120	3400
199	46	8.28	6	4.875	120	4100
200	46	9.54	6	4.875	120	3500
201	47	8.28	6	4.875	120	4500
202	47	9.54	6	4.875	120	4000

203	48	8.28	6	4.875	120	5150
204	49	8.28	6	4.875	120	4800
205	50	8.28	6	4.875	120	4700
206	51	8.28	6	4.875	120	4700
207	52	8.28	6	4.875	120	4550
208	53	8.28	6	4.875	120	4400
209	54	8.28	6	4.875	120	4100
210	55	8.28	6	4.875	120	3950
211	39	8.28	6	4.875	120	3900
212	39	9.54	6	4.875	120	3900
213	38	8.28	6	4.875	72	9300
214	38	9.54	6	4.875	72	8800
215	65	6.58	12	3.375	120	440
216	66	6.58	12	3.375	120	510
217	67	6.58	12	3.375	120	480
218	68	6.58	12	3.375	120	430
219	69	6.58	12	3.375	120	410
220	70	6.58	12	3.375	120	420
221	71	6.58	12	3.375	120	430
222	72	6.58	12	3.375	120	470
223	73	6.58	12	3.375	120	410
224	74	6.58	12	3.375	120	390
225	75	6.58	12	3.375	120	400
226	76	6.58	12	3.375	120	350
227	77	6.58	12	3.375	120	360
228	65	8.28	6	4.875	120	2100
229	66	8.28	6	4.875	120	2300
230	67	8.28	6	4.875	120	2300
231	68	8.28	6	4.875	120	2100
232	69	8.28	6	4.875	120	2000
233	70	8.28	6	4.875	120	2000
234	71	8.28	6	4.875	120	1900
235	72	8.28	6	4.875	120	1800
236	73	8.28	6	4.875	120	1400
237	74	8.28	6	4.875	120	1500
238	75	8.28	6	4.875	120	1400
239	76	8.28	6	4.875	120	1100
240	77	8.28	6	4.875	120	1100

```
Model: MODEL1
Dependent Variable: RES

                            Analysis of Variance

                      Sum of             Mean
Source       DF       Squares            Square             F Value     Prob>F

 Model       5        741778268          148355653.6        160.576     0.0001
 Error       234      216192175.33       923898.18518
 C Total     239      957970443.33

      Root MSE        961.19623          R-square      0.7743
      Dep Mean        2402.33333         Adj R-sq      0.7695
      C.V.            40.01094


Parameter Estimates

Variable      DF    Parameter          Standard          T for HO:        Prob > |T|
                    Estimate           Error             Parameter=0

INTERCEP      1     24466              1371.3388175      17.841           0.0001
WEEK          1     -82.121042         4.91124587        -16.721          0.0001
THICK         1     -469.529560        151.34271381      -3.102           0.0022
DEFECTS       1     -390.010203        27.15216949       -14.364          0.0001
DIST          1     773.206757         155.19953773      4.982            0.0001
TEMP          1     -117.387327        8.36255143        -14.037          0.0001

N = 240    Regression Models for Dependent Variable: RES
```

# in Model	R-Square	Adjusted R-Square	C(p)	Variables in Model
1	0.27121230	0.26815017	519.66452	TEMP
1	0.23710257	0.23389712	555.03217	WEEK
1	0.23505317	0.23183911	557.15715	DEFECTS
1	0.10510608	0.10134602	691.89654	THICK
1	0.05584112	0.05187407	742.97832	DIST
2	0.55462757	0.55086915	227.79724	WEEK DEFECTS
2	0.49760527	0.49336565	286.92245	DEFECTS TEMP
2	0.44890564	0.44425506	337.41806	WEEK TEMP
2	0.37774317	0.37249206	411.20492	WEEK THICK
2	0.36677353	0.36142984	422.57911	THICK TEMP
2	0.33456147	0.32894595	455.97911	WEEK DIST
2	0.32285681	0.31714252	468.11542	DIST TEMP
2	0.24195194	0.23555491	552.00396	DEFECTS DIST
2	0.23807195	0.23164218	556.02704	THICK DEFECTS
2	0.10777414	0.10024481	691.13008	THICK DIST
3	0.74748680	0.74427689	29.82558	WEEK DEFECTS TEMP
3	0.57551858	0.57012263	208.13579	WEEK DEFECTS DIST
3	0.57528889	0.56989002	208.37395	WEEK THICK TEMP
3	0.55889329	0.55328600	225.37419	WEEK THICK DEFECTS
3	0.53589244	0.52999277	249.22329	WEEK DIST TEMP
3	0.50348584	0.49717422	282.82500	DEFECTS DIST TEMP
3	0.49945043	0.49308752	287.00923	THICK DEFECTS TEMP
3	0.37782810	0.36991913	413.11686	WEEK THICK DIST
3	0.36887602	0.36085326	422.39907	THICK DIST TEMP
3	0.24246689	0.23283723	553.47003	THICK DEFECTS DIST
4	0.76504000	0.76104068	13.62505	WEEK DEFECTS DIST TEMP
4	0.75038501	0.74613624	28.82049	WEEK THICK DEFECTS TEMP
4	0.58428629	0.57721031	201.04473	WEEK THICK DEFECTS DIST
4	0.57534021	0.56811196	210.32073	WEEK THICK DIST TEMP
4	0.50467449	0.49624342	283.59252	THICK DEFECTS DIST TEMP
5	0.77432271	0.76950054	6.00000	WEEK THICK DEFECTS DIST TEMP