

Evaluation of corrosion resistance of different steel reinforcement types

by

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ABSTRACT

The corrosion of steel reinforcement in an aging highway infrastructure is a major problem currently facing the transportation engineering community, in particular bridge engineers. Further, the use of deicing salts has resulted in steady deterioration of bridge decks due to corrosion. In the United States, maintenance and replacement costs of deficient bridges are measured in billions of dollars.

These concerns have initiated continual development of protective measures. Application of corrosion-resistant steel reinforcement as an alternate reinforcement for existing mild steel reinforced concrete bridge decks has great potential due to the inherent corrosion resistant properties associated with the materials.

To investigate corrosion prevention through the use of corrosion-resistant alloys, the performance of corrosion resistance for MMFX Microcomposite steel reinforcement, a high-strength, high chromium steel reinforcement, is evaluated. The study presented herein presents the separate field and laboratory studies of ongoing research at Iowa State University to determine if in fact MMFX reinforcement provides superior corrosion resistance to epoxy-coated mild steel reinforcement in bridge decks. However, as definitive field evidence of the corrosion resistance of MMFX reinforcement may require several years of monitoring, attention transferred to investigate the reinforcement under accelerated conditions in a laboratory. In the laboratory investigation, the evaluation process was based on using ASTM and Rapid Macrocell accelerated corrosion tests.

After 40 weeks of testing, the associated ASTM ACT corrosion potentials indicate corrosion has not initiated for either MMFX or as-delivered epoxy-coated reinforcement. However, the uncoated mild steel underwent corrosion within the fifth week, while epoxy-coated reinforcement with holidays underwent corrosion between 15 and 30 weeks. Within the fifth week of testing, the Rapid Macrocell ACT produced corrosion risk potentials indicative of active corrosion for all the reinforcement types tested. For the study presented herein,

concrete powder specimens were collected at the top reinforcement depth at the first indication of corrosion initiation. For uncoated mild reinforcement, the chloride-ion concentration of 1.06 lb/yd³ was obtained. This value matches the 1.00 to 1.40 lb/yd³ commonly believed to be the chloride threshold of uncoated mild steel. For the epoxy-coated reinforcement with induced holidays the chloride-ion concentration was 1.74 lb/yd³. Chloride-ion concentrations will continue to be monitored for the remaining MMFX and as-delivered epoxy-coated reinforcement through the duration of the ASTM accelerated corrosion test.

While the limited results from 40 weeks of laboratory testing do not constitute a prediction of life expectancy and lifecycle cost, a procedure is presented to determine the life expectancy and lifecycle cost once definitive evidence is attained. The life expectancy of bridge decks constructed with different steel reinforcing systems is estimated by the two-stage, diffusion-spalling model (i.e., the time required for corrosion initiation and the subsequent time period required to cause spalling due to corrosion). Fick's Second Law of Diffusion was applied in this study to estimate the time required to corrosion initiation, while calculated corrosion rates were utilized to determine the time between initiation and spalling of the concrete. The combination of time to initiation and time between initiation and spalling results in the time to the first repair (i.e., life expectancy).

The in-place costs for the two twin, side-by-side "field" bridge decks (MMFX Bridge and Epoxy-coated Bridge) in conjunction with the calculated life expectancy (i.e., time from first constructed to spalling) can be used to compare the lifecycle costs associated with MMFX and epoxy-coated reinforcement in bridge decks constructed in the state of Iowa. However, until enough data becomes available, no direct comparison for MMFX and epoxy-coated can be obtained. Therefore, further investigation is required to determine more representative chloride threshold values and corrosion rates for the MMFX and epoxy-coated reinforcement.

CHAPTER 1 INTRODUCTION

1.1 Background

Reinforced concrete (RC) is a versatile, economical, and proven construction material. Able to be placed to a variety of shapes and finishes, reinforced concrete generally performs well throughout its service life. However, the corrosion of steel reinforcement is the primary, and costly, form of deterioration currently impacting the performance of RC bridge structures. As an example, in the United States alone this deterioration results in billions of dollars spent to maintain and replace existing bridge decks. In 1979, an estimated \$6.3 billion in federal-aid was allocated for rehabilitation due to corrosion incurred bridge damage (1). By 1986, the amount had risen to \$20 billion, and in 1992, the amount totaled \$51 billion (2, 3). With ever increasing bridge maintenance costs, protective measures to arrest chloride-induced corrosion have been actively studied for over 30 years.

Eliminating or slowing the deterioration of RC structures due to corrosion of steel reinforcement requires the use of innovative methodologies, which are commonly subdivided into two categories. First, deterioration is slowed through methods that lengthen the time it takes chloride ions to reach the steel reinforcement. The second includes methods that lengthen the time between initiation of corrosion and the end of service life (4).

Over the last three decades, the principle techniques for corrosion prevention in bridge decks incorporate increased concrete cover depth and the application of epoxy coating over the steel reinforcement. In 1976, the Iowa Department of Transportation implemented the use of epoxy-coated mild steel reinforcement for top layer of reinforcement in bridge decks. Within 10 years, bridge deck designs had integrated epoxy-coated reinforcement in both top and bottom layers of reinforcement (5).

Increasing concrete cover depth and infusing epoxy coating over uncoated mild steel reinforcement are believed to delay corrosion initiation and extend service life (4). Increased

concrete cover depth lengthens the time for chlorides to propagate to the level of the steel reinforcement and also lessens the availability of oxygen and moisture for the corrosion process. Epoxy coating, additionally, limits the exposure of the steel to chlorides, oxygen, and moisture. Even in regions with holidays (i.e., areas where the epoxy coating is absent), the corrosion process is thought to be abated due to the epoxy coating limiting the oxygen and moisture despite the chloride contact (4). However, increasing concrete cover depth increases both dead load and construction costs and is virtually unnecessary for structural reasons. Epoxy-coated reinforcement adds only slightly to the cost of bridge construction. However, holidays in the epoxy coating at cracked locations in bridge decks in combination with high chloride concentrations could result in corrosion of the steel reinforcement that affects the overall performance of the bridge. Further, it is believed that as a bridge deck ages, epoxy coatings may become brittle and eventually delaminate from the steel reinforcement under exposure to high chloride concentrations (6). Published literature reports that poorly adhering epoxy coatings have actually increased corrosion problems (7). All epoxy coatings are susceptible to delamination from the steel reinforcement. Small breaks, cracks, etc. in the epoxy coating allow the bond between the coating and steel to be lost. The epoxy coating remains generally intact, but the chloride concentration increases in the solution directly below the coating in an environment that is low in oxygen. This results in hydrochloric acid attack of the steel. An example of this occurred in 1986, six years after construction, epoxy-coated reinforcement used in bridge substructures in the Florida Keys showed signs of chloride-induced corrosion (7). This provided an initial indication that the long-term protection provided by epoxy coating may be less than was intended. Further, there is clear evidence that given enough time, even well applied epoxy coatings tend to lose adhesion by the time chlorides reach the level of steel reinforcement (6, 8).

The above concerns have initiated continual development of other protective measures. The use of dense concretes, corrosion inhibitors, and both nonmetallic and steel-alloy corrosion-resistant reinforcement are among the most common techniques being developed.

1.2 Objectives

This study presents a dual-phase investigation at Iowa State University (ISU) that is funded by the Iowa Department of Transportation (Iowa DOT) through the Federal Highway Administration (FHWA), Innovative Bridge Research and Construction (IBRC) Program. The objective of which is to determine if MMFX Microcomposite steel reinforcement will provide superior corrosion resistance to epoxy-coated mild steel reinforcement (ECR) in bridge decks. The principal reason for selecting a new reinforcement material for concrete bridge decks is to improve both the life expectancy and cost effectiveness of the structural system. A prerequisite requirement is that the material, which presumably is more expensive than the current system, provides a significant improvement in corrosion resistance compared to the current material of choice.

The investigation is comprised of both field and laboratory evaluations of MMFX, epoxy-coated, and uncoated reinforcement. Two twin, side-by-side bridge decks reinforced with MMFX and epoxy-coated steel were constructed and instrumented to investigate the “field” performance through periodic monitoring for corrosion. As the field evaluation may require several years of monitoring to make a valid comparison, procedures to accelerate corrosion in a laboratory setting are also being conducted. In the laboratory the mechanical properties and corrosion resistance performance of MMFX, epoxy-coated, and uncoated reinforcement are evaluated. Specifically, concrete specimens reinforced with MMFX, epoxy-coated, and uncoated steel were constructed and are being monitored for corrosion.

In both the field and laboratory evaluations, emphasis is placed on the corrosion resistance performance, including:

- Determining the initiation of corrosion and the intensity of corrosion growth.
- Assessing the difference in corrosion resistance between MMFX, epoxy-coated, and uncoated reinforcement.

To accomplish the aforementioned objectives, the following tasks were completed:

1. Review of related literature: Prior studies related to MMFX reinforcement and corrosion resistance performance were reviewed to provide background for evaluating the mechanical properties for this new steel reinforcement.
2. Conduct field evaluation: The program consisted of construction documentation and post construction monitoring of two bridges constructed using MMFX and epoxy-coated reinforcement. Monitoring sensors were installed in the two newly constructed concrete bridge decks in a similar manner. Periodic measurements were taken to assess the corrosion performance of the two bridges.
3. Conduct mechanical laboratory tests: The MMFX, epoxy-coated, and uncoated reinforcement were tested in the laboratory to evaluate the mechanical properties for each type of steel reinforcement.
4. Conduct corrosion resistance performance laboratory tests: ASTM G 109 and Rapid Macrocell accelerated corrosion tests were utilized to evaluate general and pit corrosion properties of the MMFX, epoxy-coated, and uncoated reinforcement. At the onset of corrosion, chloride-ion concentration analysis was performed.
5. Compile and analyze data: Macrocell and half-cell potential change measurements were compiled to statistically determine corrosion rates for each reinforcement type tested herein. Comparisons of the compiled corrosion rate and chloride concentration were made between MMFX, epoxy-coated, and uncoated reinforcement.
6. Evaluate the performance of MMFX steel as concrete reinforcement: The results of the laboratory corrosion evaluation were combined with analytical and experimental field experience in the state of Iowa to evaluate the impact of the new steel reinforcement on life expectancy and lifecycle cost of reinforced concrete bridge

decks. The life expectancy can be determined two-stage (i.e., the time to corrosion initiation and the time from corrosion initiation to spalling). Then utilizing the in-place costs, the lifecycle cost for a bridge deck is calculated by dividing the in-place costs by the life expectancy.

CHAPTER 2 LITERATURE REVIEW

2.1 Corrosion Process

Both mild and high strength steel reinforcement corrode in the presence of oxygen and water. As concrete has interconnected pores, air and moisture are ever present around the steel reinforcement. Initially, at least, the alkaline nature of the surrounding concrete prevents embedded steel reinforcement from corroding.

Microscopic pores within a concrete matrix with high concentrations of soluble calcium, sodium, and potassium oxide form hydroxide when water is present. This subsequently creates the alkaline condition (i.e., pH 12 – 13) mentioned above (9). The alkaline condition leads to the formation of a “passive” layer on the steel reinforcement surface. This passive layer is a dense, impenetrable film which, if fully established and maintained, prevents further corrosion of the steel reinforcement. A true passive layer is a very dense, thin layer of oxide that leads to a very slow rate of corrosion (9). The passive layer formed on steel reinforcement in concrete is likely part metal oxide-hydroxide and part mineral from the cement paste. There is some discussion whether the layer on the steel is a true passive layer as it is thick compared with other known passive layers and consists of more than just metal oxides. However, it behaves similarly to a passive layer and is, therefore, generally referred to as such.

Corrosion engineers try to stop the corrosion of steel by simulating the passive layer with applied protective coatings. Metals, such as zinc, or polymers, like acrylics and epoxies, are used to stop corrosive conditions from getting to steel surfaces. The true passive layer is the ideal protective coating as it will form, maintain, and repair itself as long as the alkaline environment is sustained. This is a far better situation than any artificial coating (i.e., galvanizing or fusion bonded epoxy) as artificial coatings can be consumed or damaged, allowing corrosion to proceed in damaged areas. However, in reality the passive environment is not always maintained in RC. Most notably, the chloride attack mechanism

can break down the alkaline condition in concrete resulting in a corrosion susceptible environment.

2.1.1 Chloride Attack Mechanism

In reinforced concrete (RC) structures, sources of chloride-ion can come from several sources. They can be cast into the concrete or they can be due to deliberate addition of chlorides set accelerators. Calcium chloride, CaCl_2 , was widely used until the mid-1970s as a concrete set accelerator (9). The use of sea water or sea dredged aggregate will also contaminate the concrete mix with chloride. Chlorides can also diffuse into concrete as a result of deicing salt application, marine salt spray, and storage of salts.

The diffusion of chlorides via the application of deicing salts or marine salt spray is the primary source of chlorides in most RC structures. However, the cast-in chloride must not be overlooked. A low level of cast-in chloride can lead to the rapid onset of corrosion if additional chlorides become available from the environment. This often happens in marine conditions where seawater contaminates the original concrete mix and also diffuses into the hardened concrete.

Chloride ions penetrate through concrete capillaries and can act as catalysts to corrosion when there is sufficient chloride-ion concentration at the reinforcement surface. This could break down the passive layer of oxide on the steel allowing corrosion to initiate. Chloride attack is difficult to remedy as chlorides are generally hard to eliminate once introduced into a RC structure.

2.1.2 Chloride Threshold

A “small” concentration of chloride ions in the pore water will not break down the previously described passive layer, especially if the system is effectively reestablishing itself. However, there is a “chloride threshold” for corrosion, which is given in terms of the chloride-hydroxyl

ratio, which represents the concentration of chloride-ion required to initiate corrosion. Several researchers have studied uncoated reinforcement in laboratory tests with calcium hydroxide solutions to establish a chloride threshold.

For uncoated mild steel reinforcement, when the chloride concentration exceeds 0.6 of the hydroxyl concentration, corrosion is observed (10). This approximates to a concentration of 0.4 percent chloride by weight of cement if chlorides are cast into concrete and 0.2 percent if they diffuse into concrete (11, 12). Based on an assumed 6.5 sacks of cement per cubic yard of concrete, the chloride threshold for uncoated reinforcement has been estimated to be 1.2 pounds of chloride per cubic yard of concrete (13, 14).

Unlike the case for uncoated reinforcement, no published literature presents definitive chloride threshold values for MMFX Microcomposite or epoxy-coated mild steel reinforcement. This could have been due to several factors such as uncertainties associated with the quality of the organic coating of the epoxy, damage that could have occurred during transportation or storage of the epoxy-coated reinforcement, or due to loss of adhesion between the coating and the base metal. For these reasons, a range of chloride threshold from 3.3 to 3.6 lb/yd³ and 1.2 to 3.6 lb/yd³ at reinforcement level has been suggested, respectively, for MMFX and epoxy-coated reinforcement (4, 15). The lower bound of the range for epoxy-coated reinforcement was recommended as an empirical chloride threshold for uncoated reinforcement (11, 12).

2.1.3 Corrosion Process

Corrosion of steel reinforcement in concrete can be modeled as a two-stage process. The first stage is known as the initiation, diffusion, or incubation period, in which chloride-ions migrate to the reinforcement level. In this stage the reinforcing steel experience negligible corrosion. The time required for the chloride concentration to reach the aforementioned chloride threshold value at the reinforcement level can be determined by the diffusion

process of the chloride-ion through concrete following Fick's Second Law of Diffusion (16, 17).

Once the passive layer breaks down (i.e., the chloride threshold has been reached), the second stage which is referred to as the active corrosion period of steel reinforcement occurs and propagates. The length of the second stage depends on how fast the corroded steel reinforcement deteriorates resulting in an observable distress. Although it is not an easy task to predict the length of the second stage, eventually a RC structure will reach a condition at which some type of maintenance activities must be taken.

The two-stage corrosion model is often used to assess corrosion of uncoated reinforcement (16). This concept was assumed to be applicable for estimating the service life of a bridge deck constructed using epoxy-coated reinforcement. However, to the author's knowledge, the chloride threshold for epoxy-coated reinforcement and the length of the active and deterioration stage have not been definitely determined.

When corrosion occurs, the steel reinforcement "dissolves" in the pore water, giving up electrons and forming cations (positively charged ions). The process of losing electrons is known as oxidation. The following chemical reaction is the oxidation of steel reinforcement at the anode (i.e., the location that releases electrons).



where Fe is iron, Fe^{2+} is ferrous-ion, and $2e^{-}$ are two free electrons.

In the presence of water molecules and the free electrons, oxygen is transformed from a neutral molecule to an anion which has become more negatively charged by gaining the electrons. This process is called reduction. The gain of electrons comes from losses of electrons in two substances that react with each other. The following chemical equation illustrates the cathodic reaction (i.e., the gain of electrons).

The cathodic reaction: $4e^- + 2H_2O + O_2 \rightarrow 4OH^-$, (2.2)

where O_2 is oxygen, H_2O is water, and OH^- is a hydroxyl-ion.

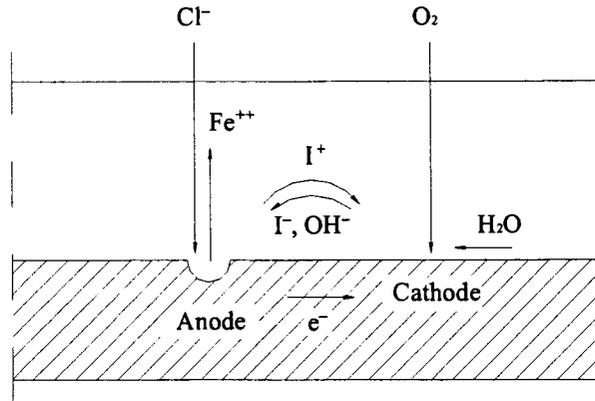


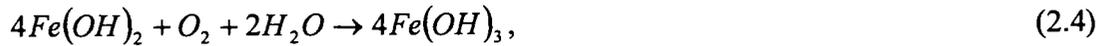
Figure 2.1 – Schematic of corrosion process.

The surface of the iron where oxidation occurs serves as an anode. The two free electrons, $2e^-$, created in the anodic reaction must be consumed elsewhere on the steel surface to preserve electrical neutrality. In other words, it is not possible for a large amount of electrical charge to build up at one location. Another chemical reaction must consume the electrons. Oxidation and reduction are coupled together as electrons are transferred between them. This reaction consumes water and oxygen.

If the iron were to just dissolve in the pore water, no cracking, delaminating, and spalling of the surrounding concrete would occur. Several more reactions occur for corrosion products to form. This can be expressed through the following steps. First, as shown in 2.3, the products of the anodic reaction, Fe^{2+} , and the cathodic reaction, OH^- , react producing Ferrous hydroxide, $Fe(OH)_2$.



In 2.4, ferrous hydroxide, $Fe(OH)_2$, is further oxidized to allow for the formation of ferric hydroxide, $Fe(OH)_3$.



As an effect of dehydration from the exposure to the environment, 2.5 shows how ferric hydroxide becomes ferric oxide, Fe_2O_3 , commonly referred to as rust.



Dehydrated ferric oxide, Fe_2O_3 , has a volume of approximately twice that of the original steel reinforcement it replaces. When it becomes hydrated, ferric oxide swells even more and becomes porous. This means that the volume increase at the steel-concrete interface is two to ten times that of the original steel reinforcement volume. This leads to the cracking, delamination, and spalling that are observed as the manifestation of corrosion of steel in concrete.

The electrical current flow described above, and the generation and consumption of electrons in the anode and cathode reactions, are used in macrocell and half-cell potential measurements to assess corrosion activity. The fact that the anodic and cathodic reactions must balance each other for corrosion to proceed is the reason that epoxy coatings protect steel reinforcement.

It has been observed that anodic and cathodic reactions are coupled mutually when corrosion is taking place. Therefore, one can possibly reduce corrosion by eliminating either one of the reactions.

2.2 Methods of Corrosion Monitoring

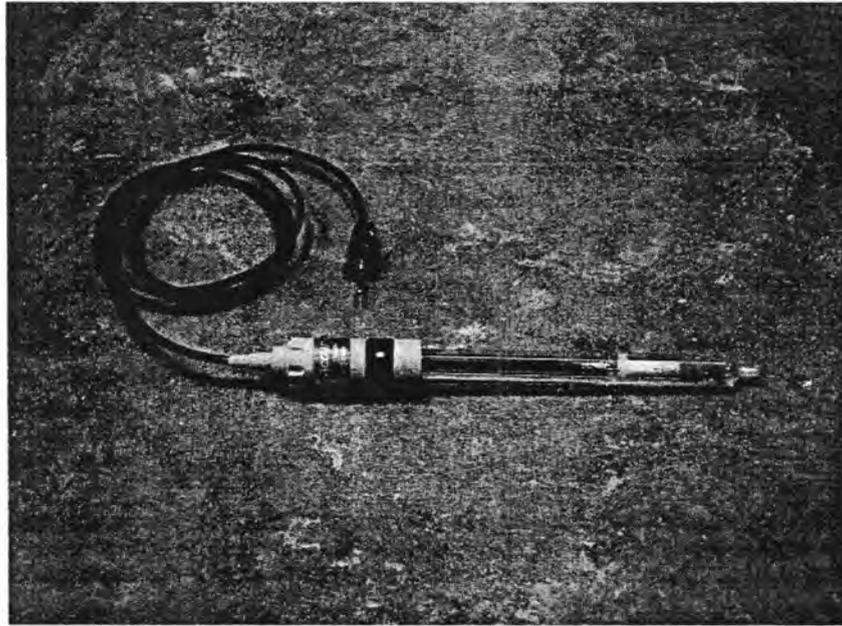
Techniques for corrosion monitoring are well established for reinforced concrete structures. Because corrosion is an electrochemical process, the collection and interpretation of data is relatively easy. As introduced, during corrosion of steel reinforcement, electrons are released as a product of the anode chemical reaction. The electrons flow from the site of corrosion (anode) to a non-corroding site (cathode). This allows for corrosion risk and corrosion rate to be evaluated through electronic means (i.e., voltmeter measurements). Among the many possible techniques for corrosion monitoring, four techniques were utilized in the study presented herein. Each of these four techniques is described in the following sections.

2.2.1 Half-cell Potential Monitoring

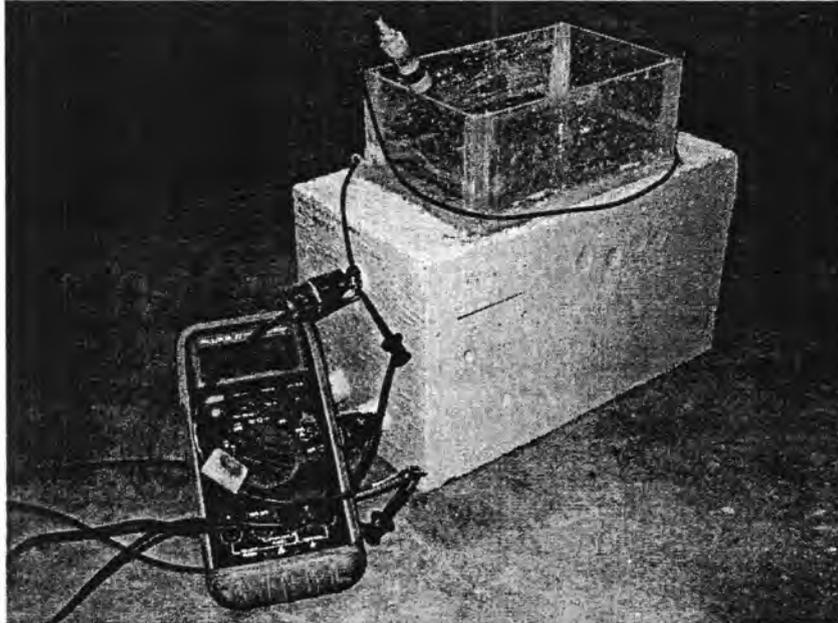
The corrosion risk of any steel reinforcement is measured by using the saturated calomel reference electrode shown in Figure 2.2. By placing the electrode on the concrete surface and connecting it via a voltmeter directly to the top or bottom reinforcement, a current will flow, and voltage is measure. The electrical potential difference (voltage) will be a function of the iron in the pore water environment. As such, the electric potential is a measurement of the corrosion risk.

2.2.2 Macrocell Corrosion Monitoring

In the case of chloride attack, the formation of anodes and cathodes are often well separated with areas of corrosion separated by areas of non-corroded steel. This is known as the macrocell phenomenon. In macrocell corrosion in bridge decks, the anode and cathode are located on different steel reinforcement, presumably the top and bottom layers.



(a) – Saturated calomel reference electrode

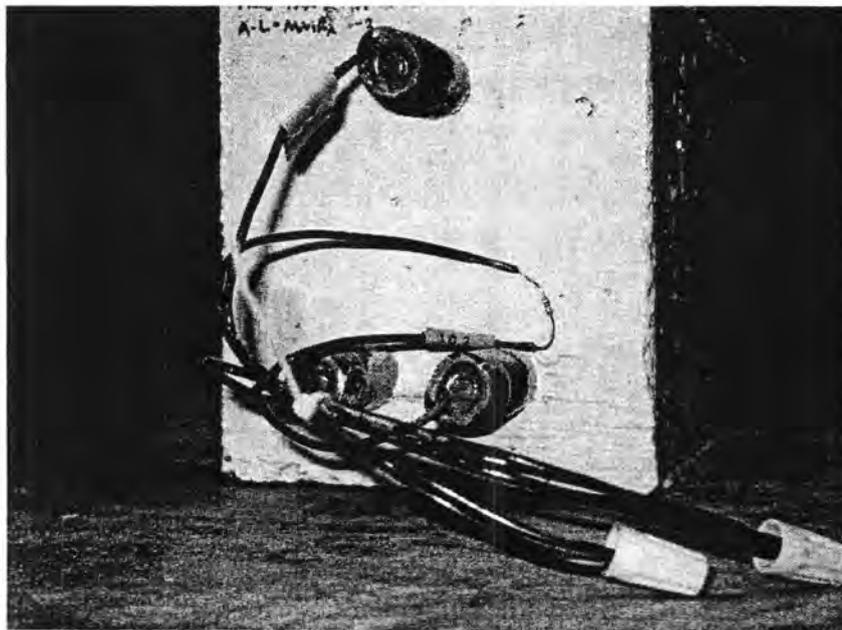


(b) – Measure of half-cell corrosion potential for the top layer of reinforcement

Figure 2.2 – Half-cell corrosion potential monitoring method.

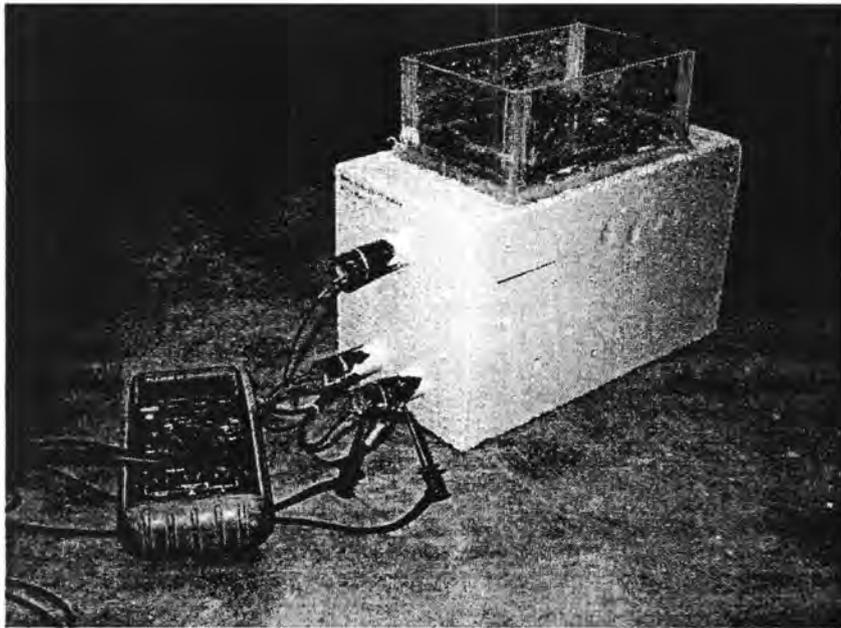
Chloride-induced corrosion, the typical type of corrosion occurring in bridge decks, is particularly prone to macrocell formation as a high level of water is usually present to carry chloride ions into the concrete. The presence of water in the pores increases the electrical conductivity of the concrete. The higher conductivity allows the separation of the anode and cathode as the chloride-ion can easily transport through the water filled/lined pores.

In North America, the macrocell is used as a way of measuring the corrosion rate. The current flow between the top and bottom steel reinforcement layers is monitored from the measure of voltage across a resistor connecting the layers of reinforcement, as illustrated in Figure 2.3. By Faraday's Law, the mass loss rate (i.e., corrosion rate) is directly proportional to the monitored corrosion current.



(a) – Top and bottom reinforcement layers connected via resistor

Figure 2.3 – Macrocell corrosion monitoring method.



(b) – Measurement of macrocell corrosion

Figure 2.3 – Continued – Macrocell corrosion monitoring method.

2.2.3 CMS V2000 Silver-Silver Chloride Electrode Monitoring

With the CMS V2000 silver-silver chloride electrode (V2000 electrode), the potential difference between the electrode and the anode steel reinforcement is measured. This is dictated by Faraday's laws and is a direct result of the relationship between dissimilar metals in the presence of an acid or alkaline substance.

However, while the electrochemical process of corrosion causes current through the flow of free electrons the V2000 electrode generates a second, independent current, as a function of the dissimilarity of the metals, the amount of moisture and chlorides present, etc. The two processes are additive.

By measuring the induced voltage and current, the corrosion risk and corrosion rate can be determined. This makes the V2000 electrode a viable, permanent embedded sensor for the long-term monitoring of bridge deck steel reinforcement.

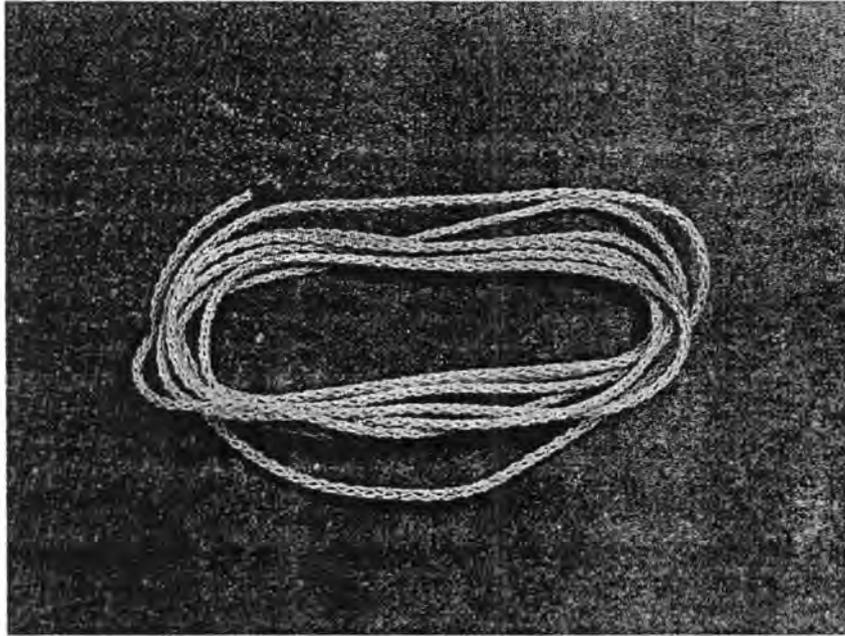


Figure 2.4 – CMS V2000 silver-silver chloride electrode.

2.2.4 Chloride-ion Concentration Monitoring

The chloride concentration in concrete at the level of reinforcement is a major factor in the corrosion of reinforcing steel. The chloride-ion migrates to the reinforcement by permeating through the concrete or by penetrating through cracks in the concrete. To initiate corrosion of steel reinforcement, the concentration of the chloride-ion must reach the previously described corrosion threshold at the steel reinforcement level.

The chloride-ion concentration of concrete can be evaluated by several different methods. The AASHTO T 260-94 Test (Sampling and Testing for Chloride-ion in Concrete and Concrete Raw Materials) suggests three procedures (Procedure A, B, and C) for determining the chloride-ion content in concrete (18). Both time consuming and complicated tests,

Procedure A determines the chloride-ion concentration potentiometric titration whereas Procedure B utilizes an atomic absorption process to determine the concentration of chloride-ion. In Procedure C, the chloride-ion concentration is determined using a specific ion probe.

An alternative to the three aforementioned procedures is the nondestructive use of X-ray fluorescence (XRF) spectroscopy to analyze the chloride-ion concentration in the powder samples. XRF spectroscopy provides an analytical means to identify and quantify the concentration of elements contained in a solid, powdered, and liquid sample (19).

2.3 MMFX Microcomposite Steel Reinforcement Research

MMFX Microcomposite steel reinforcement is publicized as a proprietary chemical composition material and known to have a unique microstructure with enhanced corrosion resistance characteristics and higher mechanical properties (yield and tensile strengths) than conventional ASTM A 615 steel. The developers claim that MMFX reinforcement is designed to delay corrosion initiation and to slow corrosion growth after initiation.

2.3.1 University of Kansas Center for Research

With no previously published study available on MMFX reinforcement performance, a study was conducted by the University of Kansas Center for Research to evaluate the performance of MMFX reinforcement with a major emphasis placed on the comparison of corrosion resistance of MMFX, epoxy-coated, and uncoated reinforcement (4). This study was conducted in cooperation with the United States Department of Transportation, Federal Highway Administration, the Kansas DOT, the South Dakota DOT, and the National Science Foundation. The Rapid Macrocell accelerated corrosion test was utilized as the principal evaluation test. The complete evaluation involved the corrosion testing, the measurement of bar deformation, the analysis of material composition, and the impact of reinforcement properties on the structural performance of bridge decks. Also, the results of the laboratory evaluation were supplemented with construction and maintenance experience in South

Dakota and other states to evaluate the impact of implementing MMFX on the life expectancy and cost effectiveness of concrete bridge decks.

From physical and mechanical tests conducted by the University of Kansas Center for Research, several issues were encountered with the implementation of MMFX reinforcement in three trial bridge deck designs in South Dakota. Based only on design, MMFX reinforcement provides few satisfactory options for replacing conventional reinforcement under the current AASHTO design procedures. MMFX reinforcement exceeds maximum allowable steel and concrete stresses, violates crack control and fatigue provisions, and exceeds the maximum allowable percentage of reinforcement (4). From laboratory corrosion testing, the epoxy-coated reinforcement was found to be more effective in corrosion performance than the MMFX steel. The mechanical properties provided by the manufacturer were found to be higher than what resulted from a series of tests conducted by the University of Kansas Center for Research, shown in Table 2.1 (20, 4). Overall, the report concluded that the use of MMFX reinforcing steel in bridge decks did not appear to be cost effective compared to the use of epoxy-coated reinforcement.

Table 2.1 – Mechanical properties of MMFX Microcomposite steel reinforcement.

Given by	Yield Strength, ksi	Tensile Strength, ksi
MMFX Steel Corporation of America	119-133	180-186
Kansas Department of Transportation	110-120	160-175

This conclusion was met with some criticism from the manufacturer of MMFX reinforcement. MMFX Steel Corporation of America submitted a critique of the laboratory evaluation conducted by the University of Kansas Center for Research presenting concerns to be addressed in future research. The study presented herein will attempt to incorporate these issues through the follows means:

- The principle differences between the Cracked Beam accelerated corrosion test and the ASTM G 109 accelerated corrosion test are that the Cracked Beam specimens used in by the University of Kansas Center for Research allows a higher concentration of chlorides (i.e., 15 percent sodium chloride) direct access to the top layer of reinforcement through a fabricated crack and a “severe” drying regime utilizing elevated temperature. While the general ASTM G 109 specimen does not employ the use of a fabricated crack, similar to the University of Kansas Center for Research, the study presented herein will also utilize a cracked specimen to accelerate the diffusion of chloride-ion through the concrete. However, beyond fabricating a crack, the study presented herein utilized the lower chloride concentration (i.e., 3 percent sodium chloride) and a non-heat drying regimen of the ASTM G 109 accelerated corrosion test.
- For the Rapid Macrocell accelerated corrosion test developed by the University of Kansas Center for Research, the MMFX Steel Corporation reported to establish realistic corrosion rates, the test would be improved by embedding the cathode in greater concrete cover. To accommodate this recommendation, for the study presented herein the forms the specimens were cast in were increased from the 38-mm (1-1/2-in.) diameter PVC pipe used by the University of Kansas Center for Research to a 76-mm (3-in.) diameter PVC pipe.
- One of the problems with determining chloride threshold concentrations from a laboratory test in which the rate of chloride ingress is greatly accelerated is that the chloride-ion concentration at the level steel reinforcement may be much higher than the threshold when corrosion activity is first detected by periodic monitoring. While period monitoring will give rise to uncertainty, by utilizing the lower chloride concentration outline by ASTM G 109 in the study presented herein, the change in chloride-ion concentration between successive measurements is believed to be minimal.

- The chloride exposure conditions performed by the University of Kansas Center of Research are believed to produce higher corrosion rates times than corrosion rates in field conditions. As previously discussed, to alleviate the uncertainty associated with the higher chloride concentration used by the University of Kansas Center for Research, the study presented herein decreased the chloride concentration to that outlined by ASTM G 109.
- None of the corrosion data collected by the University of Kansas Center for Research for the epoxy-coated reinforcement were used in the life expectancy and cost effectiveness analyses with reliance being placed solely on the field performance of epoxy-coated reinforcement. The life expectancy presented for each reinforcement type tested in the study herein was calculated based on measured laboratory chloride-ion concentrations and corrosion initiation and corrosion rates for each reinforcement type, respectively.

2.3.2 Field Evaluation by Iowa State University Center for Transportation Research and Education

This section recalls the physical and mechanical characteristics reported for the two newly constructed bridges being evaluated for the complimentary field evaluation of the study presented herein (21). The bridges are located in northeast Iowa on relocated U.S. 20 over South Beaver Creek in Grundy County, Iowa. Specifically, the layout and installation of corrosion sensors and the monitoring protocols as well as interim results of the monitoring program are presented.

2.3.2.1 Construction and Bridge Description

The subject bridges are new twin 83.5 m x 12 m, three-span prestressed concrete girder bridges constructed in 2002. The bridges have a total length of 83.5 m consisting of two 24.75 m end spans and a 34 m center span. The roadway width is 12 m allowing two traffic

lanes with a narrow shoulder on each side. Each bridge deck is a nominal 200-mm thick cast-in-place, reinforced concrete slab that includes a 13-mm integral wearing surface. Each bridge deck was constructed entirely of either MMFX Microcomposite or epoxy-coated mild steel reinforcement. MMFX reinforcement in the eastbound bridge (denoted MMFX Bridge) and epoxy-coated reinforcement in the westbound bridge (denoted Epoxy Bridge).

Both bridges were constructed identically. As shown in Figure 2.5, the top transverse reinforcement in both bridge decks was placed parallel to the top of the slab with a clear distance of 65 mm, while the bottom transverse reinforcing steel was placed parallel to the bottom of the slab with a clear distance of 25 mm. Approximately 50 mm of clear distance from the face of concrete to near reinforcing bar was used. The bridge deck is supported by five prestressed concrete beams spaced at 2,200 mm on center.

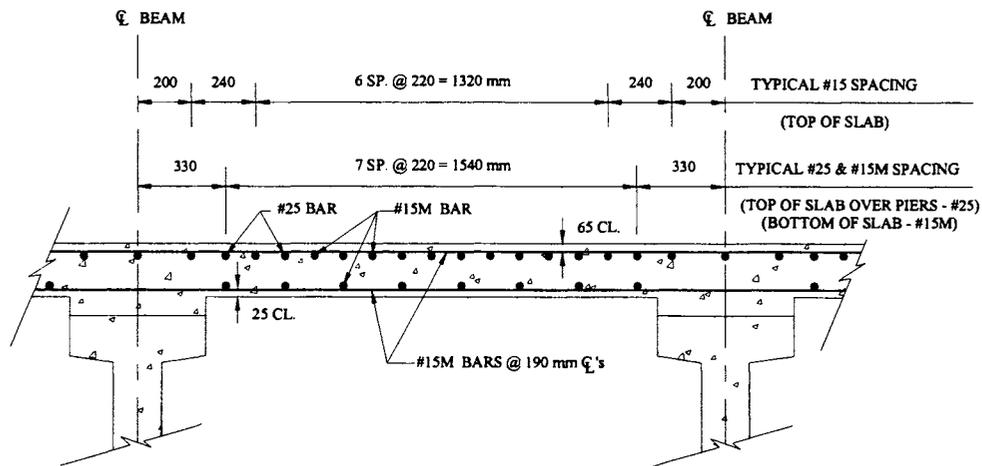
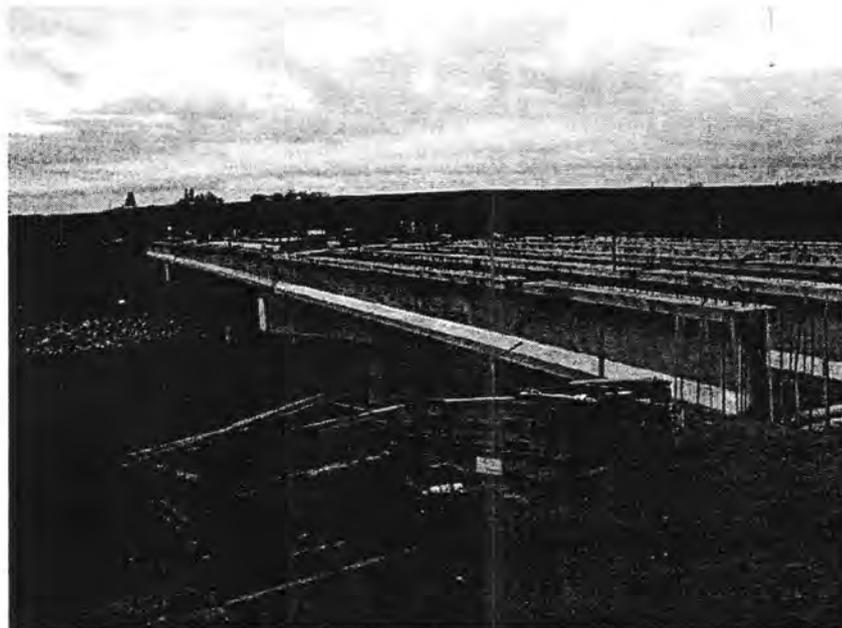


Figure 2.5 – Typical cross-section.

The concrete bridge decks were cast for both bridges in May, 2002. All slab and diaphragm reinforcing steel was tied in place and adequately supported before the concrete was cast. The pier and abutment diaphragm concrete was cast monolithically with the bridge deck slab. Moderate curbs were constructed integral with the bridge deck and concrete guardrails were

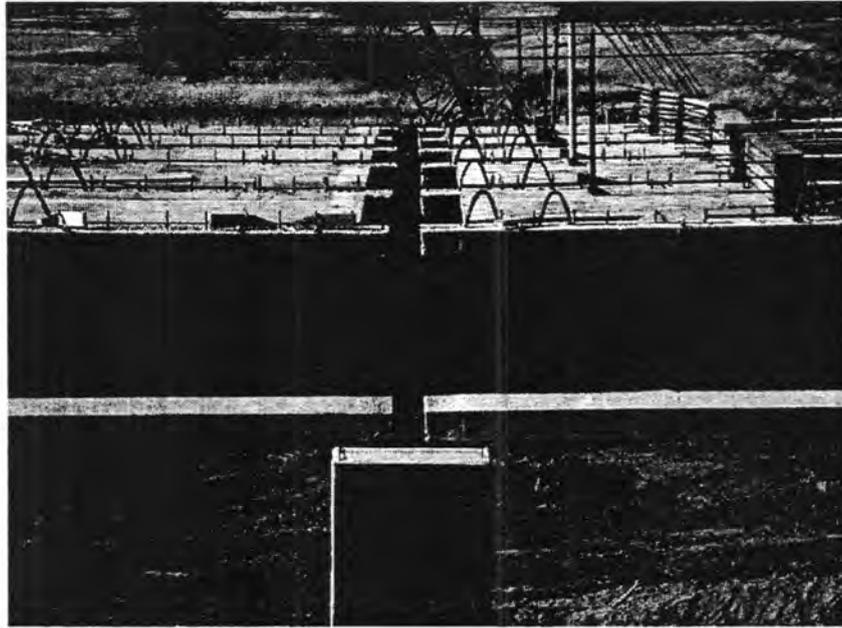
connected to the curbs. Corrosion monitoring commenced following concrete placement in May, 2002 while the bridges were open to traffic in August, 2003.

Typical photographs of subject bridges taken during construction through completion are shown in Figure 2.6. No special construction techniques were required for the MMFX Bridge. In fact, it appeared to be easier to construct the MMFX Bridge because special handling care was not needed, unlike the care needed to protect the epoxy coating on the bars in the Epoxy Bridge.

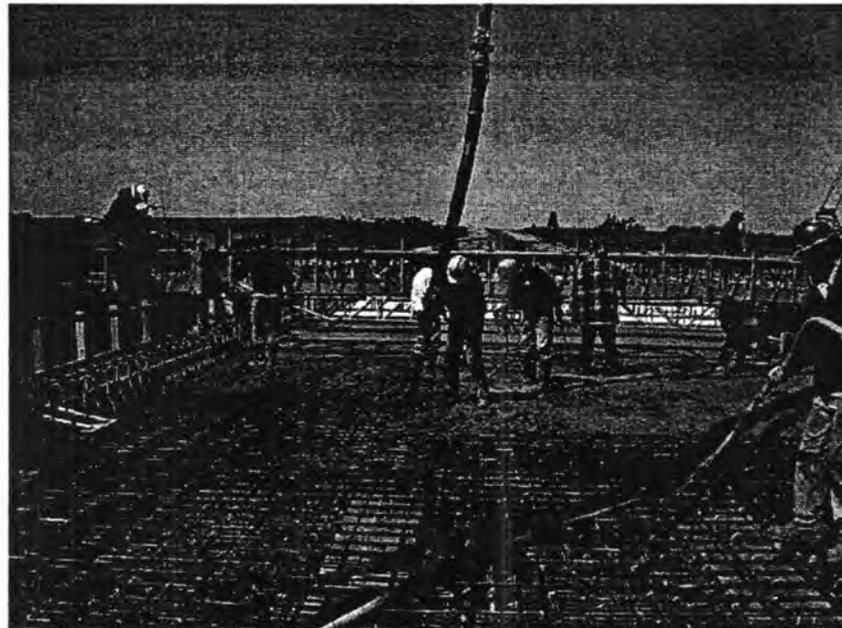


(a) – Side view of girders and piers

Figure 2.6 – Photographs of the construction process through completion.

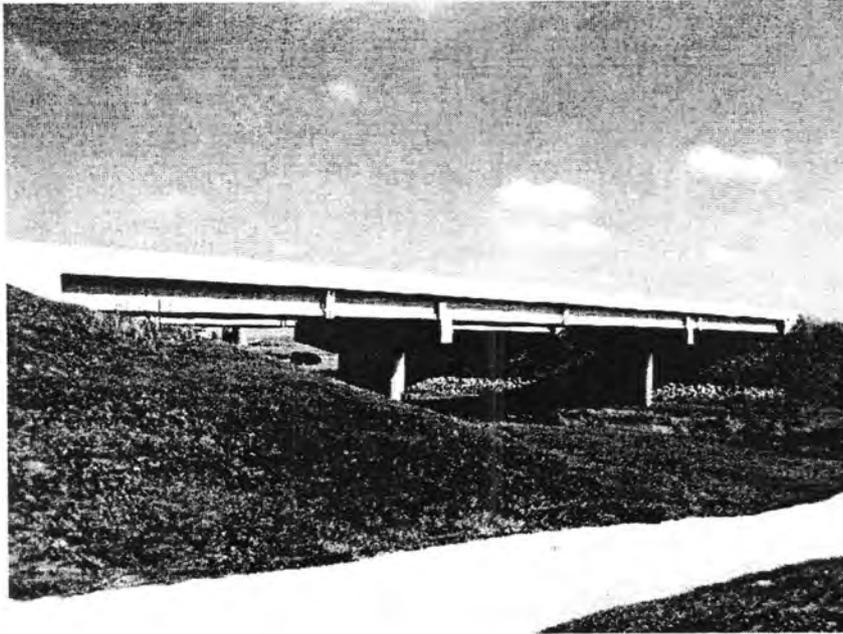


(b) – Typical girder connection at pier



(c) – Concrete placement for MMFX Bridge

Figure 2.6 – Continued – Photographs of the construction process through completion.



(d) – Completed MMFX Bridge



(e) – Completed inner span and west pier

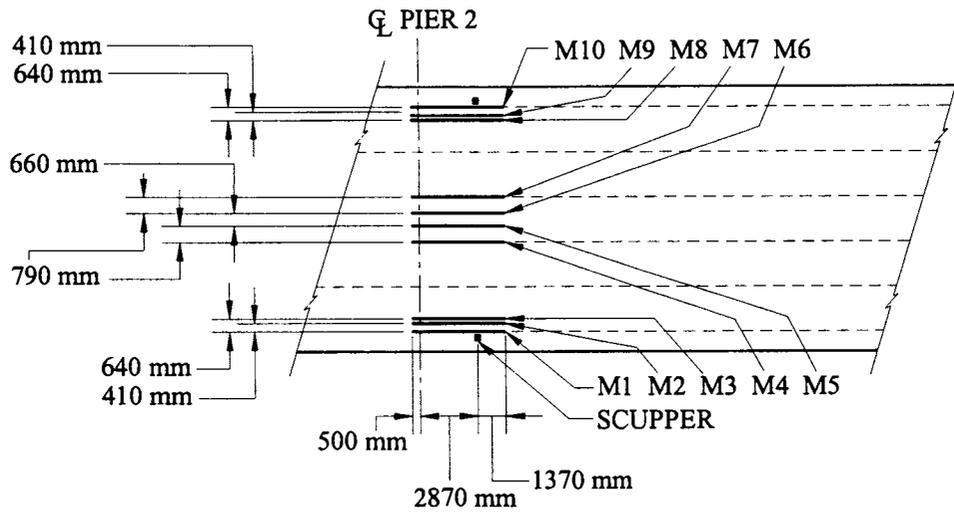
Figure 2.6 – Continued – Photographs of the construction process through completion.

2.3.2.2 Corrosion Monitoring System

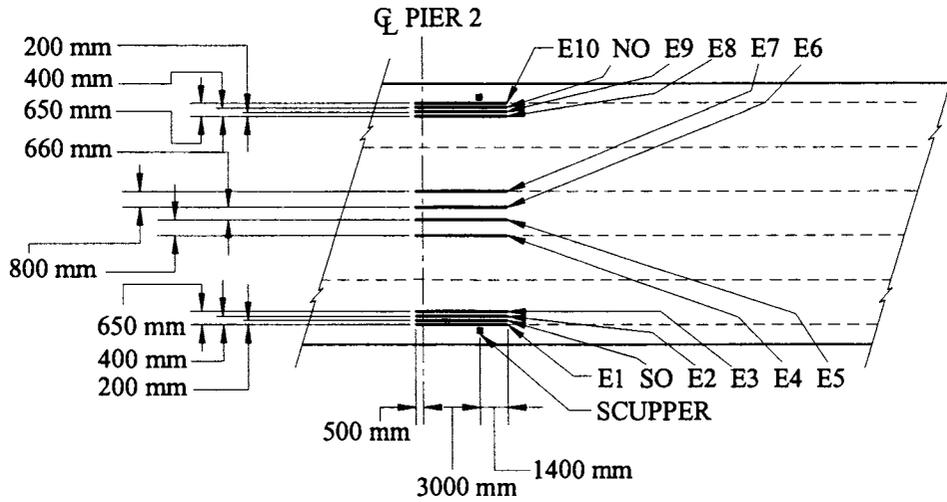
Corrosion detecting devices, the previously described V2000 electrode, were permanently embedded in both bridge decks prior to concrete placement. The use of this embedded electrode offers the ability to monitor the interior state of a structure by measuring parameters that can be used as reliable indicators of the likelihood of corrosion in the surrounding area. Although the electrode does not address the specific electrochemical mechanisms, it provides a reliable and cost-effective monitoring system for the onset of corrosion, the cessation of corrosion, and the intensity of corrosion growth.

Twenty No. 25 top bars, 10 on each bridge (denoted M1 through M10 on the MMFX Bridge and E1 through E10 on the Epoxy Bridge) in the negative bending moment region near the eastern drainage points were instrumented. On the Epoxy Bridge, two additional short sections of MMFX reinforcement were each instrumented with an electrode and tied between other epoxy-coated reinforcement (one on the north side, denoted NO, and one on the south side, denoted SO) to compare with the epoxy-coated reinforcement within the same environment. The electrode locations within each bridge deck are shown in Figure 2.7.

The electrode sensors were wound around each bar for approximately a length of 4.6 m. Each electrode was connected to a red-sheathed copper lead wire with a protected butt splice, while a black-sheathed copper lead wire was attached directly to the bar using a stainless steel clamp. Both lead wires for each monitored bar were run out of the bridge deck and used to measure internal voltage and electrical current to assess corrosion development.

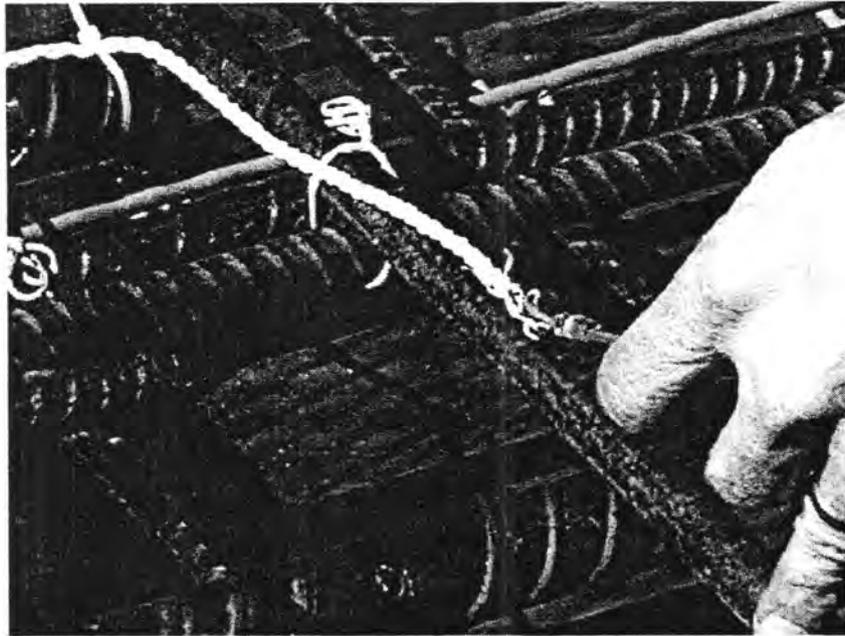


(a) – MMFX Bridge



(b) – Epoxy Bridge

Figure 2.7 – General locations of the V2000 electrode.



(a) – Connection of V2000 electrode to copper lead wire



(b) – Extending lead wires for future corrosion monitoring

Figure 2.8 – Typical photographs of the instrumentation installation.

2.3.2.3 Field Monitoring and Discussion

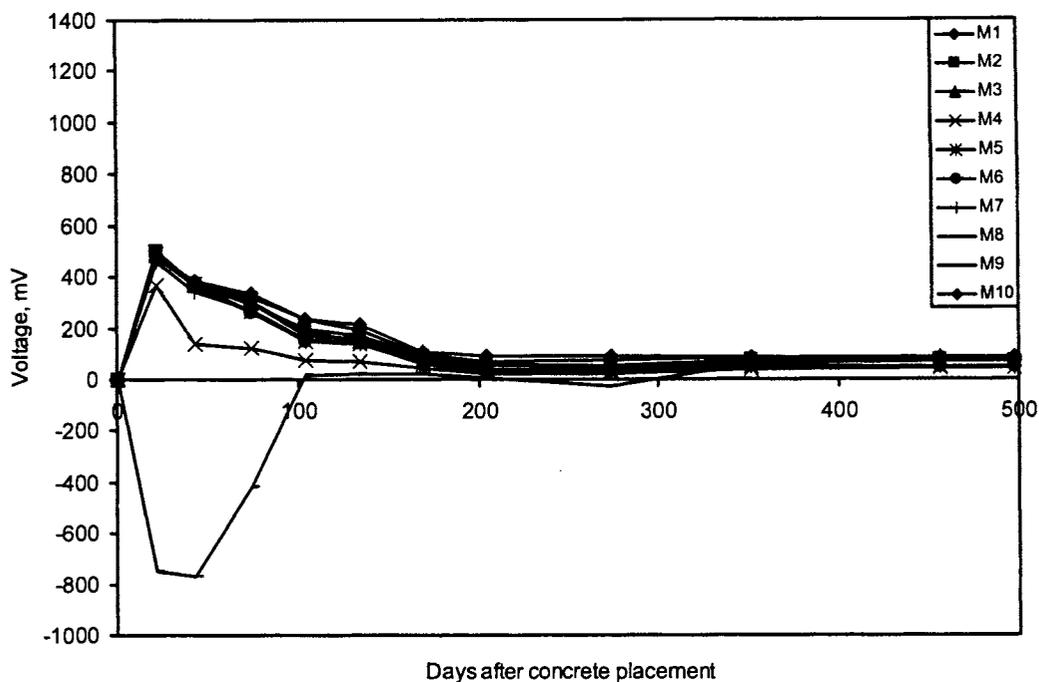
Visual inspections of both bridges have regularly been conducted to identify any sign of corrosion. No obvious external signs of damage have been observed (i.e., no signs of cracking, spalling, delamination, etc).

By measuring DC voltage (in millivolts, mV) and DC current (in microamps, μA) with a voltmeter, corrosion activity (DC voltage) and severity (DC current) can be determined. Since considerable moisture is present, high voltage levels (even over 1000 mV) shortly after concrete placement are expected. While the concrete is fresh and uncured, the bridge deck (concrete and reinforcement) is highly active and generates high output. As the concrete cures, the initial “spike” typically subsides back to within the “normal” range of less than 400 mV, indicating no occurrence of corrosion.

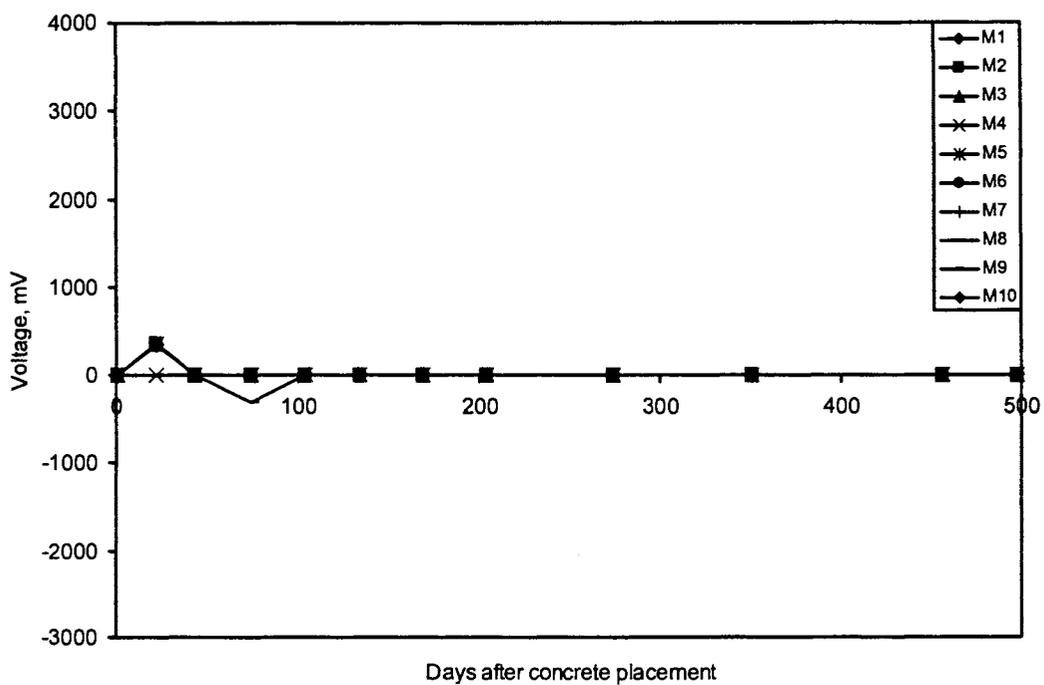
In general, current measurements below 0.100 mA (1000 μA) are considered a weak site of corrosion. When corrosion occurs, however, a natural DC electrical current begins to flow from one area to another and current measurements increase significantly. DC electrical currents in excess of 1000 μA are considered to have undergone corrosion.

Figures 2.9 and 2.10 show the internal voltage and electrical current measurements, respectively, for all instrumented steel reinforcement in both bridge decks.

All of the MMFX Bridge measurements appear as expected. Although the voltage increased above 400 mV initially, normal levels were measured after the concrete cured. Even while the voltage increased above 400 mV, the current remained below 1000 μA (i.e., indication of weak site of corrosion). Approximately 84 days (12 weeks) after concrete placement, all voltage levels for the MMFX Bridge dropped steadily and remained less than 100 mV. After 490 days (70 weeks) of monitoring, there is no indication of corrosion activity.

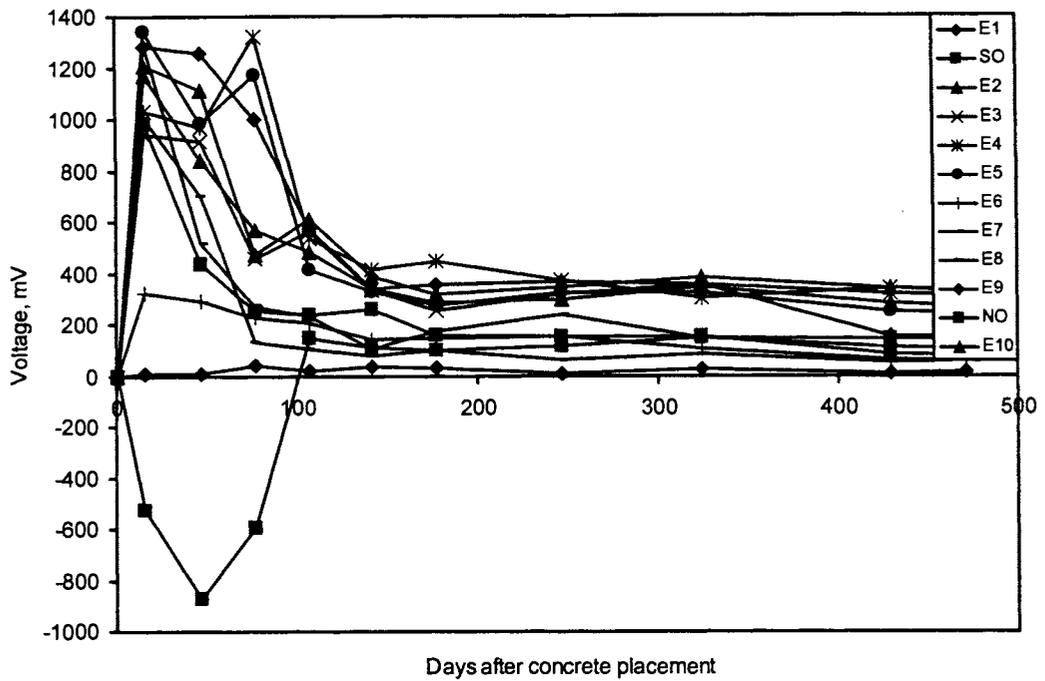


(a) Internal voltage

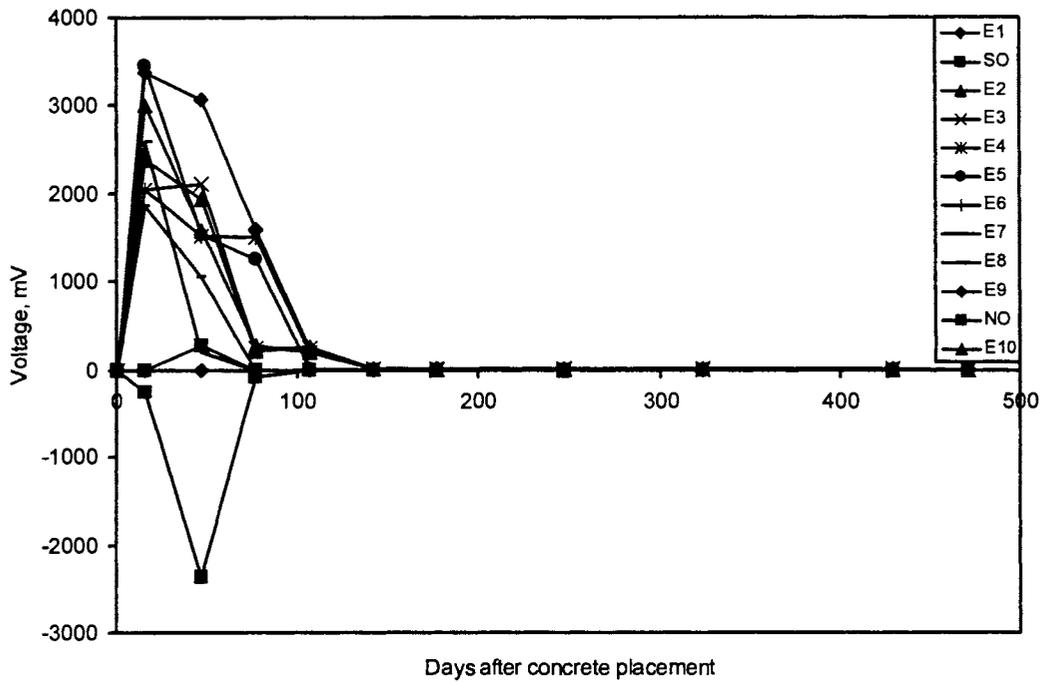


(b) Electrical current

Figure 2.9 – Measurements from V2000 electrode instrumented on the MMFX Bridge.



(a) – Internal voltage



(b) – Electrical current

Figure 2.10 – Measurements from V2000 electrode instrumented on the Epoxy Bridge.

The epoxy-coated reinforcement, however, behaved somewhat unexpectedly. Measurements on the Epoxy Bridge were higher (about two times higher than the measurements on the MMFX Bridge) than originally expected. During the initial stage, shortly after concrete placement, some of measurements increased over 1200 mV, as shown in Figure 2.9. Although these most of these high voltage measurements dropped below 400 mV, some are still above 300 mV. It is believed, if the bar is coated perfectly the measurement should be nearly zero. As shown in Figure 2.10, it is evident that contact between the steel and concrete has been made on virtually all specimens being monitored. This is an indication that there is at least one or more holidays (i.e., the coatings may have defects from the layout process of construction) in the epoxy coating on most bars.

Overall, the measurements indicate that measurements are lower for the MMFX Bridge than for the Epoxy Bridge. However, no significant corrosion has been observed in either bridge deck (i.e., electrical current measurements are currently close to zero as shown in Figure 2.10). Although not completed, through continued monitoring and evaluation, the on-going field monitoring system is expected to provide evidence of the corrosion resistance of the reinforcement monitored. So as definitive evidence of the corrosion resistance of MMFX reinforcement could not be surmised, attention transferred to investigate the reinforcement under accelerated conditions in a laboratory.

CHAPTER 3 LABORATORY TEST PROGRAM

As previously stated, the principal reason for selecting a new reinforcement material for concrete bridge decks is to improve the life expectancy and cost effectiveness of the structural system. A requirement of the material, which presumably is more expensive, is that it provides a significant improvement in corrosion resistance compared to the current material of choice, epoxy-coated mild steel reinforcement while at the same time meeting the requirements of ASTM A 775. In light of this requirement, this study compared the corrosion resistance of MMFX Microcomposite steel reinforcement with that of epoxy-coated and uncoated mild steel reinforcement using the ASTM G 109 accelerated corrosion. An additional test method introduced by the University of Kansas Center for Research is referred to as the Rapid Macrocell accelerated corrosion test was also used for the evaluation presented herein.

3.1 Material Properties

Steel reinforcement used in the laboratory test program described herein consisted of one heat of No. 16 (No.5) MMFX, epoxy-coated, and uncoated reinforcement. The MMFX reinforcement was obtained from the Iowa Department of Transportation. The epoxy-coated reinforcement was provided for evaluation by Construction Material Incorporated of Des Moines, Iowa. The uncoated reinforcement was acquired through a local distributor. A single batch of concrete was utilized to preserve uniformity among the individual test specimens and between the tests. The following paragraphs describe both the properties of the materials used in the subsequently described corrosion monitoring program.

3.1.1 Steel Reinforcement Properties

Although published data exists, the MMFX, epoxy-coated, and uncoated reinforcement used in the laboratory study was tested to determine yield strength, tensile strength, and

elongation. Three specimens of each steel type were tested to determine the mechanical properties. The tests were conducted were conducted following ASTM E8.

The results of the mechanical tests are presented in Table 3.1, including yield strength, tensile strength, and elongation for each of the steel reinforcement types tested. Yield strengths were measured based on a well-defined yield point for epoxy-coated and uncoated steels and based on the 0.2 percent offset method for MMFX steel. Data is reported for No. 16 (No. 5) MMFX, epoxy-coated, and uncoated mild reinforcement with as-delivered cross-sections. The average of the three trials per reinforcement type was also reported.

Table 3.1 – Mechanical properties of steel reinforcement.

Reinforcement Identification	Yield Strength, ksi	Tensile Strength, ksi	Elongation, percent in 24 in.
MMFX ¹ (1)	114.2	165.1	7.5
MMFX (2)	110.6	158.0	7.3
MMFX (3)	118.4	167.5	6.9
<i>MMFX Average</i>	<i>114.4</i>	<i>163.5</i>	<i>7.2</i>
UC ² (1)	58.5	96.0	16.4
UC (2)	60.1	96.0	16.6
UC (3)	60.1	95.6	16.2
<i>UC Average</i>	<i>59.6</i>	<i>95.9</i>	<i>16.4</i>
EC ³ (1)	66.7	106.6	14.3
EC (2)	67.1	106.3	13.5
EC (3)	65.7	104.1	9.8
<i>EC Average</i>	<i>66.5</i>	<i>105.7</i>	<i>12.6</i>

1 MMFX – MMFX Microcomposite steel reinforcement

2 UC – Uncoated mild steel reinforcement

3 EC – Epoxy-coated mild steel reinforcement

3.1.1.1 MMFX Microcomposite Steel Reinforcement

The No. 16 (No.5) MMFX reinforcement exhibited yield strengths between 114.2 and 118.4 ksi, with an average of 114 ksi. Tensile strengths were recorded between 158.0 and 167.5 ksi, with an average of 163 ksi. The total elongations at failure ranged between 6.9 and 7.5 percent, with an average of 7.2 percent.

These values match those obtained by the University of Kansas Center for Research and differ from those reported by MMFX Steel Corporation of America as previously shown in Table 2.1. The University of Kansas Center for Research reported a yield strength range of 110 to 120 ksi, tensile strength range from 160 to 175 ksi, and elongation range from 6.3 to 7.8 percent (4). While MMFX Steel Corporation of America reported a yield strength range from 119 to 133 ksi and tensile strength range from 180 to 186 ksi (20).

3.1.1.2 Uncoated Mild Steel Reinforcement

Yield strengths for No. 16 (No. 5) uncoated reinforcement ranged from a low of 58.5 ksi to a high of 60.1 ksi, with an average of 59.6 ksi. Tensile strengths ranged from 95.6 to 96.0 ksi, with an average of 95.9 ksi. Elongations ranged from a low of 16.2 to a high of 16.6 percent, with an average of 16.4 percent.

3.1.1.3 Epoxy-coated Mild Steel Reinforcement

The No. 16 (No. 5) epoxy-coated reinforcement exhibited yield strengths between 65.7 ksi and 67.1 ksi, with an average of 66.5 ksi. Tensile strengths were recorded between 104.1 and 106.6 ksi, with an average of 105.7 ksi. The total elongations at failure ranged between 9.8 and 14.3 percent, with an average of 12.6 percent.

3.1.2 Concrete Mix Properties

All of the test specimens, described in the following section, were constructed from a single 1-1/2 yd³ batch of ready-mix concrete to ensure that the MMFX, epoxy-coated, and uncoated steel reinforcement were subject to similar conditions. Compressive strength and modulus of rupture for the concrete mix proportions used in corrosion resistance performance tests were conducted and summarized in Table 3.2 with other pertinent properties.

Table 3.2 – Mix proportions per cubic yard and concrete properties.

Property	Quantity
Cement	500 lb
Sand	1526 lb
Course aggregate ¹	1489 lb
Water	217 lb
Fly ash	64 lb
Air-entraining agent	2 oz
Air content	5.5 percent
Unit weight	3815 pcy
Slump	3.0 in.
Average 28-day compressive strength	5964 psi
Average 28-day modulus of rupture	623 psi

¹ Course aggregate – 3/8 inch nominal maximum size

3.2 Accelerated Corrosion Test Program

Corrosion resistance performance is evaluated based accelerating the corrosion process in laboratory specimens. Changes in corrosion potential, relative corrosion rates, and chloride

concentrations needed for corrosion initiation are monitored. Additionally, interval powder samples were collected and analyzed through X-ray fluorescence spectrometry for chloride content comparison. At the conclusion of the test program, evaluation of the corrosion products on the steel reinforcement will also be performed utilizing scanning electron microscopy.

3.2.1 Accelerated Corrosion Tests

Both the ASTM and Rapid Macrocell accelerated corrosion tests induce general and pitting corrosion and are believed to provide valid comparisons using realistic exposure conditions. The study presented herein will utilize these two test methods to accelerate the corrosion of different types of steel embedded in concrete.

3.2.1.1 ASTM G 109 Accelerated Corrosion Test

Comparisons of corrosion response are made using the ASTM G 109 accelerated corrosion test (ACT), a test first developed to study the effective corrosion protection of chemical admixtures on steel reinforcement (22). While the ASTM ACT was originally developed to evaluate admixtures materials intended to inhibit chloride-induced corrosion of steel in concrete, over the past two decades, the test method has been most notably used to evaluate the corrosion response of corrosion-resistant steel reinforcement. Although the ASTM ACT typically requires one to two years for completion, the test method provides a severe corrosion environment that is believed to simulate 30 to 40 years of exposure for bridges.

The ASTM ACT is used to model the corrosion of steel reinforcement in concrete where two layers of reinforcement are present (i.e., the cross-section of a bridge deck). The test specimen consists of a small beam constructed with two layers of steel reinforcement. The top layer of reinforcement consists of one bar, while the bottom layer consists of two bars. The layers are connected electrically across a 10-ohm resistor and the sides of the concrete

are sealed with epoxy. A reservoir is secured to the beam, to retain liquid on the upper surface.

The beam subjects 229 mm (9 in.) of reinforcement below the surface to alternate cycles of wetting and drying with a 3 percent sodium chloride solution. This is illustrated in Figure 3.1. The cycles of wetting allow for chloride ingress to the reinforcement level while the cycles of drying allow for oxygen to replenish.

The half-cell corrosion potentials for the top and bottom layers are measured as an indicator for the onset of corrosion. At the initiation of corrosion, concrete powder samples are taken at the level of the top reinforcement to estimate the chloride-ion concentration required for corrosion initiation. Additionally, corrosion current and the corresponding corrosion rates are determined by measuring the voltage drop across the resistor.

3.2.1.2 Rapid Macrocell Accelerated Corrosion Test

An additional test method introduced as the Rapid Macrocell ACT compares the corrosion response of steel reinforcement. The Rapid Macrocell ACT was originally developed at the University of Kansas under the SHRP program (23, 24) and updated under the NCHRP-IDEA program (4). The goal of the technique is to obtain a realistic measure of the performance of corrosion protection systems over a shorter period of time than traditional accelerated corrosion tests (i.e., ASTM ACT).

The basic test system requires two containers and consists of either bare or mortar-clad steel reinforcement. This is illustrated in Figure 3.2. The contact surface between the mortar and the bar simulates the concrete-reinforcement interface in actual structures. A single bar, either bare or mortar-clad, is placed in a 1-quart container with a simulated pore solution, based on the ASTM G 109 ACT, containing a 3 percent concentration of sodium chloride. Two bars are placed in a second 5-quart container and immersed in simulated pore solution with no chlorides added. The solution in both containers places 76 mm (3 in.) of

reinforcement below the surface. The solutions in the two containers are connected by a salt bridge and the test specimen in the pore solution containing sodium chloride (anode) is electrically connected through a single 10-ohm resistor to the two specimens in the simulated pore solution (cathode).

Air is bubbled into the pore solution surrounding the cathode to ensure an adequate supply of oxygen is present for the cathodic reaction. The air causes some evaporation, which is countered by adding distilled water to this container to maintain a constant volume of solution.

Similar to the ASTM ACT, half-cell corrosion potentials for the anode and cathode are measured, again to establish corrosion initiation. The corrosion current and the rate of corrosion are also determined by measuring the voltage drop across the resistor.

3.2.1.3 Accelerated Corrosion Test Monitoring

Half-cell potentials are measured using a reference electrode. The steel reinforcement layers are isolated (i.e., each bar is disconnected from the resistor) before the measurement of half-cell potential to avoid the interference from the other steel elements. After the measurements are performed, the steel elements are again electrically connected through the resistor.

The half-cell corrosion potential of the anode and cathode are measured using a saturated calomel electrode. The half-cell is maintained in accordance with ASTM C 876 for the stabilization of corrosion potential. The associated corrosion condition with varying half-cell corrosion potentials from the saturated calomel electrode are listed in Table 3.3 (9). In the study presented herein, a corrosion potential more positive than 276 mV was considered as active corrosion of the metal.

Table 3.3 – ASTM criteria for corrosion of steel in concrete for the saturated calomel reference electrode (9).

Corrosion Potential	Corrosion Condition
Less than 126 mV	Low (10 percent risk of corrosion)
126 mV to 276 mV	Intermediate corrosion risk
Greater than 276 mV	High (90 percent risk of corrosion)
Greater than 426 mV	Severe corrosion risk

3.2.2 Accelerated Corrosion Test Specimens

A total of 22 specimens were tested following the ASTM ACT to study the corrosion behavior when chlorides have rapid access to the steel. Six Rapid Macrocell ACT specimens were completed for each reinforcement configuration. Table 3.4 lists the designation used to refer to all tests specimens in the following section.

In both the ASTM ACT and Rapid Macrocell ACT, the MMFX, epoxy-coated, and uncoated reinforcement were tested in the “as-delivered” condition. Two additional conditions for the epoxy-coated reinforcement were evaluated. First, the epoxy coating was breached by four 1/8-inch diameter holes drilled equidistantly in line along the reinforcement length. Epoxy-coated reinforcement specimens were also evaluated where the coating was chipped by off at random locations through the use of a razor blade removing approximately a total area of 64 mm² (2-1/2 in²) of the original coating.

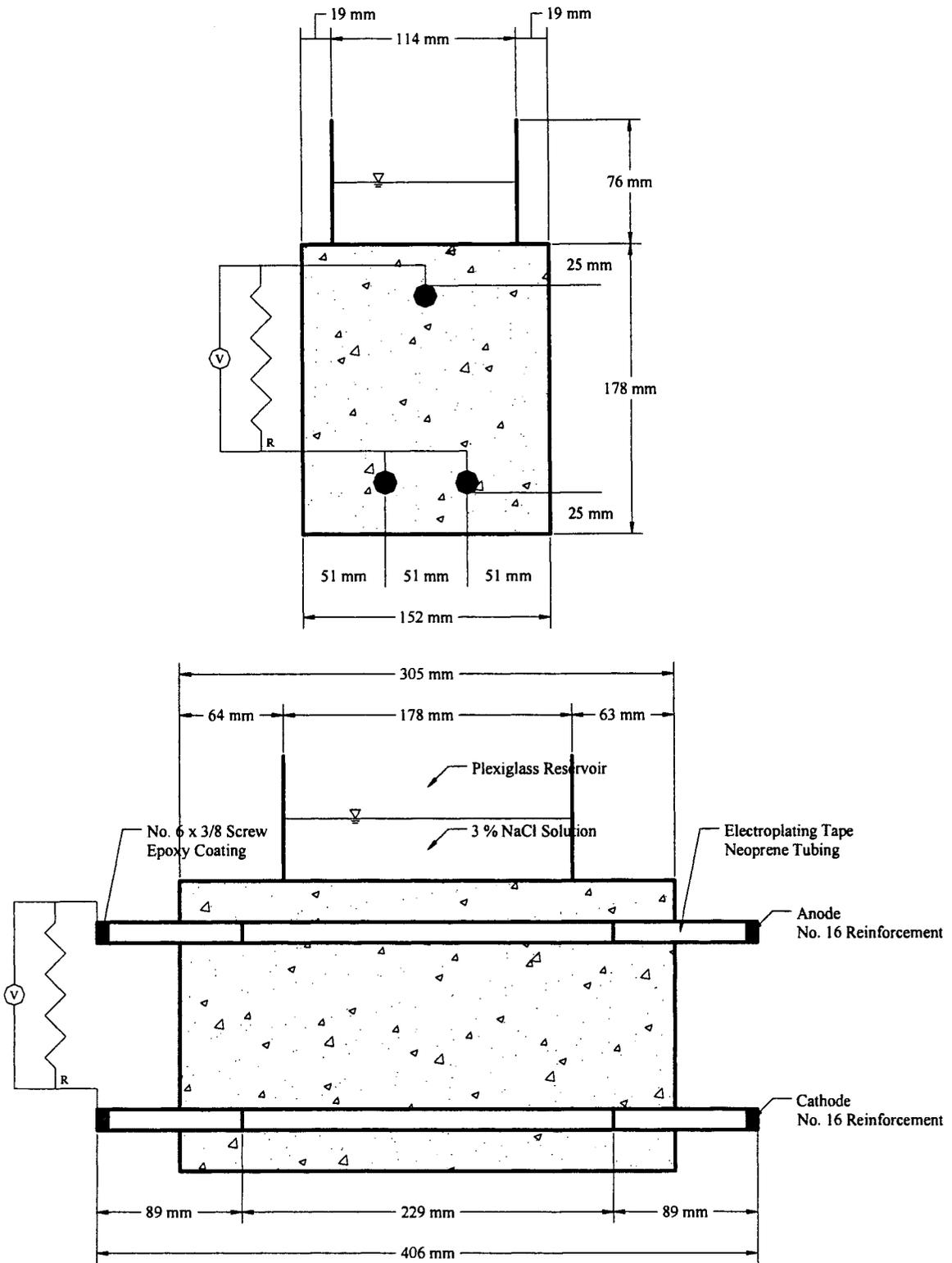
Table 3.4 –Accelerated corrosion test program.

Specimen Identification	NaCl Concentration	Number of Specimens
A ¹ -L ² -MMFX ³	3 percent	3
A-T ⁴ -MMFX	3 percent	2
A-L-UC ⁵	3 percent	3
A-T-UC	3 percent	2
A-L-EC ⁶ -AD ⁷	3 percent	3
A-T-EC-AD	3 percent	2
A-L-EC-DH ⁸	3 percent	3
A-T -EC-DH	3 percent	2
A-T -EC-CH ⁹	3 percent	2
RM ¹⁰ -MMFX	3 percent	6
RM-UC	3 percent	6
RM-EC-AD	3 percent	6
RM -EC-DH	3 percent	6

1 A – ASTM G 109 accelerated corrosion test
2 L – Artificial longitudinal crack
3 MMFX – MMFX Microcomposite steel reinforcement
4 T – Artificial Transverse cracks
5 UC – Uncoated mild steel reinforcement
6 EC – Epoxy-coated mild steel reinforcement
7 AD – As-delivered epoxy coating condition
8 DH – Drilled holiday epoxy coating condition
9 CH – Chipped holiday epoxy coating condition
10 RM – Rapid Macrocell accelerated corrosion test

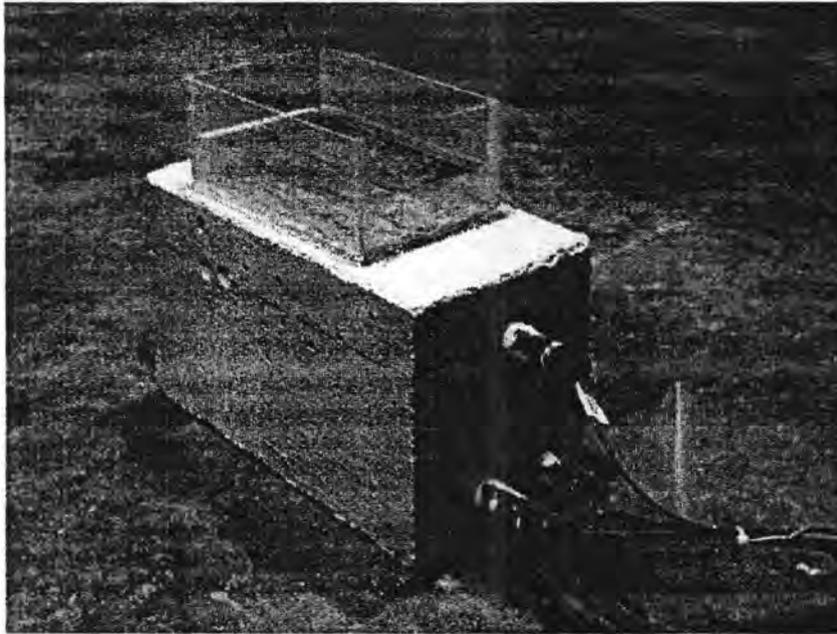
3.2.2.1 ASTM G 109 Accelerated Corrosion Test Specimen

The 22 ASTM ACT concrete beams were cast in a single set of formwork. As illustrated in Figure 3.1, the beams were cast 152 mm (6 in.) in width, 178 mm (7 in.) in height, and 305 mm (12 in.) in length. The forms were constructed with holes in the appropriate locations to position the No. 16 (No. 5) reinforcement to maintain exactly 25 mm (1 in.) of clear cover.



(a) – Schematic of ASTM G 109 ACT

Figure 3.1 – Schematic of ASTM G 109 accelerated corrosion test specimen.



(b) – Typical as-constructed ASTM G 109 ACT

Figure 3.1 – Continued – Schematic of ASTM G 109 accelerated corrosion test specimen.

Concrete was placed in two layers and consolidated by internal vibration. A float finish was applied after consolidation. Additional un-reinforced concrete beams were cast for background chloride analysis.

To leave a direct path for chlorides to the top layer of steel reinforcement, an artificial crack was fabricated in the specimen. The crack was oriented either parallel or perpendicular to, and directly above, the top steel reinforcement through the insertion and removal of a 0.3 mm (0.012 in.) stainless steel shim when the specimen was fabricated. The shim was removed within 24 hours of placement, leaving a direct path for chlorides to the steel reinforcement and simulating the effects of a settlement crack over the bar.

The No. 16 (No.5) reinforcement was cut to a length of 406 (16 in.) and both ends were ground to remove any sharp edges. One end of each bar was drilled to a depth of 13 mm (1/2 in.) to accept a self-tapping No. 6 x 3/8 in. stainless steel sheet metal screw. The bars were

soaked in hexane until clean of grease, dirt, and hydraulic fluid, and allowed to air dry. Each end of the bar was wrapped with electroplating tape so that only a 229-mm (9-in.) length in the middle of the bar was bare. A 89-mm (3-1/2-in.) length of neoprene tubing was placed over the electroplating tape at each end of the bar. The length of tubing protruding from the bar ends was filled with two applications of 3M Scotchkote 413/215 PC Patch Compound two-part epoxy to prevent the bar ends from corroding due to external interference.

The bars were placed in the forms so the 229-mm (9-in.) bare region was centered within the concrete. A single bar was placed in the top reinforcement layer, while two bars were placed in the bottom layer. The epoxy-coated mild steel reinforcement with drilled holidays was placed with the line of holes facing the top surface to simulate a worst-case scenario. Whereas the epoxy-coated mild steel reinforcement with chipped holidays was randomly breeched, no particular attention was paid to the alignment of these holidays. The two layers of steel reinforcement were connected electrically across a 10-ohm resistor.

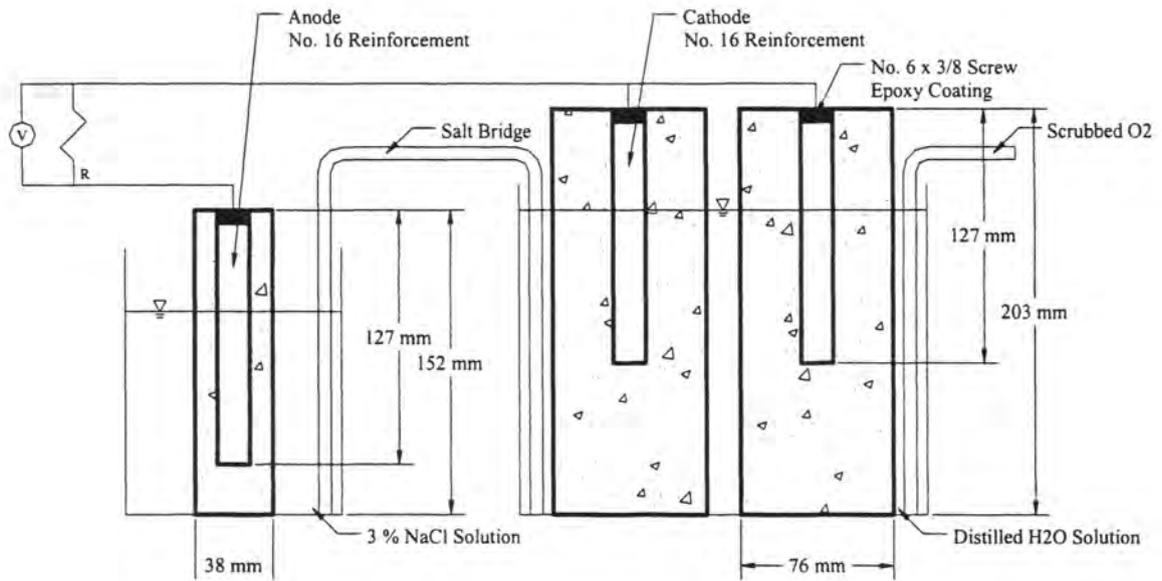
The beams were covered with wet burlap and plastic sheets for the initial curing. The specimens were then stripped from the forms and cured under the wet burlap and plastic sheets until age 21 days. The top test surfaces were slightly sanded at age 18 to 20 days to remove the portland cement skin or laitance, which normally wears off by natural weathering. At age 21 days, they were then stored, on their sides, in air in a laboratory room at 68 to 78 degrees Fahrenheit for 7 days. At approximately age 28 days, the four sides and bottom of each test specimen were coated with two applications of Sierra Performance Manufacturing two-part epoxy concrete paint to minimize lateral moisture movement in the specimens during the tests.

A Plexiglas reservoir was constructed 114 mm (4-1/2 in.) in width by 178 mm (7 in.) in length and 76 mm (3 in.) in height was attached to the specimen, as depicted in Figure 3.1. A silicone caulk was used to seal the reservoir from the outside, and the epoxy sealer was applied to the top surface of the specimen outside of the reservoir.

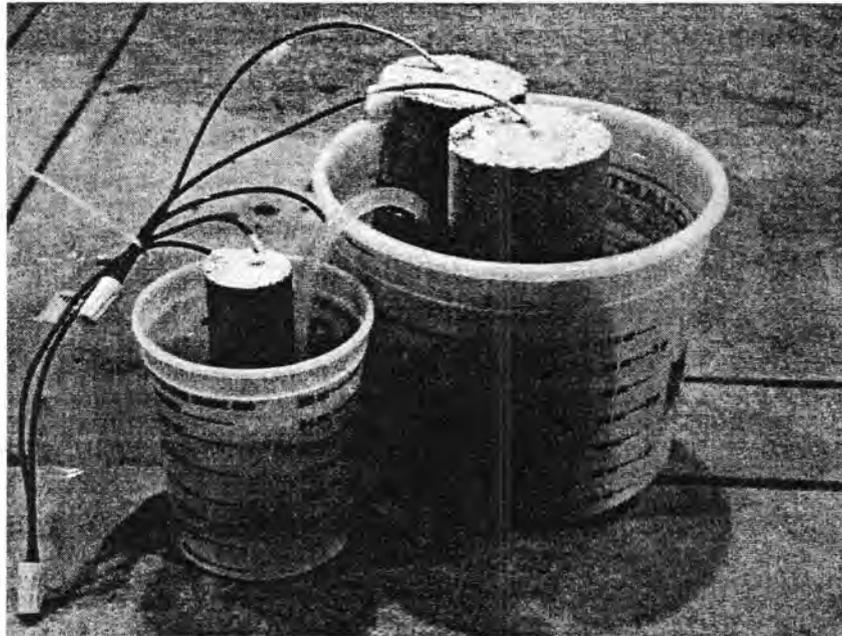
3.2.2.2 Rapid Macrocell Accelerated Corrosion Test Specimen

As illustrated in Figure 3.2, No. 16 (No. 5) reinforcement was cut to a length of 127 mm (5 in.) for the Rapid Macrocell ACT. One end of the reinforcement was then drilled to a depth of 13 mm (1/2 in.) to accommodate a self-tapping No. 6 x 3/8 stainless steel sheet metal screw and both ends were ground removing sharp edges. The bars were then soaked in hexane to remove grease, dirt, and hydraulic fluid from the surface, and dried at room temperature. The end of the epoxy-coated reinforcement that was to be submerged was protected using 3M Scotchkote 413/215 PC Patch Compound two-part epoxy.

Three cylindrical mortar-clad bars are necessary for each laboratory test system. A single mortar-clad bar was cast in a mold consisting of 152 mm (6 in.) and 38 mm (1-1/2 in.) diameter PVC pipe to serve as the system anode. The bar was centered in the mold with the mortar sheathing covering the exterior surface of the bar and projecting 25 mm (1 in.) past one end of the reinforcement. Two mortar-clad bars were cast in a molds consisting of 203 mm (8 in.) and 76 mm (3 in.) diameter PVC pipe to serve as the system cathode. Similarly, these bars were centered in the mold with the mortar sheathing covering the exterior surface of the bar. However, for the cathode the mortar sheathing projected 76 mm (3 in.) past one end of the reinforcement. The mix proportions for the “mortar” were the same as the concrete used in the ASTM ACT specimens. The concrete was placed in the cylindrical mold in two layers. Each layer was rodded 25 times using a 1/8-in. diameter rod, followed by external vibration for 30 seconds.



(a) – Schematic of Rapid Macrocell ACT



(b) – Typical as-constructed Rapid Macrocell ACT

Figure 3.2 – Schematic of Rapid Macrocell accelerated corrosion test specimen.

Specimens were cured in the molds for 24 hours and then removed from the molds and cured in saturated water for 13 days. After 14 days of curing, the specimens were dried for one day. For the mortar-clad specimens, a 14-gauge copper electrical wire was secured to the tapped end of each specimen with a self-tapping No. 6 x 3/8 stainless steel sheet metal screw. The top of the screw, exposed wire, and concrete were then coated with two applications of 3M Scotchkote 413/215 PC Patch Compound two-part epoxy and Sierra Performance Manufacturing two-part epoxy concrete paint, respectively.

3.2.2.3 Rapid Macrocell Accelerated Corrosion Test Salt Bridge

The Rapid Macrocell ACT salt bridge serves as an ionic pathway between the containers holding the specimens. Each salt bridge is made using a 0.9-m (36-in.) long flexible Tygon tube with an inner diameter of 6.4 mm (1/4 in.) and outside diameter of 9.5 mm (3/8 in.). The tube is filled with a salt gel made from 4.5 g of agar, 30 g of potassium chloride (KCl), and 100 g distilled water, a quantity of ingredients adequate to make three salt bridges.

1. The salt gel is made by thoroughly mixing the agar and potassium chloride powders together. The mixture is then combined with distilled water and placed over a Bunsen burner.
2. While heating over the Bunsen burner, the three constituents (agar, KCl, and distilled water) are stirred to a consistency of syrup. The semisolid gel (syrup consistency) is poured into the tubing.
3. At this point, the completed salt bridge is heated in a container of boiling water for approximately 4 hours. The salt bridge is taken out of the boiling water and allowed to cool at room temperature.

If the semisolid gel does not fully fill the tube (i.e., voids present) this salt bridge is discarded.

3.2.3 Chloride Exposure Protocol

The ASTM ACT chloride exposure condition was based upon a weekly cycle. The beams are subjected to a seven day ponding and drying regime. For the first 4 days of each week, the test surface was ponded with a depth of approximately 38 mm (1-1/2 in.) of 3 percent sodium chloride solution in a laboratory at 68 to 78 degrees Fahrenheit. During this period, the reservoir was covered with a plastic sheet to minimize evaporation. Following this 4 day exposure, the NaCl solution was removed, and the test surface was rinsed with distilled water and drained.

These unponded beams remained dry for three days in a laboratory at 68 to 78 degrees Fahrenheit. After this exposure the test surface was immediately reponded with the 3 percent NaCl solution. The ponding and drying regime was continued for 12 weeks where upon completion the test surface was subject to continuous ponding for 12 weeks. Following the 12-week interval of continuous ponding, the alternating ponding and drying regime was resumed. The two regimes were continued for remainder of the test period.

For the Rapid Macrocell ACT, the mortar-clad specimen was placed in a 1-quart container, along with a simulated pore solution containing a 3 percent concentration of sodium chloride for the duration of the test period. When needed, simulate pore solution was added to maintain the 3 inches of reinforcement below the surface.

CHAPTER 4 LABORATORY TEST RESULTS

The test results described in the following section describe the corrosion resistance performance of MMFX Microcomposite, epoxy-coated mild, and uncoated mild steel reinforcement. Specific findings are presented in terms of half-cell voltage (corrosion potential) for the ASTM G 109 ACT and Rapid Macrocell ACT tests. The different reinforcement type for a particular accelerated corrosion test specimens are distinguished by line type in the same figure. The designation for each reinforcement type is as listed in Table 3.4.

4.1 ASTM G 109 Accelerated Corrosion Test

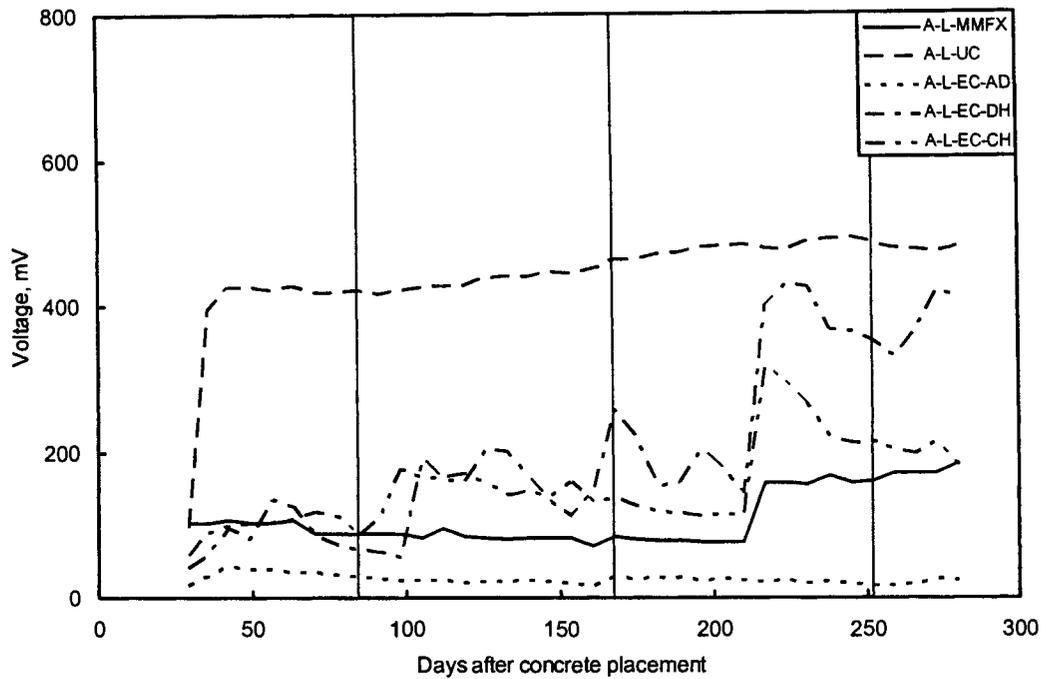
As previously discussed, to leave a direct path for chlorides to the top layer of steel reinforcement, an artificial crack, oriented either longitudinally or transversely was fabricated in the specimen.

4.1.1 Results for Longitudinally Cracked Specimens

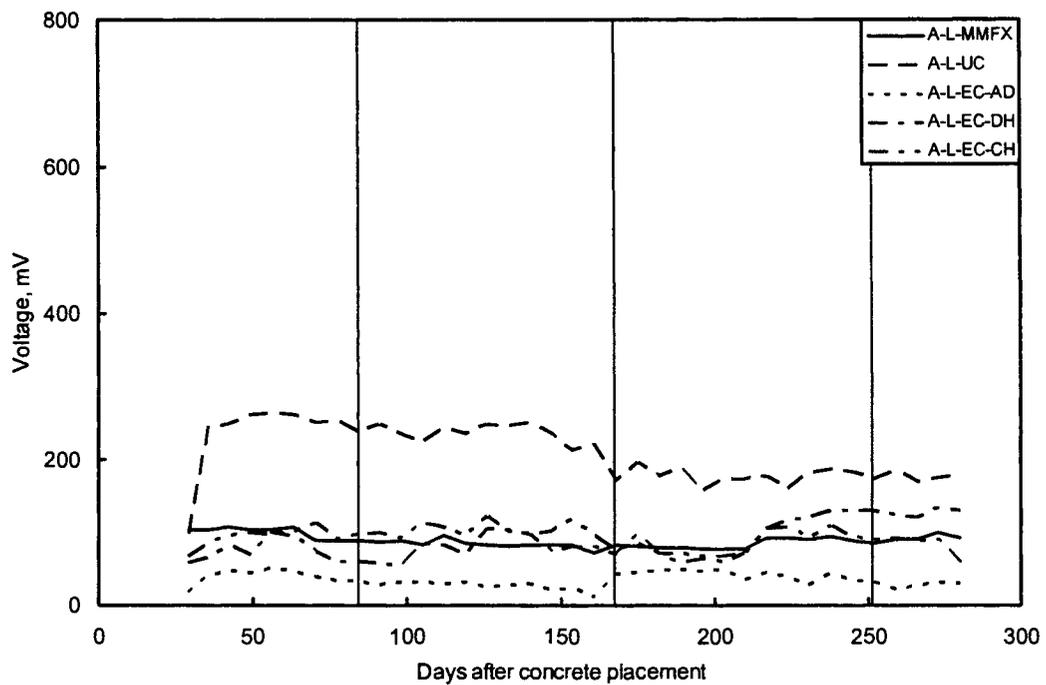
The 280-day (40-week) average anode (top reinforcement layer) and cathode (bottom reinforcement layer) corrosion potentials for specimens with a longitudinal artificial crack over the top layer of steel reinforcement are shown in Figure 4.1.

4.1.1.1 MMFX Microcomposite Steel Reinforcement

The corrosion potential for the top layer of reinforcement (anode) for the MMFX reinforcement with longitudinally cracked specimens remained at a relatively constant value of 100 mV through 217 days (31 weeks). At 217 days, a single MMFX specimen began corroding which caused the rapid increase and continued increase to 183 mV through 280 days (40 weeks).



(a) – Corrosion risk of the top layer of steel reinforcement



(b) – Corrosion risk of the bottom layer of steel reinforcement

Figure 4.1 – ASTM G 109 ACT subjected to 3 % NaCl solution through longitudinal crack.

4.1.1.2 Uncoated Mild Steel Reinforcement

The corrosion potential for all the uncoated reinforcement specimens increased beyond 276 mV (i.e., high risk of corrosion) by 35 days (5 weeks). After 35 days, the uncoated specimens experienced a corrosion potential value greater than 400 mV. The corrosion potential rose to a maximum of 493 mV at 245 days (35 weeks) and has remained constant through 280 days (40 weeks), indicating a continued severe risk for corrosion.

4.1.1.3 As-delivered Epoxy-coated Mild Steel Reinforcement

Specimens with the as-delivered epoxy-coated reinforcement exhibited a relatively constant corrosion potential value of 25 mV through 280 days (40 weeks), indicating a low risk for corrosion.

4.1.1.4 Drilled Holiday Epoxy-coated Mild Steel Reinforcement

The corrosion potential for the drilled holiday epoxy-coated reinforcement experienced spikes of 300 mV throughout days 105 to 217 (weeks 15 to 31) as the specimens began to corrode throughout the interval. The corrosion potential rose to a maximum of 430 mV at 224 days (32 weeks) and has continued indicating a severe corrosion risk through 280 days (40 weeks).

4.1.1.5 Chipped Holiday Epoxy-coated Mild Steel Reinforcement

The chipped holiday condition of the epoxy-coated reinforcement exhibited a corrosion potential of 100 mV through the first 217 days (31 weeks). At 217 days, a single specimen began corroding which caused the maximum of 316 mV. However, by 238 days (34 weeks), the corrosion potential value decreased and has remained a constant 200 mV.

The corrosion potential for the bottom layer of reinforcement (cathode) for all reinforcement types remained below 276 mV, indicating that none had undergone active corrosion. Additionally, no corrosion products were observed on the concrete surface for any reinforcement type.

4.1.2 Results for Transversely Cracked Specimens

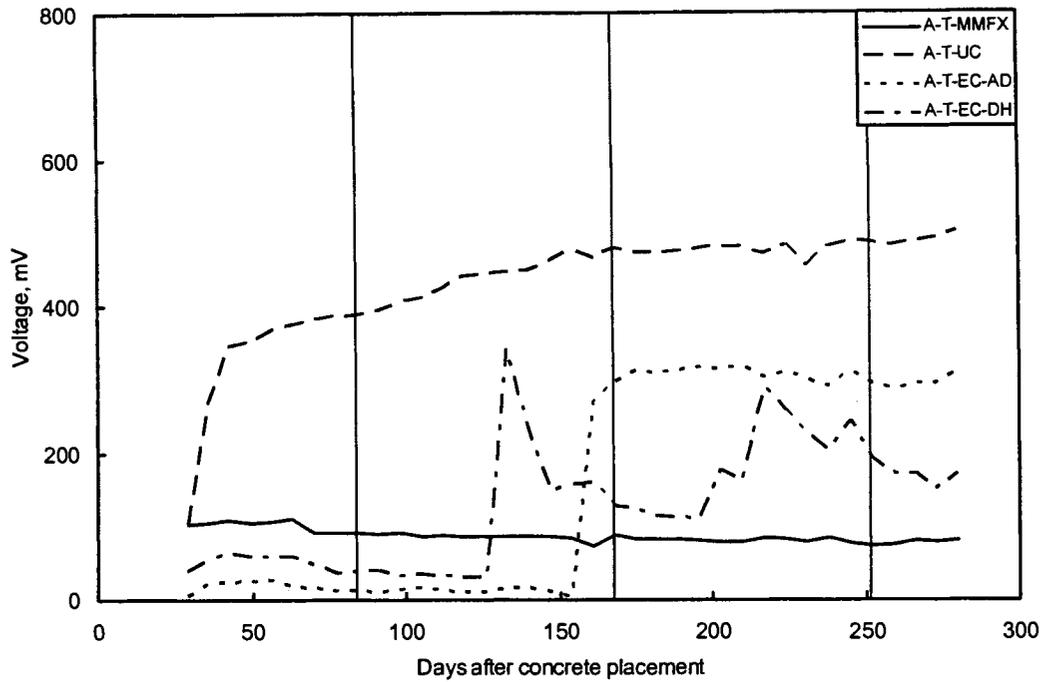
The 280-day (40-week) average anode (top reinforcement layer) and cathode (bottom reinforcement layer) corrosion potentials for specimens with transverse artificial cracks over the top layer of steel reinforcement are shown in Figure 4.2.

4.1.2.1 MMFX Microcomposite Steel Reinforcement

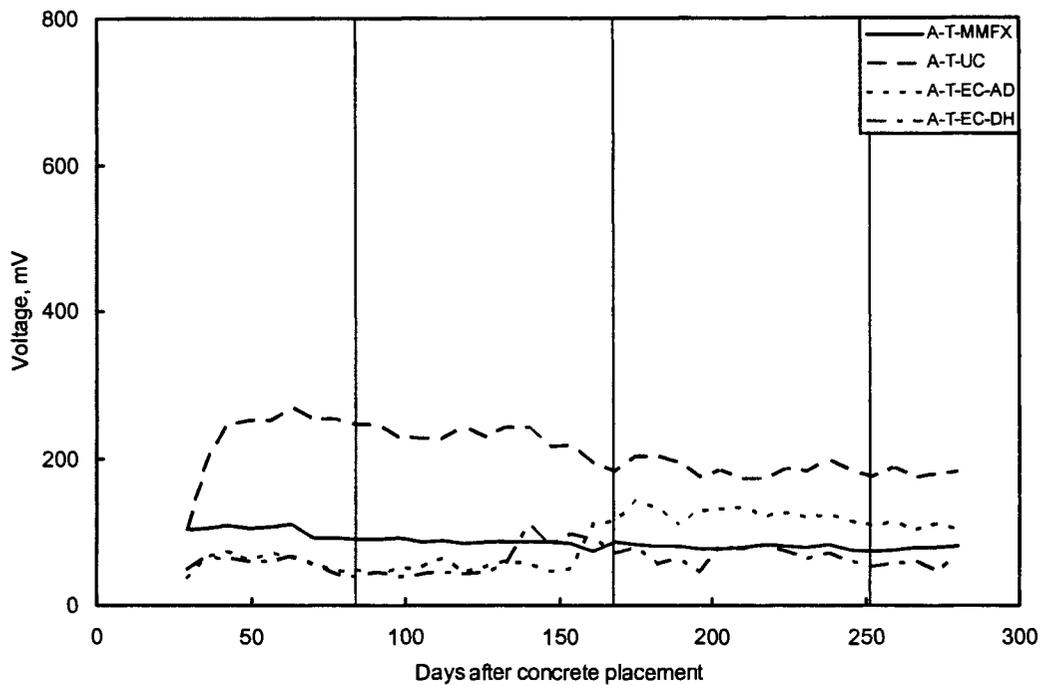
The corrosion potential for the top layer of reinforcement (anode) for the MMFX reinforcement with transversely cracked specimens remained a relatively constant value of 80 mV through 280 days (40 weeks), indicating a low risk for corrosion.

4.1.2.2 Uncoated Mild Steel Reinforcement

Similar to the longitudinally cracked specimens, the corrosion potentials for all the uncoated reinforcement specimens with transverse cracks increased beyond 276 mV by 35 days (5 weeks). By 98 days (14 weeks), the uncoated specimens experienced a corrosion potential value greater than 400 mV and continued to rise to 501 mV through 280 days (40 weeks), indicating a continued severe risk for corrosion.



(a) – Corrosion risk of the top layer of steel reinforcement



(b) – Corrosion risk of the bottom layer of steel reinforcement

Figure 4.2 – ASTM G 109 ACT subjected to 3 % NaCl solution through transverse crack.

4.1.2.3 As-delivered Epoxy-coated Mild Steel Reinforcement

Specimens with as-delivered epoxy-coated reinforcement exhibited relatively constant corrosion potential values of 20 mV through 161 days (23 weeks). At 161 days, a single specimen began corroding which caused rapid increase and the constant corrosion potential of 300 mV through 280 days (40 weeks).

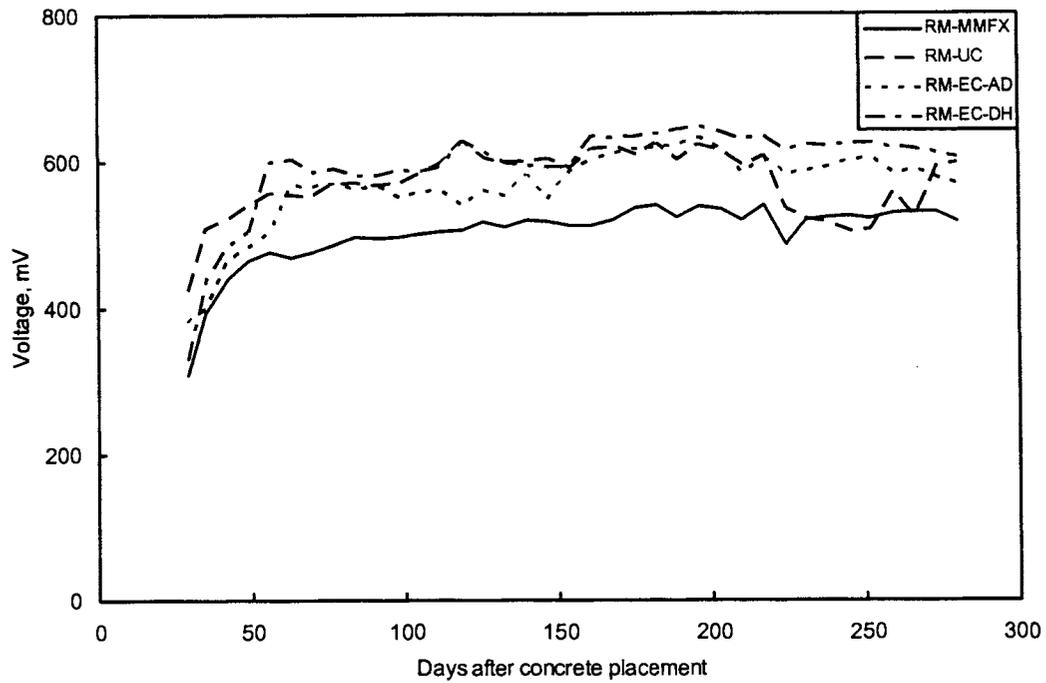
4.1.2.4 Drilled Holiday Epoxy-coated Mild Steel Reinforcement

Similar to the longitudinally cracked specimens, the drilled holiday epoxy-coated reinforcement with transverse cracks experienced spikes of 250 mV throughout days 105 to 217 (weeks 15 to 31) as the specimens began to corrode. However, due to continued decrease in corrosion potential the 280-day (40-week) value was 171 mV.

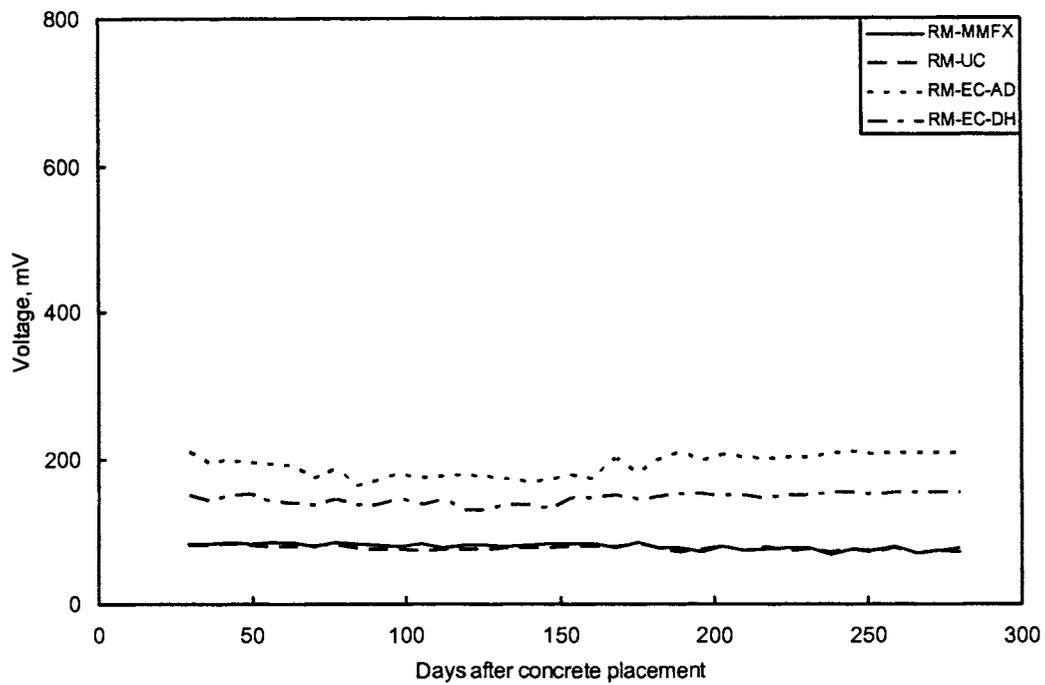
The corrosion potential for the bottom layer of reinforcement (cathode) for all reinforcement types remained below 276 mV, indicating none had undergone active corrosion. Additionally, no corrosion products were observed on the concrete surface for any reinforcement type.

4.2 Rapid Macrocell Accelerated Corrosion Test

The 280-day (40-week) average corrosion potentials for the anode (reinforcement in the container with sodium chloride solution) and cathode (reinforcement in the container with distilled water) steel reinforcement are shown in Figure 4.3.



(a) – Corrosion risk of the top layer of steel reinforcement



(b) – Corrosion risk of the bottom layer of steel reinforcement

Figure 4.3 – Rapid Macrocell ACT subjected to 3 % NaCl solution.

4.2.1 MMFX Microcomposite Steel Reinforcement

Within 35 days (5 weeks), all reinforcement types in the container with 3 percent sodium chloride solution (anode) were undergoing corrosion. Through 280 days (40 weeks), the MMFX reinforcement experienced a constant corrosion potential of 500 mV.

4.2.2 Uncoated Mild Steel Reinforcement

From 105 to 217 days (15 to 31 weeks), the uncoated reinforcement specimens exhibited a corrosion potential of 600 mV. At 217 days and continuing through 273 days (39 weeks), a single specimen ceased corroding which caused corrosion potential to decrease to 515 mV. After 273 days, the corrosion potential returned to 600 mV.

4.2.3 As-delivered Epoxy-coated Mild Steel Reinforcement

After 280 days (40 weeks), the as-delivered epoxy-coated specimens experienced a constant corrosion potential of 600 mV.

4.2.4 Drilled Holiday Epoxy-coated Mild Steel Reinforcement

Similar to the as-delivered epoxy-coated specimens, the drilled holiday epoxy-coated specimens experienced a constant corrosion potential of 600 mV.

The corrosion potential for all reinforcement types in the container with distilled water (cathode) remained below 276 mV, indicating none have undergone active corrosion. With the exception of the as-delivered and drilled holiday conditions of epoxy-coated reinforcement, corrosion products were visually observed on the mortar sheathing and within the solution of the anode.

4.3 Chloride-ion Concentration

To investigate the chloride-ion concentration in the ASTM ACT specimens, concrete powder samples were collected. Two unreinforced beams were cast at the same time as the other laboratory ACT specimens for background chloride analysis. Powder samples were collected as soon as corrosion initiated and were also collected on an interval basis.

4.3.1 Cement Mortar Powder Collection

As electrochemical investigations indicated corrosion initiation (i.e., 276 mV) for an individual ASTM ACT specimen, concrete powder samples were collected at the depth of the top reinforcement (anode) layer using a hammer drill with a stop gage.

Powder samples were obtained by use of a rotary impact drill as described by ASTM C 1152/C 1152 M and ASTM C 1218/C 1218 M (Standard Test Method for Acid-Soluble Chloride and Water-Soluble Chloride in Mortar and Concrete, respectively) (25, 26). The procedure was as follows:

1. Mark the location: The specimen was marked for two adjoining holes to be drilled to obtain a representative sample of at least 20 grams of concrete powder.
2. Drill powder from the marked location: A drill bit was selected to ensure the majority of the powder collected was cement mortar and not course aggregate (i.e., 5/8 in.; 1-1/2 times larger than the nominal course aggregate). Each of the adjoining holes was first drilled to a depth of 1/2 in. After drilling both initial holes, the powder was vacuumed from each hole and discarded and the top surface blown clean. The final 5/8 in. diameter holes were then drilled. No lubricants were used during drilling.

3. Transfer the drilled powder into a sealable plastic bag: The powder from the two adjoining holes was removed and combined into the first composite sample and the specimen ID is recorded on the bag.
4. Clean drill bit thoroughly with a brush to avoid contamination between powder samples: To prevent sample contamination, contact with sample material was also avoided with hands and other sources of perspiration. All other sampling tools were cleaned and dried prior to each sampling operation.
5. Repeat the procedure for each specimen: The process was repeated to obtain a second composite powder sample for a total of two composite samples for each ASTM ACT specimen.

From the two composite samples described above for a given ASTM ACT specimen an average specimen chloride-ion concentration was determined.

4.3.2 Chloride-ion Concentration Test

The collected powder samples were tested using the Phillips PW 2404 X-ray fluorescence (XRF) spectrometer at ISU Material Analysis and Research Laboratory. XRF spectroscopy provides a nondestructive means to identify and quantify the concentration of elements contained in a solid, powdered, and liquid sample.

4.3.3 Chloride-ion Concentration Results

The chloride-ion content data collected from the powder collected were used to determine a comparative chloride-ion concentration for each reinforcement type after the first high corrosion risk (i.e., 276 mV) was measured in the ASTM ACT specimens. The average results are shown in Table 4.1 for MMFX, epoxy-coated, and uncoated steel reinforcement, respectively. Additionally, chloride-ion concentrations of the concrete were also analyzed on

90-day intervals to determine if the rate of chloride ingress was similar among all the ASTM ACT specimens.

4.3.3.1 MMFX Microcomposite Steel Reinforcement

One specimen containing MMFX reinforcement has experienced high corrosion risk measurements (i.e., 276 mV). Subsequently, powder samples from that specimen were collected and chloride-ion concentration was measured as a weight concentration on a cubic yard basis. This MMFX reinforcement had a chloride-ion concentration of 2.73 lb/yd³. This value is lower than the chloride-ion concentrations of 3.32 and 3.60 lb/yd³ published by the University of Kansas Center for Research. At 90 days, values ranged from a low of 1.34 lb/yd³ to a high of 1.60 lb/yd³ with a two specimen average value of 1.47 lb/yd³. By 180 days, the values ranged from a low of 1.95 lb/yd³ to a high of 2.56 lb/yd³ with a two specimen average value of 2.25 lb/yd³. And at 270 days, values ranged from a low of 1.32 lb/yd³ to a high of 2.96 lb/yd³ with all five specimens averaging a value of 2.18 lb/yd³.

4.3.3.2 Uncoated Mild Steel Reinforcement

High corrosion risk has been measured for all five specimens containing uncoated reinforcement. The corresponding chloride-ion concentration values ranged from a low of 1.03 lb/yd³ to a high of 1.11 lb/yd³ with an average value of 1.06 lb/yd³. These values match those obtained in earlier studies. At 90 days, values ranged from a low of 1.34 lb/yd³ to a high of 1.35 lb/yd³ with a two specimen average value of 1.34 lb/yd³. By 180 days, the values ranged from a low of 1.74 lb/yd³ to a high of 2.10 lb/yd³ with a two specimen average value of 1.92 lb/yd³. And at 270 days, values ranged from a low of 1.85 lb/yd³ to a high of 3.34 lb/yd³ with all five specimens averaging a value of 2.37 lb/yd³.

Table 4.1 – Chloride-ion concentration at corrosion initiation and 90-day intervals.

Specimen Identification	Chloride-ion at Corrosion Initiation		90-day Chloride-ion Concentration	180-day Chloride-ion Concentration	270-day Chloride-ion Concentration
	Time, days	Concentration, pcy	Concentration, pcy	Concentration, pcy	Concentration, pcy
A-L-MMFX (1)	189	2.73	1.60	2.56	2.96
A-L-MMFX (2)			1.34	1.95	2.38
A-L-MMFX (3)					1.32
A-T-MMFX (1)					1.72
A-T-MMFX (2)					2.54
<i>MMFX Average</i>		2.73	1.47	2.25	2.18
A-L-UC (1)	7	1.05	1.34	1.74	1.85
A-L-UC (2)	7	1.11	1.35	2.10	3.34
A-L-UC (3)	7	1.03			2.00
A-T-UC (1)	7	1.07			2.14
A-T-UC (2)	14	1.03			2.52
<i>UC Average</i>		1.06	1.34	1.92	2.37
A-L-EC-AD (1)			1.35	1.83	1.93
A-L-EC-AD (2)			1.47	1.93	2.88
A-L-EC-AD (3)					2.23
A-T-EC-AD (1)	133	1.96			2.42
A-T-EC-AD (2)					2.99
<i>EC-AD Average</i>		1.96	1.41	1.88	2.49
A-L-EC-DH (1)	77	1.14	1.16	1.58	2.19
A-L-EC-DH (2)	77	1.20	1.20	1.74	2.69
A-L-EC-DH (3)	98	1.43			3.68
A-T-EC-DH (1)	189	2.10			2.61
A-T-EC-DH (2)	105	2.82			3.32
<i>EC-DH Average</i>		1.74	1.18	1.66	2.90
A-L-EC-CH (1)					2.00
A-L-EC-CH (2)	189	2.08			2.61
<i>EC-CH Average</i>		2.08			2.31

4.3.3.3 As-delivered Epoxy-coated Mild Steel Reinforcement

A single specimen containing epoxy-coated reinforcement in the as-delivered condition exhibited a high corrosion measurement. The chloride-ion concentration for this reinforcement was 1.96 lb/yd³. At 90 days, values ranged from a low of 1.35 lb/yd³ to a high of 1.47 lb/yd³ with a two specimen average value of 1.41 lb/yd³. By 180 days, the values ranged from a low of 1.83 lb/yd³ to a high of 1.93 lb/yd³ with a two specimen average value of 1.88 lb/yd³. And at 270 days, values ranged from a low of 1.93 lb/yd³ to a high of 2.99 lb/yd³ with all five specimens averaging a value of 2.49 lb/yd³.

The author believes the value of 1.96 lb/yd³ may not be representative of epoxy-coated reinforcement in a pure as-delivered condition. Since chloride-ion concentrations in as-delivered epoxy-coated specimens have been found higher than 1.96 lb/yd³ without indication of corrosion (i.e., 276 mV), the 270-day concentration was time required for chloride-ion to reach the average 270-day concentration was found. The author believes the 270-day concentration to be a lower bound, with the understanding an even higher chloride-ion concentration is expected before the remaining four specimens undergo corrosion. Strict observation of the aforementioned bar should determine whether corrosion occurred at site where a holiday was present.

4.3.3.4 Drilled Holiday Epoxy-coated Mild Steel Reinforcement

All five specimens containing the drilled holiday condition of epoxy-coated reinforcement have experienced high corrosion risk measurements. Values for the drilled holiday condition of epoxy-coated reinforcement ranged from a low of 1.14 lb/yd³ to a high of 2.82 lb/yd³ with an average value of 1.74 lb/yd³. At 90 days, values ranged from a low of 1.16 lb/yd³ to a high of 1.20 lb/yd³ with a two specimen average value of 1.18 lb/yd³. By 180 days, the values ranged from a low of 1.58 lb/yd³ to a high of 1.74 lb/yd³ with a two specimen average value of 1.66 lb/yd³. And at 270 days, values ranged from a low of 2.19 lb/yd³ to a high of 3.68 lb/yd³ with all five specimens averaging a value of 2.90 lb/yd³.

4.3.3.5 Chipped Holiday Epoxy-coated Mild Steel Reinforcement

High corrosion risk has been measured for a single specimen containing the chipped holiday condition of epoxy-coated reinforcement. The corresponding chloride-ion concentration value was 2.08 lb/yd³. At 270 days, values ranged from a low of 2.00 lb/yd³ to a high of 2.61 lb/yd³ with both specimens averaging a value of 2.31 lb/yd³.

Chloride sampling will continue throughout the duration of the ASTM accelerated corrosion tests to finalize the chloride-ion content for MMFX and as-delivered epoxy-coated reinforcement.

4.4 Discussion of Laboratory Test Results

The results from the mechanical properties testing demonstrates that MMFX Microcomposite steel reinforcement has considerably higher yield and tensile strengths, but lower elongations than mild steel reinforcement. The lower elongations obtained with MMFX steel are expected for high-strength steels. The tensile and yield strengths of MMFX steel are closer to those specified for high-strength steel reinforcement for prestressing concrete (ASTM A 722) than they are to mild steel reinforcement (ASTM A 615). For the No. 16 reinforcement, however, while the tensile strengths exceed the 150 ksi minimum required for A722 reinforcement, the yield strengths based on 0.2 percent offset do not meet the requirements for either Type I or Type II, which are 85 percent (127.5 ksi) and 80 percent (120 ksi), respectively, of the minimum tensile strength.

The following discussion for the results from the accelerated corrosion tests and chloride-ion concentration analyses should be interpreted only as preliminary findings. While these results are utilized to make preliminary comparisons of corrosion performance for MMFX, epoxy-coated, and uncoated reinforcement, the reader should be aware that a degree of uncertainty exists. Especially in the cases MMFX and as-delivered epoxy-coated

reinforcement, where corrosion has initiated for only a single ASTM ACT specimen of each reinforcement type.

At a given time, the corrosion potential for specimens containing the same reinforcement type has shown significant variation between the specimens. This variation may be caused by dissimilarities in anode and cathode locations, epoxy coating performance, and reinforcement material. The rates of consumption and renewal of the fundamental factors (i.e., chloride ions, oxygen, and water) to sustain active corrosion may also cause specimens reinforced with the same steel type to behave differently (27). However, a reasonable correlation does exist when the average of the corrosion potentials for each reinforcement type is compared under the same test conditions.

Through 280 days (40 weeks), the ASTM ACT generally showed evidence of low to intermediate corrosion risk potentials for the MMFX reinforcement, with the exception of a single longitudinally cracked specimen. This specimen began corroding at 217 days (31 weeks). The corrosion potential increased rapidly for the uncoated reinforcement, whereby after 35 days (5 weeks), all the specimens indicated corrosion had initiated. Longitudinal and transverse cracked specimens with the as-delivered epoxy-coated reinforcement exhibited the lowest corrosion potential, although a single transversely cracked specimen began corroding at 161 days (23 weeks). The corrosion potential, for epoxy-coated reinforcement with induced holidays, indicated corrosion initiated in the specimens between 105 to 217 days (15 to 31 weeks). Through 280 days (40 weeks) none of the ASTM ACT specimens showed any visual indication (i.e., deposition of corrosion products or concrete discoloration) of the state of the top reinforcement (anode).

Within the first week, the Rapid Macrocell ACT produced severe corrosion risk potentials for all the reinforcement types. The specimens with MMFX reinforcement had the least severe corrosion risk potential, while the uncoated, as-delivered condition, and drilled holiday condition of the epoxy-coated had the most severe corrosion risk potential. Since the Rapid Macrocell ACT specimens are an alteration from the ASTM ACT beam specimens, the

almost immediate severe corrosion risk potentials measured for all the reinforcement types was unexpected. To justify the different responses between the ASTM and Rapid Macrocell corrosion potentials, the author attributes the difference to the continuous renewal of oxygen to the Rapid Macrocell ACT. By continuously replenishing oxygen, the Rapid Macrocell ACT creates an environment more conducive to initiating and sustaining corrosion than the ASTM ACT, which replenishes oxygen through the previously described ponding and drying regime. Additionally, the Rapid Macrocell ACT was carried out with a plastic sheet placed over the entire test system. This maintained a high humidity environment over the portion of the cylindrical test specimen not submerged in the solution.

Since only severe corrosion risk potentials were observed, more significance was placed on the measurements obtained from the ASTM ACT specimens. However, through 280 days (40 weeks), the concrete surrounding the MMFX and uncoated reinforcement Rapid Macrocell ACT specimens had discolored due to deposition of corrosion products. And following the conclusion of the test period, visual inspection of the Rapid Macrocell reinforcement should provide reasonable comparisons based on the extent of corrosion products and damage.

For the study presented herein, a corrosion potential greater than 276 mV was employed as the indication of corrosion initiation. At the time of the first measurement greater than 276 mV, concrete powder specimens were collected at the top reinforcement depth. The chloride-ion concentration for the single specimen containing MMFX reinforcement was 2.73 lb/yd³. This value is lower than the chloride-ion concentrations of 3.32 and 3.60 lb/yd³ published by the University of Kansas Center for Research. For uncoated mild reinforcement, the chloride-ion concentration of 1.06 lb/yd³ was obtained. This value matches the 1.00 to 1.40 lb/yd³ commonly believed to be the chloride threshold of uncoated mild steel. For the single specimen containing as-delivered epoxy-coated reinforcement the chloride-ion concentration was 1.96 lb/yd³, while the chloride-ion concentration for the epoxy-coated reinforcement with induced holidays was 1.74 lb/yd³.

As previously discussed, the author believes the value of 1.96 lb/yd³ may not necessarily be representative of epoxy-coated reinforcement in a pure as-delivered condition, and strict observation of the bar should determine whether a holiday in the coating existed. Chloride-ion concentrations will continue to be monitored for the remaining MMFX and as-delivered epoxy-coated reinforcement through the duration of the ASTM accelerated corrosion test.

CHAPTER 5 LIFE EXPECTANCY AND LIFECYCLE COST

The author acknowledges that sufficient data has not yet been obtained to decisively predict the life expectancies and lifecycle costs for MMFX Microcomposite and epoxy-coated mild steel reinforcement. However, as the monitoring of corrosion for MMFX and epoxy-coated reinforcement continues, a greater degree of certainty of the measured data will be established, and a more justified conclusion can be drawn per life expectancy and lifecycle cost. While the limited results from 40 weeks of laboratory testing do not constitute a prediction of life expectancy and lifecycle cost, a procedure is presented to determine the life expectancy and lifecycle cost once definitive evidence is attained.

5.1 Life Expectancy

The life expectancy of bridge decks constructed with different steel reinforcing systems is estimated by the two-stage, diffusion-spalling model (i.e., the time required for corrosion initiation and the subsequent time period required to cause spalling due to corrosion). In the study presented herein, a procedure that can be used to estimate the life expectancy of a bridge deck is defined.

5.1.1 Time to Corrosion Initiation

The time to corrosion initiation is estimated using Fick's Second Law of Diffusion and chloride-ion concentrations at corrosion initiation, as measured in the current study.

Fick's Second Law of Diffusion is a common diffusion model used to determine the length of the initiation stage (i.e., the time required for the chloride-ion to migrate through a bridge deck to the top steel reinforcement layer and accumulate to the chloride threshold value). The model assumes that the chloride-ion diffuses through an isotropic medium (28). The fundamental second order differential equation of Fick's Second Law of Diffusion is as follows:

$$\frac{\partial C}{\partial t} = D_c \frac{\partial^2 C}{\partial x^2}, \quad (5.1)$$

where:

C = chloride concentration with depth, in.

t = time, years.

x = depth, in.

D_c = diffusion constant, in²/yr.

A closed form solution of the above differential equation for a semi-infinite bridge deck (i.e., small ratio of depth to length or width of a bridge deck) can be express as follows (29):

$$C_{(x,t)} = C_0 \left\{ 1 - \operatorname{erf} \left[\frac{x}{2\sqrt{(D_c t)}} \right] \right\}, \quad (5.2)$$

where:

$C_{(x,t)}$ = measured chloride concentration at desired depth, lb/yd³.

C_0 = constant surface concentration measured at 1/2 in. below the bridge deck surface, lb/yd³.

t = time, years.

x = depth measure from the bridge deck surface, in.

$$\operatorname{erf}(y) = \frac{2}{\sqrt{\pi}} \int_0^y e^{-(s)^2} ds, \quad (5.3)$$

The error function, $\operatorname{erf}(y)$, is the integral of the Gaussian distribution function from 0 to y .

5.1.1.1 Surface Chloride-ion Constant

The application of Fick's Second Law of Diffusion to assess the time to corrosion initiation requires the determination of the surface chloride content, C_0 , and the diffusion constant, D_c .

Investigations of the chloride-ion concentration in bridge decks concluded that the concentrations measured 1/2 in. from the bridge deck surface reached a stable condition after four to six years of service (28). For this reason, the surface chloride constant, C_s , in equation 5.2 was recommended to equal measured chloride-ion concentration at 1/2 in. from the bridge deck surface.

Corrosion of the steel reinforcement will not initiate when the chloride-ion ingress reaches the reinforcement depth, as the previously described threshold of chloride-ion concentration must be present to break down the passive layer formed by the concrete alkalinity.

5.1.1.2 Chloride Diffusion Constant

The transport of chloride-ion in bridge decks is assumed to be a one-dimensional diffusion process. However, the ingress of chloride-ion in concrete is impacted by concrete capillaries and cracking. The quality of concrete affects the phenomenon of the diffusion process in terms of time needed for chloride content to reach a certain depth and concentration. A strong correlation between the diffusion constant and the water-cement ratio has been observed in studies of controlled experimental specimens (30). Earlier research has also found temperature to have a significant impact on the diffusion process of chloride in hardened cement paste (29). The omnipresent cracking that increases the rate of chloride diffusion is affected by many factors, such as water-cement ratio, temperature fluctuation, traffic volume, and the curing and construction process. Therefore, the chloride diffusion constant, D_c , in equation 5.2 is commonly characterized by the construction practices dictated from state to state.

5.1.1.3 Reinforcement Cover Depth

Cover depth is defined as the clear distance from the surface of the bridge deck to the top of the first layer of reinforcement. A sufficient cover depth can effectively provide corrosion protection for the reinforcement. As steel reinforcement cover depth increases, the corrosion

protection increases and the time to initiation increases. Studies have shown that the chloride concentration decreases significantly along with increasing depth from the bridge deck surface (31).

Since rehabilitation will only take place after spalling or other deterioration has occurred, to calculate a realistic time for chloride-ion to reach the reinforcement depth, the full functional service life is utilized. By this reasoning, researcher recommends not using the mean values of the cover depth, but instead a statistical value for cover depth that accounts for the possibility that some reinforcement could be located at a depth less than the mean value (14). This can be calculated as:

$$x = \bar{x} + \alpha\sigma , \quad (5.4)$$

where:

\bar{x} = mean steel reinforcement cover depth, in.

α = values corresponding to a given cumulative percentage.

σ = standard deviation of the cover depth.

The alpha-value can be selected as the percent damage of the worst traffic lane. Statistical analysis of the measured cover depth taken from several bridge decks followed a normal distribution. Therefore, a standard normal cumulative probability table can be utilized to establish the alpha-value.

Previous research was conducted to analyze of the diffusion constant and the surface chloride-ion constant throughout the state of Iowa (5). This database consists of concrete power samples collected from 81 bridge decks reinforced with epoxy-coated steel. An average concrete cover depth $\bar{x} = 2.74$ in., associated with standard deviation $\sigma = 0.444$ in. was reported for bridge decks in the state of Iowa.

Research recommends using the alpha-value corresponding to 11.5 percent visual damage (corrosion or delamination) to the reinforcement of the worst traffic lane of a bridge deck, as an indication of the end of the functional service life for a bridge deck (14). Based on this assumption that 11.5 percent of the reinforcement is contaminated by chloride ions, then from a standard normal cumulative probability table the alpha-value for calculating the top reinforcement layer depth is $\alpha = -1.2$. Subsequently, the cover depth calculated from equation (5.4) is:

$$x = 2.74 + (-1.2)(0.444) = 2.21 \text{ in.}, \quad (5.5)$$

Additionally reported, bridge decks in the state of Iowa were found to have a diffusion constant $D_c = 0.05 \text{ in}^2/\text{yr}$ and a mean surface chloride constant $C_0 = 14.0 \text{ lb/yd}^3$ (5). Substituting the cover depth $x = 2.21 \text{ in.}$, the diffusion constant $D_c = 0.05 \text{ in}^2/\text{yr}$, and the mean surface chloride constant $C_0 = 14.0 \text{ lb/yd}^3$, equation (5.2) can be expressed as:

$$C_{(x,t)} = 14.0 \left\{ 1 - \operatorname{erf} \left[\frac{2.21}{2\sqrt{(0.05t)}} \right] \right\} \quad (5.6)$$

where:

$C_{(x,t)}$ = measured chloride-ion concentration for the initiation of corrosion at depth of the top layer of reinforcement, lb/yd^3 .

t = time to be calculated for the initiation of corrosion, years.

Equation 5.6 relates the time required for chloride-ion to migrate to a depth of 2.21 in. and accumulate the required concentration to initiated corrosion for a respective reinforcement type. Substituting the chloride-ion concentration at initiation in equation (5.6), the time t can be calculated. This is the time to corrosion initiation for the top layer of steel reinforcement of a bridge deck.

5.1.2 Time between Corrosion Initiation and Spalling

Published literature indicated that estimating the length of time between corrosion initiation and spalling is a difficult task. However, a research study suggested using the rate at which a given reinforcement type corrodes as a means to estimate the length of this period (32).

Equation (5.7) shows this linear relationship.

$$e = r \cdot t \quad (5.7)$$

where:

e = loss in reinforcing bar diameter, μm .

r = corrosion rate, μm loss per unit time.

t = time to be calculated between corrosion initiation and spalling, years.

The research presenting this technique also published that a critical value for the loss in reinforcing bar diameter of 0.025 mm (0.00098 in.) would result in a volume of corrosion products sufficient to crack concrete (32). Utilizing this assumption, equation (5.7) can be expressed as:

$$25 \mu\text{m} = r \cdot t \quad (5.8)$$

To determine the rate of corrosion for use in equation (5.8), the corrosion current can be determined by measuring the voltage drop across the resistor (i.e., macrocell corrosion measurement). The corrosion current is calculated from Ohm's Law, equation (5.9), which is equal to the voltage divided by the resistance.

$$I = \frac{V}{R}, \quad (5.9)$$

where:

I = corrosion current, amperes.

V = macrocell voltage, volts.

R = resistance, ohms.

The actual resistance of each 10-ohm resistor is measure separately. Once the current is calculated, the corrosion rate, in terms of metal loss, is calculated using Faraday's Law (4).

$$r = \frac{ia}{nFD}, \quad (5.10)$$

where:

r = corrosion rate, thickness loss per unit time.

i = current density, amperes per cm^2 .

a = atomic weight, 55.84 g for mole iron.

n = number of equivalents exchanged, 2 electrons transferred for Fe^{2+} .

F = Faraday's constant, 96,485 coulombs per mole.

D = density of metal, 7.87 g per cm^3 for steel.

In terms of current density, i , in μA per cm^2 , corrosion rate, r , in μm per year is:

$$r = 11.59i, \quad (5.11)$$

The calculated average 280-day (40-week) corrosion rates are shown in Table 5.1 for MMFX, epoxy-coated, and uncoated steel reinforcement, respectively.

Table 5.1 – Average corrosion rate from corrosion initiation.

Specimen Identification	Corrosion Rate, $\mu\text{m/yr}$
A-L-MMFX (1)	3.19
A-L-MMFX (2)	
A-L-MMFX (3)	
A-T-MMFX (1)	
A-T-MMFX (2)	
<i>MMFX Average</i>	<i>3.19</i>
A-L-UC (1)	17.18
A-L-UC (2)	12.92
A-L-UC (3)	8.12
A-T-UC (1)	9.04
A-T-UC (2)	13.13
<i>UC Average</i>	<i>12.08</i>
A-L-EC-AD (1)	
A-L-EC-AD (2)	
A-L-EC-AD (3)	
A-T-EC-AD (1)	0.00
A-T-EC-AD (2)	
<i>EC-AD Average</i>	<i>0.00</i>
A-L-EC-DH (1)	0.00
A-L-EC-DH (2)	0.00
A-L-EC-DH (3)	0.00
A-T-EC-DH (1)	0.00
A-T-EC-DH (2)	0.00
<i>EC-DH Average</i>	<i>0.00</i>
A-L-EC-CH (1)	0.00
A-L-EC-CH (2)	
<i>A-EC-CH Average</i>	<i>0.00</i>

From the macrocell corrosion measurement, an average corrosion rate can be determined by using equation (5.11). Substituting this corrosion rate r in equation (5.8), the period of time t between corrosion initiation and spalling can be calculated. This is the required time for corrosion products on the reinforcement to accumulate to a volume sufficient to crack the concrete in a bridge deck. Through the combination of the time to corrosion initiation from equation (5.6) and time from initiation to spalling from equation (5.8), the time to the first repair for the respective reinforcement type is calculated.

5.2 Illustrative Example to Calculate the Life Expectancy of a Bridge Deck with Uncoated Mild Steel Reinforcement

The following example uses the two-stage, diffusion-spalling model to illustrate how the above procedure can be used to estimate the life expectancy of a bridge deck reinforced with uncoated steel in the state of Iowa. The reader should recall that the uncoated reinforcement values for chloride-ion concentration at initiation (1.06 lb/yd³) and corrosion rate (12.8 μm/yr) listed in Tables 4.1 and 5.1, respectively.

5.2.1 Time to Corrosion Initiation

An average chloride-ion concentration of 1.06 lb/yd³ was obtained at the initiation of corrosion for all five specimens containing uncoated reinforcement. By substituting the chloride-ion concentration at corrosion initiation (1.06 lb/yd³) for $C_{(x,t)}$, equation (5.6) can be expressed as:

$$1.06 = 14.0 \left\{ 1 - \operatorname{erf} \left[\frac{2.21}{2\sqrt{(0.05t)}} \right] \right\}, \quad (5.12)$$

Solving for t yields a time of 15 years for the chloride-ion to migrate to the top reinforcement depth and accumulate a concentration sufficient to initiate corrosion for uncoated reinforcement.

5.2.2 Time between Corrosion Initiation and Spalling

From the macrocell corrosion measurement, an average corrosion rate of 12.08 $\mu\text{m}/\text{yr}$ over 252 days (36 weeks) was calculated for all five specimens containing uncoated reinforcement. By substituting the corrosion rate (12.8 $\mu\text{m}/\text{yr}$) for r , equation (5.8) can be expressed as:

$$25\mu\text{m} = 12.08 \frac{\mu\text{m}}{\text{yr}} \cdot t, \quad (5.13)$$

Solving for t yields a time of 2 years for corrosion products on the reinforcement to accumulate a volume sufficient to crack concrete containing uncoated reinforcement. This value matches those obtained in earlier studies (5). The combination of the calculated time to corrosion initiation and time from initiation to spalling results in 17 years to the first repair for uncoated reinforcement. This value matches those obtained in earlier studies (4, 5).

The preceding approach can also be utilized to estimate the life expectancy of bridge decks reinforced with MMFX or epoxy-coated steel. However, this is not illustrated due to the limited results obtained for these two types of steel reinforcement.

5.3 Lifecycle Cost

The in-place costs for the two twin, side-by-side “field” bridge decks (MMFX Bridge and Epoxy-coated Bridge) in conjunction with the calculated life expectancy (i.e., time from first constructed to spalling) can be used to compare the lifecycle costs associated with MMFX and epoxy-coated reinforcement in bridge decks constructed in the state of Iowa. The in-place costs for each bridge deck were obtained through personal communication with the Bridge Design Office of the Iowa DOT (33). As released by the Iowa DOT, the in-place costs are listed in Table 5.2.

Table 5.2 – MMFX Bridge and Epoxy-coated Bridge in-place costs.

Bridge Identification	In-place Bridge Deck Cost
MMFX Bridge	\$53,552.40
Epoxy-coated Bridge	\$41,247.80

Utilizing the listed in-place costs, the lifecycle cost for a bridge deck can be calculated by the following capital recovery expression:

$$A = P \left[\frac{i(1+i)^n}{(1+i)^n - 1} \right] \quad (5.14)$$

where:

A = uniform value over the interest period.

P = present value.

i = interest rate.

n = expected life of an asset.

The present value P is the listed in-place bridge deck cost and the expected life n can be calculated as defined in section 5.1. The reader is recommended to approach the life expectancy calculation with care. Until enough data becomes available, no direct comparison for MMFX and epoxy-coated can be obtained. Therefore, further investigation of MMFX and epoxy-coated reinforcement is required to determine the chloride threshold values and corrosion rates required to establish a corresponding lifecycle costs.

CHAPTER 6 SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

6.1 Summary

The corrosion of steel reinforcement in an aging highway infrastructure is a major problem currently facing the transportation engineering community, in particular bridge engineers. In the United States, maintenance and replacement costs of deficient bridges are measured in billions of dollars. Further, the use of deicing salts has resulted in steady deterioration of bridge decks due to corrosion.

These concerns have initiated continual development of protective measures. Application of corrosion-resistant steel reinforcement as an alternate reinforcement for existing mild steel reinforced concrete bridge decks has great potential due to the inherent corrosion resistant properties associated with the material.

To investigate corrosion prevention through the use of corrosion-resistant alloys, MMFX Microcomposite steel reinforcement, a high-strength, high chromium steel reinforcement, is evaluated for corrosion resistance performance. The steel is compared to epoxy-coated and uncoated mild steel reinforcement is investigated through separate field and laboratory evaluations. However, as definitive field evidence of the corrosion resistance of MMFX reinforcement may require several years of monitoring, attention transferred to investigate the reinforcement under accelerated conditions in a laboratory. In the laboratory investigation, principal emphasis is placed on corrosion performance of the MMFX, epoxy-coated, and uncoated reinforcement. The evaluation process was based on using ASTM and Rapid Macrocell accelerated corrosion tests.

While the limited results from 40 weeks of laboratory testing do not constitute a prediction of life expectancy and lifecycle cost, a procedure is presented to determine the life expectancy and lifecycle cost once definitive evidence is attained. The life expectancy of bridge decks constructed with different steel reinforcing systems is estimated by the two-stage, diffusion-

spalling model (i.e., the time required for corrosion initiation and the subsequent time period required to cause spalling due to corrosion). Fick's Second Law of Diffusion was applied in this study to estimate the time required to corrosion initiation, while calculated corrosion rates were utilized to determine the time between initiation and spalling of the concrete. The combination of time to initiation and time between initiation and spalling results in the time to the first repair (i.e., life expectancy).

The in-place costs for the two twin, side-by-side "field" bridge decks (MMFX Bridge and Epoxy-coated Bridge) in conjunction with the calculated life expectancy (i.e., time from first constructed to spalling) can be used to compare the lifecycle costs associated with MMFX and epoxy-coated reinforcement in bridge decks constructed in the state of Iowa. However, until enough data becomes available, no direct comparison for MMFX and epoxy-coated can be obtained. Therefore, further investigation is required to determine more representative chloride threshold values and corrosion rates for the MMFX and epoxy-coated reinforcement.

6.2 Conclusions

The MMFX Microcomposite reinforcement exhibit yield strengths equal to approximately twice that required for Grade 60 mild reinforcement. The results from the mechanical properties testing demonstrates that MMFX Microcomposite steel reinforcement has considerably higher yield and tensile strengths, but lower elongations than mild steel reinforcement. The lower elongations obtained with MMFX steel are expected for high-strength steels. The tensile and yield strengths of MMFX steel are closer to those specified for high-strength steel reinforcement for prestressing concrete (ASTM A 722) than they are to mild steel reinforcement (ASTM A 615). For the No. 16 reinforcement, however, while the tensile strengths exceed the 150 ksi minimum required for A722 reinforcement, the yield strengths based on 0.2 percent offset do not meet the requirements for either Type I or Type II, which are 85 percent (127.5 ksi) and 80 percent (120 ksi), respectively, of the minimum tensile strength.

The test results for the study presented herein demonstrate that MMFX Microcomposite steel reinforcement is more corrosion resistant than uncoated mild steel reinforcement, and exhibits similar corrosion resistance to epoxy-coated reinforcement meeting the requirements of ASTM A 775. Compared to uncoated reinforcement, MMFX reinforcement requires a higher chloride-ion concentration for corrosion initiation. Overall, the measurements indicate that measurements are lower for the MMFX Bridge than for the Epoxy Bridge. However, no significant corrosion has been observed in either bridge deck. Although not completed, through continued monitoring and evaluation, the on-going field monitoring system is expected to provide evidence of the corrosion resistance of the reinforcement monitored. So as definitive evidence of the corrosion resistance of MMFX reinforcement could not be surmised, attention transferred to investigate the reinforcement under accelerated conditions in a laboratory.

The test results from the accelerated corrosion tests justify the following preliminary conclusions for MMFX Microcomposite, epoxy-coated mild, and uncoated mild steel reinforcement. After 40 weeks of testing, the associated ASTM ACT corrosion potentials indicate corrosion has not initiated for either MMFX or as-delivered epoxy-coated reinforcement. However, the uncoated mild steel underwent corrosion within the fifth week, while epoxy-coated reinforcement with holidays underwent corrosion between 15 and 30 weeks. Within the fifth week of testing, the Rapid Macrocell ACT produced corrosion risk potentials indicative of active corrosion for all the reinforcement types tested. All ASTM ACT specimens had essentially identical surface appearances at 12 weeks. However, the Rapid Macrocell ACT concrete surrounding MMFX and uncoated reinforcement discolored due to deposition of corrosion products.

For the study presented herein, concrete powder specimens were collected at the top reinforcement depth at the first indication of corrosion initiation. For uncoated mild reinforcement, the chloride-ion concentration of 1.06 lb/yd^3 was obtained. This value matches the 1.00 to 1.40 lb/yd^3 commonly believed to be the chloride threshold of uncoated mild steel. For the epoxy-coated reinforcement with induced holidays the chloride-ion

concentration was 1.74 lb/yd³. Chloride-ion concentrations will continue to be monitored for the remaining MMFX and as-delivered epoxy-coated reinforcement through the duration of the ASTM accelerated corrosion test.

6.3 Recommendations

The following are recommended for future research:

- For the ASTM ACT, only a single specimen reinforced with MMFX Microcomposite steel underwent corrosion through 40 weeks of testing. Similarly, only one as-delivered epoxy-coated specimen underwent corrosion. Monitoring of MMFX and as-delivered epoxy-coated reinforcement will be continued to adequately assess the chloride-ion concentration at corrosion initiation and the corrosion rate.
- To better assess the initiation of corrosion, it would be beneficial to employ scanning electron microscopy to analyze ACT specimens immediately following the first half-cell potential measurement of 276 mV.
- Some researchers believe the two-stage process of corrosion initiation and active corrosion periods does not adequately represent epoxy-coated steel. Instead, prior to corrosion initiation, a period of epoxy-coating delamination occurs. It would be beneficial to incorporate this period where the coating delaminates from the reinforcement in the ACT specimens presented herein.
- As would be expected after only a year and a half of field monitoring, no measurements indicate corrosion initiation on either field bridge deck being monitored. Continued monitoring the field bridges will provide more accurate results, reflecting real behavior in the environmental conditions of a bridge constructed in that state Iowa.

- Through further utilization of the “field” bridge deck data, further calibration of the life expectancy procedure can be made to more accurately predict the time to corrosion initiation and the time between initiation and spalling.

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