ASSESSING THE SERVICE LIFE OF CORRSION-DETERIORATED REINFORCED CONCRETE MEMBER HIGHWAY BRIDGES IN WEST VIRGINIA

Prepared for the WVDOT / DOH RP# 234

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September 2014

1. Report No.	**2. Government Accession No.**	**3. Recipient's Catalog No.**

4. Title and Subtitle

ASSESSING THE SERVICE LIFE OF CORRSION-DETERIORATED REINFORCED CONCRETE MEMBER HIGHWAY BRIDGES IN WEST VIRGINIA

5. Report Date
September 2014

6. Performing Organization Code

7. Author**(s)**
Dr. Wael Zatar

8. Performing Organization Report No.

9. Performing Organization Name and Address

10. Work Unit No. (TRAIS)

No.

12. Sponsoring Agency Name and Address
West Virginia Department of Transportation, Division of Highways
1900 Kanawha Blvd. East, Charleston, WV 25305-0430

13. Type of Report and Period Covered

14. Sponsoring Agency Code

15. Supplementary Notes

16. Abstract

Corrosion of steel-reinforced concrete bridges is a serious problem facing the WVDOT. This paper provides an overview of techniques for evaluating the condition of reinforced concrete bridge elements; methods for modeling the remaining service life and the service life extension provided by various repair and protection methods; and a review of the various repair and protection methods. Fick’s 2nd Law of Diffusion, combined with Life-365 software, was determined the most reasonable method for service life modeling. Bridge element condition evaluation should be performed at routine intervals, depending on the condition and age of the structure being evaluated. Evaluations should use a variety of technologies and techniques, including half-cell potential measurements. This study concludes that the various types of repair, evaluation, and modeling techniques can be incorporated into any existing corrosion management through the development of a manual which outlines reliable, research-based procedures for assessing the condition of corrosion-deteriorated in-service highway bridges.

17. Key Words
Steel-reinforcement corrosion, concrete protection, service life estimation

18. Distribution Statement

19. Security Classif. (of this report) Unclassified	**20. Security Classif. (of this page)** Unclassified	**21. No. of Pages** **80**	**22. Price**

EXECUTIVE SUMMARY

Corrosion of steel reinforcement in concrete bridges is a costly and significant problem facing transportation agencies, especially those in states such as West Virginia that experience both high humidity and snowfall (the management of which involves the use of deicing salts). However, a significant body of research exists on the corrosion process, corrosion evaluation techniques, and corrosion prevention and repair. Taken together, this research can inform rational corrosion management decisions.

Chloride ingress plays a significant role in the corrosion process and should be modeled using Fick's 2nd Law of Diffusion. Though other methods exist, some require significantly more complex calculations, and at present there is no evidence that they yield more accurate results.

Life-365 service life modeling software should be used to estimate the remaining service life of reinforced concrete structures. Though other service life modeling software exists, Life-365 is both robust and user-friendly. Cracking initiation is estimated using Fick's 2nd Law; calculations can be modified based on local conditions and field observations; and it is possible to analyze life cycle costs and service life extensions among several alternatives.

Bridges should be evaluated at regular intervals. During these evaluations, bridge components should be inspected individually and systematically. The results of such an evaluation can be used to develop an overall bridge condition report. An initial corrosion evaluation should take place about four years after a new bridge is open (immediately for a bridge that has been in service longer than four years). Routine corrosion evaluations should occur during each Period Inspection, and an In-Depth Corrosion Evaluation should be performed during each In-Depth Inspection. Future evaluations should be based on the findings reported during the previous evaluation. A manual should be developed to assist WVDOH personnel in identifying reliable, research-based procedures for assessing the condition of corrosion-deteriorated bridges in West Virginia. Finally, the overall condition of and issues unique to each bridge should be used to determine which technique(s) should be deployed during bridge condition evaluation.

Several evaluation techniques are presented in Chapter 3 for consideration. Though there is no one-size-fits-all technique for field evaluations, the half-cell potential measurement technique warrants special attention. Half-cell potential measurements can both indicate the existence of corrosion and enable evaluators to determine areas with high probabilities of corrosion. To compensate for the known limitations of the half-cell potential measurement technique, a large number of closely spaced half-cell potentials should be taken.

With the use of modeling software and bridge evaluation reports, it is possible to select from the best of several alternative protection, repair, and rehabilitation techniques. Cathodic protection prevents corrosion of steel reinforcement by either deploying sacrificial corrosion nodes or sending a current through steel reinforcement. Sealers, membranes, organic inhibitors, and admixture form either an internal or external barrier that prevents chlorides from coming into contact with steel reinforcement. Patching and overlay methods can be used to repair damage and increase cover depth, thereby reducing the time to corrosion initiation. Finally, chloride extraction is an electrochemical process that can provide short-term protection against corrosion.

By using research-proven steel reinforcement corrosion management techniques, transportation decision-makers can be efficient and effective in their use of public resources, leading to a safer bridge inventory throughout the state and the nation.

ACKNOWLEDGEMENTS

This project was sponsored by the West Virginia Department of Transportation (WVDOT) Division of Highways (WVDOH). The principal investigator of the project, Wael Zatar, would like to acknowledge the financial support provided by the WVDOH. The principal investigator of the project would like to acknowledge Jimmy Wriston, Donald Williams, and Billy Varney from WVDOH, as well as Dong Nguyen and Seth Baker.

DISCLAIMER STATEMENT

Prepared for the West Virginia Department of Transportation Division of Highways, in cooperation with the United States Department of Transportation Federal Highway Administration. The contents of this report reflect the views of the authors, who are responsible for the accuracy of the information presented herein. The contents do not necessarily reflect the official views or policies of State or Federal Highway Administrations. This report does not constitute a standard, specification or regulation. Trade or manufacturers' names or models which may appear herein are cited only because they are considered essential to fulfill the objectives of this report. The United States Government and the State of West Virginia do not endorse products or manufacturers.

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CHAPTER 1 - Introduction

1.1 Introduction

Corrosion-induced deterioration of reinforced concrete bridge superstructure members is a common and costly problem in the United States. In a recent report to Congress, the Federal Highway Administration reported that of the nation's 577,000 bridges, 134,000 (23 percent of the total) were classified as structurally deficient. Structurally deficient bridges are those that are closed, restricted to light vehicles only, or that require immediate treatment to remain open. The United State Department of Transportation has estimated that $90.9 billion will be needed to repair the damage on these existing bridges. Approximately 40 percent of the current backlog of highway bridge repair and rehabilitation costs is directly attributable to the corrosion of reinforced concrete bridge components.

Concrete bridge deterioration is not a new phenomenon. There have always been concrete bridges that have failed due to of exposure to deicing salts or other environmental factors. What has changed the perspective on concrete bridge deterioration is the extent of the problem and the vast sums of money required to maintain the existing highway network. Not only does corrosion destroy the smooth riding quality of the deck; it may eventually reduce the structural integrity and safety of the bridge.

Any rational decision regarding maintenance, repair, or replacement of the deteriorated members should take into account the member's condition, the extent of deterioration, the expected remaining service life, and the impact of alternative maintenance and repair options on the service life of the members.

In making these decisions, the WVDOT —like other DOTs across the country—uses visual inspection techniques for monitoring the extent of cracking and damage of bridge members. Visual inspection, combined with concrete sounding techniques such as hammer sounding or chain dragging, may provide some information about the condition of bridge members, but newer techniques that have been developed over the years that can provide inspectors and bridge engineers with more detailed and reliable information. A few examples include chloride concentration testing from core samples, half-cell potential measurements, ultrasonic testing for assessment of cracking, ground-penetrating radar, and several other methods that will be covered later in this report.

However, in the "real world," it isn't feasible to perform every test on every structure. Yet, visual inspection does not provide an accurate, objective estimation of the remaining service life of the bridge member. Determining most effective and least cost prohibitive inspection, repair, and preservation strategies is crucial for maintaining the health of the WVDOH bridge inventory. Therefore, there is an urgent need to identify and/or develop suitable procedures for assessing the condition of corrosion-deteriorated bridge members, estimating their expected service life, and determining the effects of the various maintenance and repair options on their service life.

Once these strategies are determined, a comprehensive manual must be developed to outline the procedures for the assessment of corrosion-deteriorated reinforced concrete members of WV's highway bridges. The manual will assist WVDOH engineers in effectively managing the safety and integrity of WVDOH's bridge inventory while simultaneously maximizing returns on maintenance investments.

1.2 Research objectives

The eventual goal of this project is to develop a comprehensive manual that will be adopted by the WVDOH. This manual will identify reliable, research-proven procedures for assessing the condition of reinforced concrete bridge members subject corrosion-induced deterioration, predicting the remaining service life of these members, and quantifying the service life extension based on various maintenance and repair options.

To achieve this goal, the present study was conducted with the following objectives:

1) Review methods for evaluating concrete and steel reinforcement
2) Review methods for predicting the remaining service life of deteriorated bridge members subject to chloride-ion-induced corrosion.
3) Review methods of quantifying the service life extension expected from various maintenance and repair options.
4) Discuss the applicability and performance of steel reinforcement and concrete corrosion inhibitors.

1.3 Organization of Report

This report is divided into six chapters, including the present chapter. Chapter 2 provides an overview of the literature pertaining to the corrosion process, including sections on chloride-induced corrosion and the time to corrosion initiation in bare and epoxy-coated steel rebar. Chapter 3 provides a general overview of the bridge inspection process, and is subdivided into discussions about the evaluation and inspection of concrete and steel reinforcement in concrete, respectively. (Some overlap is necessarily present between the two sections). Chapter 4 presents several theoretical and software methods for modeling the expected service life and service life extension provided by various repair options. Protection, repair, and rehabilitation methods are covered in Chapter 5. Chapter 6 contains the study's conclusions and recommendations.

CHAPTER 2 - Literature Review

2.1 The Corrosion Process

Corrosion is defined as “the chemical or electrochemical reaction between a material, usually a metal, and its environment that produces a deterioration of the material and its properties” (ASTM terminology, G15). When reinforcement in concrete corrodes, the cross sectional area of steel reduces, which leads to failure of the concrete structures subject to tensile and flexural stresses.

On a molecular level, chemical ions occur in cells where the anode and cathode are directly adjacent to one another. These are referred to as micro-cells. Anode- and cathode-corrosion cells separated by some distance are considered macro-cells. Generally, iron (Fe) atoms pass into solutions as positively charged (Fe^{2+}) hydrated ions at the anodic site, and the liberated electrons flow through the metal to cathodic sites, where dissolved oxygen is available to consume them. The reactions that take place at the anode and cathode are presented below (Broomfield, 2007):

At the anode:

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

At the cathode:

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

The process is expressed by equations below:

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

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

$$2\ Fe(OH)_3 \rightarrow Fe_2O_3 \cdot H_2O + 2H_2O$$

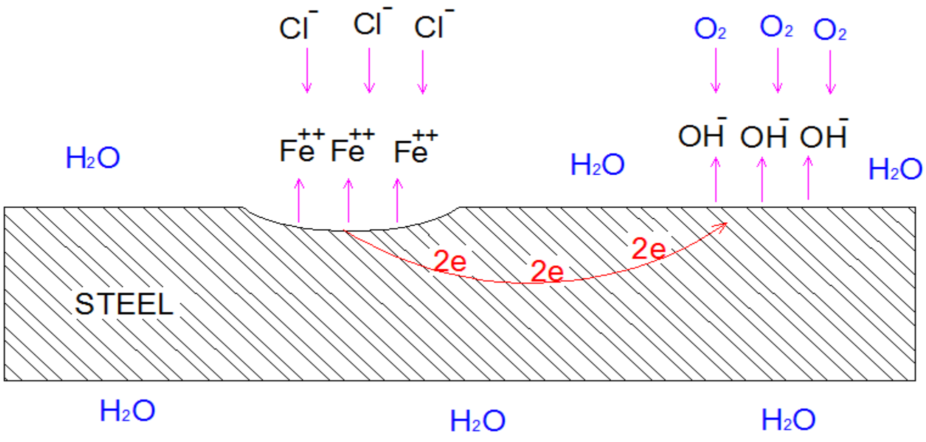


Figure 2.1 Steel Corrosion Reactions in Concrete

Hydrated ferric oxide ($Fe_2O_3 \cdot H_2O$) may have six to ten times the volume of the consumed steel that it replaces. The volumetric increase causes pressures in the concrete, which results in cracking and spalling of the concrete cover (Broomfield 2007).

2.2 Carbonation-Induced Corrosion

Carbonation is the compound resulting from the interaction of carbon dioxide gas in the atmosphere with alkaline hydroxide in the concrete (Broomfield 2007). Carbon dioxide reacts with water in concrete pores to form a carbonic acid:

$$CO_2 + H_2O \rightarrow H_2CO_3$$

Carbonic acid reacts with calcium hydroxide to form calcium carbonate:

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

Carbonation does not cause to damage to concrete. However, carbonation leads to corrosion of steel in concrete. The pH of the pore solution decreases from its normal values of pH 13 to 14 to neutral values (Bertolini, 2005). The value of pH decreases when there is a higher quantity of rust. Figure 2.2 shows the relationship between pH and the quantity of rust. A quantity of rust with a value of 0.8% has a pH value of 10; the quantity of rust will decrease if pH increases.

Carbonation damage occurs quickly where there is low concrete cover and/or an open pore structure. Low cement content, a high ratio of cement, and/or poor curing of concrete are causes of high rates of carbonation damage..

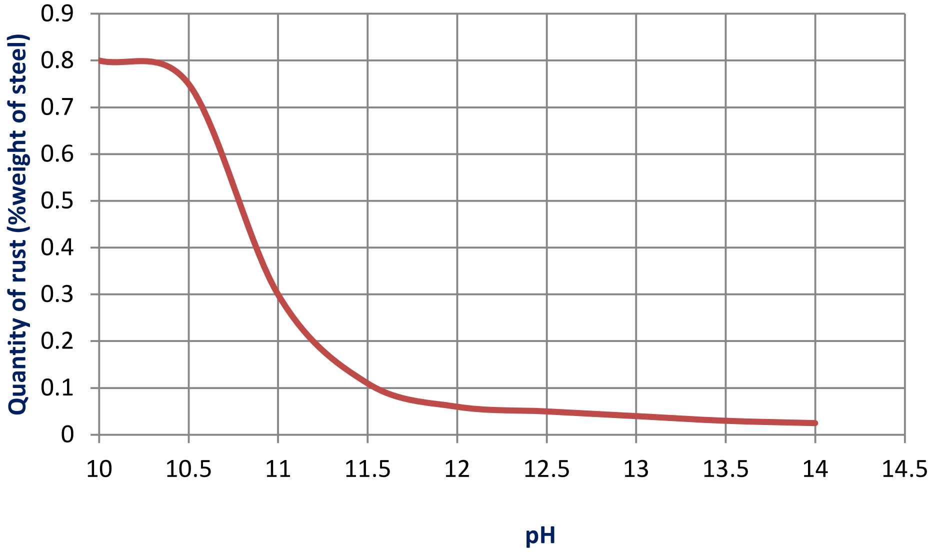


Figure 2.2 Relationship between pH and Quantity of Rust

Bertolini (2005) defined the penetration of carbonation by :

$$d = K\sqrt{t}$$

Where:

d – depth of carbonation (mm)
t – time (year)
K – carbonation coefficient corresponded to factors of concrete condition and environment.

Environmental factors as humidity, CO_2 concentration, temperature, and other factors (such as alkalinity and permeability) affect the rate of carbonation.

The higher humidity of the concrete is a result of a higher percentage of water in concrete. Carbonation reactions only occur with the presence of water; as a result, there is a low rate of reaction in dry concrete. Carbonation rate reaches its the maximum value in the humidity range of 65 – 75 percent. This is relevant for estimating the value of K; the value of K is high when humidity between 60 to 80 percent.

Carbonation concentrations vary by regions; the carbon dioxide in the atmosphere in rural environments is lower than in urban environments. Additionally, areas of high CO_2 exposure under vehicles, such as bridge decks, normally have very high carbonation concentrations.

An increase in temperature stimulates electrochemical reactions and diffusion processes. High temperatures cause a reduction of humidity and the drying up of the surface electrolyte. Lower humidity results in lower carbonation damage.

Finally, the composition of concrete has a significant effect on the permeability of concrete; higher permeability increases the penetration rate of carbon dioxide and increases the rate of the diffusion of carbon dioxide (the carbonation rate).

2.3 Chloride-Induced Corrosion

Chloride contamination of concrete causes the corrosion of steel reinforcement. Chloride ions break down the protective oxide on the steel in concrete, resulting in localized corrosion. As higher levels of chlorides reach the surface of the reinforcement, the area of corrosion increases. Chloride either casts in the concrete or diffuses outside of concrete. Chloride cast in concrete is a result of contaminated aggregates (such as the use of water with a high salt content during mixing). Diffusion outside concrete is due to deicing salts, sea salt spray, and direct seawater wetting.

The creation of passive layers in the presence of chlorides is due to localized concentrations of free chloride ions. As current flows form anodic to cathodic areas, the chloride content increases while alkalinity decreases (chlorides, being negatively charged ions, migrate to the anodic region).

As deicing salts accumulate on the surface of the bridge deck, chloride ions first cover the concrete deck, then subsequently spread to exposed caps, girders, and columns. The chloride concentration in concrete is normally higher than in the external environment because the wetting and drying of concrete raises the accumulation rate of chlorides inside the concrete (Sohanghpurwala, 2006).

Highly dry concrete resists significant corrosion rates even though chloride is present (Broomfield, 2007). “The concrete is too dry, there is no mechanism for transport of chloride into the concrete” (Sohanghpurwala, 2006). In dry environments with humidity less than 60%, chloride-induced corrosion rates are negligible (Tuuti, 1982).

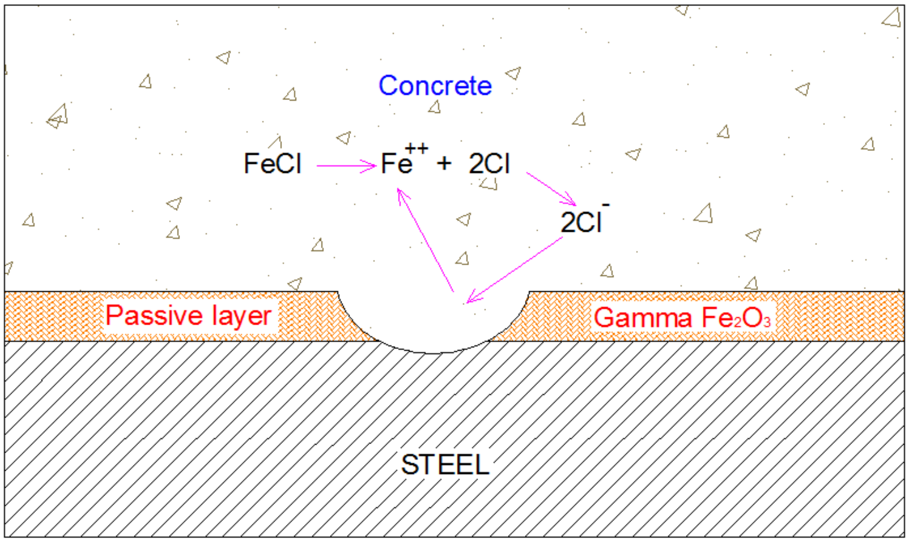


Figure 2.3 Breakdown of Passive Layer and Recycling Chlorides

In alkaline concrete, a passive layer of protective oxide is created on the reinforcement’s surface, preventing corrosion. (Broomfeild, 2007). In pitting and cracking areas, the chloride attacks the passive layer and breaks down the oxide passive layer to cause rapid corrosion (see Figure 2.3). The rate of chloride ingress is estimated by Fick’s laws of diffusion:

$$\frac{\partial C}{\partial t} = D\left(\frac{\partial^2 C}{\partial x^2}\right)$$

Where:

C = chloride ion concentration

t = time

D = diffusion coefficient

x = depth from the surface of concrete

However, in practice, the total chloride contents are measured using the error function solution:

$$C_x = C_s\left(1 - erf\,\frac{x}{2\sqrt{D_{app}.t}}\right)$$

Where:

C(x,t) = chloride concentration at depth *x* at time *t*

C_s = surface concentration (% by mass of cement or concrete at the time t (s) and the depth x from the surface of concrete)

D_{app}= diffusion coefficient for chloride (m^2/s). This value depends on the factors of exposure condition and the time of exposure.

erf = error function

The chloride concentration at the surface influences the speed of diffusion through concrete. The rate of chloride concentration is affected by the ratio of water and cement, the quantity and type of cement, and the curing conditions. In the case of one-sided pressure from water containing chloride, chloride transported with the water penetrates the concrete, causing the reaction without pressure surface as the water evaporates. The position of structure and the orientation of surface affect the concentration of chloride. In addition, environmental factors such as wind and rain also affect the chloride concentration. These effects correspond to the values of surface concentration ($C_{s)}$.

D_{app} depends on the pore structure of the concrete and factors as w/c ratio, compaction, curing and the presence of microcracking. In relation to the properties of concrete and the condition of exposure, the D_{app} has a variable value from 10^{-13} m2/s to 10^{-10} m^2/s (Bertolini, 2004).

2.4 Chloride Threshold Concentrations

It is necessary to use a threshold value of chloride concentration on the reinforcement to estimate the total section loss and to predict the concrete spalling rate. However, it's difficult to determine the exact value of the threshold concentration because there are many factors that affect corrosion initiation. These factors, such as cement composition, moisture content, W/C ratio, and temperature, influence the cement paste, the capability for corrosion to initiate based upon the electrochemical potential of the steel, the ability for localized corrosion (pitting) to form, and the passive layer's capacity to re-establish itself (Williamson, 2007).

There are many methods used to obtain the chloride concentration. A popular method is to specify the percentage of chloride by weight of cement, weight per unit concrete, and chloride-to-hydroxyl ratio (Williamson, 2007). In some reports, the chloride/hydroxyl ratio is important to obtain the specified level at which corrosion will initiate (Lambert et al; 1991). However, the hydroxyl concentration value is difficult to determine and leads to difficulty in determining the chloride-to-hydroxyl ratio. Therefore, "chloride threshold levels are best presented as total chloride contents expressed relative to the weight of cement" (Glass and Buenfeld, 1997).

The issue of free chloride concentration and the total chloride concentration is still in debate. Some argue that bound chlorides do not initiate or propagate corrosion. Other reports indicate that corrosion propagates bound chlorides, which are available to the corrosion. Williams writes that "the concentration of chlorides per unit of concrete will be the convention used in this study, as actual cement contents for the bridge decks sampled are not known and do not vary significantly in Virginia where a minimum cement content and maximum w/c are specified" (Williamson, 2007).

The measurement of chloride threshold concentration does not have an exact value because of dynamic environmental effects (such as temperature and moisture) that alter the chloride concentration rate. Further, these values are applicable only for bare steel; data on epoxy-coated or galvanized reinforcement are not available due to interpretation procedures (Sohanghpurwala, 2006).

2.5 Time to Corrosion-Induced Cracking of Cover Concrete

Time to damage is the sum total of the time of initiation period and propagation period (see Figure 1.4). Initiation period is the time required for diffusion chloride to penetrate the concrete cover and accumulate in concentrations exceeding the corrosion threshold. The propagation period is the time required for corrosion to cause spalling and eventual failure.

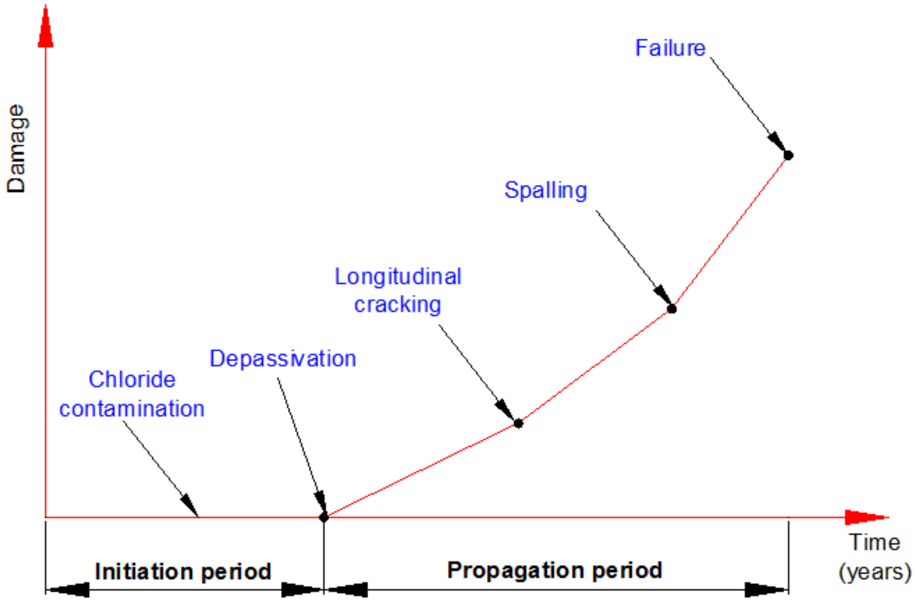


Figure 2.4: Corrosion – induced damage of reinforcement concrete structure

There are many factors affecting the corrosion rate of steel in concrete, such as the permeability of the concrete, moisture, temperature, and the availability of oxygen. As a result, the time required for corrosion to propagate significantly depends on the local climate. The propagation time in dry climate lower moisture levels is longer than in areas with high moisture levels. Many reports estimate the propagation period. Weyers (1993) gave the propagation time from 2 and 5 years for bare steel. For epoxy-coated reinforcement, propagation time has been estimated to increase the time to between 2.7 and 5.7 years (Weyers et al., 2006). In fact, in predicting the propagation period, Life-365 Service Life Prediction Model (more on this later) uses a value of 6 years.

2.6 A Brief Discussion on the Limitations of Using Epoxy-Coated Steel in Concrete

The most common solution to the corrosion deterioration problem is the use of coated steel reinforcements such as epoxy-coated steel (ECS) rebar or galvanized rebar to prevent the corrosion circuit from developing. However, the experience of other DOTs has shown that these coatings cannot completely prevent deterioration.

Epoxy-coated reinforcing steel (ECS), in which steel rebars are coated with a powdered epoxy resin, are the most widely adopted solution to the steel corrosion problem. ECS rebars were thought to be an affordable, simple, solution to the corrosion of steel rebars in concrete until 1987, when the Florida Department of Transportation reported corrosion problems with ECS rebars in marine substructures after only four to seven years of service. Because of these findings, research projects were carried out to study how long ECS rebars could prevent deterioration of bridge components exposed to chloride environments.

Epoxy coatings act as a physical barrier between aggressive chloride ions, oxygen, water and the bare rebar steel interface. As a result, the initiation period of corrosion of ECS is longer. They are less dependent upon cover depth than bare steel; however, the condition of the coating significantly affects the time to corrosion initiation.

In general, ECS defects normally occur during the fabrication process; examples include non-uniform coating or small pinholes in the coating. Further, construction can lead to nicks and cuts in the epoxy coating, leaving portions of the rebars vulnerable to chloride ion induced corrosion. At these areas without epoxy coating initiation of chloride ingress occurs quickly.. To re-establish corrosion protection, epoxy must be applied in the field, which can take up valuable time during construction. In addition, experimental tests have shown that epoxy coating significantly reduces the bond strength of rebars.

Debates continue regarding the effectiveness of epoxy coatings in reducing the corrosion rate. Some reports state that epoxy coating does not help extend the life of service in reinforcement concrete structure. “ECR will not provide any or little additional service life for concrete bridge decks in comparison to black steel” (Pyc, 1998). However, in an investigation of six 33 to 35 year-old bridges in West VirginiaLawler et al (2011) found decks that used ECR have required no rehabilitation to address corrosion-related deterioration.

In addition, Ramirez and Hasan (1995) observed six concrete bridge decks in Indiana that were reinforced with epoxy-coated steel. They concluded that “no signs of corrosion were found in the epoxy-coated steel samples extracted from cores taken in the six bridges evaluated. Evaluation of the field data revealed that the combination of adequate concrete cover and epoxy-coated steel has provided a good corrosion protection to date”(Ramirez and Hasan, 1995).

CHAPTER 3 - Evaluating the Condition of Concrete Bridge Elements

3.1 Overview of the Bridge Inspection Process

WVDOH performs bridge inspections in accordance with the National Bridge Inspection Standards (NBIS) as mandated by the Federal Highway Administration (FHWA). The NBIS are defined as “Federal regulations establishing requirements for inspection procedures, frequency of inspections, qualifications of personnel, inspection reports, and preparation and maintenance of a State bridge inventory. The NBIS apply to all structures defined as bridges located on all public roads” (FHWA, 1995).

State DOTs are required to submit NBIS data annually as part of the Federal Review Process. WVDOH collects Structure Inventory and Appraisal (SI&A) data on each structure in the bridge inventory and stores it in a database. This data includes information such as condition codes for the structure, substructure, deck, wearing surface, as well as other elements. The condition of each element is rated by the bridge inspector on a scale of zero to nine. Elements with a rating between 0-2 (critical) require immediate repair or replacement; a rating of 3-4 (poor) indicates the element is performing at a minimum of functionality, and that the element will require repair or replacement in the very near future; a rating between 5-6 (fair) may require minor repair, and/or requires rehabilitation to prevent its condition from worsening to a level such that it will not be preserved by minor maintenance; and a rating between 7-9 (good) indicates that the element is performing its intended function and does not require any repairs.

Each element must be rated accurately, as these ratings factory heavily into the overall calculation of structural deficiency, structural obsolescence, and the sufficiency rating of structures. The sufficiency rating of a structure determines its eligibility for federal funding, which is important for state DOTs when forecasting financial requirements for bridge programs.

The majority of bridge inspections in the US are performed using nondestructive evaluation techniques with hand tools or basic visual inspections, and it is not feasible (and perhaps unnecessary) to perform advanced inspection techniques on every bridge in a DOT inventory. “The Manual for Bridge Evaluation,” published by the American Association of State Highway and Transportation Officials, outlines seven different types of inspections that may be performed on a bridge over its lifetime. These inspections include:

1. Initial inspection – WVDOH refers to this inspection as an inventory inspection, since its primary purpose is to gather all SI&A data required by federal and state regulations, as well as to document other information about the bridge, such as member dimensions, material strengths, material types, etc. The second purpose of the initial inspection is to document the condition of the bridge and to establish a baseline for the structure that all subsequent inspections can be compared to. This is crucial in determining if the structure is behaving as its design anticipated.

2. Routine Inspections – WVDOH refers to this inspection as a Periodic Inspection. This inspection is generally performed once every two years. These inspections collect observations and measurements to determine whether there has been any change in condition since the previous inspection. The areas of the structure with previously documented issues should receive additional attention, and any changes should be documented with photographs and detailed sketches. An updated load rating may be required if the condition of the structure has changed enough to reduce the load carrying capacity of the element in question.

3. Damage Inspections – Required when a specific element or the entire structure has suffered damage from a collision, fire, scour, etc.

4. In-Depth Inspections – An In-Depth Inspection is a detailed inspection of the entire structure (above or below water) to identify any deficiencies not readily detectable using Routine Inspection procedures. WVDOH generally performs an In-Depth Inspection at the same time that the Initial or Inventory Inspection is performed, and every six years thereafter. An In-Depth Inspection may require non-destructive testing and will sometimes require the use of under-bridge inspection units, scaffolding, or rigging to access all parts of the structure.

5. Fracture-Critical Inspections – This type of inspection is only applicable to steel bridges and as such is outside the scope of this report.

6. Underwater Inspections –Underwater Inspections consist of a hands-on inspection of the elements of the structure below the water surface. The techniques involved in this inspection include (but aren’t limited to) diving or probing with a rod to determine the scour conditions of the substructure foundation elements.

7. Special Inspections – A Special Inspection is an inspection scheduled at the discretion of the Bridge Owner or the responsible agency. WVDOH refers to this inspection as an Interim Inspection. This type of inspection generally occurs every other year between a Routine (Periodic) Inspection. A Special Inspection would be required to monitor a known or suspected deficiency, such as source issues, major member deterioration, or structures with a low load rating. Depending on the severity of the problem, a Special Inspection may be required on a six-month, three-month, or even monthly inspection interval.

3.2 Inspection and Evaluation Methods for Concrete

Evaluation of concrete is important to predict the remaining service life of an existing structure. In a girder and beam bridge structure, concrete is designed to carry loads. Reinforced concrete is a composite material consisting of concrete and reinforcing steel. Concrete must surround embedded reinforcing steel. Concrete evaluation is important to determine the causes of damage factors such as cracking, leaching, spalling, scaling, stains, disintegration, wear, settlement, or deflection.

Emmons (1993) identified four categories of concrete evaluation:

Mechanical properties are used to evaluate for comprehensive strength, quality of concrete, tensile strength, flexural strength, abration resistance, and bond strength.

Evaluations of chemical make-up are used for electro-chemical activities (half cell potential, electrical resistivity), carbonation depth, alkali-aggregate reactions, and chloride content.

Physical condition includes evaluations of uniformity, air void system, delamination, condition of embedded steel, water and air permeability, water absorption, frost and freeze-thaw resistance, and resistance to deicing salts.

External manifestation is the process for evaluating crack/spall, deflection from loads, leakage, temperature/moisture conditions, and external geometry.

Evaluation describes some visible sign of distress which cause structural and durability concerns which could lead to poor functional and safety performance. Concrete evaluation not only studies of its physical condition, mechanical properties, chemical make-up, and external manifestation of a structure, but considers its interaction with the environment. Temperature, moisture, chemical and loading are also investigated to determine service and exposure conditions.

3.2.1 Visual Inspection

Visual inspection is normally the first step in any investigation. A visual review of conditions indicates problems related to cracking, spalling, and disintegration of the surface, scaling surface dampness leakage through joints or cracks, deflections, settlement, steel corrosion, and other problems. Visual examination, mapping the location of problems and construction records can provide a general scope of the problems and possible clues to the cause. Visual examination often results in many questions regarding the extent and details of deterioration/distress. A useful way to answer these questions is through the exploratory removal of concrete to expose hidden elements.

It is normal to record the location of problems, date time and weather conditions when doing the survey, also noting visual observations such as water or salt runs. Visual inspection equipment includes camera, notebook, hand-held computer, and/or and binoculars. Interpretation is usually based on the knowledge and experience of the engineer or technician conducting the survey.

However, there are some available technique equipments that help inspectors view cracks easily, such as new Spectroline Model ASR-260 Field Inspection Kit. This device uses a reactivity gel to instantly detect the presence of damage previously hard-to-examine places, shown in Figure 3.1.

Figure 3.1:Installing the ASR-260 Field Inspection Kit

3.2.2 Delamination Inspection

A typical form of deterioration induced by corrosion of reinforcing steel is delamination of the concrete. Ongoing corrosion, freeze-thaw cycles, and the impact of traffic increase the size of the delamination, which can impact the structural integrity of the concrete element. Delamination is the separation of concrete planes leading to tensile failure. Therefore, the purpose of a delamination inspection is to measure the amount of cracking between the rebars before it becomes apparent at the surface.

Survey equipment includes a hammer, radar, ultrasonic or infrared thermography equipment. However, sounding of concrete with a hammer is usually a quick, cheap, and accurate method for identifying delaminated areas. Good concrete with no delamination has a sharp ringing sound; delaminated areas have a puckish, hollow sound.

Electronic systems only give a general picture of the areas of delamination. They should be used only for general assessment rather than a detailed layout required for reconstruction. Infrared thermography is a popular method of detecting delaminations in bridge decks. Since concrete is a poor conductor of heat, this method works by evaluating differences in temperature of the concrete during heating and cooling. Consequently, delaminations and other incontinuities interrupt the heat transfer through the concrete due to substantial thermal gradient within the concrete.

The sounding method is specified in ASTM C4580-86, “Standard Practice for Measuring Delaminations in Concrete Bridge Decks by Sounding.” ASTM D4788 (2003) is used for covering infrared thermography of concrete and asphalt-covered concrete bridge decks. ASTM D6087-05 (2005) contains the use of ground penetrating radar to evaluate asphalt covered concrete bridge

3.2.3 Cover Depth Measurements

The depth of concrete cover over reinforcing steel affects the rate of corrosion. Shallow cover will increase the corrosion rate due to more rapid accumulation of chloride ions at the steel depth. A cover survey is essential to explain why some areas of structure are corroding and to determine areas of potential future corrosion.

A cover survey must identify three properties, including: (1) the relationship of reinforcement positions with each other, (2) the plane of the surface, and (3) the depth from the surface of reinforcement. Measurement of the rebar diameter is also necessary in the cover survey.

Logging features and digital output of magnetic cover meters are used to determine cover depth. Limitations of this method include potentially inaccurate readings when rebars are closely spaced, and the time required to gather data. Newer technology enables the scanning cover meter to generate a plot of steel layout and depth (Alternatively, spacer, radiograph, and radar can also be used to estimate rebar diameter in concrete). However, cover meters are difficult to use in deep covers because they are slow. Performance and accuracy of some designs are discussed in BS1881 part 204, Concrete Bridge Development Group Technical Guide 2 (2002).

3.2.4 Electrical Resistance & Potential Methods

Electrochemical corrosion of reinforcement occurs when the passivity of the steel is destroyed (by carbonation or chloride ingress) and electrochemical cells develop. Steel in the concrete has anodic (corroding) areas and cathodic (passive) areas.

Copper-copper sulfate (Cu-$CuSO_4$), silver/silver chloride/calcium chloride (Ag/AgCl/CaCl), manganese/manganese dioxide/sodium hydroxide (Mn/MnO_2/NaOH) double-junction electrodes are used for determining corrosion potential. The Cu-$CuSO_4$ half-cell type is the most popular due to its applicability to a wide range of operating conditions and the convenience of obtaining copper sulfate and distilled water. A copper/saturated copper sulfate reference electrode covers on the surface of concrete.

As previously mentioned, corrosion creates electrical potentials. Half-cell potential measurement is a method of detecting and categorizing these electrical potentials. (See ASTM C 876 for method and equipment requirements).

The equipment used consists of a voltmeter, reference electrode, and cables in contact with the concrete (see Figure 3.2 and Figure 3.3). The reference cell connects the concrete surface to a high-impedance voltmeter and connects the reinforcing steel to the reference electrode. Variations in chloride distribution, temperature and moisture content affect half-cell potentials over wide areas. To obtain a good result, several closely spaced half-cell potentials should be evaluated.

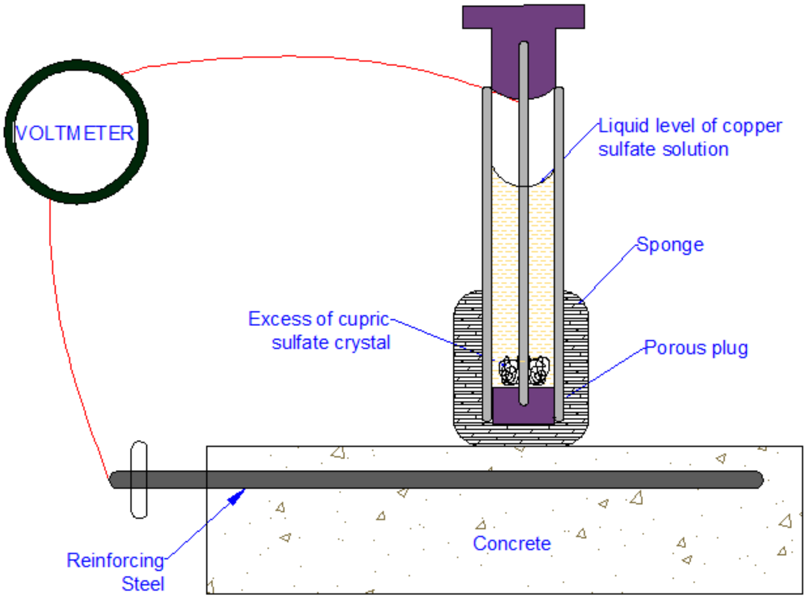


Figure 3.2 Copper-copper sulfate haft cell circuitry

Figure 3.3 Devices for Electrode (half cell) Potential Measurements

It is necessary to ensure good contact between the reference electrode and the concrete in order to minimize the ohmic drops and avoid errors (Arup, 1979).

ASTM C 876, “Standard Test Method for Half-Cell Potentials of Uncoated Reinforcing Steel in Concrete”, presents guidelines for interpreting the results of Cu-CuSO4 half-cell potentials of uncoated reinforcing bars in concrete. See Table 3.1

Table 3.1: Guidance on interpretation of results from half-cell surveys

(Adapted from ASTM C 876-91)

E_{corr} ($Cu/CuSO_4$)	Probability of Corrosion
> -0.02 V	Greater than 90% probability to no corrosion
-0.35 to -0.20 V	Corrosion activity uncertain
< -0.35 V	Greater than 90% probability of active corrosion

Collection of historical records of corrosion potentials is vital. Sohanghpurwala (2006) claimed that "if the potential survey is conducted at a regular time interval, variations in the potentials with respect to time can indicate if the corrosion activity of the steel is increasing or if the total area of steel showing active potentials is increasing."

However, Liu (1996) indicated that this method does not provide information on the amount of corrosion, since contact between steel reinforcement and concrete is a function of a large number of variables related to many factors such as composition of pore solution, degree of polarization, temperature, and cover depth. Broomfield (2007) listed nine limitations of electrode (half-cell) potential measurements:

1. Electrical continuity: poor metal to metal contact, poor reinforcement continuity within survey area, or scanning across a discontinuity such as an expansion joint.
2. Surface contact: failure to remove contamination, coatings or to wet the surface adequately.
3. Cracked and spalled concrete: this can distort the current path or give low readings due to poor contact between steel and the electrode.
4. Saturated concrete: can give very negative potentials while excluding the oxygen needed to fuel corrosion.
5. Other metals: the presence of galvanizing, galvanic anodes, or conduit in the concrete can distort potential measurements.
6. Stray currents: nearby sources of DC (such as cathodic protection or DC traction systems) can lead to potential shifts.
7. Electrochemical treatments: cathodic protection, electrochemical chloride extraction and electrochemical realkalization are designed to shift the potential of the steel. This effect may be permanent (in the case of cathodic protection) or temporary (but long-term).
8. Chemical contamination: anything that inhibits or promotes anodic or cathodic reactions on the steel surface can distort potential measurements. This may include surface-applied corrosion inhibitors.
9. Carbonation - the issues of carbonation have been discussed in the previous section. Broomfield (2007)

However, many studies such as Elsener and Böhni (1991); Berkeley and Pathmanaban (1987); and Sykes (1994) concluded that the half-cell potential technique is useful and widely applicable for the measurement of the corrosion rate of the reinforcement. To address the listed limitations,

Sohanghpurwala (2006) advised that it is important to carry out on a regular interval grid and evaluate a large number of closely spaced half-cell potentials.
Standards and guidance for Electrode (half-cell) Potential Measurements is illustrated in ASTM C876-91 (1999), Concrete Society (2004).

3.2.4.1 Mapping corrosion using half-cell potential measurements.

Half-cell potential measurements can also be used to map concrete corrosion using voltage contour line, which indicates areas for the level of corrosion (Figure 3.4). The red area shows a major problem,where potential difference (V) is greater than - 0.35. The yellow area indicates a slight problem at the locations with the potential difference (V) ranging from - 0.2 to - 0.35. Green is the area without corrosion with a potential difference (V) greater than - 0.35.

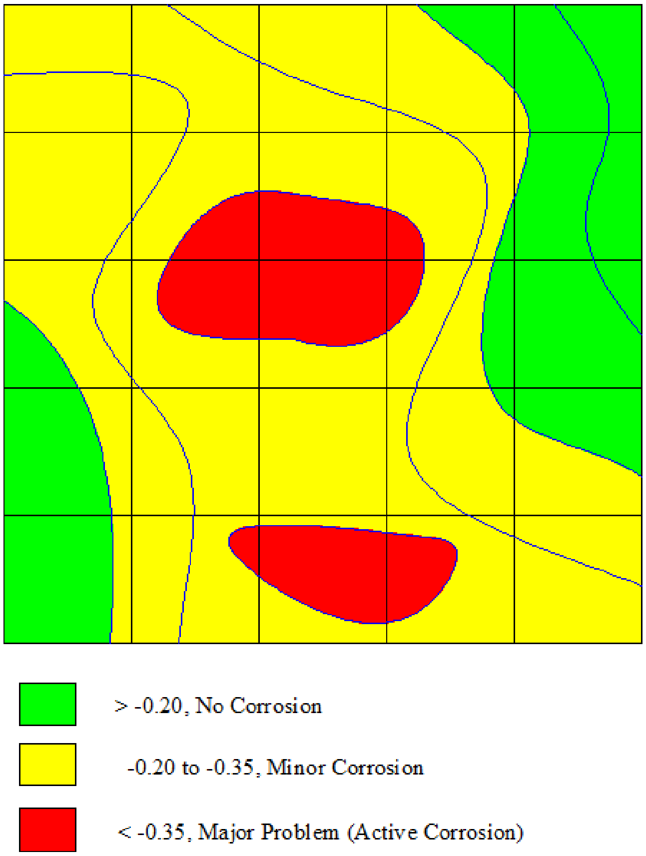
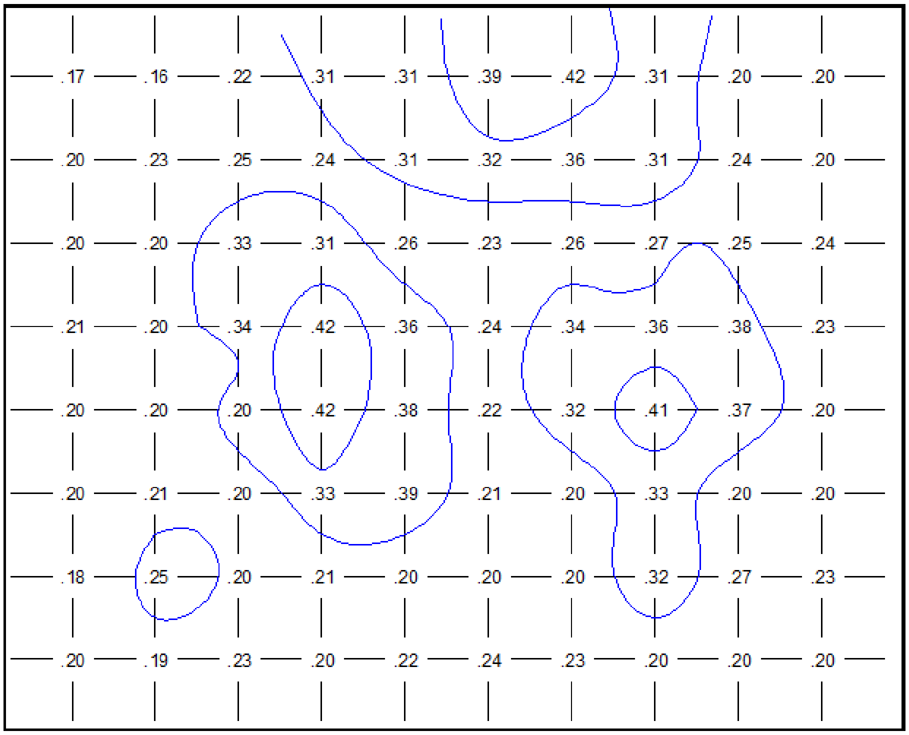


Figure 3.4: Sampling of Corrosion from Haft-Cell Potential Measurement

In general, the map is divided into 1.00 m x 1.00 m grid lines for test locations on the concrete surface. Typically, the solutions are determined on each grid of the locations. Importantly, the potential difference is recorded by the voltmeter to be used to plot the results in the contour lines as in the figure 6.2.



.17	.16	.22	.31	.31	.39	.42	.31	.20	.20
.20	.23	.25	.24	.31	.32	.36	.31	.24	.20
.20	.20	.33	.31	.26	.23	.26	.27	.25	.24
.21	.20	.34	.42	.36	.24	.34	.36	.38	.23
.20	.20	.20	.42	.38	.22	.32	.41	.37	.20
.20	.21	.20	.33	.39	.21	.20	.33	.20	.20
.18	.25	.20	.21	.20	.20	.20	.32	.27	.23
.20	.19	.23	.20	.22	.24	.23	.20	.20	.20

Figure 3.5: Contour Lines for Typical Concrete Section

3.2.5 Chloride Ion Determination

The amount of chlorides present in reinforced concrete can significantly affect its performance. Evaluation of chloride ion content is an important testing method. There are many different methods to measure chloride contents. All normal methods require series samples from drillings at different depths so that a chloride profile can be produced.

There are two forms of chloride ions in concrete: chemically bound and soluble in the concrete pore water. The chloride ion content of concrete is normally determined in the laboratory by wet chemical analysis. In fact, Sohanghpurwala (2006) stated that the total chloride (or acid soluble) test method equals to the sum total of all chemically bound and free chloride ions in the concrete. The water soluble test method is not accurate; as a result, using the acid soluble test method is only the general practice (Sohanghpurwala, 2006).

Chlorides in concrete structures can be present as cast-in chlorides or diffusing from the outside. As previously mentioned, the presence of corrosion of reinforcements only occurs once a threshold value of chloride ion content adjacent to the bars is approached this threshold value is approximately 0.2% to 0.4% by weight of cement (Broomfield, 2007). Output data of the chloride content analysis is normally either percentage chloride by weight of concrete, parts per million (ppm) of chloride ions, percentage chloride per weight of cement, or weight of chloride per volume of concrete (Sohanghpurwala, 2006). The chloride diffusion rate can be used to calculate the threshold of chloride ion concentration required to initiate corrosion. Acid-soluble content and water-soluble content are two analytical values used to determine the chloride content. Acid-soluble content is widely used to gather a measurement and proportion of chloride. The water soluble content is chloride extracted in water depending on conditions (Liu, 1996).

Samples from drillings at different depths can be collected by using a hammer drill. Brown (2002) recommended collecting samples different drill depths, and that that chloride samples should be obtained using a hollow-bore impact drill bit of 29 mm (1.125 inches) diameter, from three locations of each bridge.

In the laboratory, powdered samples are used to determine the concentration by dissolving powder samples in acid. In the field, quantab strips and specific ion electrodes are two popular methods of measuring chlorides. However, any field technique should be checked against laboratory analysis of the same samples.

The presence of the steel bar in concrete and capillary action significantly affect chloride concentration. The presence of the rebar likes a local barrier diffusing within the concrete matrix (Brown, 2002). In fact, the presence of the bar can remarkably build up the rate of chloride concentration increase at the leading face of the bar (Kranc, et al., 2002). The resulting effective diffusion rate will be dependent upon the ratios of bar diameter to clear cover, and of surface concentration to threshold initiation concentration. Capillary action influences chloride uptake during the initial ponding (Brown, 2002). However, once chloride concentration reaches saturation, the behavior more closely resembles diffusion-controlled chloride migration.

There are some relevant ASTM standards for chloride determination. ASTM C11521/C1152M-04e1 is the standard test method for acid-soluble chlorides in concrete and mortar. ASTM C1524-02a is the standard test method for water-extractable chloride firm aggregate. ASTM C12181/C1218M-99 is the standard test method for water soluble chloride in mortar and concrete. Finally, the ingress of chloride through the concrete cover to the reinforcing steel depth is normally modeled by Fick’s Second Law of Diffusion (see Chapter 4, Section 3)..

3.2.6 Electrical Resistance and Potential Methods

The resistivity of concrete is affected by concrete quality such as cement content, water/cement ratio and curing. The magnitude of resistivity varies from 100 to 100,000 Ohms.m (Bertolini, 2004). The electrical resistivity of concrete may identify the progress of corrosion of steel reinforcement. The intrinsic quality of concrete, the condition of surface/bulk deterioration caused by cracking, delamination and salts, and the distribution of reinforcement bars greatly affects the electrical resistivity of reinforced concrete. Dry concrete (low water/cement ratio) has high resistivity.

3.2.7 Petrographic Analysis

The standard method of Petrographic Analysis is covered in ASTM C 856 “Standard Practice for Petrographic Examination of Hardened Concrete.” Petrography is an excellent aid to assess the cause of deterioration as well as overall quality and durability when a Non-Destructive Evaluation (NDE) is not adequate.

Information obtainable from a petrographic analysis include: concrete condition; mix design; admixture type; w/c ratio of concrete; and air content estimate. A petrographic analysis can also be used to identify the causes of distress or deterioration of concrete structures related to chloride ion infiltration and to perform an air void analysis as described in ASTM C 457 “Standard Test Method for Microscopical Determination of Parameters of the Air-Void System in Hardened Concrete.”

3.2.8 Acoustic Emission Monitoring of Concrete

Acoustic Emission (AE) is defined as “a transient elastic wave generated by the rapid release of energy from a localized source or source within a material” (ASTM E 1316). AE can detect the onset of corrosion and also distinguish between levels of corrosion. AE is also non-intrusive and can enable global assessment of reinforced concrete structures (ElBatanouny et al, 2014).

The method is based on the transient elastic wave that results due to variations in the internal structure of the material; more succinctly, energy is released, deformations alter the wave as it propagates through the solid, and the resulting wave is measured. The size of the local deformation process affects the acoustic energy released. The process is described in Figure 3.6 and 3.7:

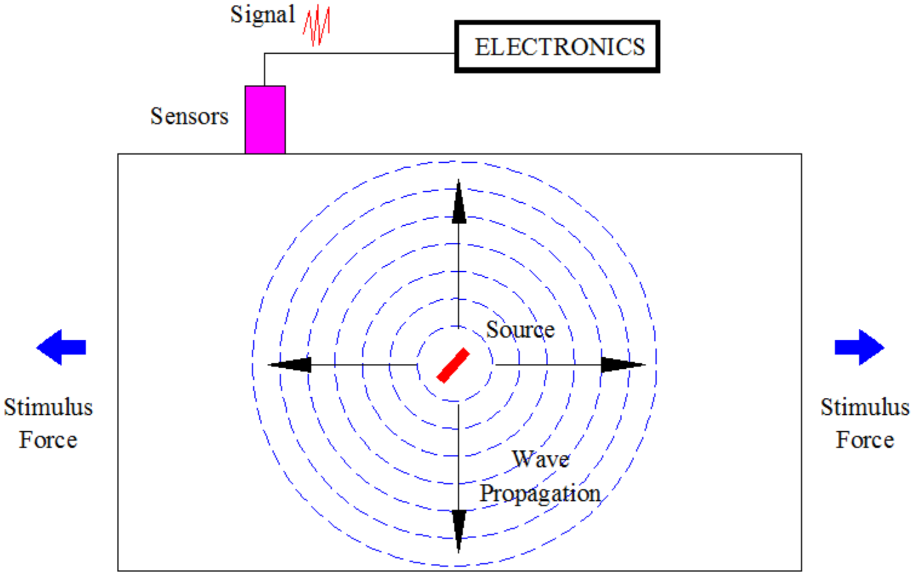


Figure 3.6 Principle of Acoustic Emission

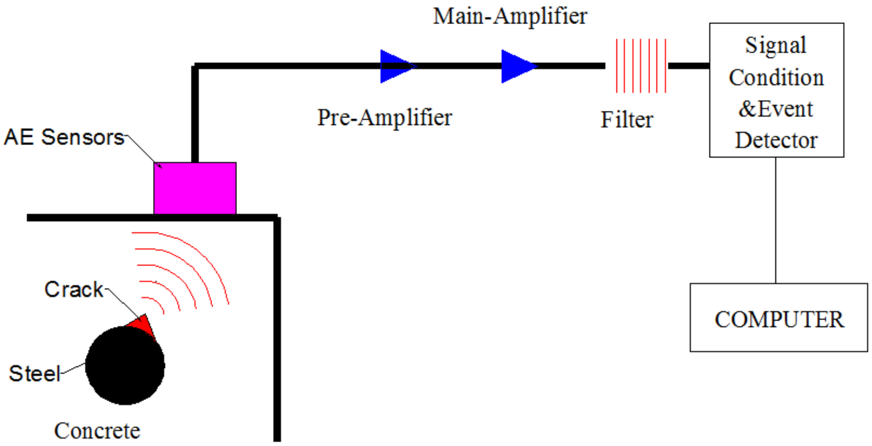


Figure 3.7 AE Measurement

- The knowledge of certain basic AE parameters is important to understand an acoustic signal. Visualized in Figure 3.8, the parameters include Maximum amplitude: is the maximum absolute value in the duration of burst signal
- Rise time: is the time period that is from the first threshold crossing to the maximum amplitude of the burst signal.
- Duration: the time period from the first to the last time the detection threshold was exceeded by a burst signal.

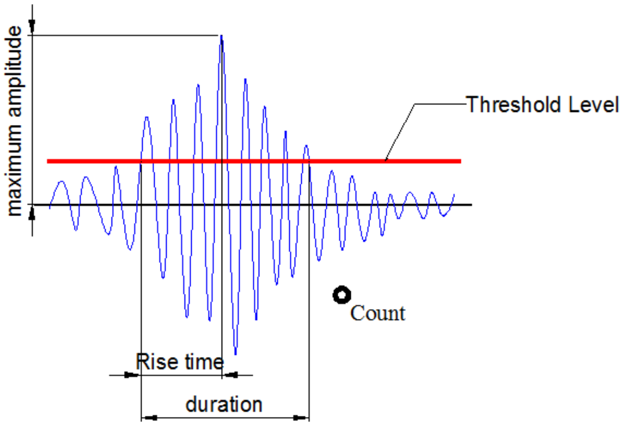


Figure 3.8: AE Signal Parameters

There are two primary types of Acoustic Emission signal: transient signals (bursts) and continuous signals. The transient signal that is associated with crack propagation is defined as having start and end points that deviate clearly from background acoustic, shown as in Figure 3.9. Continuous signal, shown in Figure 3.10, represents a typical continuous signal pattern and is used as the AE response for movements or dislocations (Vallen, 2002).

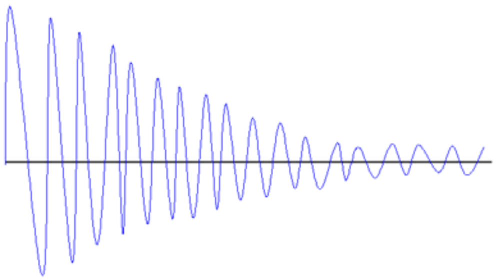


Figure 3.9: Transient Signal

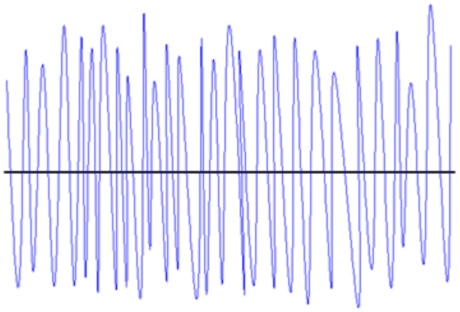


Figure 3.10: Continuous Signal

Sensors are very important in an AE test process because they are systems used to measure or respond to structural performance. The most common sensors used for concrete bridge testing are strain gauges, linear displacement transducers, load cells, tilt meters, and accelerometers. Their primary function is to correlate the AE data to the loading demand on the structure (Lovejoy, 2008). Lovejoy also recommended three ranges of crack width levels as below:

> "Level 1: Maximum previous loading / Ultimate capacity < 0.5
>
> Qualitative interpretation: Light loading with no serviceability concerns
>
> Level 2: 0.5< Maximum previous loading / Ultimate capacity < 0.7
>
> Qualitative interpretation: Moderate loading that may justify continued surveillance or refined load capacity calculations when operating at the higher end of the range.
>
> Level 3: Maximum previous loading / Ultimate capacity > 0.7
>
> Qualitative interpretation: Heavy loading is implied and will require refined load capacity calculations and possibly a full time structural health monitoring system, if the loading is not reduced or the capacity increased." (Lovejoy, 2008)

The AE response to damage levels is based on damage parameters such as the felicity ratio, calm ratio, severity, and historic index. These parameters are described below.

The felicity ratio is the ratio of load at the start of AE activity in the current load cycle and the maximum previous load. Lovejoy based the felicity ration on 42 full-sized laboratory beam tests and reached quantified three levels:

Level 1: Felicity ratios greater than 0.9
Level 2: Felicity ratios between 0.9 and 0.65
Level 3: Felicity ratios less than 0.65

However, Schumacher (2008) indicated that the felicity ratio is only applied to unloaded test beams. In addition, Lovejoy (2008) concluded that the use of the felicity ratio is not significantly popular for tests of in-service structures.

The calm ratio is defined as the ratio of total unloading AE hits and total loading AE hits. Lovejoy (2008) indicated the accumulated damage, which corresponds to the levels below:

Level 1: Calm ratios less than 0.4
Level 2: Calm ratios between 0.4 and 0.65
Level 3: Calm ratios greater than 0.65

The historic index and the severity parameters are used for intensity analysis. Intensity is used to determine the measurement of the structural significance of an AE source. The historic index, H (t), is determined by the following equation:

$$\mathrm{H}\,(\mathrm{t}) = \frac{\mathrm{N}}{\mathrm{N}-\mathrm{K}}\frac{\sum_{t=K+1}^{N} S_{0i}}{\sum_{i=1}^{N} S_{0i}}$$

where:

N is the total number of AE hits up to and including time t
S_{0i} is the signal strength of the i^{th} hit
K= empirical parameter

Lovejoy (2008) indicated that the historic index does not apply for N<200 hits. Values of K and N are determined as below:

K= 0.8N for 200 < N < 1000
K= N-200 for N> 1000

The severity index, S_r, is the average of the 50 largest peak amplitude hits striking a particular sensor, and is shown in the equation:

$$S_r = \frac{1}{J}\sum_{i=1}^{J} S_{0i}$$

where:

S_{0i}= Signal strength of i^{th} hit
J = empirical parameter ranging from 10 to 50.

Additionally, Lovejoy (2008) indicated that severity does not apply until N > 50 and concluded the following:

- At low damage locations, both H and S_r show relatively low values.
- When the diagonal tension cracks form and begin the transition from loading level 1 to level 2, both H and S_r increase in magnitude.
- When accumulated damage corresponding to levels 2 and 3, both H and S_r values drop in magnitude and then stabilize during shear crack extension and coalescing.

- When the structures are loaded at ultimate capacity, both H and S_r increase quickly.

3.3 Inspection and Evaluation Methods for Steel Reinforcement in Concrete

Corrosion mapping is very important to determine the corrosion of steel reinforcement corrosion concrete before damage manifests as cracks or surface deterioration. From these results, steel-reinforced concrete structures may be obtained to the solutions of the severity of the corrosion in concrete element.

As discussed in 3.2.4, half-cell potential measurements can provide probability estimates of corrosion.

3.3.1 Macrocell Corrosion Monitoring

A macrocell is created due to anodic and cathodic processes that take place on separate areas of the surface of the reinforcement. This macrocell can exist between active and passive areas of the reinforcement. Current circulating between the former increases as the difference in the free corrosion potential between passive and active rebars increases, and reduces as the dissipation produced by the current itself at the anodic and cathodic sites and within the concrete increases (Bertolini, 2004). The current from the anodic area flowing towards the cathodic areas creates a motion of Fe2+ ions out of the corroding site. Bertolini (2004) concluded that corrosion produced due to macrocells may have less expansive effect than corrosion produced due to microcells. As mentioned previously, anodic and cathodic areas which exist with the presence of water and oxygen is microcell corrosion. In cases of corroding steel, electrons surround passive steel. As a result, the anodic process on the corroding steel and the cathodic process on the passive steel cause a macrocell process. The ratio between anodic and cathodic sites strongly affects the increase in the corrosion rate on the active steel induced by macrocell action. Bertolini also stated that macrocell corrosion causes more than 90 % of the metal loss.

It may be assumed that there is only a passive rebar and an active rebar inside the concrete. As a result, a driving voltage for the macrocell is indicated as the following equation:

$$\Delta E = \mathrm{E}_{\mathrm{p.cor}} - \mathrm{E}_{\mathrm{a.cor}}$$

where

$E_{p,cor}$ = Voltage at passive rebar

$E_{a,cor}$ = Voltage at active rebar

Bertolini (2004) concluded that the value of voltage at the passive rebar may equal the value of voltage at the active rebar in the absence of oxygen A macrocell current (I) is formed if there is a connection between the two rebars. Bertolini described the relationship of the driving voltage ΔE, anodic and cathodic polarization, and the Ohm drop as the following:

$$\Delta E = \psi_{\mathrm{a}} + \psi_{\mathrm{c}} + \psi_{\mathrm{ohm}}$$

where:

ψ_a is the increase in potential on the active rebar, calculated by the equation:

$$\psi_a = E_a - E_{a,cor}$$

Ψ_b is the decrease on the passive rebar, calculated by the equation:

$$\psi_c = E_{p,cor} - E_c$$

Ψ_{ohm} is ohmic drop due to current circulating through the concrete between the anodic and cathodic areas, calculated by the equation:

$$E_c - E_a = \psi_{ohm}$$

In addition, Gastaldi et al (2002) discovered the equation for calculating driving voltage ΔE as below:

$$\Delta E = \psi_a(i) + \psi_c(i\,.\,r) + R\,.\,i\,.\,A_a$$

Where:

i is the current density at the anodic surface

r is the ratio between the anodic and cathodic areas

$$x = \frac{A_a}{A_c}$$

R is the resistance of the concrete between the cathodic and the anodic areas

3.3.2 CMS V2000 Silver-Silver Chloride Electrode Monitoring

In the CMS V2000 silver-silver chloride electrode (V2000 electrode), the relationship between the silver electrode and the anode steel reinforcement is measured by Faraday's laws. Since the flow of free electrons causes the electrochemical process of corrosion, the independent current is as a function of the dissimilarity of the metals (such as the amount of moisture and chlorides present, etc). The V2000 electrode generates a second and independent current as a function of the dissimilarity of the metals, the amount of moisture and chlorides present, etc. The corrosion risk and corrosion rate is determined based on both the induced voltage and current. Wipf et. al. (2006) claimed that the V2000 electrode is a permanent embedded sensor for the long-term monitoring steel reinforcement corrosion in the concrete structure.

The Vetek V2000^{TM} Corrosion Monitoring System is a commercially available system which requires the installation of the reference electrode during the manufacture of encapsulated anchors. This system determines the actual electrochemical condition of the steel elements embedded in concrete. According to the manufacturer, the V2000 monitoring system consists of a patented CMS electrode of silver wire inside a plastic braid. The braid prevents the contact of the wire and the steel anchor. When the anchors are being produced, the braided electrode is wrapped around the steel of the anchor, which is then placed into sheathing to complete the corrosion-protected anchor. The top end of the electrode is extended past the top of the anchor in order to be electrically connected to the test box on-site.

3.3.3 Corrosion Rate Measurement

Measuring the corrosion rate of reinforcement in concrete is the newest technique of all the examined methods. There are many ways to measure the rate of the corrosion, such as using AC impedance and Electrochemical methods. These methods are not convenient to apply for the reinforcement in concrete (Broomfield, 2007). However, the polarization resistance technique has become more popular in recent years. In this technique, the corrosion rate is measured as value of the corrosion current per unit area of reinforcement. The corrosion rate on the surface of the reinforcement is based on the magnitude of the corrosion current measured. Areas with high corrosion rates correspond to high magnitudes of currents.

As discussed previously, corrosion rate measurement determines the amount of steel dissolving and the amount of oxide (rust) forming.. In other words, this measurement is a measurement of the electric current generated (Broomfield, 2007) by the anodic reaction:

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

By the cathodic reaction:

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

Using the Faraday's law to convert the current flow to metal loss as the following equation:

$$m = \frac{Mit}{zF}$$

where:

m is the mass of steel consumed,
i is the current (amperes),
t is time in seconds,
z is the ionic charge (2 for Fe),
and M is the atomic mass of metal.

The linear polarization technique is use to polarize the steel with an electric current and monitor its effect on the reference electrode potential (Broomfield, 2007). The corrosion current is measured by basing on the change in the reference electrode potential related to the corrosion current by the equation (Stern and Geary, 1957):

$$I_{corr} = \frac{B}{R_p}$$

where:

B is a constant from 26 to 52 mV, depending upon the passivity or active condition of the steel surface and
R_p is the polarization resistance (in ohms)

$$R_p = \frac{\text{change in potential}}{\text{applied current}}$$

In addition, Broomfield (2007) determined the corrosion rate in μm/ year by the following equation:

$$x = \frac{11 * 10^6 * B}{R_p * A}$$

where:

A is the surface area of steel measured in cm^2

B is a constant from 26 to 52 mV depending upon the passivity or active condition of the steel surface,

R_p is the polarization resistance (in ohms).

The GECOR 6 is a meter that is used to measure the corrosion rate of steel in concrete by the “polarization resistance” or “linear polarization” technique, shown as Figure 3.11. GECOR 6 was developed by GEOCISA in collaboration with two leading Spanish Research Centres (Inst. Torroja and CENIM) under EUREKA/EUROCARE project EU-401 with the involvement of the Swedish Cement and Concrete Research Institute (CBI).GECOR 6 measures the concrete resistivity by the equation:

$$\text{Resistivity} = 2 * R * D$$

where:

R - the resistance by the "Ir drop" from a pulse between the sensor counter-electrode and the rebar network.

D - the counter- electrode diameter of the sensor.

The sensor is mounted on the concrete surface to routinely measure the corrosion current I_{corr} and the electric resistance between the reference electrode and the rebar.

Figure 3.11 Gecor 6 Device with Sensor

However, the corrosion rate measurement method has some limitations. The corrosion rate measured is only valid under particular conditions. Dynamic environmental factors such as temperature and moisture significantly influence the measured corrosion rate values. Therefore, the most active corrosion or highest corrosion rates do normally not occur at the time of the field study, and continuous monitoring over a period of time provides a more accurate appraisal of the corrosion rate (Sohanghpurwala, 2006). Finally, tests of corrosion rate measurement are not valid for coated steel such as epoxy-coated or galvanized reinforcement.

3.3.4 Radiography Method

Radiography is a method that uses high frequency electromagnetic radiation such as X-rays and γ-rays to generate images of the steel reinforcement in concrete structures. The principle of radiography is the measurement of attenuation of radiation as it passes through a material. As radiation is transmitted through concrete, it interacts with the atoms in the material. Defects are determined by basing on changes in intensity in a radiograph. These changes of intensity can be used to quickly generate images, which allow preliminary interpretation on site (Broomfield, 2007). Locations of internal cracks, voids and variation in the density of concrete are easily determined by the radiography technique.

However, Broomfield (2007) noted several limitations of radiography:

- “Access is required to both sides of structures
- The site needs to be screened and secure to provide a safe environment
- Progress rates tend to be slow so time limitations and coverage needs to be considered.
- Multiple layers of reinforcement will be superimposed and may limit interpretation
- Although interpretation can be straight forward the specialized equipment and safety considerations require expert operation” (Broomfield, 2007)

3.3.5 Ultrasonic Pulse Velocity Method

In the ultrasonic pulse velocity method, a high-frequency (greater than 20 kHz) sound wave is emitted by one of two transducers (the other transducer is a receiver (Sohanghpurwala, 2006). The travel time for the pulse between the two transducers is then determined. The travel time can be converted into velocity and, compared with typical values (sound travels at about 12,100 feet per second in concrete) if the thickness of the material is known (Sohanghpurwala, 2006). However, the analysis of direct travel time is normally used to determine the elastic modulus and/or the compressive strength of the material. It may also be used to identify defects such as cracks and voids (Broomfield, 2007).

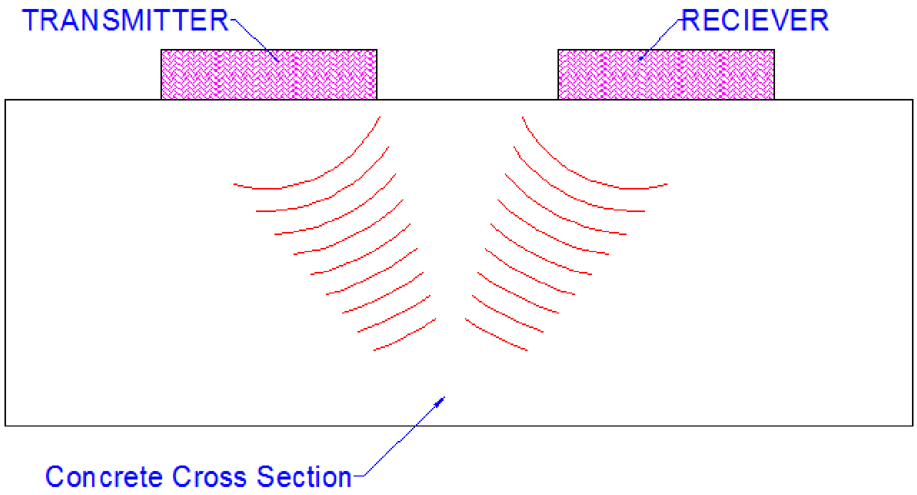


Figure 3.12: Measure of Time of Arrival and Back as Transmitter and Receiver

In the Ultrasonic Method, longitudinal pulse velocity is given by:

$$v = \frac{L}{T}$$

where:

v is the longitudinal pulse velocity,
L is the path length,
T is the time taken by the pulse to traverse that length.

Normally there are two types of simulated cracks: perpendicular (vertical crack) as Figure 3.13 and parallel (horizontal crack) as Figure 3.14 Malhotra (2004) estimated the depth of an air-filled crack using the pulse velocity method. In the case of vertical cracking, the depth of crack is determined by the following equation:

$$\mathrm{h} = \frac{\mathrm{x}}{\mathrm{T_2}} (\mathrm{T_1^2} - \mathrm{T_2^2})^2$$

where:

x = distance of transducer from the crack (note that both transducers must be placed equidistant from the crack);
T1 = transmit time around the crack;
T2 = transmit time along the surface of the same type of concrete without any crack

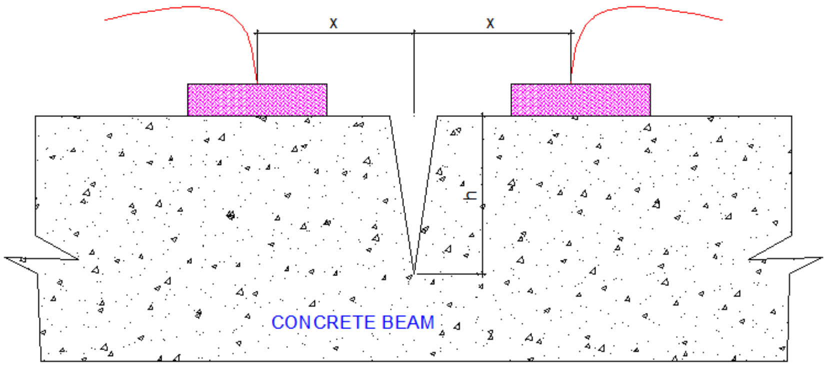


Figure 3.13 Experimental Set up for Vertical Crack Depth Estimation

In the case of horizontal cracking, the depth of the crack can be calculated by the equation:

$$y = 0.5\sqrt{(V^2T^2 - x^2)}$$

Where:

y = depth of the horizontal crack
V = velocity of the compression wave through the concrete
x = distance between the two transducers
T = transit time.

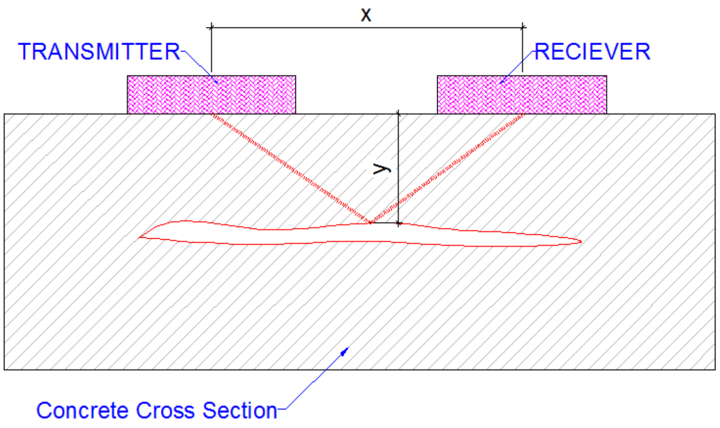


Figure 3.14: Experimental Set up for Horizontal Crack Depth Estimation

However, there are many factors that affect the accuracy of the ultrasonic pulse velocity method. Proper contact between the transducers and the surface of the concrete structure is critical (Sohanghpurwala, 2006). The surface of the structure must be smooth in order to get sufficient energy into the concrete (Broomfield, 2009). Traffic movement, temperature, and moisture content also affect the travel time of the ultrasonic pulse. Sohanghpurwala (2006) indicated that the minimum spacing between the transducers (i.e., the thickness of material being tested) should be no less than about 4 to 6 inches, depending on the velocity of the ultrasonic pulse in the material and the frequency of vibration of the piezoelectric material in the transducer.

3.3.6 Remote Sensing of Rebar Corrosion

Numerous remote sensing technologies exist for monitoring steel reinforcement corrosion in concrete. However, this paper only considers the most common types of sensors for detection corrosion.

The SensCore system is a wireless sensor network that is used to detect and predict the condition of steel reinforcement corrosion in a concrete structure. The system consists of sensors, data-loggers a measurement hub, and software for analyzing the data. Data can be transferred from several data-loggers to central database where it can be accessed by the authorized users (Inaudi, 2009). According to the manufacturer, “the SensCore Corrosion sensor is composed by 4 mild steel rebars that are anchored to a stainless steel support. The 4 dummy rebars are placed in the concrete at 4 different depths. The SensCore datalogger measures the corrosion current for each rebar separately. A zero current indicates that the corrosion is inhibited at that depth. A non-zero current indicates that the conditions for rebar corrosion are present at that depth, while the corrosion current gives an indication of the corrosion rate” (Available on link: http://www.smartec.ch/PDF/SDS%2016.1010%20SensCore%20Corrosion%20Current%20Sensor.pdf)

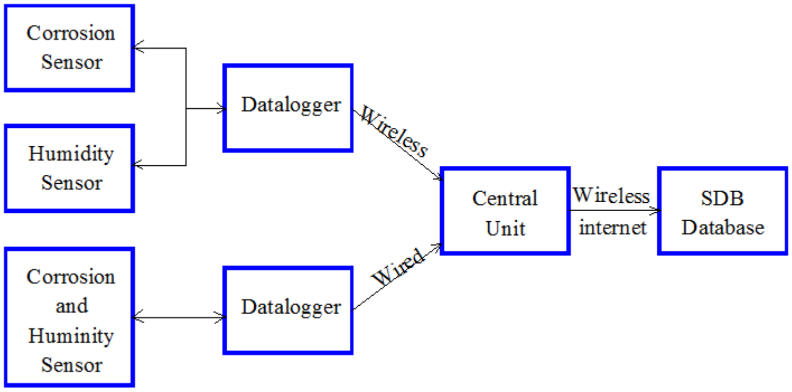


Figure 3.15: SensCore System Overview

The SensCore Corrosion Current Sensor measures two critical parameters: corrosion initiation and corrosion rate (Inaudi, 2009). This method allows the evaluation of the corrosion front progression. Inaudi also wrote that the corrosion of steel rebars can be determined quickly because there are four different depths in the two measurements. In fact, the conjunction of the SensCore Humidity Sensor can provide a complete picture of the corrosion initiation and progression in a reinforced concrete structure..

The SensCore Humidity Sensor is used to evaluate water content in concrete, as well as the ability of water penetration into concrete. This sensor collects electrical resistance and temperature measurements at different depths, typically between the concrete surface and the reinforcement bars’ depth (Inaudi, 2009). Additionally, the sensor indicates a complete picture of the corrosion initiation and progression in a reinforced concrete structure.

Inaudi (2009) also claimed that the SensCore datalogger determines the concrete resistivity across the depth determines the resistivity between pairs of bars by the measurements of the concrete resistivity across the depth.

However, the full evaluation of the corrosion state of reinforced concrete structure requires many parameters (such as corrosion initiation and rate, humidity content and temperature). Therefore, the combined humidity and corrosion sensor should be used for existing structures.

CHAPTER 4 - Service Life Modeling

4.1 Corrosion Initiation

As discussed previously, there are two main causes of steel corrosion in concrete: chloride-induced corrosion and carbonation-induced corrosion. In reinforced concrete structures, chloride-induced corrosion is primarily considered because of the high diffusion of chlorides into concrete. Therefore, carbonation-induced corrosion will not be considered when evaluating methods of service life modeling. This chapter will report applications of some models to determine potential repair and rehabilitation alternatives for bridge superstructure elements.

The service life of a corroded bridge beam and girder is understood as three distinct time periods shown in Figure 4.1:

Stage 1: Time to corrosion initiation,
Stage 2: Time from initiation to cracking, and
Stage 3: Time for corrosion damage to propagate to a limit state.

Fickian diffusion models require parameters of chloride ion content, rate of transport from surface to the reinforcement, concrete clear cover, and chloride concentration thresholds.

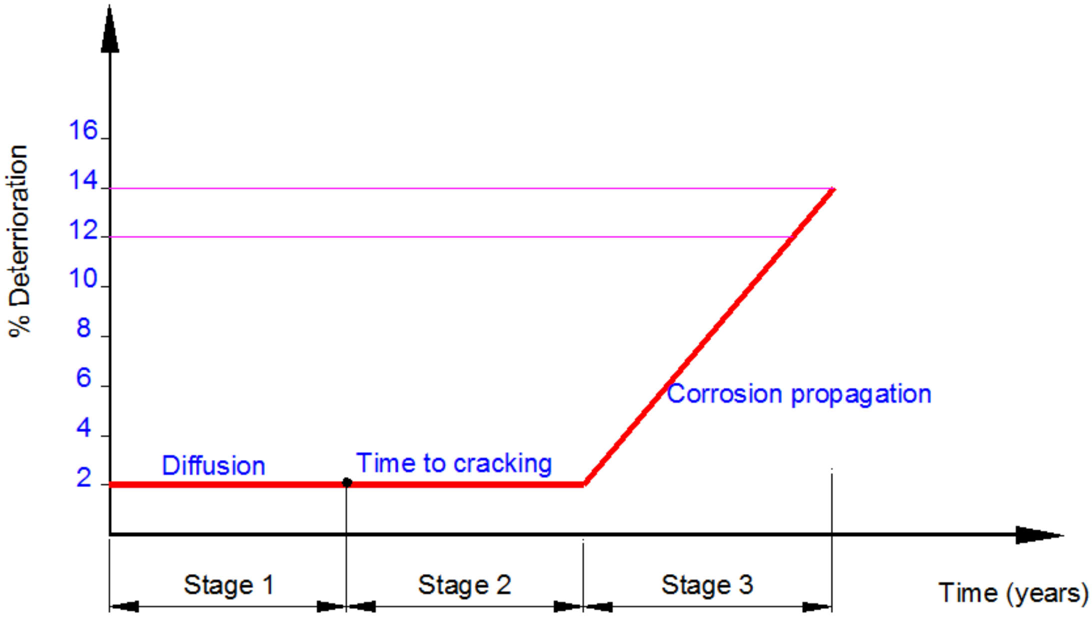


Figure 4.1: Service Life Model

The initiation period, *ti*, is the time required for sufficient chlorides to penetrate the concrete cover and accumulate in sufficient quantity at the depth of the embedded steel to initiate corrosion of the steel. Vassie (1987) took data from the series of U.K. highway bridges and described the relationship of corrosion initiation (% probability) vs. chloride concentration From Figure 4.2shows clearly that corrosion initates when chloride levels reach between 0.1% to 1.8% (by mass of cement). However, in recent studies, the chloride initiation concentration was found to be 2% (Williamson, 2007).

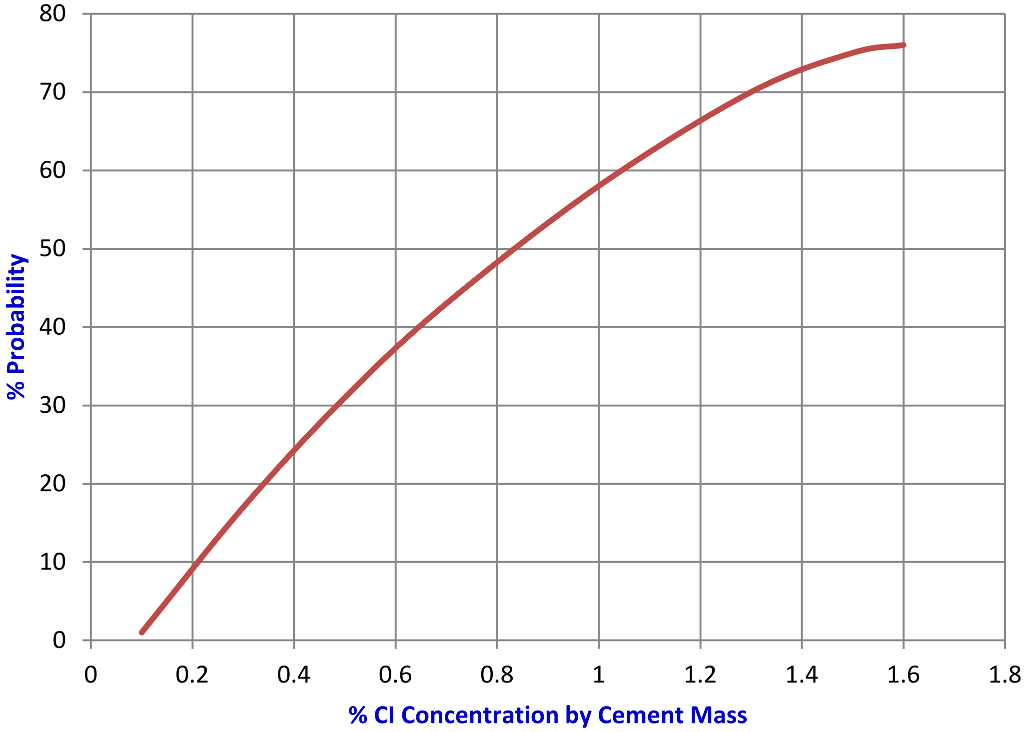


Figure 4.2 Corrosion Initiation (% probability) Vs. Chloride Concentration

Table 4.1 (adapted from Brown, 2002) indicates the average time to initiation and cracking for bare steel and epoxy-coated steel. The average time to initiation of bare steel is 0.2 years compared with 0.62 years for epoxy-coated steel. Also, the time to cracking of bare steel is 1.15 years and 1.62 years of epoxy-coated steel.

Table 4.1: Average Times to Initiation and Cracking for Bare Steel and ECR

	Time to Initiation (years)	Time to cracking (years)
Bare steel	0.20	1.15
Epoxy coating Steel	0.62	1.62

In addition, the cumulative probability of initiation of bare steel significantly increases in the 0.5 initiated years and may reach 90%. This cumulative probability of initiation is about 20% in 0.5

years and 50% in 1.3 years in the case of epoxy coated reinforcement. Figure 4.3 and Figure 4.4 show the relationship between the cumulative probability of initiation and time to corrosion initiation of epoxy-coating steel and bare steel, respectively.

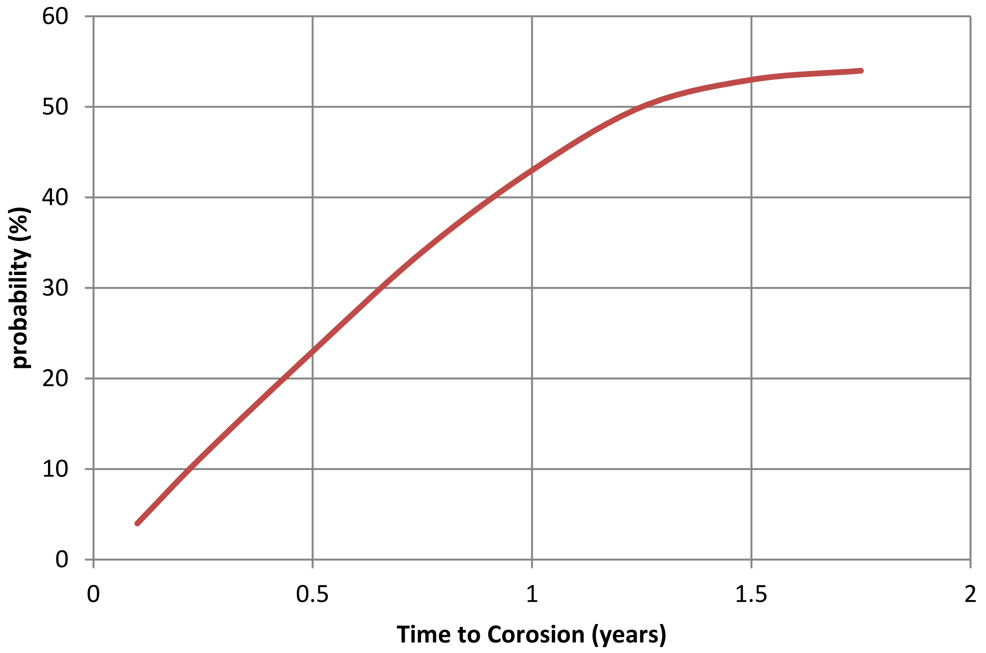


Figure 4.3 Probability Vs. Time to Corrosion Initiation of Epoxy Coating Steel

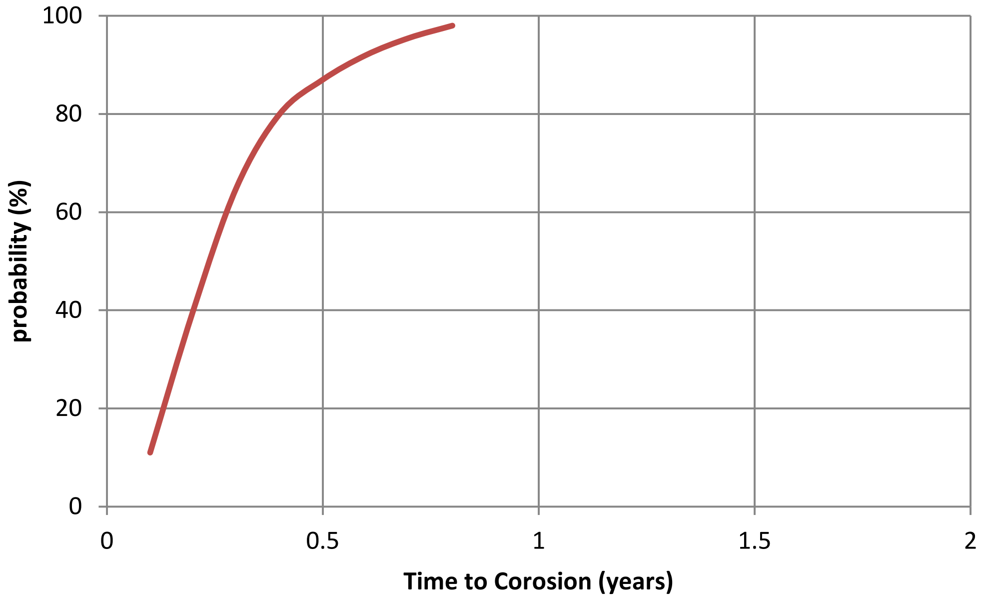


Figure 4.4 Probability Vs. Time to Corrosion Initiation of Bare Steel

In comparing of time to cracking between bare steel and epoxy coated steel, most studies show that the cumulative probability of bare steel cracking is 50% at the period of one year. Also, at this time, cracking has not yet occurred in epoxy-coated steel. At the period of 1.7 years, the cumulative probability of cracking in bare steel may nearly reach more than 90 percent compared with about 12 percent of the epoxy-coated steel. Figures 4.5 and 4.6 visualize the comparison of time to the cracking of bare steel versus epoxy-coated steel.

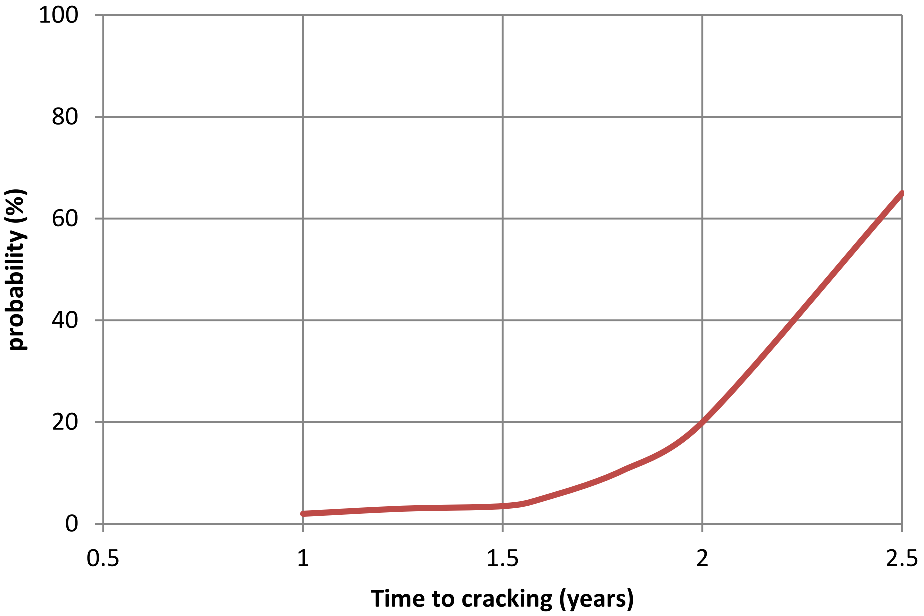


Figure 4.5: Probability Vs. Time Cracking of Epoxy Coating Steel

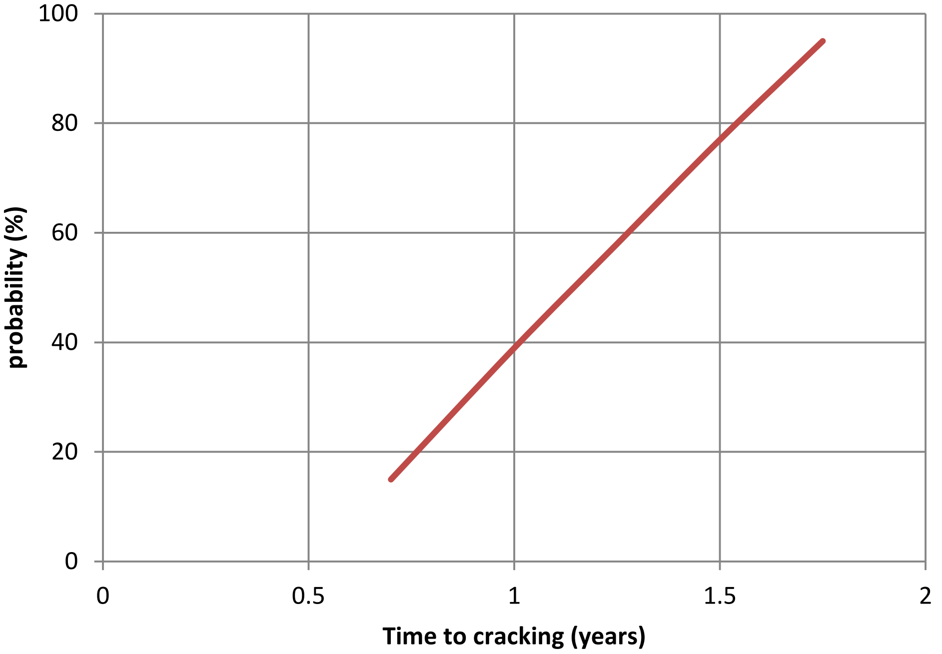


Figure 4.6 Probability Vs. Time to Cracking of Bare Steel

4.2 Reinforcement Degradation Estimations

4.2.1 Corrosion Propagation Time

The total corrosion propagation time is the time it takes for corrosion deterioration to advance from a level of 2% to a level of 12% (Williamson, 2007). Modeling the propagation of corrosion after initiation is somewhat difficult. As mentioned previously, once corrosion has begun, the corrosion will not follow the Fickian chloride diffusion model, as corrosion is a complex process of electrochemical reactions and ion transport mechanisms.

In addition, because of the difficulties in converting section loss to cracking rate, distribution of loading (especially live loading) to cracking, estimating the time to cracking, delamination, or spalling presents a series of significant problems (Williamson, 2007)

In modeling the time-to-corrosion cracking of the cover concrete in chloride-contaminated reinforced concrete structures, Liu (1996) calculated the critical amount of rust required to produce a crack based on the mechanics of the generation of rust and the resulting stress in the concrete, which is:

$$W_{crit} = \rho_{rust}\left[\pi\left(\frac{Cf_i}{E_{ef}}\left(\frac{a^2 + b^2}{b^2 - a^2} + v_c\right) + d_0\right)D + \frac{W_{st}}{\rho_{st}}\right]$$

where:

W_{crit} = the limiting amount of the corrosion products needed to induce cracking of the cover concrete

r_{rust} = density of rust
C = clear concrete cover
f_i = tensile strength of concrete
E_{ef} = effective elastic modulus of concrete
a = inner radius of the thick-walled cylinder (clear concrete cover – d_0)
b = outer radius of the thick-walled cylinder (clear concrete cover + $D/2$)
v_c= Poisson's ratio for concrete
d_0 = thickness of the porous zone around the steel-concrete interface
D = diameter of the rebar
W_{st} = mass of steel corroded
r_{st} =density of steel

It is clear that W_{crit} is mainly dependent on the quality of the concrete and the cover depth. High strength concrete with a thick cover depth is related to a higher value of W_{crit}, while low strength concrete with a thin cover depth has a lower value of W_{crit}.

By the above analysis, Liu (1996) calculates the time to cracking with the equation:

$$t_{cr} = \frac{W_{crit}^2}{2\,k_p}$$

where:

W_{crit} = the limiting amount of the corrosion products needed to induce cracking of the cover concrete.
k_p = the rate of rust production. k_p is related to the rate of metal loss, which may be expressed in terms of the corrosion rate.

$$k_p = 2.59 * 10^{-6} * \frac{1}{a} * \pi * D * i_{cor}$$

in which:

a is related to types of rust products,
D = the steel diameter (in.)
i_{cor} =the annual mean corrosion rate (µA/ft^2)

In the cases of the growth of rust products, the amount of rust products for a certain period of corrosion can be estimated using the following equation:

$$W_{rust}^2 = 2\int_0^t k_p\,dt$$

In conclusion, this model indicates that relating physical properties of concrete (elastic modulus and Poisson's ratio), clear concrete covers, rebar diameters, and corrosion rates affect the time to cracking. However, corrosion rate should be measured several times in a year and the average of the measured corrosion rates used (Andrade, 1982).

4.2.2 Rebar Corrosion Rate Measurement Model

This model uses the measurements of corrosion rate to estimate the remaining service life of reinforced concrete in which corrosion is the limiting degradation process (Klinghoffer, 2000). Andrate (1990) and Clear (1989) developed this model by using the polarization resistance technique to measure corrosion currents.

The Andrade model is considered when determining reductions in the diameter of reinforcing steel, as it relates the corrosion rate to the reduction in reinforcement diameter:

$$d(t) = d(0) – 0.023 * I_{corr} * t$$

where:

$d(t)$ - the reinforcement diameter in (mm) at time t
t - time after the beginning of propagation period (years)
$d(0)$ - the initial diameter of the reinforcement in (mm)
I_{corr} - the corrosion rate in ($\mu A/cm^2$)

Clear (1989) suggested the use of the following relationships between the corrosion rates and remaining service life as in Table 4.2

Table 4.2 Corrosion Rates and Remaining Service Life

Corrosion rate I_{corr}, ($\mu A/cm^2$)	**Remaining service life**
$I_{corr} <0.5$	no corrosion damage
$0.5 < I_{corr} <2.7$	corrosion damage in the range of 10 to 15 years
$2.7 < I_{corr} <27$	corrosion damage in the range of 2 to 10 years
$I_{corr} > 27$	corrosion damage less than 2 years

As mentioned previously, the measured corrosion rate varies based on environmental conditions such as average temperature and humidity. Therefore, the change in the corrosion rate with respect to time is linear in both the Andrade and Clear models.

To overcome this limitation, E. Poulsen (2000) suggested using the equation of the loss of reinforcement diameter as the following:

$$D = 2 * R(t) * t$$

Where:

D = loss of diameter of reinforcement
R(t) = corrosion rate at time t

4.3 Diffusion Models for Predicting Chloride Ingress

4.3.1 Fickian Diffusion Model

This model predicts the initiation period, assuming diffusion to be the dominant mechanism. Fick's second law is the governing differential equation:

$$\frac{\partial C}{\partial t} = D\left(\frac{\partial^2 C}{\partial x^2}\right)$$

Where:

C = chloride ion concentration (kg/mm^3)
t = time (years)
D = diffusion coefficient
x = depth from the surface of concrete

However, the total chloride contents are measured by the error function solution:

$$C_{(x,t)} = C_o\left(1 - erf\,\frac{x}{2\sqrt{D_c.t}}\right)$$

where:

C(x,t) = chloride concentration at depth *x (mm)* at time *t (years)*
C_0 = surface concentration (% by mass of cement or concrete at the time t (s) and the depth x from the surface of concrete)
D_c= diffusion coefficient for chloride (m^2/s). This value depends on the factors of exposure condition and the time of exposure.
t = time for diffusion (years),
x = depth (mm)
erf = statistical error function, shown in the appendix.

From the equation above, it is clear that the parameters of the surface chloride concentration and concrete diffusion coefficient need to be known prior to estimating the chloride concentration at a specified depth and time given.

It may be said that the diffusion coefficient for chloride (D_c) is used to estimate the service life of a time-dependent parameter because the rate of chloride penetration decreases with time (Bamforth, 1999). Also, the rate of reduction depends on the composition of the concrete and environmental conditions such as temperature and moisture. Williamson (2007) explained that the reduction of D_c is due to pozzolanic reactions within the concrete matrix and the presence of insoluble salts at the surface of the concrete. The water/cement ratio of the concrete, amount and type of pozzolan, and the rate of hydration significantly affect the reduction rate. Williamson also concluded that initiation of the diffusion coefficient for chloride (D_c) for a concrete mixture is relatively high. After that, there is a quick reduction from 5 to 10 years to reach a near steady condition.

While the diffusion coefficient for chloride (D_c) will decrease with time, the chloride concentration at the surface (C_o) will increase over time. This reduction was checked on 15 bridge decks in the snow-belt regions over a 15-year period by Weyers (1994).

4.3.2 Mangat and Molloy Diffusion Model

As discussed previously, the diffusion coefficient used to quantify the expected service life extension is a time-dependent parameter. The concentration of chloride ingress into concrete reduces with time. Mangat and Molloy (1994) developed a diffusion model as a time-dependent equation as the following empirical relationship:

$$D_c = D_i \, t^{-m}$$

Where:

D_c = Effective diffusion coefficient at time t (cm^2/s)

D_i = Effective diffusion coefficient at time t = 1.0 sec

t = Time (s)

m = Empirical coefficient that varies with mixture proportions

The chloride contents are measured as the following equation:

$$C_{(x,t)} = C_o \left(1 - \mathrm{erf} \left(\frac{x}{2\sqrt{\frac{D_i}{1-m} \; t^{(1-m)}}} \right) \right)$$

In which: m is the parameter depending on the ratio of water to cement (w/c) as relative below:

$$m = 2.5 \left(\frac{w}{c} \right) - 0.6$$

However, this equation of chloride content is valid if the initial diffusion coefficient and the age at the time of first exposure to chlorides are taken at time t = 1 second (Williamson, 2007).

However, the problem of Mangat and Molloy's model is the requirement of the diffusion coefficient at time equal to 1 second, which will result in the overestimation of chloride concentrations. In fact, Williamson (2007) concluded that measuring the parameter of D_c is not feasible and estimating the value of m is probably inaccurate.

4.3.3 Maheswaran and Sanjayan Diffusion Model

Maheswaran and Sanjayan (2004) developed Mangat and Molloy's solution by using the different times of chloride application and D_c measurement. Chloride content equation is presented as the following:

$$C_{(x,t)} = C_o \left[1 - \mathrm{erf} \left(\frac{x}{2\sqrt{\frac{D_{ref} (t_{ref})^m}{1-m} \; [(t)^{1-m} - (t_i)^{1-m}]}} \right) \right]$$

where:

D_{ref} = Diffusion coefficient measured at time t_{ref}
t_i = Age at first exposure to chloride
m = Empirical coefficient varying with mixture proportions, determined as Mangat and Molloy Model

Although the Maheswaran and Sanjayan model overcomes the limitations of the Mangat and Molloy's model, estimating an appropriate value for m still remains difficult (Williamson, 2007).

4.3.4 Bamforth Diffusion Model

Bamforth (1999) uses the following equation to predict chloride concentration at depth *x* at time *t*.

$$C_{(x,t)} = C_o\left[1 - \mathrm{erf}\left(\frac{x}{2\sqrt{\left[D_{ca(t_m)}\left(\frac{t}{t_m}\right)^n t\right]}}\right)\right]$$

Where: $D_{ca(tm)}$ = Apparent diffusion coefficient measured at time t_m, showed as below:

$$D_{ca} = a\, t^n$$

In which:

D_{ca} = Apparent diffusion coefficient
a = D_{ca} at t = 1 year
n = Empirical constant

4.3.5 Finite Element Model

In recent years, the Finite Element Method (FEM) has become popular in solving application of diffusion problems. FEM is a numerical technique for finding the solution to a set of partial differential equations. Williamson (2007) claimed that there are more details and parameters in the use of FEM to estimate the diffusion of chlorides into concrete than those investigated in Fickian models.

Saetta et al. (1993) applied the Finite Element Method to solve a set of nonlinear equations of the chloride penetrating partially saturated concrete. The effects of many factors in diffusion of chloride (such as a time-dependent chloride diffusion coefficient, degree of cement hydration, temperature, humidity, and moisture flux) are solved in this model.

Boddy et al. (1999) determined diffusion and convection of chloride transport by the following equation:

$$\frac{\partial C}{\partial t} = D.\frac{\partial^2 C}{\partial x^2} - v.\frac{\partial C}{\partial x} + \frac{\rho}{n}.\frac{\partial S}{\partial t}$$

where:

C = free chloride in solution at depth x after time t

D = diffusion coefficient
ρ = concrete density
n = porosity
S = bound chloride
v = average linear velocity

$$v = \frac{Q}{nA} = -\frac{k}{n}\frac{dh}{dx}$$

in which:
Q = flow rate
A = cross-sectional area
k = hydraulic conductivity
h = hydraulic head

The above equation solves the rate of change in chloride concentration, account for the effects of chloride diffusion, convective flow, and chloride binding. As a result, FEM is able to predict the time to diffusion and corrosion. However, the Finite Element Method (FEM) is difficult to handle because of the complexity of the calculations required (Williamson, 2007).

4.3.6 Limitations of Diffusion Models

The Magnat and Molloy model is limited by the fact that it requires input of a diffusion coefficient at t = 1. Additionally, the estimate of D_i requires numerous estimates of D_c over a defined time frame. The assumption that the first exposure to chlorides occurs at t = 1 second will cause one to overestimate the chloride concentrations. Making numerous D_c measurements is not feasible, and estimates of m may not be accurate (Williamson, 2007).

Maheswaran and Sanjayan attempted to solve the problems discovered with Mangat and Molloy's model. They took into account that the first chloride exposure occurs at times other than t =1 second. However, the estimation of an appropriate value for m is still cumbersome.

Bamforth's model makes use of an apparent D_c (D_{ca}) value, which is an approximation of the time-dependent variable D_c. Maheswaran and Sanjayan (2004) demonstrated numerically that the empirical constant n used in Bamforth's model is a function of time and as such cannot be used to accurately estimate a D_{ca} value.

Further, the rate at which D_c reduces is dependent upon concrete mix proportions. The rate is affected by the w/c ratio, pozzolan content, and curing conditions. Due to the high variability in mixture proportioning and curing conditions, estimating the appropriate parameters used in calculating a time-dependent D_c remains a difficult task (Williamson, 2007).

Finally, the Finite Element Model (FEM) is more detailed in its calculations and parameters than the Fickian-based diffusion models presented above. However, FEM is very difficult to use due to the complexity of the calculations required.

4.4 Software-Based Service Life Modeling

The limitations of service life modeling can be somewhat remedied by the use of service life modeling software. Two applications will be considered- the Life-365 Rebar Corrosion Rate Model and the STADIUM Software model.

4.4.1 Life-365 Rebar Corrosion Rate Model

Life-365 is software developed by ACI Committee 365 for possible adoption as an initial "standard" model. Life-365 is used to predict the service life of reinforcement concrete structures by comparing different strategies and techniques for increasing the service life of the structure.

There are four steps in the analysis of carrying out the software of Life-365:

1. Predicting the time to the onset of corrosion, commonly called the initiation period, t_i;
2. Predicting the time for corrosion to reach an unacceptable level, commonly called the propagation period, t_p; (Note that the time to first repair, t_r, is the sum of these two periods: i.e. $t_r = t_i + t_p$)
3. Determining the repair schedule after first repair
4. Estimating life-cycle costs based on the initial concrete (and other protection) costs and future repair costs.

The Life-365 calculates the initiation time based on the basics of the Fickian diffusion model. Life-365 defaults the number of chloride exposure conditions based on the type of structure, the type of exposure and the geographic location. Alternatively, users can select these parameters in input data.

When predicting the Initiation Period, input parameters required by inputting of users include:

- Geographic location;
- Type of structure and nature of exposure;
- Depth of clear concrete cover to the reinforcing steel,
- Details of each protection strategy scenario such as water-cement ratio, type and quantity of mineral admixtures or corrosion inhibitors, type of steel and coatings, presence of membranes or sealers." (Thomas and Bentz, 2000)

As a result, the Life-365 will calculate the time to corrosion by the above equation based on the input parameters. In calculating propagation period, the Life-365 program defaults t_p= 6 years. This is also suitable with a range of propagation period from 3 to 7 years in the USA (Weyers, 1998).

Life-365 shows some graphs that illustrate the performance of the concrete, by time and by the dimensions of the concrete. Figure 4.7shows the service life of each alternative mix design including the component initiation period and propagation period. Figure 4.8 shows a cross-section of the chloride concentration of the concrete mix at the point of initiation.

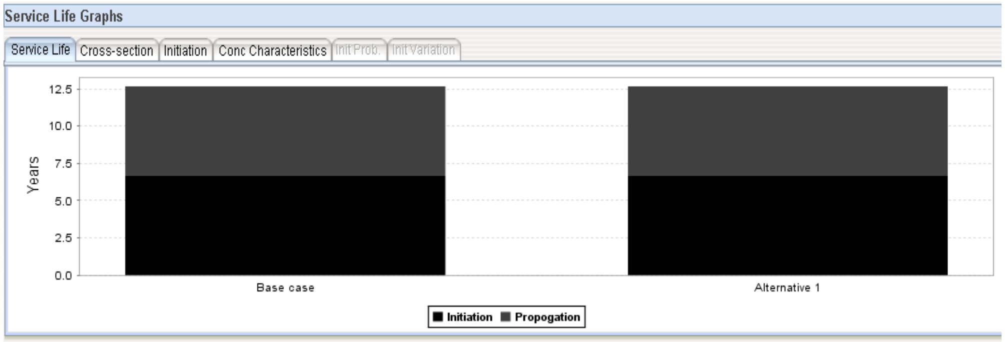


Figure 4.7 Service life

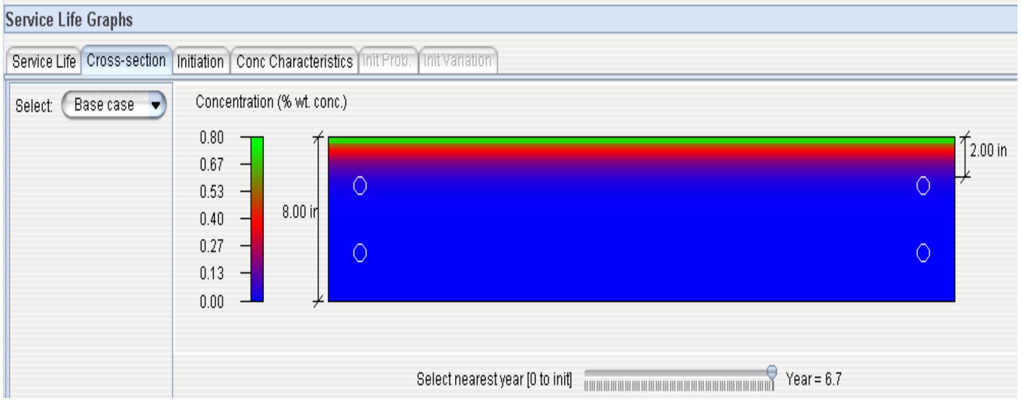


Figure 4.8 Cross-section of the chloride concentration

Life-365 can estimate the time to corrosion initiation. Figure 4.9 shows the relationship between the concentration of chlorides and structure depth. Finally, cost and service lives can be compared among several alternatives. Figure 4.11 shows life-cycle cost comparisons, while Figure 4.12 shows service life estimates for various concrete mix alternatives.

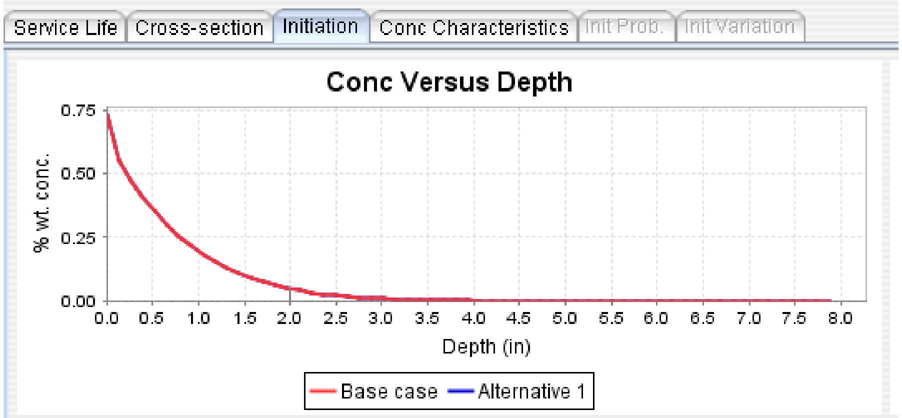


Figure 4.9 Concentration of chlorides Versus Depth of Structure

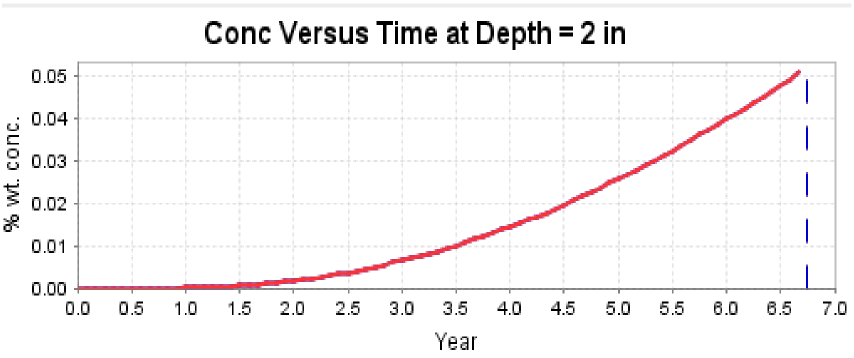
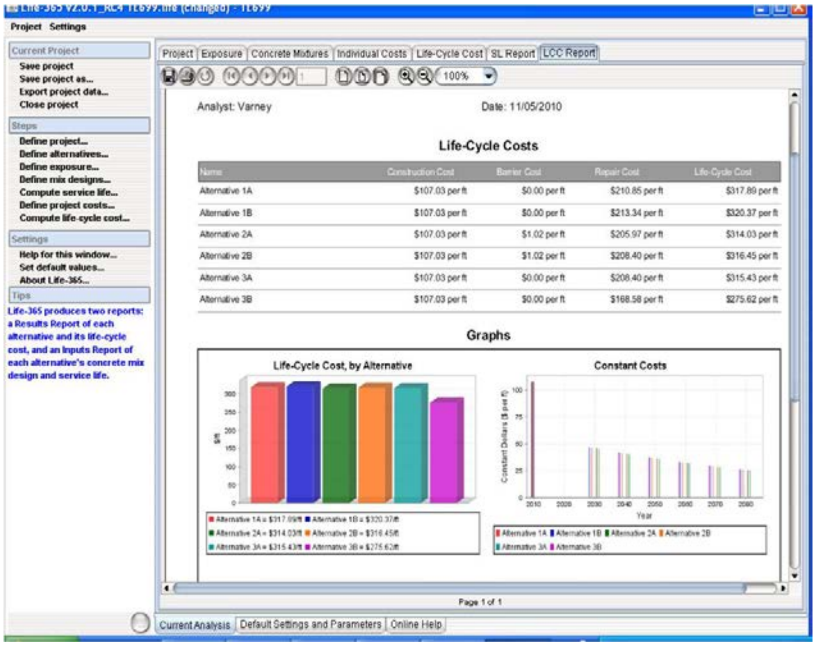


Figure 4.10 Level of chloride Vs the time up



Life-365 produces two reports: a Results Report of each alternative and its life-cycle cost, and an Inputs Report of each alternative's concrete mix design and service life.

Analyst: Varney

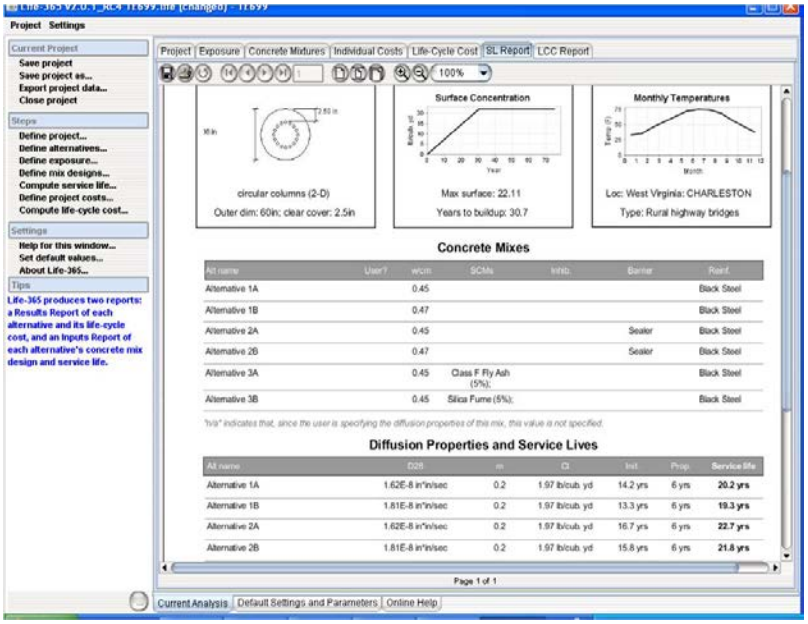
Date: 11/05/2010

Life-Cycle Costs

Name	Construction Cost	Barrier Cost	Repair Cost	Life-Cycle Cost
Alternative 1A	$107.03 per ft	$0.00 per ft	$210.85 per ft	$317.89 per ft
Alternative 1B	$107.03 per ft	$0.00 per ft	$213.34 per ft	$320.37 per ft
Alternative 2A	$107.03 per ft	$1.02 per ft	$205.97 per ft	$314.03 per ft
Alternative 2B	$107.03 per ft	$1.02 per ft	$208.40 per ft	$316.45 per ft
Alternative 3A	$107.03 per ft	$0.00 per ft	$208.40 per ft	$315.43 per ft
Alternative 3B	$107.03 per ft	$0.00 per ft	$168.58 per ft	$275.62 per ft

Graphs

Figure 4.11 Life Cycle Cost Alternatives



Life-365 produces two reports: a Results Report of each alternative and its life-cycle cost, and an Inputs Report of each alternative's concrete mix design and service life.

circular columns (2-D)
Outer dim: 60in; clear cover: 2.5in

Max surface: 22.11
Years to buildup: 30.7

Loc: West Virginia: CHARLESTON
Type: Rural highway bridges

Concrete Mixes

Alt name	User?	w/cm	SCMs	Inhib.	Barrier	Reinf.
Alternative 1A		0.45				Black Steel
Alternative 1B		0.47				Black Steel
Alternative 2A		0.45			Sealer	Black Steel
Alternative 2B		0.47			Sealer	Black Steel
Alternative 3A		0.45	Class F Fly Ash (5%);			Black Steel
Alternative 3B		0.45	Silica Fume (5%);			Black Steel

"n/a" indicates that, since the user is specifying the diffusion properties of this mix, this value is not specified.

Diffusion Properties and Service Lives

Alt name	D28	m	Ct	Init.	Prop.	Service life
Alternative 1A	1.62E-8 in*in/sec	0.2	1.97 lb/cub. yd	14.2 yrs	6 yrs	**20.2 yrs**
Alternative 1B	1.81E-8 in*in/sec	0.2	1.97 lb/cub. yd	13.3 yrs	6 yrs	**19.3 yrs**
Alternative 2A	1.62E-8 in*in/sec	0.2	1.97 lb/cub. yd	16.7 yrs	6 yrs	**22.7 yrs**
Alternative 2B	1.81E-8 in*in/sec	0.2	1.97 lb/cub. yd	15.8 yrs	6 yrs	**21.8 yrs**

Figure 4.12 Service Life Extensions for Alternative Mixes

4.4.2 STADIUM Software Model

STADIUM is computer software to program numerical simulations that may describe the mathematical, chemical, and numerical models. In other words, it is the “numerical model that predicts the mechanisms of ionic transport in unsaturated cement systems” (Marchand, 2000). According to the guidance of Stadium Software, “Stadium is a multi-ionic transport model based on a split operator approach that separates ionic movement and chemical reactions. Ionic transport is described by the extended Nernst-Planck equation applied to unsaturated media.” The software used the following equation for the electrical coupling between ionic species, chemical activity, transport due to water content gradient, and temperature effects:

$$\frac{\partial(\mathrm{wc_i})}{\partial \mathrm{t}} - \mathrm{div}\left\{(\mathrm{D_i w\,grad(c_i)} + \frac{\mathrm{D_i z_i}}{\mathrm{RT}}\mathrm{w\,c_i\,grad}(\varphi) + \mathrm{D_i wc_i\,grad}\left(\ln\gamma_i\right) + \frac{\mathrm{D_i c_i}\ln(\gamma_i \mathrm{c_i})}{\mathrm{T}}\,\mathrm{grad(T)} + \mathrm{c_i\,D_w grad(w)}\right\} = 0$$

Where:

c_i = the concentration (mmol/L)
w = the water content (m3/m3)
D_i is the diffusion coefficient (m2/s)
z_i is the valence number of the ionic species i
F is the Faraday constant (96488.46 C/mol)
y is the electrodiffusion potential (V),
R is the ideal gas constant (8.3143 J/mol/°K),
T is the temperature (°K), g_i is the activity coefficient
D_w is the water diffusivity (m2/s).

Eight ionic species are considered: OH^-, Na^+, K^+, $SO4^{2-}$, Ca^{2+}, $Al(OH)^{4-}$, Mg^{2+}, and Cl^-. In addition, the software used in the ionic transport equation is coupled to Poisson's equation as the following:

$$\mathrm{div}\left(\left(\tau w\,\mathrm{grad}(\varphi)\right) + \frac{F}{\varepsilon} w\left(\sum_{i=1}^{N} z_i c_i\right) = 0$$

Where:

Ɛ = the medium permittivity, (C/V/m)

N = the number of ions in the pore solution

The stadium software uses the following to account for water flow in the presence of water content gradients in unsaturated materials:

$$\frac{\partial w}{\partial t} - \mathrm{div}(D_w \mathrm{gad}\ (w)) = 0$$

The software determines the temperature distribution in the material by the heat condition equation as below:

$$\rho C_p \frac{\partial T}{\partial t} - \mathrm{div}(k\mathrm{gad}\ (T)) = 0$$

Where:

r - the density of the material (kg/m3),

C_p - the specific heat of the material (J/kg/°C)

k - the heat conductivity ($W/m^2/°C$).

Stadium software models the chemical equilibrium module by checking for equilibrium between the concentrations at each node of the finite element mesh as the following equation:

$$K_m = \prod_{i=1}^{N} c_i^{v_{mi}} \gamma_i^{v_{mi}} \quad \text{with } m = 1, \ldots., M$$

Where:

M is the number of solid phases

N is the number of ions,

K_m is the equilibrium constant (or solubility constant) of the solid m,

c_i is the concentration of the ionic species i,

g_i is its chemical activity coefficient, and

nmi is the stoichiometric coefficient of the ith ionic species in the mth mineral.

Stadium Software is a model with complex equations. Users need to have various input data. Williamson (2007) listed the necessary input data as the following:

- "Material density
- Paste content
- Diffusion coefficients
- Water diffusivity
- Total porosity
- Capillary porosity
- Initial values of ion concentration, volumetric water content in the pores, and electrical potential

- Initial amount of solid phases
- Equilibrium constants
- Boundary conditions for ion concentration, volumetric water content in the pores, and electrical potential
- Temperature” (Williamson, 2007)

CHAPTER 5 - Protection, Repair, and Rehabilitation Methods

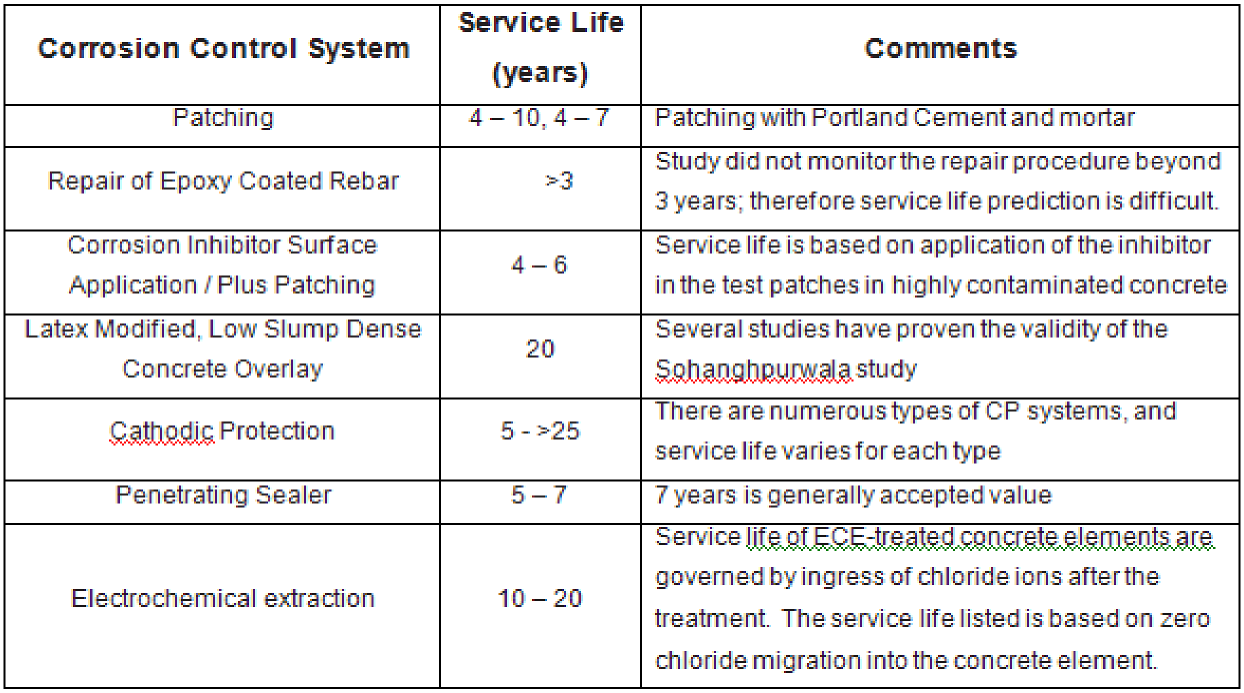
5.1 Introduction

This section addresses the protection, repair, and rehabilitation methods available, including corrosion inhibitors.

Deck protection methods are used to reduce the rate of diffusion of chlorides in the concrete, extend the corrosion propagation period, and/or prevent active corrosion from initiating.

Repair methods involving deteriorated concrete and corrosion of reinforcing rebar is used in most states to extend the service life of a bridge structure. Since it's cheap and simple, many reinforced steel concrete structures are repaired by being overlaid with protective wearing surfaces. This chapter will several common repair methods, including patching method and structure overlay method.

Expected service lives of some protection methods are outlined in Table 5.1:



Corrosion Control System	Service Life (years)	Comments
Patching	4 – 10, 4 – 7	Patching with Portland Cement and mortar
Repair of Epoxy Coated Rebar	>3	Study did not monitor the repair procedure beyond 3 years; therefore service life prediction is difficult.
Corrosion Inhibitor Surface Application / Plus Patching	4 – 6	Service life is based on application of the inhibitor in the test patches in highly contaminated concrete
Latex Modified, Low Slump Dense Concrete Overlay	20	Several studies have proven the validity of the Sohanghpurwala study
Cathodic Protection	5 - >25	There are numerous types of CP systems, and service life varies for each type
Penetrating Sealer	5 – 7	7 years is generally accepted value
Electrochemical extraction	10 – 20	Service life of ECE-treated concrete elements are governed by ingress of chloride ions after the treatment. The service life listed is based on zero chloride migration into the concrete element.

TABLE 5.1 Overview of Corrosion Control Systems

5.2 Cathodic Protection

Cathodic protection and prevention is a common and effective method used to control reinforcement corrosion by making a metal surface an anode, and installing a sacrificial metal (such as titanium or zinc) as a cathode, thus drawing corrosion away from the surface. The reduced potential in the electrochemical cell then slows or stops corrosion. (Williamson, 2007).

Bertoline (2004) describes cathodic prevention as the "application of this low current densities (<2mA/m^2) can bring the potential to values in which steel operates in conditions of "imperfect passivity" so that initiation of pitting is suppressed even if high levels of chlorides, penetrating through the cover concrete, [have] built up at the surface of steel."

Barnhart (1982) stated at the Federal Highway Administration that cathodic protection is the only rehabilitation technique that has proven to stop corrosion with high chloride concentrations in concrete. It has been suggested that a cathodic protection system is effective if it is designed, installed, and operated in the presence of corrosion (Sohanghpurwala, 2006). However, not every reinforced concrete structure requires cathodic protection (Scannell, 1993). Finally, though it is effective, cathodic protection can be expensive and requires ongoing monitoring of the system.

Though standard cathodic protection systems are expensive, Vector Concrete Technologies offers alternative methods of cathodic protection and prevention for reinforced concrete. In addition to several varieties of active and passive cathodic systems, Vector's Vectrode Ti Tape Anode is an electro-conductive adhesive that purports to be a cost-effective form of cathodic protection for non-marine environments. When applied to existing concrete structures, Vectrode Ti Tape reduces existing corrosion and mitigates initiation of new corrosion by allowing current to flow from the anode and into the structure, thus protecting reinforcing steel from corrosion without requiring significant structural modification.

5.3 Concrete Sealers

Sealers stop chloride ingress into concrete by sealing concrete pores, while still permitting water molecules to enter and exit the concrete. "This allows for the concrete deck to dry out, which increases the resistance in the corrosion cell by limiting the electrolyte (the concrete pore water)" (Williamson, 2007)

Sealer methods are widely used in the United States. But it is difficult to estimate the period of the effectiveness of a sealer because of the permeability of the original concrete. Sealer effectiveess may be measured on site using surface conductivity and initial surface absorption tests, which tests measure the uniformity and effectiveness of the sealant application (Broomfield, 2007)

5.4 Epoxy-Coated Reinforcement

Epoxy-coated reinforcement (ECR) is widely highway bridges in the United States. Epoxy coatings prevent the penetration of aggressive chloride ions, oxygen, and water from the bare rebar steel interface, thus lengthening the initiation period of. ECR is also less dependent upon cover depth than bare steel. However, the condition of the coating significantly effects the corroded initiation, and ECR does somewhat reduce the bond strength between reinforcement and concrete. Further, ECR is approximately twice as expensive as uncoated steel. See Section 2.7 for a more in-depth discussion.

5.5 Chemical Corrosion Inhibitors

Corrosion inhibitors such as sodium nitrite, potassium dichromate, sodium benzoate, and stannous chloride are chemical compounds or formulations that can provide corrosion control for steel reinforcement. They have been used extensively since the 1960's. Some inhibitors include sodium nitrite and the sodium and potassium salts of chromate and benzoate, but sodium and potassium salts give mixed results on corrosion inhibition and have been shown to reduce the strength of concrete (Kepler, 2000).

Many different types of corrosion inhibitors exist for specific applications, such as water treatment, petroleum refining, or other chemical processing applications. Corrosion inhibitors control corrosion in the following ways:

> "a) by a competitive surface adsorption process of inhibitor and chloride ions
>
> b) by increasing and buffering of the pH in the local (pit) environment, and
>
> c) by competitive migration of inhibitor and chloride ions into the pit (so that the low pH and high chloride contents necessary to sustain pit growth cannot develop)" (Bertolini et al, 2004).

Williamson (2007) explained that the various types of corrosion inhibition operate either the anodic reaction site, the cathodic reaction site, or both, and may be categorized as cathodic inhibitors, anodic inhibitors, and organic inhibitors. The purpose of an anodic inhibitor is to minimize the anodic part of the corrosion reaction. An anodic inhibitor is normally made up of nitrites, molybdates, silicates, alkali phosphates, carbonates, and so on. Cathodic inhibitors are less commonly used than anodic inhibitors. They are made from zinc, salts of antimony, magnesium, manganese, and nickel, and form an insoluble film on the cathodic surfaces of the steel (Kepler el all, 2000). Organic inhibitors are formulations which include a water-based, organic amine and an oxygenated hydrocarbon. Calcium nitrite (inorganic) and organic amine-based inhibitors have been developed as an effective inhibitor for reinforced concrete (Sohanghpurwala, 2006).

5.5.1 Calcium Nitrite Inhibitors

The Federal Highway Administration (FHWA) and DOTs in Idaho, Indiana, Pennsylvania, and Virginia indicated the benefits of using calcium nitrite after evaluations of their use in concrete used in new construction (Powers, 1999). The Idaho DOT stated that calcium nitrite reduced corrosion and deserved further study after evaluation in the laboratory (Cosho, 1980). The Indiana DOT, based on cracked beam and other testing performed in accordance with ASTM G109, indicated that calcium nitrite was effective (Belew, 1996). The Pennsylvania DOT used two products, calcium nitrite and an organic inhibitor, in two bridges and concluded that no construction problems were encountered (Sukley, 1998). Ozyildirim (1992) studied the effects of calcium nitrite on the properties of concrete used in prestressed piles, beams, and bridge decks and concluded that the properties of the concrete were acceptable.

Calcium nitrite (inorganic) is a liquid admixture containing 30% calcium nitrite and a 15% solution of calcium nitrite (Sohanghpurwala, 2006). The calcium nitrite chemically oxidizes the surface of the reinforcement according to the following reaction (Gaidis and Rosenburg, 1987).

$$2Fe^{2+} + 2OH^{-} + 2NO^{2-} \rightarrow 2NO + Fe_2O_3 + H_2O$$

In this equation, nitric oxide, ferric oxide, and water are formed by the reaction of ferrous, hydroxyl, and nitrite ions. Hime and Erlin (1987) determined the reaction between ferrous ions and a chloride ion, which is shown below:

$$Fe^{2+} + 6Cl^{-} \rightarrow FeCl_6^{-4}$$

$$FeCl_6^{-4} + 2OH^{-} \rightarrow Fe(OH)_2 + 6Cl^{-}$$

Generally, chloride ions react with ferrous ions to form $FeCl_6^{-4}$ and then react with hydroxyl ions to form the compound of ferrous hydroxide, Fe(OH)2. The chloride ions (Cl^{-}) are released back into the solution for further reaction with additional ferrous ions (Sohanghpurwala, 2006).

The ratio of the nitrite to chloride ions in the concrete significantly contributes to the effectiveness of the nitrite inhibitor. Therefore, dosage rates are normally recommended by the manufacturer. In addition, the recommended dosage levels may be added from 33% to 67% more than the instruction of the manufacturer to offer additional protection Virmani and Clemena (1998) indicated that, depending on the expected chloride exposure level, the typical dosage ranged from 10 to 30 l/m^3 (2 to 6 gal/yd^3) of concrete. In fact, Bentur et al. (1997) found a relationship between calcium nitrite dosage and chloride ion concentration at the level of reinforcement as in the following Table 5.2

Table 5.2 Dosage Required for Protection against Chloride-Induced Corrosion

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

The table gives the recommended calcium nitrite dosages required for protecting reinforcement from corrosion for different chloride ion contents. A higher chloride ion concentration requires a higher dosage of level of calcium nitrite.

Sohanghpurwala (2006) listed advantages and disadvantages of cacium nitrite as a corrosion inhibitor for reinforcement embedded in concrete. The advantages include the following:

- "When used as an admixture in new construction, effective corrosion protection is given to both reinforcement bar mats, thereby providing extensive system longevity compared with unprotected reinforced concrete in chloride laden environments.
- Nitrite is fully compatible with portland cement and can be used during batch mixing or at the job site.
- Nitrite requires no maintenance after installation.
- Nitrite can be used in reinforced and prestressed concrete elements.
- Nitrite can act as an accelerator and inhibitor.
- Nitrite compares favorably with other corrosion protection systems in protection and cost-effectiveness, versatility, and east of application.
- In solution form, nitrite can be used in chloride-contaminated concrete without extensive surface preparation or exposure of corroded reinforcement bars" (Sohanghpurwala, 2006).

Disadvantages to using nitrite as a corrosion inhibitor include the following:

- "Accelerating properties of calcium nitrite can create difficulties in placing, finishing, and curing concrete under certain conditions (such as high ambient temperatures and use of cements with a low C3A content).
- Nitrite may cause an increase in slump.
- Although an increase in strength may be seen, nitrite also has the detrimental effect of retempering due to stiffening of the treated concrete.
- Because the amount of nitrite required depends on the amount of chlorides in the concrete, careful estimates of these quantities are required (nitrite may lead to an over dosage when used as an admixture).
- When nitrite is used in solution form, the concrete cover should not be more than 0.5 inch" (Sohanghpurwala, 2006).

Organic Inhibitors 5.5.2

Organic products are formulations that include a water-based, organic amine and an oxygenated hydrocarbon. The amine based inhibitors are currently marketed as a liquid admixture and a surface-applied inhibitor (Sohanghpurwala, 2006). There are many categories of organic inhibitors such as amines, esters, and sulfonates that block both the anodic and cathodic reaction on the entire surface of the metal (Virmani and Clemena 1998). Also, organic corrosion inhibitors have been used in the oil and gas industries for years. (Nmai et al. 1992). Organic inhibitors form a protective barrier on the surface of the reinforcement to prevent chloride from contacting with the reinforcement. Unlike nitrite inhibitors, the ratio of inhibitor to chloride in the concrete does not alter the effectiveness of organic inhibitors (Sohanghpurwala, 2006).

Various types of organic corrosion inhibition have been developed. This paper enumerates several types of popular organic inhibitors cited in the literature. See Appendix A.

Sohanghpurwala (2006) listed advantages and disadvantages of an organic inhibitor as a corrosion inhibitor for reinforcement embedded in concrete. The advantages include the following:

- Organic inhibitors can be used as either admixtures or penetrating coatings.

- Organic inhibitors are cost-effective and easy to apply.
- When organic inhibitors are used as an admixture, they do not have any detrimental effects on setting time, slump, and stiffness.
- When organic inhibitors are used as an admixture, they greatly reduce the ingress of chloride, sulfate, and other aggressive ions.
- When organic inhibitors are used as a coating, no concrete removal is required (however, the surface must be cleaned).
- When organic inhibitors are used as a coating, the product can be spray or brush applied.
- Organic inhibitors are water based, easy to handle and use, and are environmentally safe" (Sohanghpurwala, 2006).

There are some disadvantages to using organic inhibitors include the following:

- "Because of a more compact pore structure, organic inhibitors reportedly are not as effective when used with concretes having a low water-to-cement ratio.
- Organic inhibitors produce a slight decrease in concrete strength.
- When organic inhibitors are used as a coating, they cannot be applied below 36°F, nor should they be applied if the temperature is expected to fall below 36°F within 12 hours of application.
- The concrete surface should be as dry as possible before applying the organic inhibitor product" (Sohanghpurwala, 2006).

5.6 Patching Method

Depending on the size, location, and the general function of bridge components, various materials may be used for patching. Material selection is normally based on compatibility of the material to the original concrete, environmental considerations, cost effectiveness, expected service life, availability, and familiarity of contractors with the product.

Common patching materials include latex-modified concrete and mortar, epoxy patching compounds, polyester resin, acrylic concrete and mortar, polymer-modified cement based materials, pozzolanic modified concrete, high alumina cement compounds, magnesium phosphates, molten sulfur, calcium sulfate based materials, non-shrink quick setting mortar cement based polymer concrete, and pneumatically applied mortar. Portland cement binders and well-proportioned aggregates are widely used in deep-repair of patching, since "durability for these materials can be increased using special pozzolans (microsilica), polymers (Oatex), or admixtures that reduce permeability", Arockiasamy (1998).However, patching does not completely prevent corrosion in concrete. Williamson (2007) stated that "the patched area will contain very low chloride concentrations and will therefore become cathodic, which will in turn drive the adjacent actively corroding anodic sites." Sacrificial zinc has been used to counter this, but the effect is temporary. Again, Williamson: "as the zinc is consumed, the reinforcement in the deck will once more become anodic and corrosion will progress just as before."

5.7 Overlay Method

Overlays are placed on the surface of sound concrete to increase the effective cover depth over the reinforcing steel. This cover layer prevents chloride ions from reaching the reinforcement in concrete. Overlay materials include latex modified concrete, low-slump dense concrete, polymer concrete (epoxy), micro-silica concrete, hot-mix asphalt concrete, and membrane systems The most common application of overlay systems involves the use of multiple layers of polymer concrete (Williamson, 2007).

Overlay material requires adequate compressive creep, modulus of elasticity, and strength. Overlays should be designated low shrinkage, low permeability, and low water/cement ratio to prevent moisture and chloride penetration. In addition, an overlay system must be strong enough to confront variable temperatures, moisture, high traffic, and chemical exposure. The overlay method has several limitations. Overlay increases dead load on structures, which will decrease the live load carrying capacity of beams or girders in bridges. Structures already containing critical level of chlorides in overlay require additional attention, since this increases the corrosion rate of reinforcement in concrete. Additionally, problems may develop due to debonding and shrinkage of the overlay.

5.8 Other Reinforcement Methods

5.8.1 Stainless Steel Reinforcement

Stainless steel has a much higher corrosion resistance than carbon steel because of its higher chloride threshold. The threshold value for stainless steel is 10.4 times more than carbon steel (Clemena, 2003). Therefore, the main benefit of stainless steel reinforcement is to extend the lifetime of concrete structures in corrosive environments and reduce the maintenance cost. (For concrete structures using stainless steel, less concrete cover may be used).

Stainless steel can be 6 to 10 times more cost-effective than that of carbon steel (Williamson, 2007). Therefore, stainless steel may be considered for structures erected in high-chloride environment or those with long expected service lives. To reduce cost, it is possible to use stainless-steel-clad reinforcement, which contains a mild steel core and a 1-2mm stainless outer layer. Williamson (2007) stated stainless-steel-clad reinforcement costs about 3 times less than completely stainless steel reinforcement. Though the cost is reduced, the service life is still long; stainless steel clad bars can be used for structures with an expected service life of 100 years (Williamson, 2007).

5.8.2 Galvanized Steel Reinforcement

Hot dip galvanization is one of the most effective methods of corrosion risk reduction. Galvanizing steel reinforcement is a process that coats steel bars with metallic zinc or a zinc alloy (Bertolini, 2004). The layer of zinc prevents the penetration of chloride, thus preventing corrosion.

Galvanized steel reinforcement has a 2-4 times higher chloride threshold than uncoated steel (American Galvanizers Association, 2006). Williamson (2007) showed that galvanized steel has a period of corrosion initiation that is 4-5 times longer than that of carbon steel. The thickness of the zinc coating normally ranges from 100 to 150 mm (0.0039 to 0059 in.) (Bertolini, 2004). The corrosion rate of galvanized steel bars in carbonated concrete is approximately 0.5-0.8 mm per year (Metal Handbook); therefore, the durability time of zinc coating thickness of 100 mm is more than 120 years.

However, several limitations of galvanized steel reinforcement exist. Like epoxy-coated reinforcement steel, the zinc oxide coating may be damaged during construction, and these damaged regions will corrode like normal carbon steel. Welding or cutting of the steel in construction creates exposure of unprotected edges. Finally, the price of galvanized steel is 2 to 2.5 times more expensive than normal black steel.

5.9 Chloride Extraction

Sohanghpurwala (2006) verified in the laboratory the effectiveness of electrochemical chloride extraction on bridge superstructure elements, substructure elements, and slabs. He reported that slabs treated with an electrochemical chloride extraction process had not exhibited any signs of corrosion ten years after the treatment, and concluded that another ten years of protection could be expected. Chloride extraction is considered a short-term protection measure, and should be used in conjunction with other treatment measures such as sealants or electrochemical protection.

CHAPTER 6 - Conclusions and Recommendations

6.1 Conclusions

Corrosion is an electrochemical process. The causes of corrosion of steel reinforcement in concrete bridge structures can be attributed primarily to either carbonation or chloride ingress. Carbonation — a compound formed from the interaction of carbon dioxide in air and alkaline hydroxide in concrete —results in damage in areas with low concrete cover or in areas with high humidity levels. Chloride contamination creates an electrochemical circuit in which chlorides (negatively charged ions) migrate to the anodic regions, in this case steel reinforcement. As the chloride content increases, alkalinity decreases, which breaks down the passive layer of protective oxide that normally exists on the surface of the reinforcement, and corrosion may begin.

Several factors influence the corrosion rate of steel in concrete, and as a result the time required for corrosion propagation depends significantly upon the local climate. High moisture levels, both in the concrete and the surrounding environment, decrease the corrosion propagation time period, as does a high concentration of free chlorides. The rate of chloride ingress can be estimated using Fick’s Second Law of Diffusion.

A thorough understanding of the corrosion process can inform the inspection of both steel reinforcement and the concrete surrounding it. Proven tools and existing processes can be supplemented with several techniques, including half-cell potential measurements, acoustic emissions monitoring, macrocell corrosion monitoring, ultrasonic pulse methods, wireless sensors, polarization resistance measurements, and several others. These techniques can identify areas of steel corrosion and/or concrete damage, as well as high chloride concentrations. Though no one of these techniques is appropriate or necessary for all inspections, some situations may warrant the level of detail and insight they can provide.

With information gathered from an ongoing and rational evaluation program, it is possible to generate an estimate of the remaining service life. Several theoretical models exist for predicting both degradation and chloride ingress, though some (such as the Finite Element Model) require complex calculations. Computer-based modeling software can simplify this process somewhat, while simultaneously providing a more detailed estimate of the remaining service life. These estimates can be expanded to include effects of the various repair and protection techniques and technologies.

These techniques and technologies can be grouped into several broad categories: cathodic protection, which is an electrochemical method to neutralize chloride ingress; sealers, membranes, organic inhibitors (see Appendix A), and admixtures form a protective barrier that prevent chlorides from coming into contact with the reinforcement; patching and overlay methods using a variety of materials can be used to repair damage and increase cover depth, both of which will reduce the time to corrosion initiation, but these should be used in conjunction with corrosion inhibitors and/or cathodic protection. Chloride extraction has also shown to be an effective short-term protection measure.

6.2 Recommendations

Based on the findings presented here, several recommendations can be made.

6.2.1 Theoretical Modeling of Chloride Ingress – Use Fick's 2nd Law of Diffusion

It's cumbersome to estimate the appropriate values for the parameter m that involves the calculation of an empirical equation that varies with mix proportions included in the more complex Fickian Diffusion Models. It's also clear that estimating appropriate parameters for calculating a time-dependent D_c remains a difficult task (Williamson, 2007). While the FEM is more detailed that the Fickian-based diffusion models presented here, in its calculations and parameters FEM is difficult to use because of the complexity of the calculations required. Therefore, Fick's 2nd Law of Diffusion remains the best choice due to the ease of ease and the lack of evidence that other methods are significantly more accurate.

6.2.2 Measure Corrosion Rate with Half-Cell Potential Measurement Technique

Due to its modest technical requirements, the half-cell potential measurement technique should be used in conjunction with other techniques and methods to determine the probability of corrosion. This technique can provide detailed mapping of areas of high probabilities of corrosion, allowing inspectors and engineers to devote limited attention and resources to areas known to be troublesome. To mitigate the limitations, Sohanghpurwala (2006) advised that it is important to carry out on measurements on a regularly spaced grid and to evaluate a large number of closely spaced half-cell potentials.

6.2.3 Software Modeling – Use Life-365

STADIUM modeling software is a powerful tool for estimating service life and analyzing life cycle costs. However, it includes complex equations for its computations, and gathering the required input data necessitates a number of tests and calculations be performed (Marchand, 2000). Therefore, it is recommended that Life-365 be used for quantifying the service life extension expected from various maintenance and repair options. The interface is user friendly; it uses Fick's 2nd Law of Diffusion to calculate time to cracking initiation; it allows the user to define propagation periods based on local experience; it allows the user to adjust concrete mix parameters and analyze the use of corrosion inhibitors and members; and it performs a very detailed life cycle cost analysis for up to six alternatives.

6.2.4 Perform Field Evaluations Using a Variety of Suitable Techniques.

Field evaluation techniques should be developed to assist WVDOH engineers, inspectors, and maintenance personnel to easily determine what service life extension techniques would be most beneficial for the specific issues that may exist for each particular bridge. However, these techniques will not be a one-size-fits-all solution. Each bridge component should be evaluated independently and preservation techniques should be recommended for each based on the evaluation. For instance, some structures may require only hammer sounding, while the condition of others necessitates a half-cell potential measurement to determine the probability of corrosion for various superstructure elements, which can provide more detailed locations for the collection of core samples collected for more detailed evaluation.

6.2.5 Follow Corrosion Evaluation Schedule

An initial corrosion condition evaluation should take place approximately four years after a new bridge is open, and immediately for a bridge that has been in service longer than four years and has not been properly evaluated for corrosion potential.

Routine corrosion condition evaluations should be performed at each Period Inspection (2 years) and an In-Depth Corrosion Evaluation should be performed during each In-Depth Inspection (6 years). If the remaining service life is estimated to be greater than 15 years, it would not be necessary to perform an In-Depth Corrosion Inspection every six

The extent of the evaluation should be based on the extent of damage that is discovered from less intensive evaluation methods. For example, if a structure is in low-to-poor condition (a condition rating of 3 or 4), rehabilitation may not be a good idea. Alternatively, if a structure is in good condition (condition rating 7), minor rehabilitation may be required to keep the element serviceable. For bridges with a rating of 5 or 6, a more detailed evaluation should be employed.

The most efficient method would be to develop procedures for selecting when various inspection types occur based on condition coding, since this concords with the WBDOH Bridge Management System currently in use.

6.2.6 Develop a Manual

To systematize the various types of evaluation, repair, and modeling techniques outlined in this report, a manual should be developed that outlines reliable, research-based procedures for assessing the condition of corrosion-deteriorated in-service highway bridges in West Virginia. The manual will assist WVDOH personnel in identifying the most sound preventative maintenance activities for effectively managing their bridge inventory.

The manual must be easy to use and contain detailed procedures for assessing, in both field and office, the condition of reinforced concrete bridge elements subjected to corrosion-induced damage. The manual should also describe at what frequency corrosion condition evaluations should be performed.

6.2.7 Final Words

The ultimate conclusion of this report is that we should keep good bridges good. To achieve this end, we should be proactive instead of reactive, share best practices gained from previous experiences, and employ techniques based on research. Being more efficient with the resources available will lead to a safer bridge inventory in West Virginia and nationwide.

REFERENCES

AASHTO LRFD Bridge Design Specifications, Interim Revision, 2005.

AASHTO LRFD Bridge Design Specifications, 3rd Edition, 2004.

AASHTO LRFD Bridge Construction Specifications, 2nd Edition, 2004.

AASHTO Standard Specifications of Highway Bridges, 17th Edition, 2002.

ACI Committee 318, "Building Code Requirements for Reinforced Concrete, ACI 318-05, and Commentary, ACI 318R-05," American Concrete Institute, Detroit, MI, 2005, 443 pp.

American Concrete Institute (2005), "Guide for the Design and Construction of Concrete Reinforced with FRP Bars," ACI 440.1R-05, ACI, Farmington Hills, MI.

ACI Control of Deflection in Concrete Structures, ACI 435R-95 (1995), American Concrete Institute Committee 435, Farmington Hills, MI.

ACI Committee 201 (1994) "Guide to Durable Concrete, Chapter 4 – Corrosion of Steel and Other Materials Embedded in Concrete", Manual of Concrete Practice, Part I.

Andrade, Alonso, Gonzalez: "An Initial Effort to Use the Corrosion Rate Measurements to Estimating Rebar Durability", ASTM STP Corrosion Rate of Steel in Concrete, 1990, pp. 29-37.

Andrade, C., and Gonzalez, J. A., "Effect of Carbonation, Chlorides and Relative Ambient Humidity on the Corrosion of Galvanized Rebars Embedded in Concrete," British Corrosion Journal. 1982, Vol. 17.

Andrade, Maribona, Feliu, Gonzalez, Feliu "The effect of macrocells between active and passive areas of steel reinforcements, Corrosion Science, 1992, 33, 237-249.

Arup, H. Danish Corrosion Centre. - Newsletter, 1979, No. 2.

Arockiasamy, M.,Evaluation of Conventional Repair Techniques for Concrete Bridges. 1998.

Broomfield, J. P. "Corrosion of Steel in Concrete—Understanding, Investigation, and Repair" E & FN Spon, 2007.

Brown, M; "Corrosion protection service life of epoxy coated reinforcing steel in Virginia Bridge Decks", Virginia Polytechnic Institute and State University, 2002.

Bertolini, L., B. Elsener, P. Pedeferri, and R. Polder, "Corrosion of Steel in Concrete: Prevention, Diagnosis, Repair", Wiley and Sons, 2004.

Boddy, A., E. Bentz, M.D.A. Thomas, and R.D. Hooton, "An overview and sensitivity study of a multimechanistic chloride transport model," Cement and Concrete Research, Vol. 29, pp. 827-837, 1999.

Berkeley, K.G.C., and Pathmanaban S. "Practical Potentials Monitoring in Concrete," UK CORROSION'87, Brighton, UK, 1987, pp. 115-131.

Berke, N, Aldykiewicz, A and Li, L; "What's new in Corrosion Inhibitor", Concrete Reinforcing Steel Institute, 2003.

Barnhart, R. A;"FHWA Position on Cathodic Protection," Federal Highway Administration: Washington, DC, 1982.

Bentur, Arnon, Diamond, Sidney, and Berke, Neal S; "Steel Corrosion in Concrete" E and FN Spon, New York, 1997, 201 pp.

Belew, Y.K; "New Products Evaluation Committee Laboratory Investigation of Five Corrosion Inhibitors," Indiana Department of Transportation, Indianapolis, 1996.

Burubai, W. and Dagogo, G.; "Comparative Study of Inhibitors on the Corrosion of Mild Steel Reinforcement in Concrete", River State University of Science and Technology, Nigeria, 2007.

Batis, G, Rakanta, E; "Effects of Migrating Corrosion Inhibitors on Reinforced Lightweight and Common Mortars", National Technical University, Greece.

Bavarian, B, Reiner, L; "Corrosion Protection of Steel Rebar in Concrete with Optimal Application of Migrating Corrosion Inhibitors, MCI 2022", The Cortec Corporation, 2003.

Clemena, Kukreja, and Napier, "Trial Use of a Stainless Steel-Clad Steel Bar in a New Concrete Bridge Deck in Virginia," Virginia Transportation Research Council, Report # VTRC 04-R5, Dec. 2003.

Clear, "Measuring Rate of Corrosion of Steel in Field Concrete Structures", paper No. 88-0324, 68-th Annual Transportation Research Meeting, 1989.

Cosho, J.L. Reducing Corrosion of Reinforcing Steel in Concrete Bridge Decks. Research Project 88. Idaho Department of Highways, Boise, 1980.

Darwin, David D. and Hadje-Ghaffari, Hossain, "Effects of PE-217 on Steel-Concrete Bond Strength," Report, University of Kansas, Lawrence, Kansas, 1990.

Detwiler, Kojundic, and Fidjestol; "Evaluation of Bridge Deck Overlays", Magazine of the American Concrete Institute, 1997.

Dillard, J. G., Glanville, J. O., Collins,W. D.,Weyers, R. E., and Al- Qadi, I. L., "Concrete Bridge Protection and Rehabilitation: Chemical and Physical Techniques, Feasibility Studies of New Rehabilitation Techniques" Strategic Highway Research Program report SHRP-S-665, Transportation Research Board: Washington, DC, 1993. (Available at link: http://onlinepubs.trb.org/onlinepubs/shrp/SHRP-C-665.pdf.)

Ehlen, M, Thomas, M and Bentz, E; "Life-365 Service Life Prediction Model Version 2.0", American Concrete Institute, 2009.

ElBatanouny, M., Abdelrahman, M., and Ziehl, P. "Review of Acoustic Emission Corrosion Monitoring of Prestressed Concrete Bridges." Paper presented at the 2014 PCI Convention and National Bridge Conference, National Harbor, MD. September, 2014.

ElBatanouny, M., Mangual, J., Ziehl, P., and Matta, F. "Early Corrosion Detection in Prestressed Concrete Girders Using Acoustic Emission." J. Mater. Civ. Eng., Vol. 26, No. 3, 2014.

Elsener, Klinghoffer, Frølund, Rislund, Schiegg, Böhni: "Assessment of Reinforcement Corrosion by Means of Galvanostatic pulse Technique", Proceedings of the International Conference: "Repair of Concrete Structures. From Theory to Practice in a Marine Environment", Svolvar, Norway, May 1997, pp. 391-400

Elsener, B., Buchler, M., and Bohni, H., "Corrosion Inhibitors for Steel in Concrete," Corrosion of Reinforcement in Concrete-Monitoring, Prevention and Rehabilitation, European Federation of Corrosion Publications No. 25, Eds. J. Meitz, B. Elsener, and R. Polder, 1997.

Frohnsdorff, G; "Modelling Service Life and Life-Cycle Cost of Steel – Reinforced Concrete', National Institute of Standards and technology, 1999.

Glass, G. K. and N. R. Buenfeld, "The Presentation of the Chloride Threshold Level for Corrosion of Steel in Concrete," Corrosion Science, Vol. 39, No. 5, 1997.

Gaidis, J. M., and Rosenberg, A., "The Inhibition of Chloride- Induced Corrosion in Reinforced Concrete by Calcium Nitrite," Cement, Concrete, and Aggregate. 1987.

Gaidis, J. M., and Rosenberg, A; "Using Corrosion Inhibitors to Avoid Damage to Reinforced Concrete", Concrete Corrosion Inhibitors Association, 2004.

Guthrie, S and hema, J; "Concrete Bridge Deck Condition Assessment Guidelines"; Utah Department of Transportation Research, 2005.

Gastaldi, Bertolini, Pedeferri, and Redaelli, "Prevention of steel corrosion in concrete exposed to seawater with submerged sacrificial anodes" Conosion Science, 2002.

Glaser, S; Wang, M; Ou, J and Lynch, J; "Sensor technology innovation for the advancement of structural health monitoring: a strategic program of US-China research for the next decade" Smart Structures and Systems, Vol. 3, No. 2 (2007) 221-244

Hime,W., and Erlin, B; "Some Chemical and Physical Aspects of Phenomena Associated with Chloride-Induced Corrosion," American Concrete Institute: Detroit, MI, 1987.

Isgor, B and Razaqpur, G; "Modelling steel corrosion in concrete structures", journal Materials and Structures (2006)

Inaudi, D, manetti, L, and Glisic, B "Reinforced Concrete Corrosion Wireless Monitoring System", 4th International Conference on Structural Health Monitoring on Intelligent Infrastructure (SHMII-4) 2009, Zurich, Switzerland.

Kepler; Darwin; Locke. 2000. "Evaluation of corrosion protection methods for reinforced concrete highway structure", The National Science Foundation.

Kranc, S. C., A. A. Sagüés, and F. J. Presuel-Moreno, “Decreased Corrosion Initiation Time of Steel in Concrete due to Reinforcing Bar Obstruction of Diffusional Flow,” ACI Materials Journal, 2002.

Kirkpatrick, Weyers, and Sprinkel, “Service Life of Concrete Bridges in Virginia With and Without Epoxy Coated Reinforcement” Cement and Concrete Research, 2000.

Kirkpatrick, T. J. et al., "Impact of Specification Changes on Chloride Induced Corrosion Service Life of Bridge Decks," submitted for publication to Cement and Concrete Research (2001a).

Kirkpatrick, T. J. et al., "Probabilistic Model for the Chloride Induced Corrosion Service Life of Bridge Decks," submitted for publication to Cement and Concrete Research (2001b).

Klinghoffer, el al.; “Rebar Corrosion Rate Measurements for Service Life Estimates”, Force Institute, Copenhagen, Denmark, 2000.

Kessler, R, Powers, K, Cerlanek W, and Saques, A; “Corrosion Inhibitore in Concrete”, NACE International, Houston, 2003.

Kumar, A and Santhanam, M; “Detection of Concrete Damage Using Ultrasonic Pulse Velocity Method,” National Seminar on Non-Destructive Evaluation, India, 2006.

Lambert, C.L.P. and Vassie, P.R.W., “Investigations of Reinforcement Corrosion”, Materials and Structures, 1991.

Lawler, J., et al. “Condition Survey of Older West Virginia Bridge Decks Constructed with Epoxy-Coated Reinforcing Bars.” Transportation Research Board of the National Academies, 2011.

Liu, Y., “Modeling the Time-to-Corrosion Cracking of the Cover Concrete in Chloride Contaminated Reinforced Concrete Structures,” Virginia Polytechnic Institute and State University, 1996.

Lundquist, J. T., Rosenburg, A. M., and Gaidis, J. M., “Calcium Nitrite as an Inhibitor of Rebar Corrosion in Chloride Containing Concrete,” Materials Performance, 1979.

Lovejoy, S “Acoustic Emisson Testing of In-service conventionally Reinforced Concrete Deck Girder Superstructures on Highway Bridges,” Federal Highway Administration, 2008.

Manual of “Life-365 Service Life Prediction Mode for reinforced Concrete Exposed to Chloride”, January 8, 2008.

Mejlbro, Poulsen: “On a Model of Chloride Ingress into Concrete Exposed to De-icing Salts Containing Chloride”, International Rilem Workshop on Testing and Modelling Chloride Ingress into Concrete, Paris, 2000.

Mangat, P.S., and B.T. Molloy, “Prediction of long term chloride concentration in concrete,” Materials and Structures, Vol. 27, pp. 338-346, 1994.

Maheswaran, T. and J.G. Sanjayan, "A semi-closed-form solution for chloride diffusion in concrete with time-varying parameters," Magazine of Concrete Research, Vol. 56, No. 6, pp. 359-366, 2004.

Marchand, J., "Modeling the behavior of unsaturated cement systems exposed to aggressive chemical environments," Materials and Structures, vol. 34, pp. 195-200, 2000.

Metal handbook, 9th Edn., ASM Intern., Metal Part, Ohio, Vol.13 1037

Malhotra V. M. and Carino N. J., "Handbook on nondestructive testing of concrete", second edition, ASTM International, CRC press, Chapter 8, 2004.

Malhotra V. M., "Testing hardened concrete: Nondestructive methods", ACI/IOWA state university press, USA, Chapter 5, 74-102, 2004.

McLaskey, Glaser, and Grosse; "Acoustic emission beamforming for enhanced damage detection" University of California in USA and University of Stuttgart in Germany, 2008

Nmai, Charles K., Farrington, Stephen A., and Bobrowske, Gregory S, "Organic-Based Corrosion-Inhibiting Admixture for Reinforced Concrete," Concrete International, 1992.

Naik, T. R., and Malhotra,V. M; 'Handbook on Nondestructive Testing of Concrete. Chapter 7-The Ultrasonic Pulse Velocity Method" Boca Raton, FL: CRC Press. pp. 169-188, 1991.

Nmai, C. K.,"Inhibiting Corrosion Organically," The Concrete Specifier. 1991.

Ozyildirim, C. "Effects of Calcium Nitrite on the Properties of Concrete Used in Prestressed Piles and Beams," Virginia Transportation Research Council, Charlottesville, 1992.

Poulsen, "Prediction of Service Life from Rebar Corrosion Rate Measurements", Internal report, October 2000.

Pyc, W.A., "Field Performance of Epoxy-Coated Reinforcing Steel in Virginia Bridge Decks" Virginia Polytechnic Institute and State University, Blacksburg, Virginia, 1998.

Powers, R.G., Sagues, A.A., Cerlanek, W.D., Kasper, C.A., and Lianfang, L. "Time to Corrosion of Reinforcing Steel in Concrete Containing Calcium Nitrite," Federal Highway Administration, Washington, D.C., 1999.

Qian, S; "Preventing rebar corrosion in concrete structure," National Research Council Canada, 2004.

Ramirez and Hasan. "Behavior of Concrete Bridge Decks and Slabs Reinforced with Epoxy Coated Steel" 1995, Available at: http://works.bepress.com/hhasan/3.

Saetta, A.V., R.V. Scotta, and R.V. Vitaliani, "Analysis of Chloride Diffusion into Partially Saturated Concrete," ACI Materials Journal, V. 90, No. 5, 1993.

Scannell,W. T., and Sohanghpurwala, A. A., "Cathodic Protection as a Corrosion Control Alternative," Concrete Repair Bulletin. 1993.

Sharp, S; “Evaluation of Two Corrosion Inhibitors Using Two Surface Application Methods for Reinforced Concrete Structure”, Virginia Transportation Research Council, 2004.

Sohanghpurwala, A. A; “Manual on Service Life of corrosion-Damaged Reinforced Concrete Bridge Superstructure Elements” National Cooperative Highway Research Program Report 558, VA, 2006.

Sohanghpurwala, A. A.,“Long-Term Effectiveness of Galvanic Current Cathodic Protection on Reinforced Concrete Highway Structures,” NACE International, 2004

Sohanghpurwala, A. A., “Service Life of Corrosion-Damaged Reinforced Concrete Bridge Superstructure Elements” Transportation Research Board:Washington, DC, 2006.

Sohanghpurwala, A. A, “Long-Term Effectiveness of Corrosion Inhibitors Used in Repair of Reinforced Concrete Bridge Components,” Paper Number 03286, Corrosion 2003, 2003.

Stern, M. and Geary, A.L. “Electrochemical polarisation. I. A theoretical analysis of the shape of polarisation curve” J. Electrochem. Soc; 1957.

“Standard Test Method for Half Cell Potentials of Reinforcing Steel in Concrete,” ASTM C 876-91, Annual Book of ASTM Standards, Volume 04.02.

“Standard Test Method for Water-Soluble Chloride in Mortar and Concrete,” ASTM C 1218, Annual Book of ASTM Standards, American Society for Testing and Materials, Philadelphia, PA. (1992).

“Standard Test Method for Acid-Soluble Chloride in Mortar and Concrete,” ASTM C 1152, Annual Book of ASTM Standards, American Society for Testing and Materials, Philadelphia, PA. (1992).

Spellman, D. L., and Stratfull R. F. “Concrete Variables and Corrosion Testing,” Highway Research Record 423, 1973.

Stratfull, R.F. “Half Cell Potentials and the Corrosion of Steel in Concrete,” Highway Research Record 433, 1973.

Sykes, J.M. “Electrochemical Studies on Steel in Concrete,” Electrochemical Methods in Corrosion Research, Proceedings of 5th International Symposium, Sosimbra, Portugal, Sept., pp. 833-842, 1994.

Sherman, Matthew R., Carrasquillo, R. L., and Fowler, D. W. “Field Evaluation of Bridge Corrosion Protection Measures,” Texas Department of Transportation, Austin, Texas, 1993.

Sukley, R., and Bartoski, T., “Evaluation of Corrosion-Inhibiting Concrete Admixtures,” Pennsylvania Department of Transportation, Harrisburg, 1998

Sprinkel, M; “Evaluation of corrosion inhibitors for concrete bridge deck patches and overlay,” Virginia Transportation Research Council, 2003.

Sagues, A; “Corrosion of epoxy coated rebar in Florida Bridge”, Florida DOT, 1994.

Schiegg, Y, Hunkeler, F and Ungricht, H; “The Effectiveness of Corrosion Inhibitor,” Congress of Iabse, Switzerland, 2000.

Schumacher, Th., and Higgins, Ch., “Acoustic Emission Techniques Applied to Conventionally Reinforced Concrete Bridge Girders,” Research Report, May 2008

Tuutti, K., “Corrosion of Steel in Concrete” CBI Forskning Research, 1982.

Tang, P; “Phenomenology Study of HERMES Ground-Penentrating Rdar Technology for Detection and Identification of Common Bridge Deck Feature” U.S Department of Transportation, 2001.

Teng, P; “Materials and methods for Corrosion Control of Reinforced and Prestressed Concrete Structures in New Construction,” Federal Highway Admission, 2000.

Tawhed, W and Gassman, S; “Damage Assessment of Concrete Bridge Decks using Impact-Echo Method”, ACI Materials Journal, 2002.

Vassie, P. R; “the Chloride Concentration and Resistivity of Eight Reinforced Concrete Bridge Decks after 50 Years Service,” Transport and Road Research Laboratory: Crowethorne, UK, 1987.

Virmani, Yash Paul and Clemena, Gerardo G. “Corrosion Protection- Concrete Bridges,” Report No. FHWA-RD-98-088, Federal Highway Administration, Washington, D.C, 1998.

Verhulst, Fuentes, Jirsa, Fowler, Wheat, and Moon; “Evaluation and Performance Monitoring of Corrosion Protection by Fiber-Reinforced Composite Wrapping”, Federal Highway Administration, 2001.

Vallen, D.I.H; “AE Testing Fundamentals,Equipment, Applications”, NDT.net - September 2002.

Williamson, G, “Service Life Modeling of Virginia Bridge Decks,” Virginia Polytechnic Institute and State University, 2007.

Weyers, R.E., M.M. Sprinkel, and M.C. Brown, “Summary Report of the Performance of Epoxy-Coated Reinforcing Steel in Virginia,” VTRC 06-R29, Virginia Transportation Research Council, Charlottesville, VA, June 2006.

Weyers, R.E., M.G. Fitch, E.P. Larsen, I.L. Al-Qadi, W.P. Chamerlin, and P.C. Hoffman; “Concrete Bridge Protection and Rehabilitation: Chemical and Physical Techniques: Service Life Estimates,” Strategic Highway Research Program, Report SHRP-S-668, National Research Council, 1994.

Weyers, R.E., W. Pyc, M. Sprinkel, and J. Zemajtis, “Corrosion Protection Performance of ECR in Solution and Field Structures: Similarities and Differences,” Paper presented at the 77th Annual Meeting of the Transportation Research Board, January 11-15, Washington D. C, 1998.

Weyers, R.E., B.D. Prowell, M.M. Sprinkel, and M. Vorster, “Concrete Bridge Protection, Repair, and Rehabilitation Relative to Reinforcement Corrosion: A Methods Application Manual,” Strategic Highway Research Program, Report # SHRP-S-360, Washington D.C., 1993.

Weyers, R.E, “Service life model for concrete structures in chloride laden environments,” ACI Materials Journal, Vol. 95 (4), pp. 445-453, 1998.

Weyers, R.E., Pyc,W., and Sprinkel, M.M, "Estimating the Service Life of Epoxy Coated Reinforcing Steel", ACI Materials Journal, Vol. 95 (5), pp. 546-557, 1998.

Wipt, Phares, Fanous, Lee, and Jolley; “Evaluation of Corrosion Resistance of Different Steel Reinforcement Types,” Center for Transportation Research and Education, 2006.

Withiam, Fishman, and Gaus; “Recommended Practice for Evaluation of Metal-Tensioned Systems in Geotechnical Applications”, National Research Council, 2002.

Wiss, Janney, Elstner Associates, Inc. “Corrosion Protection Tests on 3M Scotch Kote 911 Concrete Corrosion Inhibitor,” WJE Report No. 840058. 1986.

Xi, Abu-Hejleh, Asiz and Suwito; “Performance Evaluation of Various Corrosion Protection Systems of Bridges in Colorado,” Colorado Department of Transportation Research, 2004.

APPENDIX A

The organic inhibitor Rheocrete 222+ (previously Rheocrete 222) is made by Master Builders. The active ingredient is a water-based combination of amines and esters (Kepler, 2000). Rheocrete 222+ prevents the ingress of chlorides and moisture into the concrete. According to the manufacturer, Rheocrete 222+ provides two levels of corrosion protection, making it the most effective corrosion inhibiting admixture available. The protective barrier is effective due to the formation of a protective film on the surface of the reinforcing Rheocrete 222+ has no effect on plastic properties of concrete, slump or the temperature development profile of concrete, and concrete to steel bond strength. The recommended dosage of Rheocrete 222+ for all applications and corrosion environments is the rate of 5 l/m3 (l gal/yd3) of concrete (Virmani and Clemena 1998).

Some studies have indicated that organic corrosion inhibitors alter the properties of concrete. Nmai et al. (1992) showed that the addition of Rheocrete 222+ affects the compressive strength of concrete. This reduction of strength of concrete is due to the addition of more air-entraining agents to achieve the desired air content. The repoted reduction of the compressive strength of concrete cylinders ranges from 15 to 20% after 7 days when using Rheocrete 222+ (Darwin and Hadje-Ghaffari, 1990).

FerroGard 901 and 903, manufactured by SIKA, are water-based blends of surfactants and dimethyl ethanolamine, an amino alcohol based on Sika FerroGard Technology FerroGard 901 is a liquid admixture. FerroGard 903 is a treatment for concrete surfaces which contains a penetrating corrosion inhibitor.

According to the manufacturer, FerroGard 901 is mixed with gauging water, added into the concrete mixer, or added to the concrete in the transit mixer at the point of discharge. The recommended dosage of Ferrogard 901 is normally 10 l/m3 (2 gal/yd3) of concrete; in cases of severe marine exposure, the recommended dosage is 15 l/m3 (3 gal/yd3). The recommended dosage of FerroGard 903 is 0.27 - 0.41 l/m2 (0.06 – 0.09 gal/yd2) for structures with chloride ion concentrations at the reinforcement level up to 3.5 kg/m3 (6 lb/yd3) (Kepler, 2000).

Armatec 2000, also manufactured by SIKA, is an organic inhibitor made up of a mixture of alcohol and amine. According to the manufacturer, the inhibitors protect the steel reinforcement in concrete by forming a continuous mono molecular film on the surface of the steel reinforcement. The recommended dosage for Armatec 2000 is 2.5 l/m3 (0.5 gal/yd3) of concrete (Pyc et al. 1999).

Catexol 1000CI, manufactured and marketed by Axim Concrete Technologies, Inc, is a water-based formulation of amine derivative. The manufacturer states that the protection of steel reinforcement is due to the formation of a protective film. Virmani and Clemena (1998) discovered that Catexol is a combination of both an organic and a nitrite inhibitor; however, specific data has not been released by the manufacturer.