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16. Abstract

The reinforcing steel within concrete bridge decks is susceptible to corrosion caused most often by the intrusion of deicing salts applied to the surface during freezing weather conditions. The corrosion products cause the steel to expand and produce stresses in the surrounding concrete that eventually fracture the concrete and damage the bridge. This study involves the development of a comprehensive field test program to detect corrosion activity in its early stages, thereby providing a warning that corrective measures are needed to prevent or slow the progress of further damage. The field test program consisted of optimizing the use of several test methods and techniques available today in a way that a true assessment of the corrosion condition of the structure is made with minimum interruption of service. A methodology is presented that will suggest the necessary steps to determine the state of corrosion activity within concrete decks.

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DEVELOPMENT OF A FIELD CORROSION DETECTION AND MONITORING PROGRAM FOR REINFORCED CONCRETE BRIDGES

by

David W. Fowler
R. L. Carrasquillo
Gerald Freytag

Research Report Number 1300-3

Research Project 0-1300
Evaluations of Current Corrosion Protection Measures for Bridge Decks

conducted for the

Texas Department of Transportation

in cooperation with the

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Federal Highway Administration

by the

CENTER FOR TRANSPORTATION RESEARCH
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IMPLEMENTATION STATEMENT

This report outlines the procedures, test methods, and the equipment needed to detect and monitor corrosion of reinforcing steel within a concrete bridge deck. The information contained within this report is intended to provide guidance for the establishment of a regular corrosion inspection program. The corrosion condition inspection may be part of a program that evaluates the conditions of all of the bridges on a particular system (network-level survey). However, because of the time and manpower demands, the detection and monitoring methods described herein may be feasible only at the project level. The information provided here, which is based on experience both in the field and in the laboratory, provides insight into the time, personnel, and equipment required to monitor corrosion.

This survey technique should be used to supplement the routine bridge inspections that lack the ability to detect hidden corrosion. With the information the survey should provide, bridge inspectors will be able to determine if corrosion protection strategies are needed or are working. Further research is needed to establish the proper maintenance alternatives that are needed to slow or abate the corrosion process.

Inherent to the corrosion condition survey is a general increase in the personnel requirements and the traffic control effort over that of the existing Bridge Inspection and Appraisal Program (BRINSAP). To ensure the efficient use of maintenance funds and personnel, and to ensure that a survey of this nature is practical on a routine basis, further study is needed to determine the severity of the reinforcing steel corrosion problem in all parts of Texas.

Prepared in cooperation with the Texas Department of Transportation and the U.S. Department of Transportation, Federal Highway Administration.

DISCLAIMERS

The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Federal Highway Administration or the Texas Department of Transportation. This report does not constitute a standard, specification, or regulation.

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SUMMARY

The reinforcing steel within concrete bridge decks is susceptible to corrosion caused most often by the intrusion of deicing salts applied to the surface during freezing weather conditions. The corrosion products cause the steel to expand and produce stresses in the surrounding concrete that eventually fracture the concrete and damage the bridge. This study involves the development of a comprehensive field test program to detect corrosion activity in its early stages, thereby providing a warning that corrective measures are needed to prevent or slow the progress of further damage. The field test program consisted of optimizing the use of several test methods and techniques available today in a way that a true assessment of the corrosion condition of the structure is made with minimum interruption of service. A methodology is presented that will suggest the necessary steps to determine the state of corrosion activity within concrete decks.

CHAPTER 1. INTRODUCTION

For the highway bridges within Texas, there exists at present no system or program for assessing the extent or severity of corrosion activity within reinforced concrete. Nor is there a program for monitoring corrosion activity that could provide a means of evaluating corrosion prevention measures. This study examines some of the methods commonly used by other states and jurisdictions to assess the amount of corrosion activity that occurs in reinforced concrete bridges. Through field testing, researchers have gained insight into the effort required to efficiently perform corrosion testing and evaluations. The purpose of this report is to provide information about the requirements of instituting and running a field corrosion evaluation program. Considerations for implementation should include not only the technical aspects presented herein, but also the findings of a cost/benefit analysis that reflects the priorities of each state department of transportation (DOT).

1.1 RESEARCH SIGNIFICANCE

The corrosion of reinforcing steel damages structures by causing the embedded metal to expand, resulting in stresses that often exceed the strength of the concrete. The resulting delamination and spalling of the concrete surface then make further ingress of moisture, chlorides, and oxygen possible (thus increasing the severity of the damage). Extreme unchecked cases of reinforcing steel corrosion can result in significant loss of cross-section of the steel, which can then lead to structural failure. Most corrosion is often detected too late and only after it manifests itself in the ways described above.

Chlorides contained in deicing salts and sea water induce the corrosion activity about which this study is made. Salt is applied to bridge decks in the northern part of the state during icing conditions in order to keep the bridges open to traffic. These salt applications have been made since the early 1960's and have created a relatively new corrosion problem for the Texas infrastructure. Although the problems associated with corrosion induced by salt from sea water have been known and addressed for a longer period of time than the deicing salt problems, there remains much to be learned about the most effective methods for preventing corrosion in this environment. Where the use of deicing salt has been discontinued, there remains the threat of corrosion and the expense of periodic repairs made necessary as sodium chloride already contained within the concrete continues to be carried down to the steel. The primary thrust of the field testing within this study involved bridge decks, but other structural elements such as piers and piling exposed to sea water were considered for inclusion in a detection and monitoring program. The conduct of this study was necessary to evaluate the benefits of using a corrosion assessment and monitoring program and to determine the amount of effort required to prevent premature deterioration of those structures that were designed with insufficient protection against a corrosive environment. This study should also assist in increasing understanding about the construction of corrosion resistant structures in the future.

Cost estimates of corrosion damage to the nation's bridges are readily available, but could be misleading when the root cause of deterioration is sought. A corrosion damage estimate for Texas is not presented because of the difficulty associated with assigning the cause for bridge condition failures that may or may not have been corrosion induced. Also, corrosion damage is usually assessed only after it is physically seen. There is little probability that damage estimates will be correct when much of the concrete surface area is covered with asphalt seal coats and asphaltic concrete which mask spalling and make the detection of delaminations difficult. Corrosion commonly inflicts damage after another distress has occurred which allowed the initiation of corrosion. Concrete that has been subjected to sulfate attack or alkali-silica reactivity will suffer distresses that will allow corrosion activity to initiate and cause further damage. In this situation, the owners will want to correct the cause of the damage which is not primarily the result of corrosion. However, because rust staining is likely visible, this situation will be defined as a corrosion problem, and the damage estimate will erroneously show corrosion as the primary cause. An accurate corrosion damage estimate is not known for Texas bridges and did not drive this study. There is a strong belief, however, that corrosion is significantly shortening the life of some reinforced concrete structures. As infrastructure management systems become established, the insertion of corrosion condition aspects in regularly performed condition surveys should be considered.

The problem of reinforcing steel corrosion within bridges will continue to plague the builders and maintainers of infrastructure systems because the problem's primary instigator, deicing salt, has no cost effective substitute (1, 2). Substructures founded in sea water, although they are now constructed with epoxy-coated reinforcing steel and have corrosion inhibitors admixed into the concrete, will continue to be cautiously observed because of the uncertainty of the protection system's effectiveness. Though it is too late to stop the initiation of corrosion in some structures, there is the possibility of early detection and slowing the deterioration rate to an acceptable level, thereby extending the life of the structure.

It can be stated that the corrosion of reinforcing steel is not a problem in every part of the state, and because the distribution of the problem is not uniform, the state may need to be prepared to shift funding to areas that have corrosion abatement and prevention in major contention for money allocated for maintenance. The program presented in this study is expected to assist in determining the future funding needs. Testing procedures within the program will indicate to the bridge owner whether corrosion activity makes options such as the retention and widening economically feasible. The tests will also call attention to the need for any corrosion abatement procedures necessary in the normal maintenance scheme.

1.2 RESEARCH OBJECTIVES

The corrosion of reinforcing steel in concrete bridge decks is a relatively new problem for the state of Texas. The research involved in this study was deemed necessary to assist in building a cost effective and reliable program for determining the corrosion condition of concrete bridges. In order for a program to be instituted in the face of personnel cutbacks and competitive funding, it must be an efficient program that produces reliable results and recommendations for effective prevention and repair. The program or system must be worthy of continuation in order to build a data base of information about a problem of significance. Conducting a one-time survey would give an assessment of the current corrosion condition of the structures in the state, but would fail in building a data base that may yield valuable information over time concerning the effectiveness of protection systems and maintenance strategies. Without emphasis and support from all levels of supervision within the performing organization, a program has little chance of successfully achieving its objectives.

A study was needed to assimilate information concerning the use of state-of-the-art corrosion assessment methods and equipment and to practice their use on existing structures and gain familiarity. A gain in experience in using the equipment effectively was needed to assist in building confidence in the test methods and the results of the testing. The experience gained by employing the tests on various structural members in various environments is necessary in the formulation of manpower and cost requirements for an assessment or monitoring program.

The practical use of the test methods in the study was not conducted to necessarily increase the understanding of the corrosion mechanism. However, the testing and evaluation of test results continue to increase knowledge about why corrosion does or does not occur.

The effectiveness of the various corrosion protection measures and strategies in use today has not been extensively evaluated. One objective of this study includes the identification of test methods used to monitor the effect that various concrete sealants, corrosion inhibitors, and maintenance practices have on preventing or slowing the rate of corrosion. Historically, there have not been extensive evaluations of the protection systems tried and used in Texas; rather, there has been a dependence on receiving information from the experience gained by other states.

Bridge Management Systems (BMS) are being instituted in state transportation agencies as mandated by federal legislation. The effectiveness of the BMS will depend in part on accurate condition data being provided on a timely basis. The corrosion condition of a structure can be assessed after an effective corrosion survey has been performed; the survey may then impact decisions suggested by the BMS. This study was undertaken to assist in acquiring knowledge about specific input for the Bridge Management System.

1.3 REPORT ORGANIZATION

Chapter 2 presents the arguments for and against the institution of a program that detects the activity of corrosion and monitors changes in the activity over time. Chapter 3 reviews the corrosion mechanism and discusses various aspects that affect the probability of occurrence and the rate of corrosion. The data acquisition requirements of the Bridge Management System to be employed in Texas are covered in Chapter 4, as well as the recommendations on this subject provided by the Strategic Highway Research Program and the Federal Highway Administration. Chapter 5 discusses the various tests recommended for the program with justification based on case studies encountered in field testing. Chapter 6 presents implementation guidelines. The final chapter summarizes the findings and recommendations made within the presentation, and discusses the future work necessary for continuous improvement of the program. An appendix contains some of the corrosion condition survey reports of the structures tested and described in

Chapter 5, which will serve as examples of the type of information necessary for inclusion in the total bridge condition survey files.

CHAPTER 2. CONSIDERATIONS FOR PROGRAM IMPLEMENTATION

2.1 INTRODUCTION

The purpose of this study was to identify an efficient and effective corrosion detection and condition assessment program. Some aspects that may be examined when considering a full-scale program are presented in an attempt to provoke thought toward making the correct decision for or against implementing such a program. A system is needed because the employment of management systems may ultimately call for a program that detects the potential for corrosion damage. Furthermore, a cost-effective replacement for deicing salt is not presently available, and the corrosion problem is likely to continue. However, because new construction includes corrosion prevention measures and old deteriorated structures are being replaced, the corrosion of reinforcing steel may not be as severe in the future as it is today. The severity of the problem may subside without the establishment of a detection and monitoring program. Some structures cannot be evaluated using all the tests of the program because of some materials they contain.

2.2 JUSTIFICATION FOR INSTITUTING A PROGRAM

An evaluation such as the one presented herein will help the owners of most current systems by keeping them from relying on visual inspection only. Usually damage is detected too late by present systems to allow the installation of protection measures and achieve maximum benefit. A regular survey for corrosion condition will allow employment of proactive maintenance measures rather than reactive ones.

The Intermodal Surface Transportation Efficiency Act (ISTEA) of 1991 mandates a Bridge Management System (BMS) be in place in all states by 1998. A key feature of a BMS is its ability to provide more and better information for planning and scheduling maintenance and rehabilitation. A program or sub-system that detects and monitors the corrosion activity within a structure, when interconnected with a parent overall condition survey and ultimately the BMS, will provide the bridge owner with knowledge of the condition of a bridge and the appropriate action necessary to correct deficiencies. Potential damage resulting from corrosion activity occurring on reinforcing steel may prompt maintenance action that would not have been considered under a system that did not include a corrosion condition survey. Knowing that the structure is in need of a corrosion protection measure may allow the owner to plan for its installation in conjunction with other maintenance. This results in the saving of time and money due to a reduction in number of structure or lane closures to traffic. It is planned maintenance rather than impulsive reactions that is strived for with use of a BMS.

Because there is not a cost-affordable substitute for sodium chloride as a deicing agent, chlorides will continue to accumulate in concrete and eventually cause corrosion if it has not already initiated the process. Calcium magnesium acetate (CMA) has been clearly identified as an effective deicing chemical which inhibits corrosion and is harmless to the environment. However, it was determined that if CMA was used as a deicer in Michigan, the cost of the material used in one year would be 9 times the total annual maintenance budget of the state (1). A system that detects and monitors corrosion will be needed for a long time in the future.

A corrosion assessment program will increase understanding of the corrosion process and will bring the users knowledge of the most effective means of slowing corrosion activity. The owners of the bridge will have a valuable tool to evaluate the effectiveness of maintenance strategies and protection systems. Data will be accumulated that should assist in determining which concrete surface sealants work better than others, their optimum rate of application, and their optimum reapplication intervals. Maintenance strategies such as the "wash and sweep" can be readily evaluated and compared with others when the program is established.

Justification for instituting a corrosion detection program is also seen in the capability of providing early warning of impending corrosion induced distress. If corrosion activity is detected or suspected, maintenance measures can be employed that will slow the rate of corrosion so that the life of the structure is extended. A large amount of maintenance action that has been deferred through the years because of a lack of funds is overdue. Knowing the location of the corrosion activity, however, may help prevent unnecessary or excessive rehabilitation of a structure. Partial repairs at locations that are detected by tests within a program may be all that is necessary to adequately salvage the structure and minimize the disruption of service. Therefore, unnecessary maintenance expenditures and subsequent loss of service may be avoided by employing a corrosion detection and assessment program.

Arguments follow for not establishing a program. One case presented against embarking on a program of this nature is that more structures today than before have corrosion prevention measures included by design. It should be remembered that not all of these measures are proven in the field, and a survey program run over time is the best way to evaluate their effectiveness.

2.3 JUSTIFICATION FOR NOT INSTITUTING A PROGRAM

One argument against launching a corrosion detection program is that it is possible that many of the structures found in the worst corrosion condition are those that are scheduled for replacement for other reasons. It may be reasoned that the bridges suffering the most severe corrosion induced damage are those that were built long ago with little concern for the ramifications of the chloride ion's penetration. Before embarking on a program for detecting corrosion, the bridge owner should research whether the deterioration from corrosion is primarily occurring on the structures that have exceeded their service life expectancy. Apart from their corrosion condition, many old structures are load zoned and are destined for replacement because of their obsolete and deficient load carrying design. With this in mind, the testing of many structures on low volume roadways may not be justified because the structures will be replaced as funds become available.

Another argument against exerting the effort to fund and train for a corrosion detection program includes the fact that, today, bridges are being constructed with ample consideration for corrosion prevention. The specification of better materials and construction practices make many of the bridges on the system today more capable of resisting corrosion than the older bridges. No longer are chloride-containing admixtures allowed in the concrete used in bridges. The combination of potentially reactive aggregates with certain cements is guarded against to reduce the likelihood of distress caused by alkali-silica reaction (ASR). The damage from ASR or any other primary damage to the concrete permits the intrusion of chlorides into the concrete that will promote the secondary distress of corrosion. Today's construction makes prominent use of fly ash in concrete that significantly reduces the permeability of the concrete and deters the ingress of chlorides. It may be argued that once the deteriorated structures, the majority of which are very old, are replaced, the second generation will last longer because they will be constructed with corrosion protection elements included by design.

Being aware of the threat of reinforcing steel corrosion has led to the development and use of improved concrete sealants that are more likely to stop chloride and moisture intrusion while allowing the concrete to transmit water vapors from within. It is known that sealants must be reapplied on a regular basis to be effective, and a maintenance program that includes regular reapplications of sealants should slow the corrosion process within all structures, and will preclude the need for a corrosion monitoring program. The cost of reapplying sealants may be equitable to the cost of regular testing. Therefore, it can be argued that reapplication should be done without testing to determine its necessity.

The value of testing the substructures in sea water can be realistically questioned because of the lack of rehabilitation alternatives feasible for concrete piers and piling. Corrosion of non-coated steel is almost certain at some time within the splash zone of all submerged substructures, and testing to determine the level of activity may eventually be considered pointless. If corrosion of substructures in sea water is imminent, and there are few alternatives for remedy, testing is not needed to detect nor indicate the need for maintenance action.

Further argument against adding another facet of inspection to the already extensive duties of condition surveys is that too many of the bridges in Texas are not easily tested by some methods used in this study. The technology for testing a large portion of the state's bridges is not available. The electrical tests that are the core of a corrosion assessment program are essentially useless on structures that contain epoxy-coated reinforcing steel. The dielectric coating on the steel does not allow for electrical continuity beyond the single bar on which the ground connection is made. Consequently, a large area of deck, for example, cannot be surveyed using the half-cell and rate-of-corrosion tests that are dependent upon achieving electrical continuity throughout the reinforcing steel mat. Because new bridges within the deicing zones are built with epoxy-coated steel, the application of the recommended testing regime will be limited on these structures. Fewer tests result in greater uncertainty when drawing conclusions. The performance of the above mentioned tests is difficult at best on decks with an asphalt seal or overlay. Covering decks with asphaltic concrete or seal coats seems to be an acceptable practice to provide aesthetics and a better ride and prevent corrosion. Unless the decks are uncovered, the full extent of testing cannot be accomplished. The technology exists to detect delaminations under asphalt coverings; however, this reverts to detecting damage after the fact. A final summary case against the institution of a major corrosion assessment program in Texas may be that the newer generation bridges will out-perform their predecessors, and there are too many bridges on which only limited testing is applicable.

2.4 SUMMARY

Immediately upon considering the implementation of a corrosion condition survey program, the owner of the system must determine how many bridges are at risk of suffering reinforcing steel corrosion and how many can be fully evaluated with the testing program. The age of structures should be considered because the merits of a corrosion condition survey will not be fully revealed until a data base is built and the bridges on the program have been tested two or more times. It may be determined that the remaining life of a structure is not long enough to receive the full benefits of the inspection program. The corrosion prevention attributes of epoxy-coated steel have not been fully established, but the use of the coated steel will continue because the cost of this material has decreased to the point that practically any benefit derived from its use is considered cost effective. Because all the recommended tests cannot be employed on structures that contain electrically insulated reinforcing bars, the results may be less definite and conclusive.

CHAPTER 3. FUNDAMENTAL THEORY OF THE CORROSION OF STEEL IN REINFORCED CONCRETE

3.1 INTRODUCTION

Corrosion is the destructive result of chemical reaction between a metal and its environment. While corroding, a metal essentially returns to a state of chemical composition very similar to that of the mineral from which it was extracted, and therefore has been termed "extractive metallurgy in reverse" (3). Corrosion of reinforcing steel embedded in concrete is almost certain to occur, and in order to prevent its detrimental effects, the designer, builder, and maintainer may find that it is necessary to install controls to force the corrosion activity to occur at an acceptable rate. To detect, monitor and slow the effects of corrosion, it is necessary to understand the corrosion mechanism, those factors that affect the tendency for corrosion to occur, and the kinetic aspects of the process that will determine the rate at which corroding steel dissolves.

3.2 CORROSION PRINCIPLES AND MECHANISM

Four basic components make up the electrochemical corrosion cell. For corrosion to occur, it is essential that each of the components — anode, cathode, electrolyte, and electron path — be present. When a metal corrodes in an aqueous solution, anodic and cathodic sites develop on the metal surface. Current is carried between the sites as electrons travel within the conductive path and ions migrate within the electrolyte.

At the site of the anode, electrons are given off, and metallic ions go into solution. Metal loss, or dissolution, therefore occurs at the anode. The loss of electrons and subsequent ion release are associated with the oxidation reaction which occurs at the anode. The positively charged ions released from the anodic metal surface combine with negatively charged ions released from the cathode. At the onset of corrosion, anodic and cathodic sites are generated on the metal surface, and current begins to flow.

The cathode consumes the electrons given off at the anode. The gaining of electrons is involved in the reduction reaction, that occurs simultaneously with the oxidation, or electron-loss reaction at the anode. Unless the electrons generated from the anode are consumed, the anodic reaction cannot continue to occur and the corrosion process will not continue. The most common cathodic reaction involved in the corrosion of reinforcing steel involves the reduction of dissolved oxygen. The cathode's release of negatively-charged ions which combine with the positively-charged ions released by the anode is seen in the schematic representation of a corrosion cell in Figure 3.1.

The electrolyte, or solution previously mentioned, is the transporter of the released ions. In the reinforcing steel corrosion cell, it is the concrete surrounding the metal that serves as the electrolyte, its transporting capabilities, or electrical conductivity, affected by the presence or absence of moisture and chlorides. Within the electrolyte, current is carried in the form of ions liberated from the anode and cathode. The total or net current in the electrolyte is always equivalent to the current carried within the metallic path by electrons alone (5).

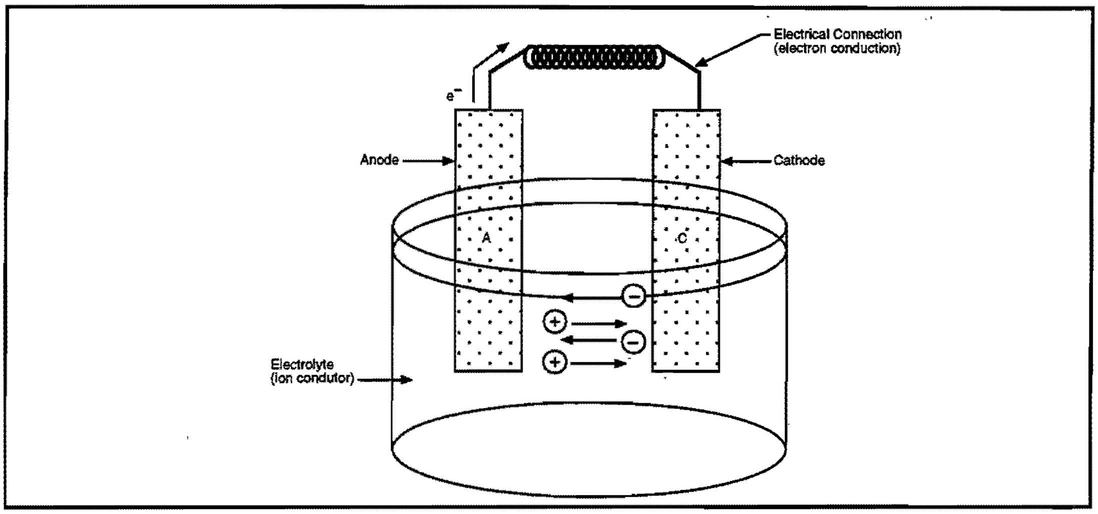


Figure 3.1. Components of an Electrochemical Corrosion Cell (4)

The electron conductive path between the anode and cathode is needed for a complete corrosion cell. When the anode and cathode are occupying the same piece of metal, the path is the metal itself. If the anode and cathode are not on the same metal, such as when the top mat of bridge deck reinforcing steel serves as the anode, and the bottom the cathode, the path for electron flow between them is supplied by the steel chairs and connecting wire ties.

For further analysis, the corrosion cell may be divided into two sub-cells or half-cells. Separate reactions occur within each of the anodic and cathodic half-cells. The anodic half-cell reaction in which the anode metal M corrodes and frees positively-charged ions into the electrolyte is described by the following oxidation reaction:

$$M \rightarrow M^{n+} + ne^-$$

where n is the number of electrons given off by the metal. The metallic ion, M^{n+} is released into the electrolyte and the released electrons flow through the conductive path to the cathode. In the corrosion or dissolution of reinforcing steel, the loss of iron in the anodic reaction is specifically described by:

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

Unlike the reaction at the anode, the metal is not involved in the reaction at the cathode. In the corrosion of reinforcing steel, the predominant cathodic reaction involves the reduction of water and oxygen and is given by:

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

Therefore, it can be seen that the amount of oxygen available at the cathode is important in driving the reactions within the corrosion cell. With the consumption of electrons , the cathodic half-cell reaction releases negatively-charged hydroxyl (OH^-) ions into the electrolyte.

The anodic and cathodic reactions occur simultaneously, and in metallic corrosion, the rate of oxidation equals the rate of reduction. Therefore, the total cell reaction is a combination of the anodic and cathodic half-cell reactions and is given as:

$$2Fe + 2H_2O + O_2 \rightarrow 2Fe^{2+} + 4OH^- \rightarrow 2Fe(OH)_2 \downarrow$$

The product of this reaction, ferrous hydroxide, is an unstable compound, which seeks to stabilize itself with yet another reaction with oxygen:

$$2Fe(OH)_2 + \frac{1}{2}O_2 \rightarrow H_2O + Fe_2O_3 \bullet H2O \downarrow$$

A product of this stabilizing reaction is then the familiar reddish brown rust. The oxygen used in the above secondary reaction at the anode is not to be confused with that required in the reduction reaction at the cathode.

Water within the concrete is needed for effective ion transport as well as for a reactant at the cathode. Consequently, keeping reinforced concrete as dry as possible is a prime consideration for the deterrence of corrosion activity. Providing a low permeability concrete medium prevents oxygen and water from infiltrating to the cathode for reaction, at the same time limiting chloride intrusion.

3.3 ELECTROCHEMICAL THERMODYNAMICS AND ELECTRODE POTENTIAL

3.3.1 Theory

When corrosion occurs on the surface of a metal, work is performed and absorbed within the corrosion cell. This work is represented by a change in the free energy, G, of the cell:

$$G = G\text{ (products)} - G\text{ (reactants)}$$

For a reaction to occur spontaneously, G must be negative, or the free energy of the reactants must be greater than the free energy of the products. The performance of work within the cell is indicated by a loss of free energy in the system. The change in free energy is directly related to the driving force of the reaction, as will be shown later.

Within each half-cell, there is a certain tendency, or potential, for a reaction to take place. The anodic half-cell undergoes a change in energy as electrons are given off. Likewise, as the cathodic reaction consumes electrons, the cathodic half-cell undergoes an energy change. The energy change associated with each of the anodic and cathodic reactions has a corresponding half-cell electrode potential, e_a and e_c, respectively, and:

$$E = e_a + e_c$$

where E is the potential or electromotive force (emf) of the cell.

3.3.2 The Half-Cell Potential Electrode

It is impossible to measure the value of a single half-cell electrode potential. It must be measured with the use of another half-cell that is used as a standard reference. The flow of electrons from the anode to the cathode coupled with the ionic flow from the cathode to the anode through the electrolyte forms an electric circuit within the corrosion cell. The driving force, or emf, measured in volts, is the potential difference between the anode and cathode. To measure only the half-cell electrode potential of either the anode or cathode, a standard electrode, one that has a relatively fixed value of potential regardless of the environment in which it is used, is employed to create and drive a reaction for which the potential is measured. In most instances, it is the anodic half-cell potential that is measured against the known potential of the reference electrode providing the cathodic reaction. The measured difference in potential between the standard and either of the half-cell electrodes is the usual means of quantifying the potential of any half-cell. Any change in the emf is because of change occurring in potential of the electrode under observation and not of the reference electrode.

Corrosion studies involving reinforcing steel commonly use the copper/copper sulfate electrode (CSE) as a reference to obtain the structure-to-electrolyte potentials which as discussed later, will indicate the probability of corrosion activity occurring within the reinforced concrete. The CSE is essentially a copper rod immersed in a saturated copper sulfate solution and not considered as accurate as other references such as the standard hydrogen electrode (SHE), the saturated calomel (SCE), or silver chloride electrodes. However, it has the distinct advantage of being resistant to shock, and its accuracy is considered adequate for the corrosion investigations of reinforced concrete. Figure 3.2 is a schematic representation of a copper/copper sulfate electrode in contact with an iron electrode in a solution of ferrous ions.

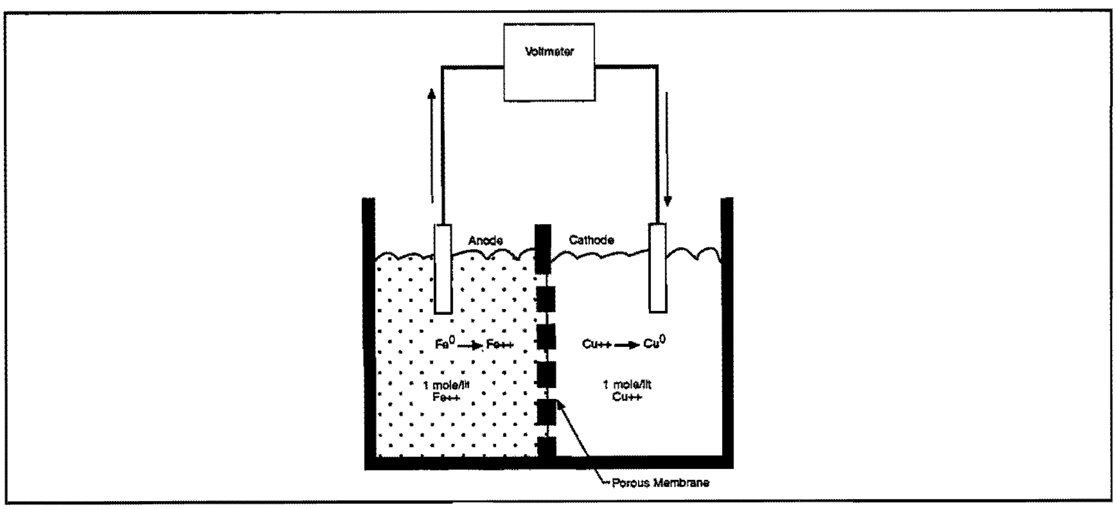


Figure 3.2 Half Cell Schematic (6)

When the potential is measured between the reference electrode, the CSE in particular, and the iron half-cell electrode in concrete, the concentration of Fe^{2+} ions in relation to the concentration of Cu^{2+} ions is considered. The Fe^{2+} ion, which is given off from the anodic areas of the steel, is found in the electrolyte surrounding the steel electrode. The Cu^{2+} ions are in the copper sulfate solution surrounding the copper electrode in the reference cell.

The half-cell potential survey as conducted in accordance with ASTM C876 "Standard Test Method for Half-Cell Potentials of Uncoated Reinforcing Steel in Concrete" (27) is a qualitative evaluation of the corrosion condition of plain steel bars in concrete. A survey as recommended by the ASTM procedure consists of placing the reference electrode in contact with the concrete at regular distance intervals at many locations to receive an indication of the location of the most highly anodic areas. By obtaining a voltage reading of the driving force between the reference electrode and the anodic reinforcing steel, there is an indication of the concentration of Fe^{2+} ions in the electrolyte, or concrete surrounding the embedded steel. By convention, the voltage readings are negative, and the voltmeter is connected between the electrodes so that negative values are normally read. From this voltage, or indirect reading of the ferrous ion concentration, the probability of corrosion occurring can be determined. ASTM assigns three categories of probability. If potential readings are:

- more positive than -0.20 V. CSE, there is less than a 10 percent probability that corrosion is occurring;
- between -0.20 and -0.35 V. CSE, there is an uncertain probability of corrosion occurring;
- more negative than -0.35 V. CSE, there is a 90 percent probability of corrosion occurring.

For the remainder of this presentation, and in reporting on half-cell survey results, the above ranges of potential readings will be termed Low, Uncertain, and High respectively, and the reporting of actual probability percentages will not usually be given.

The concentration of the ferrous ion within the concrete and in the vicinity of the steel is different at different areas of the concrete because the steel is more or less anodic to the reference electrode at those locations. The voltage readings differ because the Fe^{2+} concentration differs while the concentration of the cuprous ion within the CSE reference remains constant. The measurement of the potential of a particular area on a structure is an instantaneous look at the concentration level of ions and is an indicator of the corrosion activity probability.

Measuring the half-cell potential is a valuable tool for the detection and monitoring of corrosion activity, but the results should not be taken as a direct indication of the corrosion rate of the steel. Some researchers have suggested that the ranges of probability suggested by ASTM are not always reliable (7). Other tests, such as the rate-of-corrosion test, compliment the half-cell test, and together, a corrosion condition of a structure can be better identified.

3.4 ELECTROCHEMICAL KINETICS OF CORROSION

3.4.1 Rate-of-Corrosion Theory

The concept of corrosion tendency is based on principles of dynamic equilibrium, where the rate of reaction at the anode is equal to rate of reaction at the cathode. At equilibrium, there is no net current flowing to or from the surface of the metal. A net current causes one half-cell reaction to be accelerated while the other is decelerated. The amount of damage, or metal loss from corrosion, is directly proportional to the amount of current flowing through the cell.

It was stated previously that there must be a negative change in free energy in order for there to be a spontaneous reaction within the corrosion cell. However, a large negative value of G may or may not be accompanied by a high corrosion rate. If, however, G is positive, it would be safe to assume that the reaction will not take place at all. If G is negative, the rate that the reaction occurs may be fast or slow depending on various factors, one of which is the amount of dissolved oxygen available at the cathode.

Faraday's Law expresses the relationship between the mass of metal loss, m, and the current, I, flowing in the cell:

$$m = \frac{Ita}{nF}$$

where t is time, a is the atomic number, n is the number of electrons transferred in the reaction, and F is Faraday's Constant (96,500 coulombs/electron transferred). A further derivation of Faraday's Law shows that the corrosion rate is proportional to the current density, i, or amount of current per unit area:

$$r = \frac{ia}{nF}$$

The corrosion rate, r, is usually given in an amount of metal loss per unit time, such as mils per year (mpy) or milligrams per square decimeter per day (MDD).

When current flows to or from an electrode's surface, its potential is altered by an amount dependent upon the magnitude of the current. Potential change caused by the net current to or from an electrode is called overpotential, or polarization.

3.4.2 Rate-of-Corrosion Testing

To determine the rate of corrosion, the amount of current flowing in the corrosion cell is measured. Polarization, or overpotential was defined earlier as a shift in potential caused by the flow of current. When the potential of either of the half-cell electrodes changes, there is anodic polarization $_a$, or cathodic polarization, $_c$, occurring separately or simultaneously:

$$c = c \log \frac{i_c}{i_o} \qquad\qquad a = a \log \frac{i_a}{i_o}$$

where i_o is the exchange current density or the reversible rate at equilibrium, and i_c and i_a are the current densities of the cathodic and anodic reactions respectively. $_a$ and $_c$ are the Tafel Constants for each of the half-cell reactions. These variables and the polarization concept are described further schematically in Figure 3.3.

When reinforcing steel is corroding in a moist environment with oxygen available, both the anodic and the cathodic half-cell reactions occur simultaneously on the surface of the steel. Each reaction has its own exchange current density and half-cell electrode potential. Because of their occurring on the electrically conductive surface of the steel, the two half-cell potentials, e_a and e_c cannot co-exist separately. Each must change to a common intermediate potential at a value termed the corrosion potential E_{corr}. As the anodic reaction, $Fe \rightarrow Fe^{2+} + 2e^-$ and the cathodic reaction $O_2 + 2H_2O + 4e^- \rightarrow 4OH^-$ polarize on the same surface, the half-cell electrode potentials change by $_a$, and $_c$, respectively until they become equal at E_{corr}. At E_{corr}, the rate of anodic dissolution, i_a, is the same as the corrosion rate i_{corr}. The corrosion rate is inversely proportional to the polarization resistance, R_p.

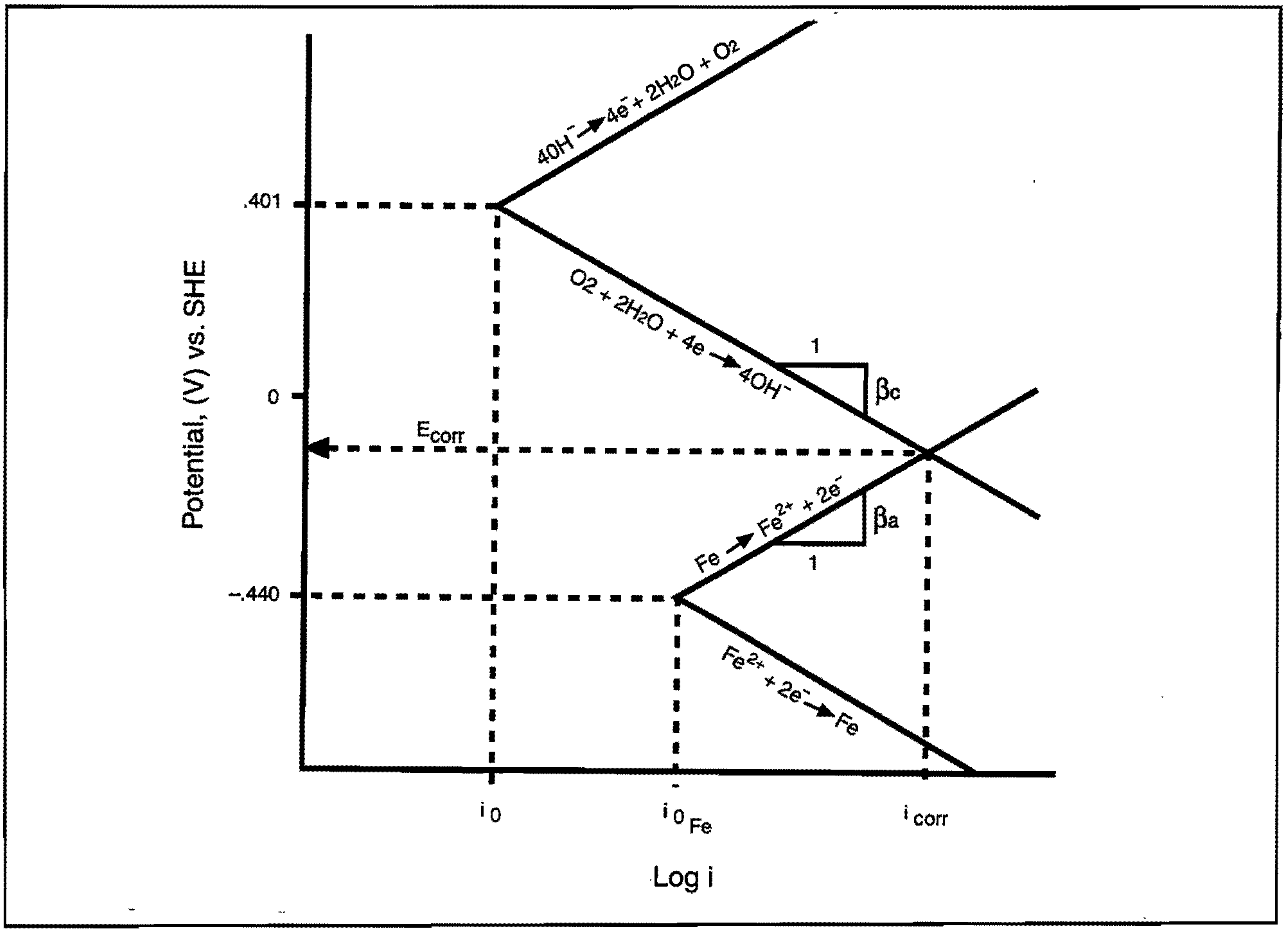


Figure 3.3 -Polarization Diagram for Reinforced Concrete

The Stern-Geary equation shows the relationship between i_{corr} and the amount of current required to polarize to a new potential:

$$i_{corr} = \frac{\beta_a\beta_c\Delta I}{2.3(\beta_a+\beta_c)\Delta E} = \frac{B}{R_p}$$

Here, B is a constant that is estimated from experience with reinforced concrete to equal 41 (8).

The most common test for the rate of corrosion in the field employs the measurement of polarization resistance. This method is also often called the linear polarization technique. The test involves the application of a small current to a small area of embedded reinforcing steel, and a corresponding voltage and response is recorded (I). Essentially, if a high current is required to polarize to the desired overpotential levels required by the testing procedure, the corrosion rate is high. Details of corrosion rate testing with one device that employs the linear polarization technique are found elsewhere in this presentation. The standard test method is as specified by the manufacturer of the testing device.

The rate of corrosion test, like the half-cell survey, is an instantaneous evaluation of one facet of the corrosion condition of a structure. To gain more knowledge of the rate, the test must be run again at regular intervals, and the results evaluated over time. There is a reliable relationship between i_{corr} values and the dissolution rates of metals. For iron, 1 A/cm^2 is equal to a penetration rate of 0.46 mpy (3). Also from experience, some thresholds of corrosion rate for reinforced concrete have been established to assist in relating i_{corr} to deterioration (8):

- i_{corr} less than 0.20 mA per sq. ft. - no corrosion damage expected;
- i_{corr} between 0.20 and 1.0 mA per sq. ft. - corrosion damage possible in the range of 10 to 15 years;
- i_{corr} between 1.0 and 10 mA per sq. ft. - corrosion damage expected in 2 to 10 years;
- i_{corr} in excess of 10 mA per sq. ft. - corrosion damage expected in 2 years or less.

As insoluble corrosion products form on the surface of the steel, they may slow the release of Fe^{2+} ions at the anode and limit the supply of oxygen to the cathode. Therefore, it is possible that corrosion rates will decrease with time because of this activity-limiting aspect.

3.5 THE ROLE OF CHEMICAL ELEMENTS AND CONCRETE PROPERTIES

3.5.1 Chlorides

The corrosion of reinforcing steel proceeds at a much faster rate in the presence of chloride ions. Although it is agreed that chlorides are the primary destroyers of the protective environment that concrete provides, the mechanism by which this is done is a matter of debate. There is also a lack of agreement on the amount of chlorides necessary to initiate corrosion activity. Not all chlorides within the surrounding concrete are free to cause corrosion, and this causes further confusion in understanding and predicting the action of this element.

It is agreed that chlorides decrease the resistivity of the electrolyte, and thereby allow an easier flow of ionic current. It is believed by most researchers that chlorides are instrumental in destroying the passivating film that protects the steel in the highly alkaline environment that concrete provides. The means by which the surface of the steel is depassivated is a point of debate among some researchers (9). The most common theory is that chlorides reduce the pH of the concrete medium causing the destruction of the thin iron oxide film on the surface of the steel. Others have stated that pH is not reduced, and that the chloride ion actually breaks the film by migrating through (10). Yet another theory is that chlorides form a complex ion which pulls the Fe^{2+} ions off the steel surface (11).

The chlorides may be introduced into the concrete from admixed or external sources. External sources, such as deicing salts and sea water, are the major contributors to chloride contamination of bridges. Admixed chlorides will be uniformly distributed within a concrete mixture and are potentially less harmful than the alien or external source chlorides which are commonly differentially distributed. The uneven distribution of chlorides causes concentration cells to form, which increases potential between areas on the steel.

Some of the admixed chlorides combine with cement during the hydration process and become unavailable to depassivate the steel or initiate corrosion. Proper curing will tie up the admixed chlorides further and render them ineffective in starting the corrosion process. The chloride ions must be unbound, or free to be catalysts for the corrosion process. However, because admixed chlorides can be bound at one point in the life of a structure and then free at another point, (11) the total chloride ion content is commonly sought in testing.

3.5.1.1 Testing for Chloride Ion Content: The total chloride content is commonly referred to as the acid-soluble chloride content. The acid-soluble test was used in this study and is faster and more reproducible than the water-soluble test. The specific ion probe technique is used to determine the total chloride content of powdered concrete samples that have been digested in a 15 percent acetic acid solution. The test, as run according to the manufacturer's instructions, is similar in some respects to the ASTM C114 and AASHTO T260 standards. In this test, four powdered concrete samples are collected at 1.25 cm intervals from the surface to 5.08 cm of depth. The powdered samples from numerous locations are processed with the acetic acid digestor, and results in units of percent by weight of concrete are generated in order to obtain a chloride depth profile.

3.5.1.2 Chloride Ion Thresholds for Corrosion: The threshold of chloride content to initiate corrosion continues to be an item of debate. The threshold is commonly sought in terms of the critical Cl^-/OH^- ratio, with consideration that the chloride ion destroys the passive layer and the hydroxyl ion repairs it (12). This chloride threshold is defined by the American Concrete Institute (ACI) for various exposure conditions and is expressed as a percentage of the cement in the concrete mixture. ACI 318-89 (13) sets the limit on the acid soluble chloride content at 0.20 percent of the weight of cement. Because it has been estimated that approximately 75 percent of the total chloride is water-soluble, ACI sets the limit on water-soluble chlorides at 0.15 percent of the weight of cement. If a six-sack mix of concrete is assumed, these limits equate to 0.66 and 0.50 kg/m^3. Some researchers believe the ACI code is not conservative enough in its specified chloride limits (14). For the purposes of this study, the threshold will be considered as 0.71 kg/m^3

to coincide with the threshold set in earlier portions of this research project (15), which is closely in accordance with the ACI limits for fresh concrete.

3.5.2 Permeability of Concrete

The ease by which a material can flow through concrete is a measure of its permeability. Concrete material selection and consolidation practices affect the permeability. Low water/cement ratios and sufficient cover as well as proper placement and finishing procedures are defenses against high permeability. Counter acting high permeability is done to reduce the intrusion rate of water, oxygen and chlorides.

3.5.2.1 Permeability Testing: The AASHTO T277 Rapid Chloride Ion Permeability Test procedure has been discussed in previous presentations of work in this research project. This test is run in the laboratory to measure the permeability of cores secured from the sample area. A study conducted as part of the Strategic Highway Research Program (SHRP) evaluated and recommends another procedure for which coring is not required (16). Permeability testing is not recommended as one of the procedures usually performed in a corrosion assessment program for the following reasons:

(1) The permeability varies considerably across the area of a deck due to differences in finish and consolidation practices;
(2) The validity of the results from the AASHTO T277 procedure are questionable (17, 18) and the test is time consuming;
(3) Keeping coring to a minimum was an objective of the study in developing an efficient program.

Unless the method evaluated by SHRP later proves to be a worthy component of a test program, regular permeability testing and monitoring is not recommended nor anticipated.

3.5.3 Cracking, Delaminations, and Spalls in Concrete

When corrosion products build on the surface of the corroding steel, the cross sectional area of the rebar increases causing pressures within the concrete that may exceed the strength of the concrete and cracking, then delamination, and finally spalling will occur. Cracking may be a direct result of the damage caused by corrosion or it may be caused by damage from concrete shrinkage or live loading. Longitudinal cracks are not generally structural, but are usually caused by the restraint of subsidence of fresh concrete. Transverse cracking is a form of structural distress and is commonly found when concrete or steel beams or girders are supporting the deck. Transverse cracks on steel beam decks are usually uniformly spaced over the entire length of the deck. Concrete beam or girder structures however, tend to have closely spaced transverse cracking over the negative moment areas of the deck and only a relatively small amount of cracking in the positive moment areas. Differential movement between the girder and slab, especially in steel I-beam structures, caused by thermal volume changes is also a factor in the development of transverse cracks. Whatever the cause, cracks on the surface of the deck can allow chloride intrusion at an increased rate; but these cracks, if left unattended, may become sealed with debris and may not be a permanent pathway for chlorides into the concrete (15).

The build-up of corrosion products on the top mat of reinforcing steel eventually causes the concrete layer above each bar to be pushed up and away. Cracks that occur and radiate from the corroding bar are many times interconnected with those of adjacent bars and a layer of concrete cover is released from the deck first as a delamination, and then a spall. Delaminations will allow the intrusion of chlorides into the deck and will keep moisture conditions closer than that of the spall to the optimum required for corrosion, since the spall allows drying to occur. The bridge inspector is immediately alerted to a problem structure when spalling is seen. No electrical-type testing is usually necessary to ascertain that the structure exhibiting spalling is one that deserves attention immediately.

Scaling, though not as problematic as spalling or cracking, is not a direct result of corrosion damage. An inadequate air void system along with improper drainage are causes of scaling. Due to the lack of a proper stress relief system, the paste freezes and disintegrates. Like cracking, scaling provides a means for chlorides to reach the reinforcing steel quickly and decreases the time-to-corrosion period of a deck.

Clear cover is now required by TxDOT to be at least 5.08 cm (19). Drying shrinkage cracks will not generally extend to the cover depth, and are not considered as serious a problem as the structural or subsidence cracking mentioned previously. The depth of cover, then, is important to reduce the intrusion of moisture, oxygen, and chlorides and is one aspect of the deck cross section that is most critical to prolonging time-to-corrosion.

When spalling is observed on bent caps, the inspector is alerted to the deck being susceptible to the same. The cap's horizontal top surface captures the salty water runoff from the deck, and because of a slower drying process occurring underneath the deck than on its surface, corrosion activity is likely to be more advanced on the substructure.

A delamination survey should be performed during inspections on old decks with deicing exposure. This test method usually consists of sounding out the deck by tapping or dragging a steel instrument over the deck and listening for a differentiation of sounds as the loose concrete is encountered. The use of a chain drag (broom) is recommended because of its simplicity to use, its reliability, and the speed of travel as compared to tapping a hammer or dropping a steel bar. The survey cannot be performed, however, when ambient noise levels are high such as from passing traffic or nearby industrial activity. This survey is seldom used for this reason on heavily traveled roadways. Other more sophisticated and expensive survey methods such as pulse radar are available (20), but were not tested in this study.

CHAPTER 4. PROGRAM DEVELOPMENT FROM OTHER MODELS

4.1 INTRODUCTION

Currently, Texas has a bridge condition rating system that assists the state in determining which structures are to be rehabilitated or replaced. This system is similar to many of those employed by other states. However, due to the passage of the Intermodal Surface Transportation Act of 1991 (ISTEA), Texas will be changing the system in the way it gathers data, processes the data, and the way it uses the results. ISTEA mandates that by 1998, all states must have an approved Bridge Management System (BMS) implemented in at least its initial stages (21). Texas' present system, like those of many other states, will not qualify as a BMS, as defined by the legislation.

A Bridge Management System is a decision-making assistance program for the legislative, administrative, and technical personnel involved with infrastructure construction and maintenance. Specifically, it is:

> a rational and systematic approach to organizing and carrying out the activities related to planning, designing, constructing, maintaining, rehabilitating, and replacing bridges vital to the transportation infrastructure. A BMS should assist decision makers to select optimum cost-effective alternatives needed to achieve desired levels of service within the allocated funds and to identify future funding requirements (22).

Texas current rating system, the Bridge Inspection Appraisal Program (BRINSAP), is not capable of optimizing or notifying the agency of detailed funding requirements based on the condition of the bridges.

Implementation of a BMS will allow the user agency the following benefits:

1. An increase in knowledge is gained of the condition of bridges at a network level.
2. A list of bridges in prioritized order needing Maintenance, Rehabilitation, and Repair (MR&R) actions is produced.
3. A cost estimate for the projected MR&R activities is for the life cycle of the improvements.
4. The performance of a structure as well as its deterioration is projected, by accumulating historical data and data that quantifies the effectiveness of MR&R strategies.
5. The planning for the use of limited funds is rationalized.
6. Minor maintenance scheduling is improved.

A BMS provides improved information and a new method for bridge management, to not only technical personnel, but also to those that make administrative as well as legislative decisions.

A new system that meets the ISTEA standards for a BMS that Texas and other states will adopt is called PONTIS. The development of this program was funded by the Federal Highway Administration and was designed by a consulting firm especially for the state of California. With PONTIS now on line there, it is available for adoption by other states. PONTIS will be instituted in Texas in its original form (23). Although changes to the program are imminent, TxDOT will make them after becoming familiar with operations under this new system.

4.2 CONDITION SURVEYS OF OTHER PROGRAMS

PONTIS is an optimization and planning system for the network level (24) which incorporates “dynamic, probabilistic models and a data base to predict maintenance and improvement needs, recommend optimal policies, and schedule projects within budget and policy constraints”. This BMS differs from BRINSAP primarily because it requires information that will describe the condition of a bridge in more detail. Furthermore, PONTIS will use the survey data to generate information that is derived from built-in decision rules. It contains models that operate on the network or entire state bridge inventory, rather than on an individual bridge. Basically, PONTIS consists of a data base and three models, each containing other sub-models.

The data base contains the condition survey and inventory information. Some data will be transferred or derived from present BRINSAP information, but additional data will be required and gathered during the regular condition surveys. The Maintenance, Repair, and Rehabilitation (MR&R) optimization model, the first of the basic models, uses maintenance costs and prediction models to arrive at optimum maintenance strategies for as many as 160 bridge elements. The second of the basic models is the Improvement model which prioritizes potential actions such as widening or replacement, and bases decisions on level-of-service standards and user-cost savings. The third basic model is the Integration model, which combines the Improvement and MR&R optimization results from their respective models, into a single program. The first two models generate needs and provide information to prioritize those needs. The Integration model then schedules the projects with budget considerations. Figure 4.1 is a schematic representation of the PONTIS components.

The most prominent feature of PONTIS or any BMS is its optimization capability. One of its main improvements over the present rating system is that it will not combine all information gathered and process it into one rating, thus reducing the value of the information. The present system provides the rating as information to decision makers. Fault is found with the present system, however, because the rating does not produce an accompanying action required to correct deficiencies. PONTIS quantifies the condition of each element of a bridge by considering the condition state it is given by the inspector. For each bridge, the percentage of each element in each condition state is determined. Shown in Figure 4.2 is an example of a listing of condition state descriptions for a single element, in this case a deck of a particular type. The condition state that is chosen by the inspector may reflect some consideration for corrosion, as the example does in Figure 4.2. However, just as the condition survey in use at the present time bases condition on the observance of distress after it has occurred, the PONTIS inspection does likewise. A corrosion condition survey recommended herein should provide information that is used to determine within

PONTIS the corrosion condition states of various elements. This, of course, will require a modification of PONTIS with the user producing a menu of condition states applicable to such parameters as exposure severity, chloride content, half-cell potential survey results, and rate-of-corrosion information.

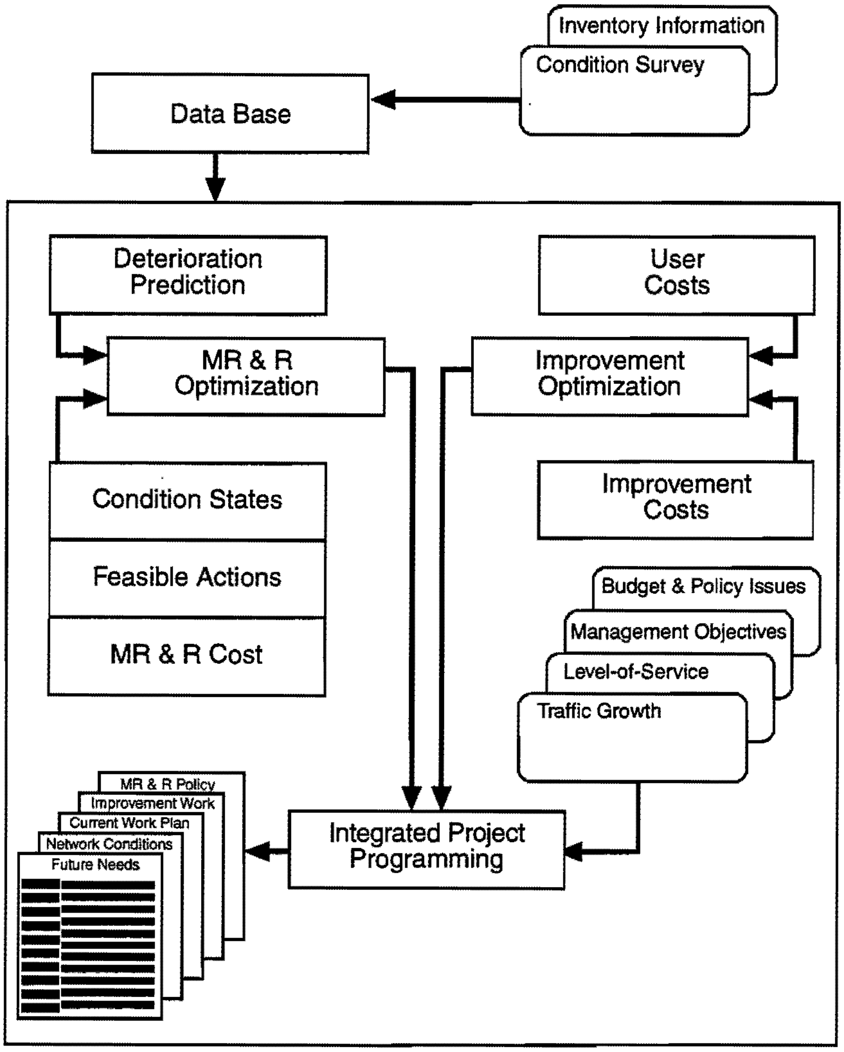


Figure 4.1 PONTIS Major System Components (25)

4.3 CURRENT RESEARCH

In the conduct of this study, the work of others was examined to assist in providing a means of integrating corrosion condition information into a program for monitoring, controlling, or preventing deterioration. The program should use the information to help bridge maintenance personnel to choose alternatives for prevention or repair. This presentation will not give detailed instructions for the modification of PONTIS, but will provide information about how to gather the data for inclusion in the data base.

Concrete Deck — Bare

Units: EA but unit costs in $/SF

This element defines those concrete bridge decks with no surface protection of any type that are constructed with uncoated reinforcement. Report the condition state that represents the condition of the entire deck.

Condition States Descriptions

1. The surface of the deck has no repaired areas and there are no spalls/delaminations in the deck surface.

Feasible actions: DN Add a protective system

2. Repaired areas and/or spalls /delaminations exist in the deck surface. The combined distressed area is 2% or less of the deck area.

Feasible actions: DN Repair spalled/delam area Add a protective system

3. Repaired areas and/or spalled/delaminations exist in the deck surface. The combined area of distress is more than 2% and less than or equal to 10% of the total deck area.

Feasible actions: DN Repair spalled areas Repair spalled areas and add protective system on entire deck

4. Repaired areas and/or spalls/delaminations exist in the deck surface. The combined areas of distress is more than 10% but less than or equal to 25% of the total deck area.

Feasible actions: DN Repair spalled areas Repair spalled areas and add protective system on entire deck

5. Repaired areas and/or spalls/delaminations exist. The combined area of distress is more than 25% of the total deck area.

Feasible actions: DN Repair spalled areas Replace deck and/or add protective system on entire deck

Figure 4.2 PONTIS Condition States Example (24)

4.3.1 Strategic Highway Research Program

The Strategic Highway Research Program (SHRP) developed a procedure for assessing the condition of bridges under project C-101. A series of 8 manuals produced at the conclusion of this recent study provides detailed directions for performing bridge condition surveys. Though the emphasis of the study was on detecting and analyzing corrosion of reinforced concrete structures, a complete distress survey method resulted. Figure 4.3 is a description of the procedure in flowchart form.

Work in this study involved only the corrosion-related testing; therefore, only parts of the procedure shown in Figure 4.3 are recommended for implementation. The reasons for not recommending certain parts of the SHRP procedure are briefly explained. The Initial (Baseline) Evaluation Survey as defined in the flow chart has little direct connection to determining the current state of corrosion activity within a structure. Some of the information such as concrete strength can be gathered from construction records, and performing testing at the present is not necessary. The rebar cover survey was tried but discontinued because of a concern for a lack of accuracy of the results. The accuracy of the pachometer or covermeter used in this study was not considered adequate for documenting steel depth. SHRP recommends that permeability tests be run and suggests a field test method that does not require coring. Although the SHRP procedure for measuring permeability may have potential for implementation at a later date, reasons for omission of any permeability test are based on experience with the AASHTO T277 method. Those reasons are delineated in Chapter 3 of this report.

This study focused primarily on the Subsequent Evaluation Surveys within the SHRP flowcharted procedure. PONTIS does not contain many of the prior-to-distress-type tests shown in this portion of the SHRP recommendation, but it is within the scope of this study to provide guidance for the inclusion of tests that detect corrosion activity before major damage occurs.

The SHRP procedure has one other survey, that for Special Conditions. In this survey, the testing effort is modified to exclude the electrical tests performed in the other parts of the survey, such as the half-cell and rate-of-corrosion tests for structures that have rigid deck overlays or polymer impregnated concrete. Several bridges with these materials were surveyed in this study, and initial indications are that the electrical tests can be performed with valid results. It is recommended that efforts be made to continue testing structures of this nature with the same survey procedures used on normal concrete.

4.3.2 Federal Highway Administration Demonstration Project No. 84

This demonstration project tested equipment and methods of testing used in the performance corrosion condition surveys. A manual that was produced as a result of the project is an excellent reference for reinforced concrete corrosion testing. Similar to the SHRP C-101 manuals, there is information on tests other than those discussed in this report. The Federal Highway Administration, at the time of this writing, is offering some corrosion detection equipment to states on a loan basis. The rate-of-corrosion meter used in this study was acquired in this manner. The bridge owner considering implementation of a corrosion testing regime should consider the use of loaned equipment to assist in choosing devices for purchase.

4.3.3 Previous Work in This Research Project

Sherman (15) recommended procedures to be included within a corrosion testing program after evaluating some state-of -the-art testing methods in the initial stages of this research. Although not all of the tests that were evaluated in his work are recommended for every bridge in service, the accounts of work of predecessors in this project are a valuable information resource. The thrust of the latest research in this project, that of the field testing to develop a procedure for a detection and monitoring program, emphasized the optimization of the testing efforts.

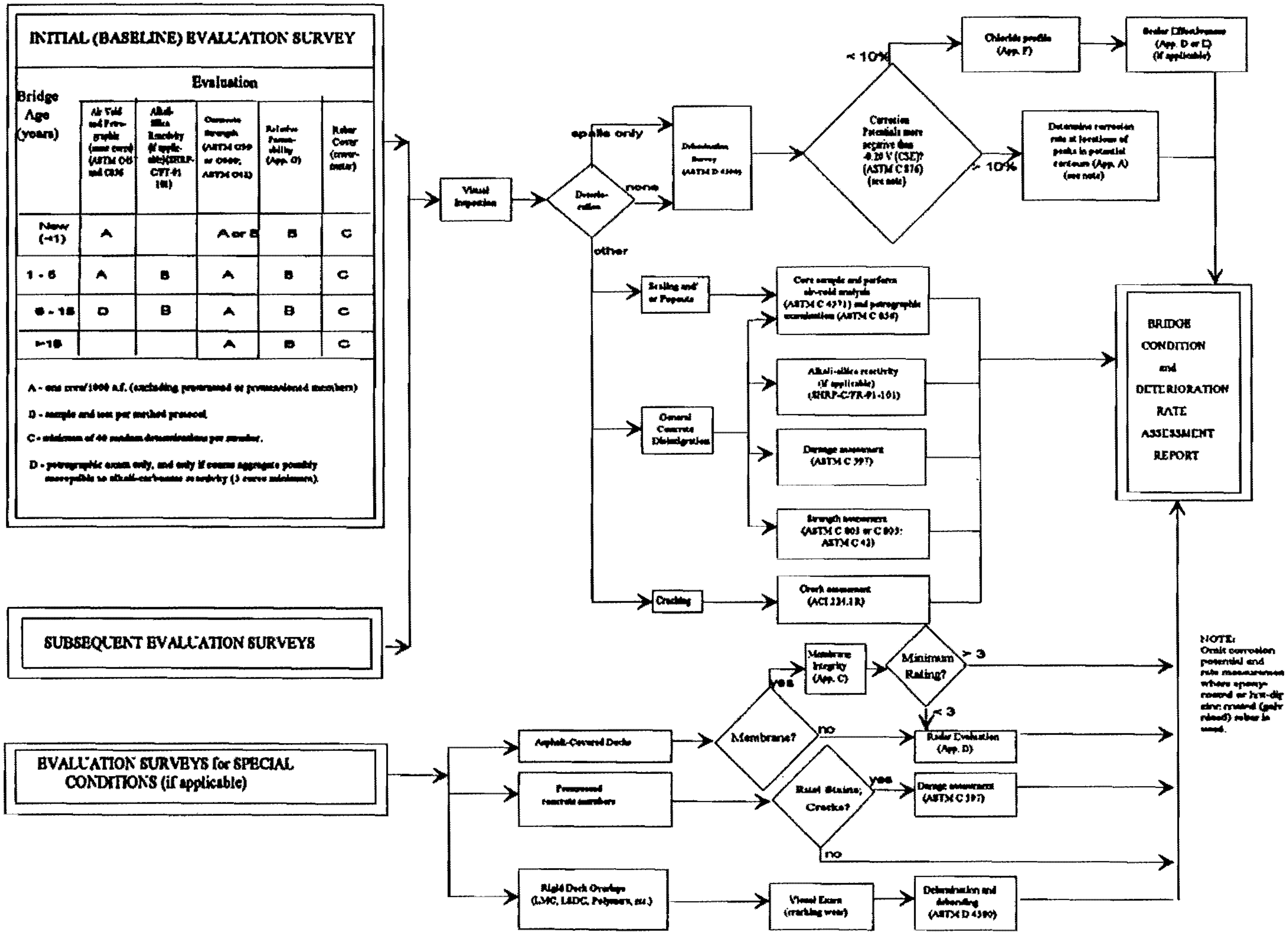


Figure 4.3 Description of the procedure in flowchart form

CHAPTER 5. TESTING PROCEDURES AND CASE STUDIES

5.1 INTRODUCTION

A procedure for performing a corrosion condition survey is presented in this chapter with the primary goal of establishing a program that is efficient in providing useful information on a regular basis to the structure owner. This procedure, as it becomes the basis for a corrosion monitoring program, should be accommodated easily into a Bridge Management System to provide another tool for planning maintenance, rehabilitation, and replacements. If the testing regime suggested does not become a part of the BMS, it should remain as a tool to assist in determining the feasibility of widening or repairing a structure whose corrosion condition is suspect. The tests within the set described will make up the majority of a simple and reliable means of evaluating the effectiveness of corrosion protection measures especially surface sealants, corrosion inhibitors, and special concrete mix designs. With information and confidence gained from recent experience with these tests during trials in the field, this collection of tests is recommended as the most efficient way to begin to assess the condition of the state's bridges exposed to chloride induced corrosion. If it is found that any of the procedures recommended herein fail to be a cost effective means of determining the corrosion condition of a bridge at any particular time, the procedures should be altered to make the most efficient use of funds and personnel and avoid any unnecessary interruption of service of a facility.

5.1.1 Field Survey Sites

In the conduct of this study, the structures shown in Table 5.1 were sampled and tested for corrosion activity. In choosing test sites for this study, various exposure environments, concrete materials, protective systems, and structural elements were considered. The structures surveyed are referenced in case studies presented throughout the remainder of this report.

In addition to the testing of existing structures, some new bridges were tested with the half-cell survey and a rate of corrosion evaluation. Electrical connections to the uncoated reinforcing steel of the deck were made prior to the placement of the concrete. These bridges will be monitored for corrosion activity over time to evaluate the effectiveness of the various protective measures used and the maintenance measures employed to prevent corrosion. No coring for the grounding of electrical test instruments will be necessary. Table 5.2 lists the structures that are fitted or are proposed to be fitted with the necessary connections for easy instrument hook-up.

5.1.2 Testing Equipment

A thorough list of equipment and supplies needed for the conduct of condition surveys is included in a previous report of this research project (15). During the course of field work of this study, the researchers discovered a need for some improved features of some of the test devices. As the survey team becomes familiar with its work, and equipment is improved or new technology becomes available, due consideration should be given to changing the equipment to maximize efficiency in testing and reporting results.

Table 5.1 Bridge Sites Visited During Field Testing

Location	Length/ Description	Age (Years)	Exposure Condition	Component Evaluated	District	Contact Person
I-20 at US 283 UP, Callahan County	252'/ Slab & Girder Unit	28	Deicing Salt	Deck, Abut.	Abilene	Marvin Rodgers
FM 2085 at Bull Creek, Scurry County	230' Cont. I-Beam Unit	37	Deicing Salt	Deck, Abut	Abilene	Otis Jones
I-45 at Cottage St. UP, Harris County	177'/ I-Beam Units and Spans	29	Deicing Salt	Deck	Houston	Gene Day
LP 494 at Caney Creek, Montgomery County	1067'/ I-Beam spans	45	Deicing Salt	Deck	Houston	James Hebert
I-610 at Wallisville Rd. OP, Harris County	180'/ Prestr. Conc. Beam Spans (Dense Conc. Overlay)	33	Deicing Salt	Deck	Houston	Gene Day
US 67 at Pecos River, Pecos County	312'/ I-Beam Spans	60	Saline River	Piers	Odessa	Charles Webb
SH 18 at Pecos River, Ward County	275'/ I-Beam Spans	59	Saline River	Piers	Odessa	Charles Webb
SH 273 at FW&D RR, Gray County	195' Cont. I-Beam Unit	31	Deicing Salt	Deck	Amarillo	Martin Rodin
SH 152 at Bear Creek, Hutchinson	182'/ Pan Girder Spans.	34	Deicing Salt	Deck	Amarillo	George Moore
I-27 NBL at Keuka St., Lubbock County	207' Cont. Prestr. Conc. Beam Unit (Poly. Impr.)	15	Deicing Salt	Deck	Lubbock	Ron Seal
I-27 SBL at Keuka St., Lubbock County	207' Cont. Prestr. Conc. Beam Unit (Dns. Conc. O'lay)	15	Deicing Salt	Deck	Lubbock	Ron Seal
SH 350 at Owens St., Howard County	750'/ Prestr. Conc. Beam (Poly. Impr.)	16	Deicing Salt	Deck	Abilene	Cary Lloyd
US 181 Nueces Bay Causeway, Nueces County	9637'/ Prestr. Conc. Beam	31	Sea Water	Deck Underside, Piling	Corpus Christi	Thomas Bell

Table 5.2 New Structures Fitted With Connections for Future Testing

Location	Contact Person	District	Date of First Survey
I 35 SB Frontage Rd. at Onion Creek, Travis County	Willie Haverland	Austin	Summer 1992
US 77 at Chambers Creek Relief, Ellis County	Hal Stanford	Dallas	Fall 1993
FM 1082 (Two Structures), Jones County	Joe Higgins	Abilene	Spring 1994
SH 361 at Redfish Bay / Morris and Cummings Cut, Nueces County	Thomas Bell	Corpus Christi	1996
FM 1541, Randall County	Martin Rodin	Amarillo	Summer 1994
LP 335, Potter County	Martin Rodin	Amarillo	Summer 1994

Experience gained from the field tests prompts a recommendation for improvement in operational features of two of the test devices use in this study. Before any equipment is purchased to begin a corrosion detection program, responsible personnel are urged to consider the features of the half-cell testing device and the rate of corrosion meter and to seek enhancements that provide accuracy and ease of reporting results. It is believed that half-cell potential mapping will be achieved easier with a different data acquisition unit or software application than used in this study. Because the half-cell test is considered the most important test of the corrosion condition survey, state-of-the-art equipment should be sought. It will benefit the bridge owner to investigate, for example, a half-cell device that will generate its own potential plots without manual manipulation of spread sheet data. These devices are commercially available but were not purchased for this relatively short-term, small-scale study.

The rate-of-corrosion meter used in this study is only one of several on the market that have been tested and evaluated. In this portion of the study, there was no time to become familiar with the operation of more than one piece of rate testing equipment. The operational shortfalls of the device used are discussed elsewhere and the potential user is urged to examine the SHRP evaluation of corrosion rate testing meters.

5.1.3 Sampling the Structure for Testing

The survey procedure is recommended with the premise that sampling, or testing only a representative or critical portion of the entire bridge, will be performed to derive an assessment of the corrosion condition. To determine the size of the sample, the inspectors must first determine the work production capabilities of the testing team and the amount of time the bridge deck is eligible to be taken out of service. The sample size is also affected by demands of the half-cell test.

The sample is generally chosen to be a certain width of an independent slab while running its entire length. To avoid confusion, one sample, later described as a grid, should not continue across an open joint. The half-cell testing cannot continue across the joint without an attachment of the voltmeter to another ground connection. Generally, the first day's testing on a structure should occupy all the time allotted, and the sample size should only be limited by:

1. The number of hours of daylight available for testing, or
2. The length of time the traffic control team is available to keep the test lanes closed, or
3. The length of time that the nature and distribution of traffic permits a lane closure, or
4. The difficulty of performing tests. When it is required that additional preparation be made on a structure such as removing an asphalt seal to allow chloride content sampling or half-cell potential testing, less time can be devoted to testing. Likewise, if testing is performed from an inspection platform or snooper, the inspection and testing will progress at a comparatively slow rate.

Depending on the results of the testing of the first day and on the size of the structure, a second day's testing may be necessary to obtain a clearer picture of the structure's corrosion condition. Often, it will not be until after the first day's testing that the sample size will be finally decided. If the inspector judges that the results of the delamination or the half-cell potential surveys are not representative of the entire structure, he may call for further sampling and testing. The sample areas should then be chosen both in size and location to further reflect the condition of the entire structure.

All tests of the set are performed and the results evaluated with respect to the same sample area. Within a sample gridded for reference purposes, chloride content sample collection is done at locations to reflect the areas most critical for chloride build-up and intrusion. For chloride content sampling, the low side of a deck may seem to be the most critical for locating high concentration areas. This has not been fully established, however, and although due consideration is given to sample the lowest elevations on a deck, the tests are not concentrated there, but usually distributed over the sample area.

On subsequent surveys, at least fifty percent of the sampling should result in a repeat of the previous survey. By performing a survey on a portion of the structure that was tested previously, trends are drawn from results, and the effectiveness of any protection measure applied since the last survey can be evaluated. Likewise, the need for corrosion protection can be predicted and planned for if the results of the latest survey indicate an increase of corrosion activity.

5.1.3.1 Case Studies: Table 5.3 shows relationship between sample and total area of the bridge decks surveyed in this study. At the conclusion of the survey of each structure, the amount of sample area shown was considered adequate to obtain an assessment of the overall corrosion condition.

Table 5.3 Sample Area vs. Total Roadway Area of Surveyed Bridges

Structure	Roadway Area (Sq. Ft.)	Sample Area (Sq. Ft.)	Sample Area / Rdwy Area, %
I-20 @ US 283 UP	11,066	6656	60
Lp 494 @ Caney Crk.	24,530	4357	18
SH 350 / Owens St. OP	48,064	5120	11
I- 27 @ Keuka St. NBL	8694	4968	57
I- 27 @ Keuka St. SBL	8694	4968	57
I-45 / Cottage St. UP	4796	2968	62

1 sq. foot = 0.304 sq. m

In the case of the I-20 / US 283 Underpass, the beginning of the first day's survey involved obtaining half-cell testing results that indicated relatively good condition on one end of the structure. However, as testing progressed in another sample area, more delaminations were found, and half-cell testing indicated a relatively poorer deck condition. At the end of the first testing day, it was decided to sample more of the structure and continue testing at another time. Further testing indicated that delaminations were more severe at the centerline of the structure, than at the low side, and that more delaminations and corrosion activity were occurring on one end. Further sampling was done after it was judged necessary by the inspectors. Judgment, based on knowledge of the corrosion process and testing, as it improves with experience, should be exercised by inspectors to determine sample size.

The Lp 494 structure required two days of testing to complete the survey. The total sample area shown in Table 5.3 represents seven separate sample areas each requiring marking and coring for grounding the instruments.. The SH 350 survey involved testing in only three sample areas. The testing was accomplished in one day because no significant variance in half-cell test results was evident as the survey progressed in different parts of the deck. More deck area was tested in less time on SH 350 than on Lp 494 because the SH 350 structure contained long continuous deck units instead of short independent spans.

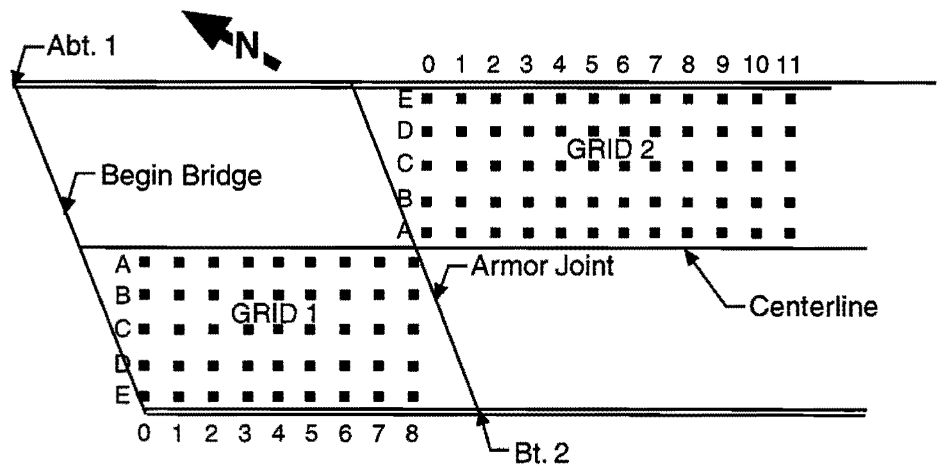
5.1.4 Reference and Grid System

A reference system is needed to provide a means of identifying the locations of test sample sites and results. The use of a standardized reference system, usually in the form of a gridded layout of points, is essential in examining the results of various tests with respect to their locations within the sample. Standardization will streamline effort and speed the very important but time-consuming pre-survey task of establishing a grid. Whenever possible, the beginning of the bridge should serve as the baseline for the longitudinal reference of the sample. This practice allows the grid numbering to increase along with the permanent bridge stationing found and referenced in the as-built plans. Though it was not done in this study, the roadway centerline of the bridge should serve as the baseline for the transverse reference, because the centerline serves as the baseline for transverse positioning in many TxDOT surveys and layouts. Ideally, the grid should be laid out in the same manner with the same conventions for all grids and for all structures to promote uniformity and familiarity for all that use and process the survey data and information.

Points in a grid at a predetermined spacing are assigned a coordinate to provide a unique identity to each point. To accomplish this, beginning with the letter designation "A" to the column of points nearest the roadway centerline, columns are assigned a letter designation as they are established toward the traffic rail. The rows of points in the grid are numbered beginning with row "zero" on the end of the test area closest to the beginning of the bridge. It is recommended that the first or last row of grid points be approximately 0.6 m from an armor joint because the half-cell readings may be affected disproportionately by the exposed corroding armor steel. Likewise, in the transverse direction, the outside column of points should be no closer than approximately 0.6 m from a concrete bridge rail. Regardless of the reference system used, some sort of standardization in laying out the grid can save time and confusion to the inspectors, the record keepers, and the users of the information.

Regardless of whether a grid pattern for a particular sample is set to a standard, the grid must always be adequately referenced to known points. This reference should always be clearly shown in all depictions of results such as in the half-cell potential contour plots. Shown in Figure

5.1 is a typical grid system for a skewed bridge deck showing some pertinent known points as references.



Grid 1: Point AQ 6 feet 9 inches off Begin Bridge; Column A 2 feet off centerline
Grid 2: Point AO 2 feet off joint; Column A 2 feet off centerline

Figure 5.1 Typical Grid Reference System

A grid with points set on 1.2-m intervals in each direction has been used satisfactorily in trials of the testing program and is as suggested by ASTM C876 for the half-cell potential survey grid. That testing procedure states that the spacing of the grid points should be controlled by the results received from the half-cell survey and that points may be spaced so that the minimum spacing generally provides at least a 100 mV difference between readings (27). During the course of this study, the 1.2-m grid spacing was used throughout for bridge deck evaluations. Spacing of points was generally less for grids on vertical members such as columns, webwalls, and piling, simply because the testing was being done to detect the physical location limits of corrosion activity. Decreasing the spacing to less than 1.2 m would increase the amount of time needed to grid a sample area and would increase the time required to perform the half-cell survey. The decreased spacing and greater number of data points may be advantageous toward achieving a better defined and more accurate potential plot than one that was derived from a grid with greater spacing. The 1.2-m grid is suitable when it is necessary to estimate to the nearest foot the boundaries of delaminated areas.

The marking of the grid was done with a 100-ft. tape and spray paint or chalk or lumber crayons. Three persons laying out the grid worked best for achieving speed and accuracy, with two holding the tape and one moving along the tape applying the marking at the 1.2-m intervals. A standard, adopted by the testing team during this study, was to establish a square grid pattern, or one that has its rows running perpendicular to the centerline on all decks, with or without a skew.

This is done for the purposes of attaining standardization, as the skew need not usually be accounted for in any of this testing.

The inspector is always at liberty to receive information from the survey that is not recorded. If he desires to get additional information to reinforce results of the official survey, the half-cell probe may be placed at points other than the established grid points. Readings obtained from the additional points may give the inspector additional confidence in the accuracy of the final report.

The importance of referencing the grid to known points is emphasized because there are occasions when the survey team tests a structure for which there are no available as-built plans. On occasions, the team may survey a bridge without being fully prepared. Without a review of the as-built plans, the grid may not be established according to the standard recommended previously. Proper referencing to points whose locations are known after the as-built plans are received, should prevent confusion, even though grid stationing may not coincide with the standard.

It is not always practical to lay out an entire grid on a structure. For example, when an attempt is made to survey a deck that is covered with an asphalt seal coat, random locations may be chosen for testing to represent the entire deck. On three occasions during the course of this study, an attempt was made to gain some insight into the corrosion condition of an asphalt sealed deck. Areas of asphalt were removed at randomly chosen points in order to perform chloride sampling and half-cell testing. At the conclusion of testing, each point was individually measured for reference of location.

With respect to asphalt-sealed decks, the sample size can be expected to be much smaller than those of plain deck surveys. The time required to remove and replace the asphalt leaves less time for testing. This forces the inspector to draw conclusions about the corrosion condition of an entire structure from a relatively small number of tested points. The other options in regard to an asphalt covered bridge are to forego testing entirely, or spend more time uncovering all points for usual grid spacing while inducing increased damage to the integrity of the seal.

In the placement of the grid reference system, white paint is usually the preferred material for marking the point locations. The paint will render the measuring tape generally useless for any applications other than grid marking. Some grades of lumber crayons produce markings that have a tendency to wash off during the repeated wetting of points during the half-cell survey. A steel tape is preferred over a cloth tape because of its weight, since wind from the atmosphere or passing traffic makes the use of a lightweight tape difficult. Care must be taken, however, to insure that the steel tape does not fall into open slab joints since kinking and breaking is almost certain. For this reason, the testing team of this study was unable to maintain a steel tape for a period long enough to justify its extra cost. It is recommended that a cloth tape be used to accomplish a grid whenever possible.

Practically all bridge decks surveyed as a part of this study had a grid reference system that began or ended on a open slab joint. The usual practice of placing the first row of points on the joint proved to be a mistake when armor was embedded in the edges of the slab. The half-cell potential readings here were usually high as compared to the remainder of the sample. Because the high readings were likely influenced by the exposed corroding plate steel in the immediate vicinity, the results of the survey for that row of points is suspect and probably not representative of the reinforcing steel in the majority of the slab. Half-cell readings at points at an armor joint were generally disregarded in this study and should not have been taken.

5.1.4.1 Case Studies: On a survey of the Bull Creek Bridge, FM 2085, a grid was not used as a reference system because the deck was covered with an asphalt seal. The seal was old and did not exhibit adhesion to the old deck surface, so the seal was easily removed in randomly selected areas of about 15.2 cm in diameter. Uncovering the deck with this size of openings in the seal coat gave surveyors space over the point to set the sponge of the half-cell apparatus and the chloride content collection plate in complete contact with concrete deck. The chloride content sampling was done after the half-cell reading was obtained to prevent the resulting 5.1-cm deep hole from affecting the readings.

The asphalt seal was so easily removed that the inspectors had to take care to insure that a larger patch than necessary for testing was not removed. As a result of this membrane covering the deck showing no adhesion, it is evident that if the seal were torn or opened, moisture would easily penetrate and move horizontally under the seal. The visual inspection of the bridge easily revealed staining and cracking on the underside of the deck. The survey was performed on the deck to gain experience with the performance of a survey under difficult testing conditions and to determine if decisions about the corrosion condition of a structure could be made from a survey that was limited in scope.

Twenty random points were selected throughout the west bound lane extending the entire length of the bridge. Also, a single column of points spaced at 1.2 m was spotted in the gutter line of the test area. This line of points was not covered by asphalt, making the lowest point of the deck cross section easy to test.

A more adhesive seal was found on two other structures on which a survey limited in scope was performed. On SH 273 at the abandoned FW&D Railroad, a new seal coat of less than three weeks of age was encountered. The adhesion of the asphalt to the concrete was good and only twelve locations were cleared for chloride content sampling. The testing team was hesitant to remove and patch any more than the above areas because of concern for damaging the newly placed seal. It was decided to test for chloride content only, and depending on the results of that testing, an expanded survey could proceed at another time. The seal coat construction was so new, that thorough testing with many areas of asphalt removed would harm the new construction and lower its effectiveness at a very early age. The covered decks make necessary the step-wise process of corrosion condition surveying which will be discussed further elsewhere.

The SH 152 structure showed staining and spalling underneath the pan girder superstructure. The structure had been accumulating numerous repair patches on the deck. The deck had asphaltic concrete of approximately 2.54 cm or less in thickness on all but the areas that were recently patched with a rapid-set concrete. On this structure, three grids of reference were established, but the points surveyed were generally the bare patch locations. Figure 5.2 shows the reference system and half-cell readings in reference to the patch areas. After reviewing the chloride content, half-cell, and rate of corrosion testing results, it was decided that sufficient testing was done on this structure to confirm the implications from the visual inspection. Corrosion is active in many portions of this deck, chloride contents are higher than the threshold set, and the rate of corrosion is high enough to cause damage in 2 to 10 years.

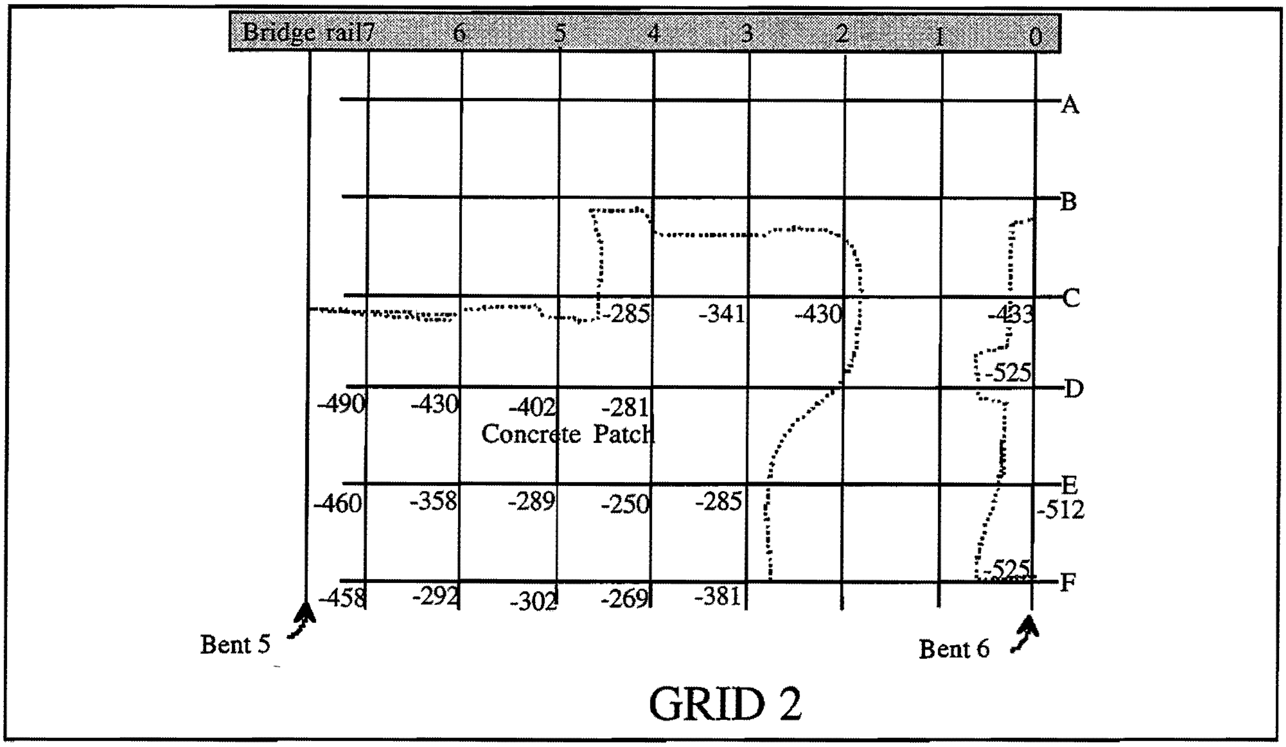


Figure 5.2 Random Point Survey Results on SH 152 Structure

5.2 THE HALF-CELL POTENTIAL SURVEY

The half-cell potential survey is the most important procedure used in determining the corrosion condition of plain steel reinforcement in concrete. This test can tell investigators more about the corrosion activity within a structure than any other single test. The test is one that gives immediate results, is readily applied to a large surface area in a rapid fashion, and serves to indicate how extensive remaining tests of the survey need to be carried out. If funding or personnel restraints allowed only a portion of the recommended program to be established, the half-cell testing would be the first test recommended for inclusion. If conditions on the day of the testing were such that only one test could be run on a structure, the half-cell test is recommended.

It must be emphasized that not all active corrosion occurs rapidly and not all corrosion activity produces red rust. The half-cell test indicates the tendency for corrosion to occur and could therefore cause too much concern for activity that is occurring at a slow and acceptable rate. However, the most important reason for performing the test is to alert the bridge owner of activity before it is damaging to the concrete and detectable with the eye .

Based on empirical relationships between the voltage readings obtained and the visually observed levels of corrosion activity on specimens in the laboratory, this test indicates to the surveyor the likelihood of corrosion occurring. The rate of dissolution of the steel is not known from performing this test, so to optimize the amount of information gained from the condition survey, the half-cell potential survey is complimented with the rate of corrosion test. The half-cell test indicates to the investigator the areas that the rate tests should be performed. Because the value of the potential indicates the probability of corrosion activity occurring at the time of the test, the half-cell survey is recommended to be part of every condition survey performed to present a corrosion activity profile over time.

5.2.1 Equipment

ASTM C876 specifies the properties and capabilities of the equipment needed to perform half-cell testing. To promote expeditious data gathering and easy and efficient reporting of results, this paper will recommend additional features than the minimum specified by ASTM. The equipment used in the half-cell survey will vary based on the user's preference and budget. The reference cell purchased for this study was a single probe device that had two features that were particularly valuable in gaining quality data in a rapid fashion. One feature is a data acquisition unit that records information taken at regular intervals. The other is a self-contained prewetting system that conveniently and neatly applies water onto the concrete surface immediately beneath the probe.

The voltmeter is housed in a hand-held unit that also contains a computerized data logger, the first feature that is recommended for the primary half-cell testing device in any program As the reference probe is placed over a prewetted grid point, the operator punches the recording trigger each time the digital reading stabilizes. This feature of the half-cell equipment is most important in allowing one person to both operate and record data quickly, so that the size of sample area can be maximized. This device has the capability of recording data from a regular layout of points and storing the data until it can be downloaded into a microcomputer for further processing using commercial software. Data can be referenced to points on the grid by typing in comments on the unit's keyboard. This referencing of data, however, slows the data gathering process considerably and is only done periodically on surveys with standardized grids. The referencing by comment is helpful when data points off the regular grid are inserted within the data matrix and are marked for later extraction.

A second feature of convenience that is recommended for purchase with the primary half-cell testing device is attached prewetting capability. With the half-cell probe used primarily in this study, a surfactant bottle encapsulated the reference cell and an attached sponge transferred the surfactant to the concrete surface immediately beneath the probe. Because the prewetting aids in obtaining stabilized readings quickly, this capability is essentially a time saving feature.

A second half-cell reference probe should be on hand to periodically perform accuracy checks against the primary testing device. It serves a backup in case the primary unit malfunctions. In consideration of budget, it should not necessarily contain the additional features that the primary unit has. The second half-cell testing device in the possession of a testing team should be physically smaller than the first piece in order to be used on caps and deck undersides and other locations in where working space is limited.

Data was downloaded daily at the completion of testing and was immediately placed into a file within a commercial spreadsheet software program where it was arranged and sorted as needed. The data can easily be stored on floppy disks and manipulated for plotting and summarizing results. There are data acquisition units available with software applications that perform potential mapping immediately and no arranging or manipulation of the data is necessary after downloading. A system of this type was not used for this research effort because of the extra cost involved with purchasing this technology. However, the purchase of these capabilities should be explored. The extra cost involved in purchasing equipment that produces reports faster than the currently used equipment may be offset by time savings realized by the users.

Reference cells are generally the probe type, or those that employ a tube electrode cell in single or multiple configurations. The one used in this study was a single probe reference cell that is capable of obtaining a reading only at a single point at any one time. Multiple probe testing devices are not necessary for bridge deck half-cell surveying. For those occasions when readings at close intervals are desired, multi-probe testers are designed to save time by obtaining multiple readings within a 1.8 to 2.4 m width with each placement of the device. This close spacing of readings inches apart as afforded by such devices is not considered necessary nor useful in most bridge surveys. ASTM C876 suggests a usual 1.2-m spacing of points, because of the relatively small chance that concentration cells occur and go undetected on a bridge deck. The use of multi-probe devices will require more maintenance and will present more transportation difficulty than the easily filled and carried single-probed tester.

Another half-cell testing device is the wheeled electrode which has a small CSE cell within a hollow wheel that is rolled over the concrete to obtain readings of the potential at small distance intervals. These wheels are also mounted on a bar in multiple arrangements. Designed to increase the speed and decrease the interval at which readings are taken, this equipment must obtain a very rapid stabilization of the reading as the wheel passes over the test area without stopping. This project was able to obtain a single wheeled electrode on a loan basis for a very short time period. After a very limited test of the device, its ability to accomplish stability of the readings as it moves over the concrete surface was not proven. Since prewetting of the concrete is usually required, and when potential readings are never obtained instantaneously even with the single probe placed over a soaked sponge, the validity of results from a moving wheel that employs a strip of wetted felt (28) to gain electrical contact was found to be questionable. Although trials of at least one potential wheel (29) resulted in general acceptance of this type of device for corrosion surveys, the use of this device effectively on tined bridge surfaces is especially questioned where adequate prewetting and electrical contact is difficult. Before wheeled devices are considered for purchase, it is recommended that the user be assured of obtaining results comparable in accuracy to a single tube probe.

5.2.2 Grounding the Voltmeter

The voltmeter must be electrically connected to the reinforcing steel to complete an electrical circuit when the probe is placed on the surface of the concrete. To accomplish this, a small area of reinforcing steel is exposed, and contact is made by connecting a wire between the voltmeter and the steel. When surveying a bridge deck, an area on the top mat of steel is exposed for grounding. Sufficient concrete is removed to allow the inspector to make the contact on the steel with any one of several connection methods. For deck surveys, a coring machine is used to remove a 10-cm core of concrete to the depth of the steel. On vertical faces, a hammer drill was used with a 2-cm bit to drill 2 or 3 holes to the depth of the steel. A chisel bit attached to the hammer drill was used to complete the removal of the cover concrete. The chisel bit is also needed to complete the removal of concrete around the steel that was exposed by the coring. Enough steel should be exposed to allow a clip to achieve solid contact with a single steel bar or prestressing strand. If coring is not available or preferred, a hammer drill with proper attachments will suffice for exposing the steel for a ground connection. If coring is done to expose the steel, the cores should be marked and kept as samples of the concrete for possible testing at a later point in time. These samples may be necessary for the performance of other tests that may provide information about a particular distress such alkali-silica reactivity (ASR) or sulfate attack.

The area of concrete removal can be minimized if a self-tapping screw is inserted into a hole drilled into the mild steel to bring the clip connection point closer to the concrete surface. Methods of attachment other than with the spring loaded clip, include using a pair of modified vice-grip pliers that contain a banana clip receptacle.

Every independent slab or grid should have at least two grounding locations to allow the inspector to check for electrical continuity throughout the mat of steel surveyed. Without continuity throughout the surveyed unit, the survey would be valid for only the areas that are electrically continuous with the bar connected to the voltmeter. The inspector may not realize he is taking readings over steel that is not continuous with the ground, but a lack of stabilization as compared to the grounded readings, should be an indicator. If steel is uncovered at more than two locations in each survey area, a better assurance of having electrical continuity throughout the grid can be obtained.

5.2.2.1 Case Study: The inspector should consider minimizing the number holes placed in the structure to lessen the chance of moisture and chloride intrusion to the steel after patching. Coring is usually difficult when concrete containing siliceous river gravel aggregates must be penetrated. From limited experience, it is estimated that coring may take two to four times longer in river gravel concretes than those made with stone aggregates. Although more holes would expose more steel for continuity checks and therefore more assurance of continuity, only two ground locations were usually provided for operations on bridge decks of this study. With the two point test as the indicator, it is believed that all points on all grids were electrically continuous.

If a deck survey within a continuous unit or single span can only be completed by switching traffic to different portions of the survey area, the wire and connection cannot be allowed to be damaged by traffic. Therefore, when the survey area will include both sides of centerline, or lanes that may be alternately tested then reopened, consideration should be given to providing a grounding location that may be used throughout the survey area, but will remain out of reach of potentially damaging traffic. If sufficient wire is available, the wire can extend from the connection over or through the traffic rail, under the structure to the test side of the bridge. There will be occasions, however, such as on surveys of narrow two lane structures, when new ground connections must be must be made when traffic is switched and surveying continues on another lane.

At the completion of testing, all cored or drilled holes should cleaned and prepared for patching. The survey team should be responsible for the patching because of their knowledge of the location of all holes. For this reason, the testing team is less likely to omit a repair. The patch material should be easy to mix and apply and capable of adhering to the old concrete on the sides of the hole. Within the study, on horizontal surfaces, a fast-setting, small coarse aggregate mix was used. The patch material may be a commercially-available mixture, or may be a hand-mixed grout to which latex is added to improve adhesion. The sides of the holes should be primed with the latex to further promote the patch material adhering to the old concrete. Vertical faces must be patched with a material that is applied with a tube gun or syringe to displace the air that becomes trapped in the hole. Using a trowel or spatula to apply the material to the hole usually results in the presence of trapped air causing a major void in the patch. Viscous epoxy has been tried and is expected to provide a satisfactory patch provided that the air from within the hole can be displaced. Not tried in this study, but having potential for success on vertical surfaces, is a tube-gun-applied silicone sealant.

On new structures built with uncoated reinforcing steel, a structure can be fitted with grounding connections that will make coring to expose grounding sites unnecessary in the future testing of the new bridge. The wire is attached to the top mat of reinforcing steel and allowed to protrude outside the deck surface where it can be attached to electrical test instruments as needed in the future.

On US 77 at the Chambers Creek Relief, Ellis County a 97.5-m continuous prestressed beam unit was built with black steel to allow this project to study the effectiveness of the linseed oil surface sealant and various reapplication strategies. The remainder of the span was built with epoxy coated steel. The uncoated steel unit had four wiring connections installed to accommodate future half-cell and rate of corrosion testing. This slab was constructed with permanent metal deck forms and the effects that this method of construction may have on the corrosion process may be evaluated.

Two of the connections were made by drilling a hole into the steel and inserting a self tapping screw. The wire to which a crimp type end connector was attached, was grounded to the steel by tightening the screw down on the connector. A wire grounding clamp used in making the two other connections is an acceptable substitute in some situations for the screw connection and is easier to install. A sketch of this connection is shown in Figure 5.3.

The clamp connection is done in consideration of convenience for the installer when project personnel are not able to be on-site at the opportune time to drill and screw into the steel. Ideally, all grounding connectors should be of material identical to the reinforcing steel, which makes the steel screw type connection preferable over the ground clamp. Steel clamps could not be located for purchase and a clamp of another material will have to be used. A die-cast zinc clamp is readily available but is likely to corrode at a different rate than the steel because of galvanic effects. However, the zinc clamp, for this same reason, is preferable over a brass clamp.

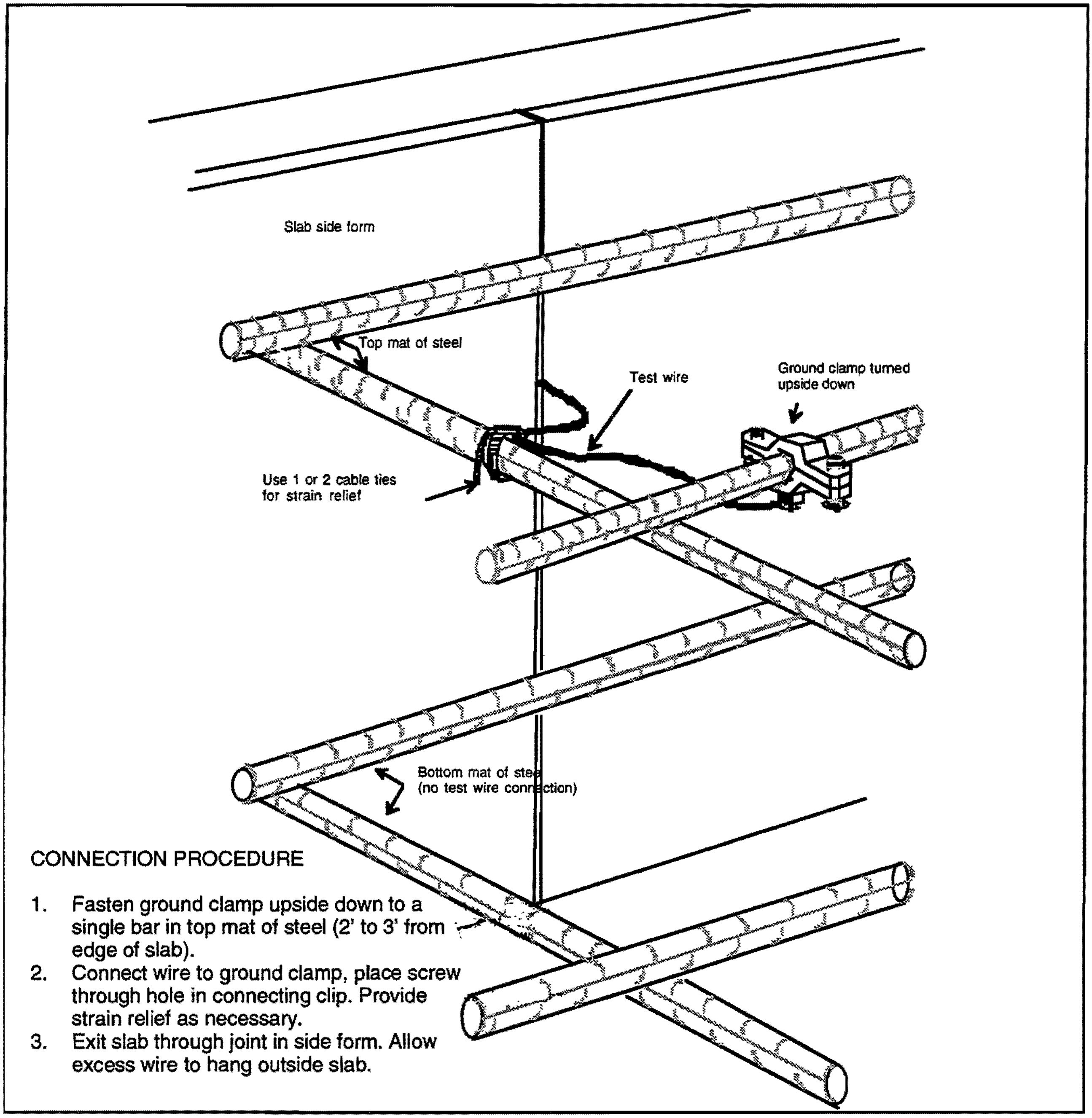


Figure 5.3 Half-Cell and Rate of Corrosion Permanent Grounding Connection

5.2.3 Checking for Electrical Continuity

To perform a check for electrical continuity throughout the survey area, ground contact is made to the voltmeter and a stabilized reading is obtained and recorded for any single point within the survey grid. The test probe is held in contact with the concrete at that point and not moved until the grounding connection can be moved to the second and each of the other ground locations. Continuity is ascertained when the readings differ by less than one millivolt (27) as the ground connection is moved among all ground points. The location of each ground point or steel exposure should be recorded so that on subsequent studies of the same grid, the electrical connections can be made at the same locations.

The grounding connections for the rate-of-corrosion testing that may follow the half-cell survey should be made at the same locations as the half-cell test. It should not usually be necessary to expose more steel for connections to the rate test devices.

5.2.4 Stability of Readings

Once the ground connection is made and electrical continuity is known for the survey area, the condition of the concrete is examined for its ability to provide a stable potential reading. If the electrical resistance of the concrete is too great, or if there are stray currents flowing through the steel being tested, a stable reading will not be achieved. To lower the resistance of the concrete, wetting for a time before reading is necessary. The surface of the concrete is essentially brought to a moisture condition that promotes intimate contact between the reference probe and the concrete. ASTM C876 stipulates that the concrete be prewetted if a voltage reading at a particular point does not remain within 20 mV for a five minute period. In this study, the concrete was always prewetted and a soapy-watered sponge was always placed between the reference probe and the concrete. Prewetting is recommended because it is easily accomplished by a team member in conjunction with other assigned duties. Obtaining stability was difficult on two occasions, but the survey continued, which is contrary to the ASTM C876 recommendation.

5.2.4.1 Case Studies: On the Nueces Bay Causeway, the readings on the underside of the deck did not stabilize sufficiently to meet the ASTM criteria, but all were fluctuating consistently in a low negative value range. The same range of readings was obtained on the I-610/Wallisville Road structure. Observing and recording these readings did give an indication of the corrosion activity of the structure and precludes the inspector from returning to the structure at another time when the cause of fluctuation may not exist. The cause of the fluctuations on I-610 is unknown, but it is believed that the high traffic in this urban area could more likely produce stray currents than other environments. The problem with instability on the causeway is believed to be a lack of wetting which is difficult to achieve on the underside of a deck.

5.2.5 Repeatability Check

The repeatability or the ability of the operator to obtain consistent readings is checked by performing an exercise before the survey for record commences and again after record testing is completed. Readings are taken on ten random points within the grid before surveying for record.

After the survey on the grid is complete, the 10 random points are read again. The second reading of each point should be within 20 mV CSE of the first reading in order for the record survey to be considered valid (15). In this study, the record survey was done again if more than one of the random point readings were not consistent. The data from both record surveys was then examined for major differences and adjusted by deleting the values of either survey that obviously did not fit. This exercise serves to give the inspector information on the quality of his data gathering effort.

5.2.6 Deck Surveys

After exposing the steel for grounding, a team member becomes available to assist with the prewetting of grid points. Prewetting the points was always done in the study to aid in obtaining stability of the readings quickly. The prewetting was applied with a garden pump sprayer that contained water only. Soap was used with water as a surfactant in the container surrounding the reference electrode in accordance with the ASTM procedure. The points may require wetting three or more times to gain sufficient moisture for stability and to keep the surface wet until the readings are obtained. The inspector taking the half-cell readings should not wet the points, because his switching duties may distract him from his primary task of gathering accurate and consistent data, and will unnecessarily delay the completion of the survey.

The actual conduct of the survey is accomplished by moving down one column of points at a time. The readings are recorded in the data logger, but each is seen to be stable before the recorder is triggered to document the reading for the point. The inspector should insert comments to annotate locations on the grid periodically, such as when he moves to another column . In this study, the comments were helpful in arranging the points for plotting after they were downloaded in the spreadsheet file used by this particular data acquisition unit.

The bent caps under open joints in the deck may be surveyed with a half-cell test, but the use of a smaller voltmeter probe may be more convenient. For grounding, the longitudinal bars that are larger than 1 inch (2.5 cm) are easily located. Connection to this steel will require a large clamp or the modified vice grip pliers for connection. Caps under continuous slabs should not be usually susceptible to corrosion and will not generally require testing.

The inspection team should be aware of factors and conditions that affect the voltage readings in the half-cell potential survey. Although some of the factors create only small affects, the inspector should be cognizant of them when evaluating different readings between surveys taken on a particular area at different times.

With increasing depth of cover, the potential readings are increasingly more positive. Most cover depths in bridge decks are approximately 4 to 8 cm. The standard reference cell used in surveys of this type is designed with the capability of obtaining readings over areas of typical steel cover. As long as the cover depth does not differ greatly from these normal expected ranges, the effects of cover depth should not cause concern for an application of correction factors. An important point for the surveyor to remember is that the potential measured is of the steel nearest the reference cell. The half-cell potential survey indirectly measures ion concentration. Therefore, to get an accurate indication of the concentration at the most critical level, the reference cell is placed nearest the steel most likely to be the most anodic. For this reason, bridge decks are usually not surveyed from the bottom, because the top layer of steel is most likely to be anodic. The top mat of steel usually becomes depassivated before the bottom mat because chlorides from deicing salts reach that steel first. If the reference cell was placed against the bottom surface of the deck, the reading would indicate potential influenced primarily by the bottom mat of steel, which is not a true indication of the worst steel corrosion condition. Although a survey from the bottom of the deck would avoid the traffic disruption problem involved with the testing of most bridges, this method is not preferred for the reasons given above.

The electrical resistivity of the cover layer will likewise cause a difference in readings, but like the cover depth, no expected difference has been quantified. It is expected (6, 7) that the potential lines emanating from the steel are diffracted by a highly resistant layer such as a dense concrete overlay, causing the voltmeter to register lower voltage readings.

Surveyors should be aware of any source of electrical current that could stray into reinforced concrete and affect the results of electrical tests. Because of this stray current, such as may be produced from electrical lines attached to the structure, the half-cell and the rate of corrosion testing may be affected by an inability to obtain stable readings. This occurred once in the performance of the rate of corrosion test during a survey where the half-cell test was not affected. Similarly, overhead electric lines may pose a problem with the performance of the electrical tests during these surveys.

The copper/copper sulfate reference electrode is sensitive to the temperature at which the test is performed. When comparing the readings from two surveys, it may be necessary to apply a factor that accounts for the difference in temperature under which the two tests may have been performed. The ASTM procedure stipulates a correction be applied if the test is not run within a temperature range of $22.2 \pm 5.5°C$. In this study, the temperature of the water in the surfactant bottle was considered the test temperature, or temperature of the reference cell.

The copper sulfate solution surrounding the copper electrode of the reference cell must be kept at a saturated concentration to avoid obtaining erroneous test results. This is easily done by insuring that there are always undissolved crystals of copper sulfate in the solution. During this study, the reference cell was checked daily to insure that the reservoir surrounding the copper electrode was full of the saturated copper sulfate solution.

A half-cell survey may be done underwater but with results that are of limited value. ASTM C876 addresses the underwater survey and emphasizes that the contamination of the copper sulfate solution must be avoided. Theoretically, an underwater test will give results that do not change across the surface of the structure, because of the water providing a surrounding electrical continuity. Therefore, the exact location of corrosion activity cannot be determined in an underwater survey, but the magnitude of the uniform readings will accurately indicate the absence or presence of the activity. For piling and columns, this limitation of not knowing the exact location of the activity is not a major problem because of the usual piling repair strategy. With a large underwater retaining wall, however, there may be more interest in repairing only the highly anodic areas, and pinpointing the distress is therefore difficult.

There are reports of the ASTM thresholds having the appearance of being incorrect (7). The affect of the chloride ion in producing an effect on the voltage read in reference to the CSE may be substantial. However, in this study, and in any testing program that may ensue, the ASTM thresholds are recommended until they are substantially further challenged.

5.2.6.1 Case Studies: Two structures surveyed in this study had dense concrete overlays applied to the decks. The I-610 Overpass at Wallisville Road in Houston showed a low probability of corrosion (100 percent of readings) as did the I-27 southbound structure over Keuka Street in Lubbock (94 percent).

The structure in Houston has had no salt applied since the overlay was placed. The chloride-contaminated concrete had been removed before the overlay was installed. The concrete was not removed to the level of the steel, however, and it is possible that corrosion could continue to occur if most chlorides were not eliminated. A large number of positive readings were obtained, and this could be due to an inability to achieve thorough wetting of the dense material. The half-cell readings were stable, but the rate-of-corrosion static potential readings were sometimes fluctuating. Low potential readings obtained on this structure are interpreted to indicate a low probability of corrosion. It is theorized that the removal of the chloride-contaminated concrete and subsequent overlay with a dense material slowed the intrusion of water and oxygen that support the cathodic reaction in the corrosion process. The low voltage readings may be lower than expected from other decks of normal concrete because the probe must sense through the dense electrically-resistant overlay material as well as a cover depth that exceeds the usual 5.1 cm found on most decks surveyed.

The Lubbock structure showed the majority of half-cell readings to be between -100 mV to -200 mV, and rate tests indicated a relatively slow rate of corrosion. Of particular interest is that the chloride content of the overlay material is very high at the 5.1-cm maximum depth tested, while the half-cell potential readings indicate only a low probability of corrosion. The conclusion may be drawn that the chlorides have not reached the top mat of steel, and the steel remains in a passive state. More information concerning this case will follow in a discussion on chlorides.

The presence of dielectric materials as either a coating on the steel or as a filler in the concrete would theoretically cause the voltage readings to differ from those obtained from uncoated steel and plain concrete. During the survey of two structures with polymer-impregnated decks, readings were generally uniform and the routine performance of the testing was not changed nor considered more difficult than other surveys. The northbound structure of I-27 over Keuka Street in Lubbock was built in 1978 and was an experimental project for polymer impregnation. The SH 350 Owens Street Overpass in Big Spring was built in 1978. It is possible that these structures did not get the high degree of impregnation that was expected, and therefore appear to be no different than any plain concrete structure when tested. The Big Spring structure exhibited a low probability of corrosion on 100 percent of the sample area. The Lubbock bridge had half-cell readings that showed a low probability for approximately half of the sample and an uncertain probability for the remaining half. SHRP classifies bridges with rigid deck overlays and polymer-impregnated decks as those needing Evaluation Surveys for Special Conditions (26). From the results of the two structures surveyed in this study, it appears that dense concrete overlays are testable with the half-cell potential survey, and the results need no special treatment or corrections applied.

The testing results of the FM 2085 structure showed that the gutter area of 57 half-cell test points had 37 percent of the points showing high probability of corrosion and 63 percent uncertain. The gutter area was not covered with asphalt. The twenty points that were previously covered by the asphalt show that 85 percent are high probability areas and 15 percent are uncertain.

5.2.7 Testing in a Sea Water Environment

Performing the half-cell survey on a substructure that is in water is especially difficult because of the lack of maneuverability that the inspector is likely to have in reaching all the survey points. The retractable arm of the snooper can place its inspection platform at almost any position, but positioning takes more time than the testing itself. Wave action affects the ability to work effectively out of a boat, and therefore wind conditions will probably almost always be a factor in choosing the right time to inspect from a boat. Dropping any tool can usually mean its permanent loss. The inspector is likely to work slower in these conditions because each action is more deliberate and carefully planned than would be necessary on the deck.

Working from the snooper platform is difficult because of crowded conditions. It is convenient in that there are electrical plugs readily available and no gas powered generator is necessary to power the hammer drill. Coring, of course, is not practical, and exposing steel for the grounding of the voltmeter is more difficult in the sea water survey conditions.

Though half-cell readings were not done under water in this study, a reference probe of good construction should be capable of performing suitably if needed. Testing under these conditions, requires that the technician insure a watertight seal of the tube containing the copper sulfate solution and to be alert for contamination of the solution. When the probe is held in contact with a submerged surface, the readings will be uniform regardless of vertical position on the member. This effect is discussed elsewhere in this chapter. At this juncture, the performance of under water half-cell surveys during routine inspections is not considered necessary or practical.

5.2.7.1 Case Study: Noteworthy in the results of the small amount of testing done in this study on sea water structures is the uncertainty of corrosion occurring within parts of the structure other than the submerged piling or columns. A half-cell potential survey was performed on the underside of a portion of the deck of the Nueces Bay Causeway to determine if the salt spray and salt air has any corrosion-promoting effect on this part of the structure. A single span of the original portion of the deck built in 1961 was surveyed with the usual grid pattern between two of the prestressed beams, which are spaced at 2.4 m. The deck had been formed with plywood and cast full depth. This is contrary to the construction method used on the subsequent widening in 1987, where precast panels make up the lower portion of the slab thickness. The panels have a very slick finish exposed on the underside of the deck, which should serve to repel the intrusion of chlorides and moisture better that than the comparatively open texture of the wood formed surface of the old deck. Because the widened portion of the deck also has epoxy-coated reinforcement within, it was not considered for half-cell potential surveying. Although obtaining a stable reading was not as easy as anticipated, the some unstable readings ranged from 0 to -78 mV. The difficulty in getting stable readings may stem from the difficulty in prewetting the underside of the deck. Water applied as a spray and within the sponge at the tip of the reference probe does not penetrate well without the help of gravity. If the readings in fact do mean a low probability of corrosion occurring in the bottom mat of steel on decks such as this one, then it is probable that this detection and monitoring program need not include portions of a structure that are not submerged or in the splash zone. Before this assumption is made, however, it is recommended that undersides of decks that have low clearances over salt water be thoroughly tested.

When testing piling, the presence of barnacles and algae prevent obtaining readings within the splash zone. Although it was not tried in the study, the readings could be taken within the splash zone if the affected area were cleaned of the marine life. That is not considered worth the effort, because an indication of the corrosion condition can be obtained by plotting a half-cell profile and making some predictions about the magnitude of readings that would be obtained under the growth. The profile is accomplished by obtaining readings at regularly spaced intervals along a vertical line beginning at a point as near the algae as possible and continuing up to an arbitrarily determined stopping point. The data in Table 5.4 shows the effect obtaining readings at various distances from the algae and barnacle line. The readings were taken at the Nueces Bay Causeway on a piling that has been in sea water since 1962. Readings were taken on the center of three faces of one piling at various heights above the barnacle line. By observing the trend of the potential readings toward the splash zone, it can be safely predicted that below the barnacle line the potential reading will indicate a high probability of corrosion.

Table 5.4 Comparison of Half-Cell Potentials on Sides of One Piling of the Nueces Bay Causeway

Distance up from Barnacle Line	West Face Potential (mV)	South Face Potential (mV)	East Face Potential (mV)
1.2 m	-247	-256	-231
0.9 m	-280	-281	-270
0.6 m	-329	-337	-328
0.3 m	-389	-409	-372
At Barnacle Line	-420	-453	-408

5.2.8 Reporting Results

Information reported from the half-cell survey includes information on each grid. Then, if in the opinion of the surveyor there is not significant difference among the results obtained from each grid, the results may be combined to provide a single report from all the half-cell data collected. Results that should be noted for each grid and ultimately the entire structure in summary are:

- Sample size versus bridge member size;
- Grid spacing; total number of readings;

- Percent readings in each probability category:
  - Low for readings less negative than -0.20 V,
  - Uncertain for readings between -0.20 V and -0.35 V,
  - High for readings more negative than -.35 V;
- Extremes or range of readings;
- Comments on particular readings, such as high probability at a joint if armor steel exists;
- A potential plot;
- Backup data - all readings.

The potential plot can be done in several ways. Varying shading or coloring may be used within the contours to show various potential levels. The method used in this study is shown in Figure 5.4 and was considered the most efficient for capabilities of the equipment on hand. Other equipment and software is capable of providing more information with less manipulation than that used in this study.

LP 494 at Caney Creek Half-cell Potential Survey July 28,1993

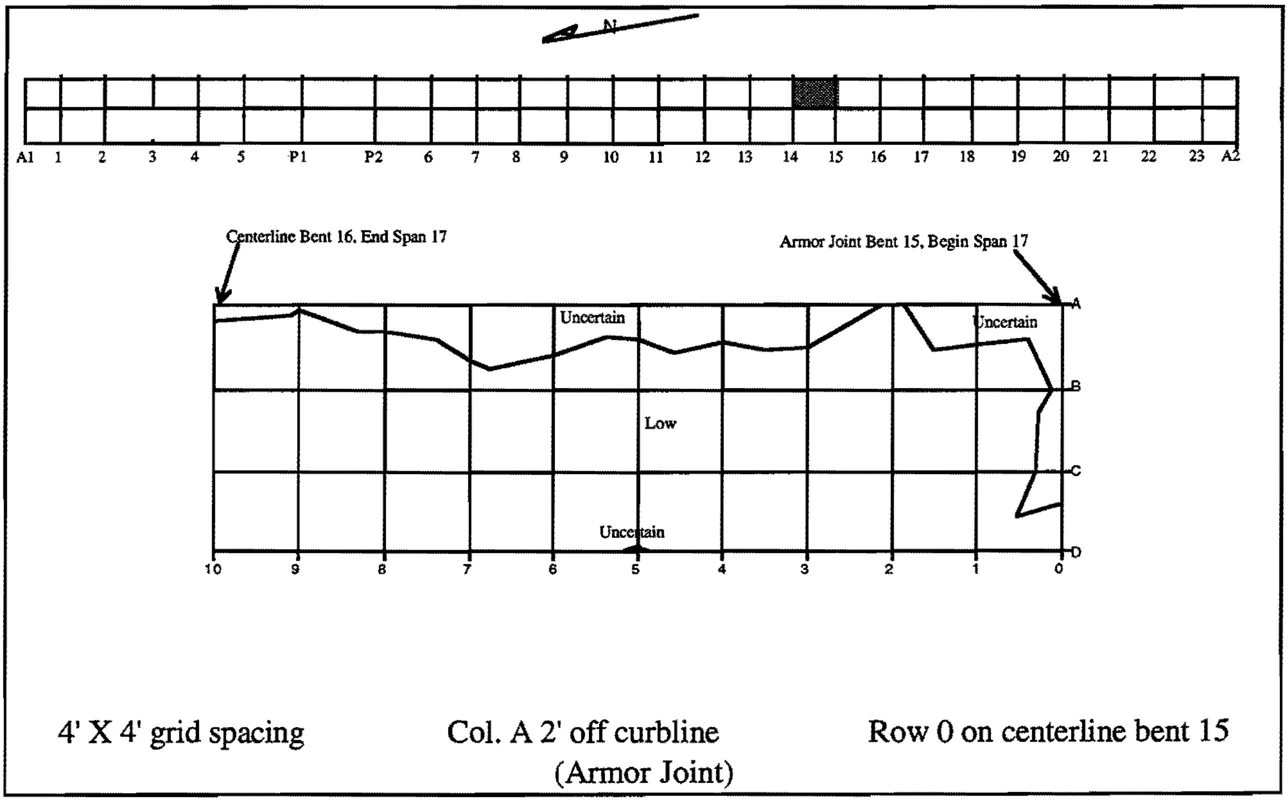


Figure 5.4 Sample Potential Plot (1 foot=0.304 m)

5.3 CORROSION RATE TESTING

5.3.1 Equipment

Only one corrosion rate meter which employed the polarization resistance or linear polarization technique was used in this study. This meter was one of three evaluated by SHRP. This device was simple to operate and transport and was deemed a useful device for accurately estimating the instantaneous corrosion rate of a particular piece of reinforcing steel within a structure (30).

The device uses essentially three electrodes to determine the resistance to polarization from which the corrosion rate is calculated. Figure 5.5 presents the device schematically.

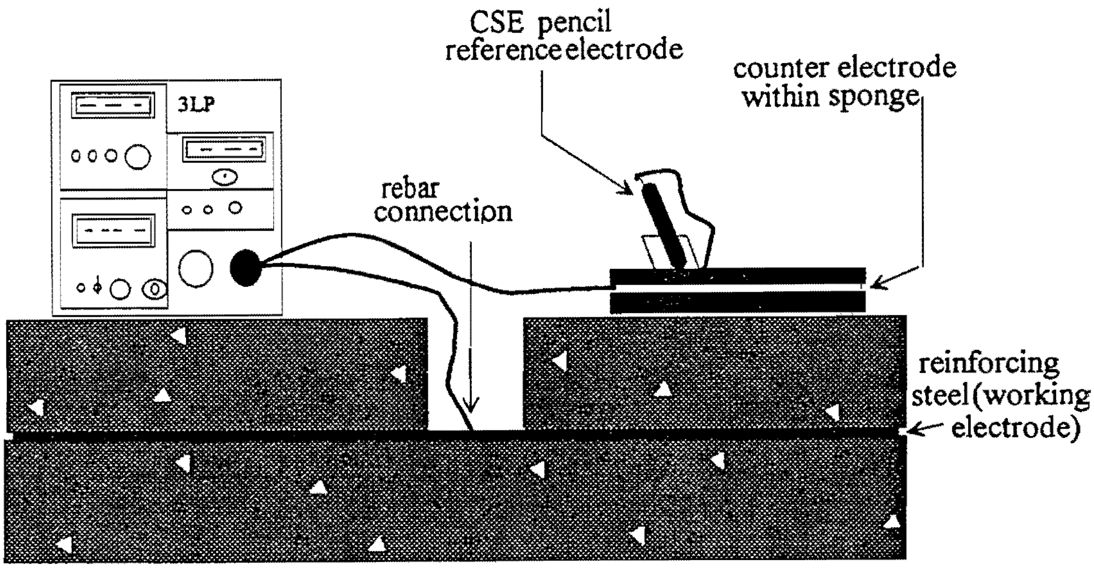


Figure 5.5 Schematic Representation of Corrosion Rate Device Used in This Study (31)

The three electrodes of this testing device are:

- The working electrode (WE) which is the reinforcing steel;
- The reference electrode (RE) which is a copper/copper sulfate reference cell that senses the potential and its changes as the steel becomes polarized;
- The counter electrode (CE) which polarizes the steel over which it is placed.

This device essentially defines the current necessary to cathodically polarize the steel by 4, 8, and 12 mV. The change in current required to achieve each of the new potentials (E + E) is recorded and used in the Stern-Geary equation as shown in Chapter 3 to determine the polarization resistance (R_p) encountered. By knowing R_p, the rate of corrosion, i_{corr} can be calculated.

intended spacing is helpful when searching for the steel with the pachometer. An individual bar was usually chosen; however, avoiding the placement of the CE over crossing bars is not always possible, and multiple bars may be marked on the concrete surface. The CE which consists of a 19.0-cm x 7.6-cm copper wire mesh housed in a sponge of equal area, is placed over the steel as marked. For the purposes of this test, the length of the steel in line with the longitudinal axis of the CE is 19.0 cm; any transverse steel is 7.6 cm in length.

The rate-of-corrosion test device is usually grounded at the same locations as the half-cell test so no further exposure of the reinforcing steel is usually needed for this testing. All of the devices currently used will require this connection with the mat of steel whose condition is tested. If the existence of electrical continuity is checked in the half-cell survey, the process need not be repeated.

The wetted sponge that houses the CE has also the reference electrode (RE) in contact. The pencil reference cell sits atop the sponge and measures the static potential or potential before polarizing just as the half-cell testing device would if it were used. However, this cell with this test method will also measure the new voltage levels of the steel as they are achieved by polarization.

Once the static potential is recorded, the polarization begins with the operator applying a current at a steady rate with the turning of a dial on the meter. The current applied at the point when the potential moves 4, 8, and 12 mV away from the static potential is recorded. Testing using this device takes approximately ten minutes per test per location. If results are suspect, or if confirmation on the results is desired, the test on the same location may be run again after a two-minute waiting period.

After the test is run, the data are used in the Stern-Geary equation to calculate the corrosion rate. The calculation is conveniently performed with a software application and may be done in the field on a portable computer to produce results within minutes of running the test.

5.3.2 Test Location

In this study, corrosion rate tests were performed nearest locations that presented the highest probability of corrosion activity as per the half-cell potential survey. Though this is sometimes the prescribed method for choosing rate test points (26), the method given by the manufacturer of the rate-of-corrosion meter used in this study is one of sampling a cross-section of the deck as represented by half-cell readings in all potential ranges (9). Because the potential at a particular location is said to be independent of the corrosion rate, the latter method would seem to be the better alternative to achieving a representative sample. Regardless of which points are ultimately chosen for rate testing, the preliminary choice of points is done during the half-cell survey. As the half-cell test was being conducted, markers were placed at points that exhibit some of the most negative readings encountered during the survey. The half-cell potential was written with chalk beside candidate points on the surface of the concrete. At the end of the half-cell survey, approximately 10 points were chosen for corrosion rate test locations. Rate tests were then run on as many of the points as possible at locations chosen best representing the sample area. If the marked high potential points were confined to a single area, the test was not run solely at the locations within that restricted area of the sample. When the rate test begins the inspector is able to verify his potential reading, because although the rate test seldom lands over the exact location that the half-cell reading was taken, the static potential recorded for the rate test should be similar.

The size and number of bars over which the counter electrode (CE) is positioned during the test must be known for later calculations of the corrosion rate. The as-built plans are consulted for the size and layout of the steel. With the use of a covermeter, the steel is located to allow the placement of the CE centered directly above. It is assumed that the CE polarizes only the steel embedded directly beneath it, and it is this assumption that costs the test with this device some of the credibility given other meters. With guard ring technology, a method of confining the polarizing field to the steel intended allows for more accurate results. It is recommended that testing devices be considered for purchase that possess this technology.

5.3.3 Performing Rate Testing — Case Studies

Shown in Figure 5.6 is the relationship between the potential, precisely, the static potential or potential before the polarization current is applied, and the corrosion current or i_{corr} as determined in the rate test during surveys of different structures within this study. It can be seen from this figure, that rate is not necessarily equated with magnitude of potential.

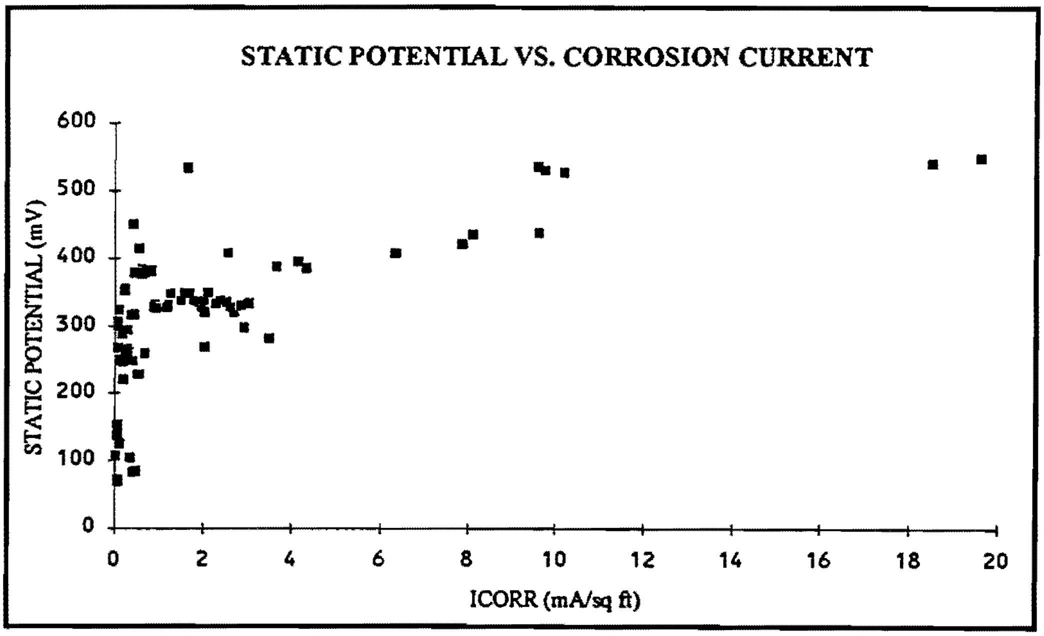


Figure 5.6 Rate of corrosion (i_{corr}) at Various Points of Static Potential

5.3.3.1 Deck Surveys: Not shown in Figure 5.6 are the highest corrosion rates measured during the conduct of this study. On FM 2085 at Bull Creek, i_{corr} values of 592 mA/m^2 and 430 mA/m^2 for which static potentials of -553 and -375 mV respectively were recorded. These high rates confirmed all suspicions of widespread corrosion activity occurring at the time of the survey.

On the US 283 / I-20 overpass, the rate test was not run on the first day of testing because a stable static potential could not be obtained. When the testing team returned on the second day of testing, the rate test was not attempted, because it was incorrectly assumed that stray current was within the bridge and obtaining a stable reading was impossible. The inspection team did not consider the performance of the half-cell as an impossibility, but that consideration would have been logical. The half-cell testing was performed on the second test day without any instability of readings. The rate test attempted on the previous day was tried after the half-cell survey was completed for the day. The inspection team was notified that a radar speed trap had operated near the structure at the time of the rate testing. It was later ascertained that the radar was the probable blame for the instability of readings. The speed trap was probably not in operation at the time of the half-cell surveying on the first day of testing, and probably commenced operations after that testing was complete and the rate testing began.

Notable lower-than-anticipated corrosion rates occurred on the Pecos River substructures which were tested because of the presumed high chloride content of the river water. Although spalling was evident and chloride contents were high, the rate of corrosion was low. A corrosion rate of 7.0 mA/m^2 with a static potential of -378 mV CSE was the highest rate recorded on SH 18. On the US 67 structure, the highest rate was again approximately 0.65 mA/SF with a static potential of -383 mV.

At I-610 at the Wallisville Road Overpass, half-cell readings were somewhat unstable but all in the low probability range. The rate of corrosion testing, however, did not experience an instability of static potential readings and revealed a low corrosion rate 5.9 mA/m^2 with a static potential of -229 mV CSE. An explanation for the stable static potential reading during the rate test may be that thorough wetting of the concrete was achieved only during that test. The performance of the faster moving half-cell test, performed carelessly, may not allow enough time for proper wetting of the concrete. Sometimes patience is required in order to provide ample time for wetting and better electrical continuity.

5.3.3.2 Testing Substructures in a Sea Water Environment: The rate test, like other testing over water and from a snooper, is cumbersome and more difficult in this environment than performing tests on a deck. While testing piling on the Nueces Bay Causeway, attachment of the counter electrode to the piling surface was accomplished with one inspector physically holding the sponge against the pile. A high potential (-407 mV) and a calculated high rate of over 75 mA/m^2 was obtained after performing only one test, consisting of two sets of readings, in this time-inefficient mode of testing. The dampness of the concrete as well as algae and barnacle growth prevent tape from working to hold the CE and RE in its required position. It is recommended that an elastic cord or strap with velcro be used to attach the sponge to the wet surfaces.

5.3.4 Rate Calculations

An assumption of linearity is made for the potential versus current plot in the vicinity of the static potential at points that are 4, 8, and 12 mV removed from that value. The slope of the curve, E/I, is proportional the polarization resistance, R_p which is inversely proportional to the corrosion rate, i_{corr}. The calculation of i_{corr} is performed most easily with the use of software furnished by the manufacturer of the corrosion rate testing device used in this study.

5.3.5 Reporting Results

When reporting rate-of-corrosion test results, the range of i_{corr} should be reported as well as average rates if appropriate. A report is made for each grid along with a summary report with comments regarding the rate of corrosion at points over the entire bridge.

5.4 CHLORIDE CONTENT TESTING

5.4.1 Equipment

The electric hammer drill is the most important piece of equipment for chloride content sample collection. The drill works best when equipped with the depth gauging attachment that quickly sets the drill to penetrate to only the required depth. A back-up drill is desirable because the inspectors in this study once encountered a problem that limited the number of samples that could be gathered.

The testing kit should allow the expedient testing of the maximum number of samples possible. SHRP has evaluated the test method used in this study, the specific ion probe, and has reported excellent results (32). The chloride ion content as a percentage of by weight of concrete is read directly from the meter once the probe is inserted in a digested or extracted mixture of crushed contaminated concrete and an acetic acid solution.

5.4.2 Sample Locations

Samples should be taken across the physical cross section of the sample area unless there is a concern in a specific area of the structure. Wherever samples are taken, it is recommended that as many locations as possible be sampled. To help accomplish this, it is recommended that powdered samples be collected from only two depths instead of the usual four recommended by the test procedure.

5.4.3 Performing Chloride Content Testing

5.4.3.1 Field Sampling: It is recommended that only two samples be collected at each location - one at the surface of the concrete and one at the 3.8-cm to 5.1-cm depth. By collecting a sample at the lower depth, knowledge is gained about the chloride ion content at the usual depth of the steel. From the sample near the surface, an insight is gained into the potential for more chlorides permeating to the steel level in the future.

The SHRP procedure as shown in the flowchart of Chapter 4 recommends chloride testing if less than 10 percent of the half-cell potential readings are more negative than -200 mV. This is logical, because if the half-cell potential survey does not indicate the likelihood of corrosion activity, then chloride testing is done to see if subsequent testing or monitoring will be necessary. This is based on the chlorides presence affecting corrosion activity in the future.

Chloride content testing is recommended in the initial corrosion condition survey regardless of results from other tests. The SHRP procedure may be more closely followed in subsequent tests, but the gathering of chloride information regardless of half-cell results during the initial survey is recommended to assist in obtaining an overall indication of condition of the concrete's reinforcement.

The chloride sampling can be accomplished at any time after the grid and reference system is established. The locations of the chloride testing are recorded as the nearest grid point from which they are taken. The sample location and depth are recorded on the sample bottle.

Chloride contents were high on the FM 2085 structure which was covered with asphalt. The chloride ion contents range from a high of to 3.0 kg/m^3 in the bare gutter line and 3.9 kg/m^3 under the asphalt.

The SH 273 structure, which was also covered, had chloride contents that averaged 0.7 kg/m^3 at all levels deeper than 1.25 cm. At these chloride contents, corrosion could be occurring, and further testing should be done to obtain a better estimation of the corrosion condition of the structure. The half-cell survey and rate of corrosion test will provide more information to the bridge owner, but by expanding the survey, more damage to the new seal coat is expected.

The Pecos River superstructures surveyed contained high chlorides although a chloride content test performed on samples of the river water did not indicate a corrosive environment. Chloride contents at steel level exceeded 5.9 kg/m^3 at some locations. Practically all chloride contents were in excess of 1.8 kg/m^3. Water samples showed chloride contents of 0.3 and 0.55 percent. This is compared to sea water which can be expected to have total chlorides of over 3 percent. After almost sixty years of exposure the chlorides have accumulated even though the levels of salts in the water are sometimes relatively low.

5.4.3.2 Laboratory Testing: Although the chloride testing can be done entirely in the field, it is recommended that the samples be weighed and digested in the secure environment of the laboratory. There is no urgency in knowing the results on the day of the test, and to test for them while in the field and before leaving the bridge site would probably require a fourth person on the testing team.

Bottles of sufficient size are needed to hold the 3 grams of powdered sample and 20 ml of extraction fluid that is added. A reaction between the powder and the fluid sometimes causes an effervescing action and bottles of insufficient size will overflow. It is recommended that bottles for mixing be 50 ml or larger.

5.4.4 Reporting Results

Chloride content test results needed in the report of the corrosion condition of a structure include the range of results as well as the average at each depth taken. The percentage of readings over the established threshold is also reported.

5.5 DELAMINATION SURVEY

5.5.1 Equipment

The chain broom is an essential piece of delamination testing equipment because it allows a swift survey of a large area. This simple testing device can be fabricated easily by personnel with basic welding skills. A hammer or steel rod or pipe may also be used to tap the structure surface to produce a sound that indicates either solid or separated concrete. Substructure or vertical member delamination testing is usually done with a hammer. Delamination surveys on decks can also be done with pulse radar and impact echo technology, but these methods were not evaluated in this study and will not be discussed.

5.5.2 Delamination Testing

Delamination testing in this study was done with the chain broom. The test was always performed by one person; however, if two persons are available, the test can be performed easier as one team member drags the chain broom and marks the delaminated areas and another records their boundaries on a sketch with the grid superimposed. A single person testing a deck may find that comments and delamination boundaries are recorded easiest with the use of a hand-held voice recorder.

The testing could not be accomplished on the structures covered with an asphalt seal or asphaltic concrete. Likewise on the structures that carry heavy traffic, such as the twin I-27 structures north of Lubbock, the performance of the survey was impossible. Traffic noise prevented a thorough survey on those structures. The inspector should attempt to obtain some estimate of size of delamination areas. The accuracy of defining the delamination limits need not be extremely precise because a rehabilitation measure would likely call for a resurvey to insure that information is current and all areas are known.

On the US 283 / I-20 overpass structure delaminations were found primarily along the center line. Spalls did not reveal reinforcing steel because the depth of spalling was less than the clear cover for the steel. Half-cell readings did not strongly suggest that corrosion activity is causing the delaminations and spalling. A test to detect alkali-silica reactivity will be performed.

5.5.3 Reporting Results

The delamination survey report should include the percentage of the grid or sample area delaminated and a plot of the affected area. A sample of plotted delaminations is presented in Figure 5.7 which was drawn for a structure surveyed in this study.

5.6 PREPARATION OF REPORT

A summary report that combines the results obtained from all sample areas should be prepared with the goal of providing sufficient information to the bridge owner so that corrective of protective actions can be taken. Attached to the report should be all raw data and grid reference information. The appendix of this report contains a sample reporting form that may be used as a summary document for the corrosion condition survey of the entire bridge. Remarks on this form should help to explain conclusions drawn about the condition and protective measures and repair strategies. The FHWA manual (31) includes forms for recording data and summarizing information and these were often used during the surveys of this study. The reports on several structures surveyed in this study are also found in the appendix of this report.

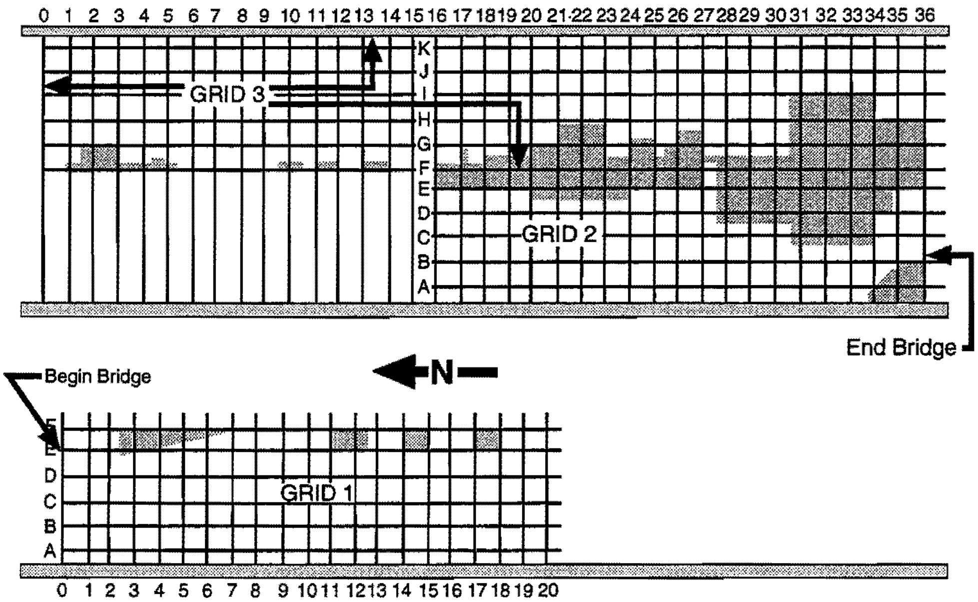


Figure 5.7 A Sample Delamination Area Plot

CHAPTER 6. IMPLEMENTATION

6.1 INTRODUCTION

The testing procedure discussed in Chapter 5 may be implemented to establish a program of work that may ultimately be chosen to perform one or both of two functions:

- Provide for all corrosion condition inspections at regular intervals as part of, or separate from, the total condition survey that is executed to provide information for the Bridge Management System;
- Provide for the testing involved with monitoring at regular intervals the performance of corrosion protection measures.

Regardless of whether or not a program of routine surveys is established, the procedures suggested herein may be adopted to fulfill the need of testing for Special Situations, as described in Chapter 5. This function could be accomplished with the employment of one team traveling statewide when called.

Specifically, a procedure was sought that would give the bridge owner sufficient information about the corrosion condition of a structure so that adequate protection or abatement actions could be taken if necessary. This was to be accomplished with the optimum amount of effort and cost and the least possible disruption of the structure's service. Some of the testing recommended in previous work was done to confirm results of other tests. At the present stage of the research, it is felt that fewer confirmation tests are needed, and it is possible that even more tests may be eliminated from the survey procedure if an increase in efficiency can be realized.

6.1.1 Status of Equipment

It will be necessary for the Texas Department of Transportation (TxDOT) to purchase or rent half-cell potential and rate of corrosion testing equipment because no-state-of-the-art equipment is on hand. The pachometer or covermeter, chloride content test kit, half-cell potential testing equipment, and rate-of-corrosion meter will cost from $25,000 to $35,000. For a pilot or trial testing program, it is possible at this time to receive and use some of this equipment on a loan basis from the Federal Highway Administration.

Each testing team will need an electrical generator to power coring and drilling equipment. Computers are assumed to be in adequate supply, but there are advantages associated with each testing team having lap top models, and these machines may need to be bought for this program. Coring machines, hammer drills, cover meters, and possibly chloride testing equipment are likely on hand or readily obtainable. With proper care and coordination, each testing team could be provided equipment such as the generator and coring rig by the hosting district until funding could allow the complete equipping of each team. A complete list of equipment as well as expendable supplies is found in reports submitted previously for this research project (15).

6.1.2 Personnel Requirements

Of the twenty-five highway districts within the Texas Department of Transportation, it is estimated that approximately half experience corrosion in reinforced concrete structures caused by exposure to deicing salts or sea water. For the structures in those districts, it must be emphasized that not all can be tested to the fullest extent by the set of tests recommended in this study. Because the application of the testing regime is limited in scope on bridges that are covered with asphalt or contain epoxy coated steel, the number of structures that are subject to all tests is significantly reduced from the total number subject to corrosion. Therefore, funding and training a team for each affected district may not be necessary. The assignment of a single testing team for the five districts with sea coast frontage may be considered, as well as one team per two districts that use deicing salt. This would indicate that approximately four teams would need to be trained and equipped statewide. Four persons per team are recommended in this presentation, and from this, it can be seen that approximately sixteen to twenty personnel are necessary to conduct the testing and provide reports for the program. A better estimate of the staffing needs for the program could be made after a pilot program is staged and executed.

6.1.3 Districts Included in the Program

From information gathered at meetings and through telephone conversations with TxDOT personnel, it is rationalized that the following Texas highway districts be included initially in the testing program because of their having structures founded in salt or brackish water or because of their continued use of deicing salts during freezing conditions:

- Abilene, Amarillo, Childress, Dallas, Fort Worth, Lubbock, Wichita Falls,
- Beaumont, Corpus Christi, Houston, Pharr, Yoakum

Consideration may be given to testing structures in districts where deicing salt was once used but discontinued. In these districts, it may be necessary to select a sample of bridges for testing to determine the severity of the corrosion problem and the need for further testing and permanent inclusion in the program. Of the districts listed above with deicing exposures, some have a significant number of structures that are not easily evaluated with all tests of the recommended program due to their having decks covered with seal coats or asphaltic concrete. A summary of the quantity of covered decks on bridge class structures in affected districts is shown in Table 6.1. The information was obtained from BRINSAP information files for the state's On-System bridges.

6.1.4 The Testing Team

There will be a difference in the staffing of testing teams dependent upon whether they will perform tests on the top side of the deck of a structure or whether they will be working underneath the bridge from an inspection platform. The difference in work space will necessitate this difference in staffing. The testing done from an inspection bucket or platform of a bridge inspection vehicle, or "snooper," progresses much slower than the testing that can be accomplished on top of a deck.

Table 6.1 Summary of Bridge-Class Structures Within Deicing Zones

District	Total Number	Number Covered	Percent Covered
Abilene	1371	546	40
Amarillo	726	385	53
Childress	706	176	25
Dallas	3761	1085	29
Ft. Worth	2076	1171	56
Lubbock	438	116	26
Wichita Falls	1031	339	33
Total of Above	**10109**	**3818**	**38**
Statewide Totals	33416	11390	34

For a deck survey team, four personnel are recommended to adequately accomplish the detection and monitoring duties set forth in this presentation. The field testing team should consist of no less than three qualified personnel accustomed to working long days in the elements. One person in the laboratory and office is needed to perform chloride content testing and to produce reports. It is recommended that the field team have duties generally divided between the three personnel as follows:

Team member #1:

1. Perform coordination duties with owner district and with traffic control personnel;
2. From the as-built plans, designate orientation of the location reference system and familiarize the team with the reinforcement scheme;
3. Locate the reinforcing steel for exposure for electrical ground;
4. Perform half-cell testing with periodic assistance from Team Member #3;
5. Locate and isolate steel for rate-of-corrosion testing;
6. Perform rate-of-corrosion test with the assistance of Team Member #3.

Team Member #2:

1. Collect chloride content samples;
2. Patch holes resulting from chloride content sampling and coring;
3. Perform delamination survey and record using hand held voice recorder.

Team Member #3:

1. Perform all coring and exposure of reinforcing steel for electrical ground connections;
2. Prewet all points on half-cell potential survey;
3. Assist Team Member #1 with electrical continuity testing and random point testing for test repeatability;
4. Assist Team Member #1 with rate-of-corrosion testing.

Experience with this division of duties showed that all team members were constantly occupied and all testing duties concluded at approximately the same time. Prior to the performance of the above duties, all team members are needed to mark the location reference system and clear any debris that may interfere with testing. The team concept of organization is emphasized to promote each team member knowing each other's job. This not only keeps the work interesting, but allows the team to continue to perform in the absence of a member.

The testing of structures in sea water may require the full time use of a snooper bridge inspection vehicle and operator. The snoopers currently in the TxDOT inventory have space on the inspection platform for the operator and two testing personnel. The size of the testing team involved with the testing of substructures in sea water may be reduced to two persons because there is no room for more testing personnel and equipment. Because of the reduced working capability caused by the of lack of room, fewer personnel, and slow maneuverability, the results of testing accomplished from a snooper will be fewer than that of a survey conducted within the same amount of time on a bridge deck.

Team members should not be responsible for performing traffic control duties and this function should be reserved for the host district which furnishes all traffic control personnel, signing, and channelizing devices. The personnel of the host district are familiar with the traffic patterns of the area and with standard traffic control procedures.

The field testing team should be responsible for patching all holes made while testing. This is because the team members are familiar with the location of all holes and will be given full responsibility for furnishing the best patch possible.

6.2 RECOMMENDED PROCEDURE

In the development of a field corrosion detection and monitoring procedure, test methods were chosen for their ability to provide the information that is seen as being most important to indicate the probability of corrosion occurring, the rate of corrosion, and the probability of a continuation of the activity. Tests were chosen that will reliably assist in the evaluation of corrosion protection systems.

6.2.1 Initial Deck Survey

After the completion of an initial bridge deck corrosion condition survey, the following questions should be answered:

- What percentage of the entire deck area is in each of the three probability ranges - low, uncertain, and high?
- How fast is metal dissolution occurring within a sampling of each of the areas of probability?
- What is the extent of corrosion related damage to this date as indicated by spalls and delaminations?
- What level of corrosion activity is likely to occur in the short and long term future based on the chloride ion content at the 0 to _ in. (0 to 1.27 cm) level and the 1_ to 2 in. (3.75 to 5.08 cm) level?

The following procedure is suggested for performance by a three man testing team on the initial survey of a bridge deck:

Visual inspection - The inspector should record the amount and general location of spalling, delamination and staining that has occurred as a result of corrosion activity. The effects of cracking may not necessarily be particularly harmful (15), so a detailed crack survey is not recommended. As inspectors become proficient at their duties, they may be readily able to recognize the cause of the distress to be non-corrosion induced, such as alkali-silica reaction (ASR). Whenever this is known to be the case, the inspectors may terminate the inspection and report the findings, because corrosion is likely to occur or continue if the primary cause of the distress is not addressed. Cracking in general should be examined and comments recorded.

Half-Cell Survey — On a 1.2-m grid spacing, obtain an indication of the probability level of active corrosion. Choose test locations for the rate test that follows by placing a marker and writing the potential at points representing each of the three probability ranges and the physical cross-section of the deck. Choose approximately three points per probability range.

Rate-of-Corrosion Test — At the locations chosen during the half-cell survey, perform the rate test on as many of the points as possible.

Chloride Content Sampling — At random points across the physical cross-section of the deck, obtain chloride samples at two depths per location chosen. It is recommended that the two depths be 0 to 1.27 cm and 3.75 to 5.08 cm. Obtain as many samples as possible, usually a minimum of 40 locations (80 samples) should be tested.

Delamination Survey — With a chain drag broom test the sample area and more of the deck area outside the sample area if determined necessary.

6.2.2 Subsequent Surveys

After the completion of a second survey on a bridge deck after approximately two years since the initial inspection, the following questions should be answered:

- Is the percentage of the deck area changing with respect to each of the probability ranges?
- What amount of metal loss has occurred since the last survey based on the average of two or more rate tests? At this rate, what is the loss expected to be in the future?

- Has deicing activity affected the chloride ion levels since the last survey?
- Has delamination continued since the survey or repair?

All tests performed in the initial survey are repeated in surveys performed subsequently to the initial survey. As stated previously, 50 percent of the area sampled during the survey prior should be tested again on the next survey. This allows for a thorough evaluation the trends in the corrosion process and of the effectiveness of any corrosion abatement measures applied since the first survey.

6.2.3 Surveys for Special Situations

A special survey may be required to determine the feasibility of rehabilitating a structure. The results of a survey of this type would not necessarily be entered into the BMS data base. Primary interest may be in the chloride content, with the complete replacement of the structure hinging on a particular threshold. Also, the chloride content at certain depths may be sought to determine the required depth of milling prior to the placement of a deck overlay. The testing team responsible for the performance of routine surveys should also be called upon to perform this type of testing.

6.3 PILOT PROGRAM

If the owners of the transportation network considering the adoption of a program have reservations about building and maintaining a comprehensive program, it is suggested that a trial or pilot program be instituted. Establishing a program that is smaller in size than that required for accomplishing corrosion condition surveys statewide will provide manageable amounts of information to assist in making further implementation decisions.

An initial program may be oriented toward one facet of the reinforced concrete bridge corrosion problem. TxDOT may implement the small-scale version of a program to gain more information about corrosion condition surveys and their use with PONTIS. The Department may also elect to concentrate its information gathering effort on a particular environment. The emphasis may be placed on the survey of decks in deicing areas because the testing of substructures in sea water may be initially impractical because of the requirement of the full time use of a snooper. A pilot program for use within the salt water environment may be best used as a monitoring tool, where the corrosion activity within piling and piers may be tracked over time. This would create an opportunity to evaluate the effectiveness of corrosion inhibiting admixtures, piling jackets, and epoxy coated steel in slowing or preventing corrosion.

The use of corrosion detection procedures to provide input for PONTIS may be best done on a small scale, as the condition states will need establishment as will feasible actions for remediation. An abbreviated program will allow familiarization with equipment and may assist in determining the equipment needs of a permanent full scale program.

CHAPTER 7. SUMMARY, CONCLUSIONS, AND FUTURE WORK

7.1 SUMMARY

The corrosion of reinforcing steel in bridge structures caused by chloride-bearing environments (e.g., deicing salt applications and exposure to marine environments for approximately the past 30 years) is now presenting itself as a serious problem. The requirement of implementing a Bridge Management System (BMS) also highlights the corrosion of reinforcing steel as an aspect of bridge maintenance that needs to be managed. A corrosion condition evaluation of existing structures as recommended in this report will detect corrosion activity before any major damage is done. The main disadvantage of the currently used methods of performing condition surveys in existing structures is that by the time damage is noted, it is too late to implement any management program and costly repairs are the only alternative.

Chlorides are the catalysts for corrosion activity within reinforced concrete bridges. The threshold level of the chloride ion and the exact mechanism for destruction of passivation continues to be debated. Reinforcing steel undergoes corrosion when there are four elements in place within a corrosion cell. The anode, cathode, conductive electron path, and the electrolyte must all be present or corrosion cannot occur. The thermodynamic potential of an area of reinforcing steel describes its tendency toward corrosion activity and is read as a voltage in the half-cell potential survey. The rate of corrosion involves kinetics, and is indicative of the speed of which electrons are being freed from the surface of the steel.

Two recent studies have provided extensive information concerning corrosion condition surveys. The Strategic Highway Research Program (SHRP) C-101 study produced an extensive, very detailed and labor intensive procedure for conducting a bridge condition survey including an in-depth evaluation of corrosion condition equipment. The survey recommended in the study presented herein is not as labor intensive, can be performed quickly and is as effective to the engineer as the procedure outlined by SHRP, but in consideration of personnel reductions and a budget constraints in the public sector, it is reduced in size. The Bridge Management System (BMS) that will be used in Texas is due to be established by 1998. In order for corrosion condition information to be input and used by this system, this BMS will require modifications to include information gathered from corrosion condition surveys.

Rather than instituting a full-scale program for detecting and monitoring corrosion statewide, the Texas Department of Transportation may consider a program on a smaller scale. A small scale pilot program should assist the state in learning more about the extent of the corrosion problem and the logistics of equipping and staffing several testing teams. After evaluating the effectiveness of the pilot program, decisions concerning further direction of a program tailored to the needs of the State may be made.

7.2 CONCLUSIONS

An objective of this research was to develop from field trials a testing regime that would adequately inform the bridge owner of the corrosion condition at any point in time. From a wide range of state-of-the-art testing methods, four tests and a visual inspection regime were chosen to provide results that, when used together, provide an indication of the probability of corrosion occurring, the rate of dissolution, the level of chlorides that may cause a continuance of activity, and an assessment of the extent of severe damage already having occurred. The limitations of using the entire testing program on all bridges were made clear during the course of the study. In fact, this program is intended to provide the DOT engineer with the needed information for him to decide on the maintenance, repair or replacement of an existing bridge structure. Basic findings of this study are listed below:

1. A method was found that is capable of detecting corrosion activity in its early stages before it is severe enough to cause delaminations and spalling. The method is considered cost efficient because, through trials in the field, it could easily be conducted within several hours with minimum disruption of traffic while optimizing the amount of equipment and number of personnel to conserve resources while producing adequate information about the corrosion condition.
2. The corrosion detection program developed in this study is based on a visual examination, a delamination survey, a half-cell survey, chloride ion content testing, and rate-of-corrosion testing. However, depending on the condition of each specific structure , not all steps in the program need to be conducted.
3. Surveying a sample area instead of the entire structure should give adequate information concerning the corrosion condition, but judgment by the inspectors must be used in determining the sample size. The use of selected sampling saves inspection time and reduces user costs by minimizing the amount of time the section of deck or bridge being evaluated is out of service.
4. Because corrosion condition is dynamic as compared to other aspects of an overall condition survey, the test methods presented provide for an instantaneous evaluation of the condition of the steel within the concrete. Benefits from testing are expected to increase with the building of a data base on each structure after further testing on a regular basis is accomplished.
5. A testing regime that uses electrical tests such as the half-cell or polarization resistance corrosion rate testing must be modified when bridges are encountered that are covered with asphalt or contain epoxy-coated steel. Because the electrical tests demand that a testing probe contact bare concrete surfaces, the removal of the cover is required. These tests, which are important in providing the instantaneous corrosion condition of a structure, may have to be eliminated or reduced in number if removing the covering is not feasible. Covering decks with asphalt to correct ride and appearance problems is a common practice, but the use of such as a corrosion prevention measure has been found to promote corrosion and thus its use should be questioned. The corrosion problem in Texas is not effectively quantified because much of the damage is masked under asphalt overlays or seals.

6. If electrical tests are difficult or impossible to perform, such as on asphalt-covered decks and on those that contain epoxy-coated steel, the survey may be done in two or more stages. More thorough testing with the electrical tests may be opted for after an initial survey containing a visual inspection and chloride content testing indicate the possibility of corrosion. The half-cell test may be performed on a grid spacing of greater than the usual 1.2-m spacing on this second step of the survey to prevent the destruction of the asphalt covering. On structures that are critical in terms of replacement cost or user cost related to interrupted service, the extent of asphalt removal may be increased to allow more half-cell and corrosion rate testing and better assess the condition of the structure.
7. The results of two thorough research efforts are available to assist in the implementation of a program. The recently completely SHRP C-101 study and the FHWA Demonstration Project 84 have information on equipment that was evaluated in field testing programs.
8. Texas will adopt a Bridge Management System for which the use of corrosion condition information is not currently provided for. PONTIS, the new BMS, does not call for before-damage corrosion data, and must be fitted with condition states and feasible actions for rehabilitation if it is to be used with a corrosion condition program.

7.3 FUTURE WORK

7.3.1 Materials

Asphaltic concrete is usually applied to bridge decks to provide a better ride and aesthetics and is a common practice across Texas. The detrimental effects that this practice has on the corrosion condition of the deck should be addressed. The detection of the delaminations can be detected without the removal of the overlay by using methods not addressed in this study. The durability of impermeable membranes other than an asphalt seal between the deck and the asphaltic concrete should be evaluated.

The effectiveness of corrosion-inhibiting admixtures and epoxy-coated reinforcing steel in substructures in sea water should be evaluated. The accomplishment of this task has begun with a proposal to install four dummy piling within bent lines of the Redfish Bay Bridge replacement project. These piling will be fitted with wire connections for half-cell and corrosion rate testing. Although the half-cell survey has no thresholds of probability established for readings obtained from corrosion activity over epoxy-coated steel, the testing may indicate corrosion in relative terms.

The attachment of voltmeter grounding wires to the reinforcing steel prior to concrete placement in the decks of new structures will continue to allow the monitoring of corrosion activity to assist in evaluating the effectiveness of various corrosion protection measures. This has been initiated, and by mid-1994, there will be six black steel structures across the state that will have electrical connections available for easy access and testing.

7.3.2 General Testing and Monitoring

Before implementing a corrosion detection and monitoring program, a cost / benefit analysis should be performed. Estimating the economic losses due to corrosion is extremely difficult, which makes a determination of the cost effectiveness of an assessment program difficult as well. A testing program has been identified in this study but its value will not be known until the amount of risk imposed by corrosion is learned.

An urgent need exists to incorporate corrosion condition surveys into a BMS. This keeps the condition survey efforts concentrated and focused and coordinated even though the functional purposes of the overall condition surveys and the corrosion condition surveys may differ. For proper inclusion in PONTIS, condition states as defined by corrosion condition need to be established and weighted properly. TxDOT should seek assistance in modifying its BMS to accept and use corrosion condition data once the decision is made to implement procedures in a pilot or regular program.

From empirical information, fixing a range or threshold of half-cell potential readings to indicate a probability of corrosion occurring in epoxy-coated reinforcing steel is needed. With this information, at least some reinforcing steel in a piling or pier can be monitored for corrosion activity using electrical tests.

In future work, constant improvement in the efficiency of the testing program should be is pursued. As the number of tested structures increases and the data base is expanded, and the experience level of the inspectors is increased, the procedures may be changed to gain more information or avoid any redundancies that become evident.

7.3.3. Testing Structures in Sea Water

Information on the severity and extent of corrosion of structures on the Texas Gulf Coast should be gathered and analyzed to determine the future direction of regular testing on structures in this environment. Further study is needed to ascertain whether decks and caps beyond a certain clearance of the water are affected by spray. If it is determined that elements above the water are not generally susceptible to corrosion , the prevention efforts maybe directed solely to the splash zone. A need also exists to determine if substructure elements can forego testing because of the certainty of corrosion occurring. If the rate of corrosion is similar in substructures of the similar ages, testing them in a regular program may be unnecessary. These elements, because of their similarity and constancy of exposure, should then have common corrosion abatement measures applied and their performance monitored.

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APPENDIX A

CORROSION CONDITION SURVEY REPORT

REPORT OF CORROSION CONDITION SURVEY
Highway: IH 27
Date: 8/18/93
Location: at Keuka Street North of Lubbock; (NBL structure; outside lane & shoulder)

Test Personnel: Barrios, Lopez, Freytag **Weather:** P/C, 87, Windy

Chloride Content Testing

Total number of samples/tests: 15 holes produced 4 samples per hole

Chloride Contents in Pounds of Chloride per Cubic Yard of Concrete at Various Depths			
Depth	Minimum	Maximum	Average
0-_"	0	8.2	4.2
_-1"	0	5.5	1.9
1-1_"	0	3.5	1.1
1_-2"	0	3.9	1.5

1 in.=2.54 cm

Remarks: Higher chloride contents generally appeared in the cracks. The polymer impregnation may be working well outside the cracking where good penetration was obtained.

Half-Cell Potential Survey

Number of Deck Sample Grids 1
Number of Survey Points 312

% More Positive than -200 mV (low probability) 48
% Between -200 and -350 mV (uncertain probability) 48
% More Negative than -350 mV (high probability) 4

Rate-of-Corrosion Testing

Number of Test Locations 9

Location in Grid	Static Potential (mV)	Icorr (mA/SF)
E26-E27	301	0.07
A2-B2	450	0.43
B2-3/C2-3	324	0.11
C11-D11	533	1.65
B11-12/C11-12	288	0.17
A11-A12	295	0.29
C41-D41	414	0.55
B35-C35	351	0.23
C47	379	0.45

Delamination Survey

Not performed due to traffic noise

Rate-of-Corrosion Testing

Number of Test Locations: 9

Location in Grid	Static Potential (mV)	Icorr (mA/SF)
E26-E27	301	0.07
A2-B2	450	0.43
B2-3/C2-3	324	0.11
C11-D11	533	1.65
B11-12/C11-12	288	0.17
A11-A12	295	0.29
C41-D41	414	0.55
B35-C35	351	0.23
C47	379	0.45

Delamination Survey

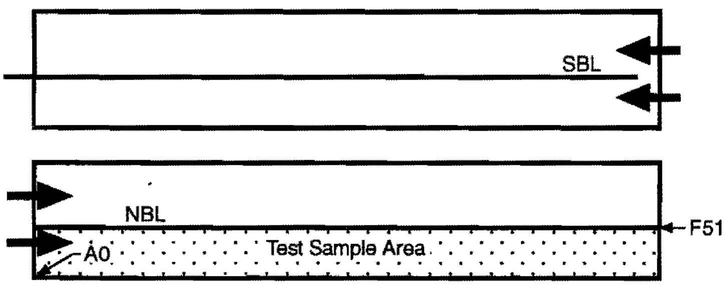
Not performed due to traffic noise

Visual Distress Survey

No detailed mapping of cracks was performed this date. Predominant existing cracks have been documented since completion of construction and were thought to be caused by the heat process used with the impregnation procedure.

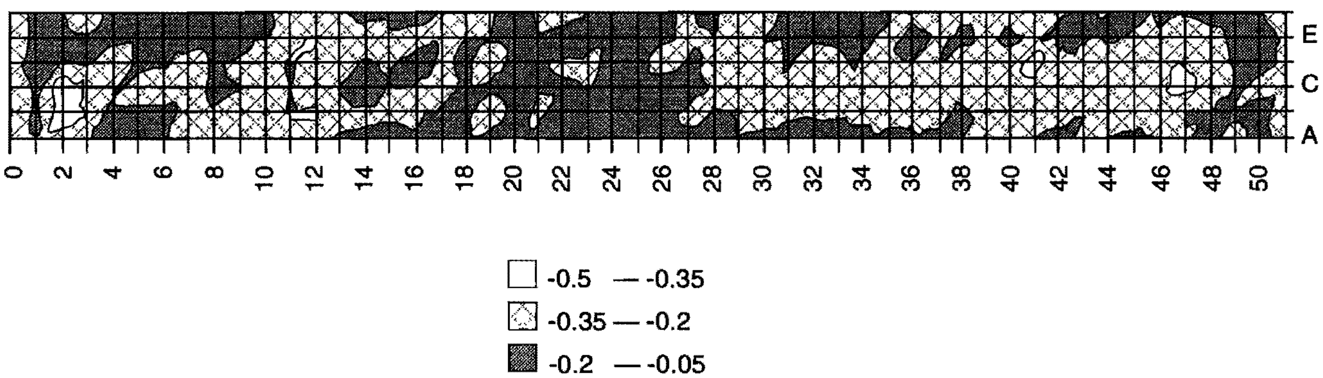
Half-Cell Potential Map

Reference: The outside lane and shoulder of the Northbound Lanes were gridded so that Line A was on the shoulder and 0.6 m from the base of the rail. Row zero was at the armor joint over the south abutment (Abutment 1).



Half-Cell Potential Map I-27 at Keuka Street (Northbound Lanes)

HALF-CELL POTENTIAL PLOT
I-27 at Keuka Street (NBL Bridge)



Summary of Test Results

The chlorides appear to be present in sufficient quantity to cause corrosion damage in the future, possibly 10 years or less. Based on the sample results, the corrosion is occurring at a relatively slow rate in the few places of probable activity. It is doubtful that any significant damage has occurred. The chlorides may have only recently reached the level and depth to initiate corrosion, and now maintenance measures are necessary to prevent moisture intrusion that will carry the chlorides downward to even greater concentrations. The deck needs a new application of sealant to assist the polymer impregnation which is probably not providing uniform protection. Furthermore, the cracks which probably came about as a result of the heat used in the impregnation process, need sealing. The best sealing alternative may be a combined application of linseed oil, or a silane or siloxane to seal the general surface porosity with a methacrylate to seal the cracks. Half-cell survey and chloride content test raw data are attached.

	A	B	C	D	E	F
	TEST STARTING DATE: 08/18/93					
	TEST STARTING TIME: 10:19:50					
	NBL I27 OUTSIDE SHOULDER AND LANE					
0	-0.258	-0.251	-0.284	-0.246	-0.242	-0.288
1	-0.184	-0.172	-0.182	-0.124	-0.178	-0.181
2	-0.191	-0.470	-0.460	-0.195	-0.165	-0.181
3	-0.206	-0.356	-0.257	-0.160	-0.199	-0.291
4	-0.184	-0.136	-0.212	-0.374	-0.176	-0.187
5	-0.191	-0.133	-0.313	-0.148	-0.124	-0.154
6	-0.179	-0.120	-0.317	-0.203	-0.150	-0.204
7	-0.195	-0.365	-0.302	-0.325	-0.121	-0.161
8	-0.197	-0.219	-0.160	-0.187	-0.145	-0.205
9	-0.203	-0.234	-0.221	-0.251	-0.164	-0.180
10	-0.240	-0.239	-0.272	-0.235	-0.194	-0.170
11	-0.257	-0.366	-0.345	-0.363	-0.251	-0.261
12	-0.257	-0.365	-0.236	-0.310	-0.361	-0.213
13	-0.176	-0.231	-0.186	-0.308	-0.278	-0.228
14	-0.127	-0.253	-0.162	-0.177	-0.330	-0.157
15	-0.164	-0.225	-0.194	-0.221	-0.229	-0.178
16	-0.193	-0.222	-0.254	-0.143	-0.273	-0.139
17	-0.147	-0.142	-0.250	-0.226	-0.218	-0.230
18	-0.177	-0.192	-0.168	-0.170	-0.205	-0.279
19	-0.169	-0.262	-0.187	-0.263	-0.209	-0.240
20	-0.154	-0.150	-0.156	-0.161	-0.150	-0.150
21	-0.131	-0.235	-0.175	-0.187	-0.189	-0.218
22	-0.098	-0.096	-0.177	-0.215	-0.094	-0.328
23	-0.095	-0.140	-0.196	-0.242	-0.134	-0.175
24	-0.088	-0.170	-0.158	-0.168	-0.166	-0.161
25	-0.126	-0.141	-0.142	-0.151	-0.193	-0.133
26	-0.135	-0.132	-0.133	-0.256	-0.183	-0.189
27	-0.118	-0.239	-0.164	-0.188	-0.213	-0.215
28	-0.117	-0.186	-0.211	-0.220	-0.208	-0.186
29	-0.152	-0.193	-0.246	-0.255	-0.216	-0.249
30	-0.159	-0.340	-0.249	-0.279	-0.211	-0.228
31	-0.134	-0.237	-0.221	-0.190	-0.182	-0.181
32	-0.172	-0.213	-0.209	-0.266	-0.183	-0.208
33	-0.152	-0.217	-0.215	-0.213	-0.152	-0.178
34	-0.142	-0.216	-0.231	-0.195	-0.162	-0.173
35	-0.152	-0.276	-0.287	-0.226	-0.264	-0.188
36	-0.179	-0.254	-0.271	-0.197	-0.171	-0.289
37	-0.144	-0.294	-0.223	-0.200	-0.226	-0.252
38	-0.196	-0.182	-0.227	-0.251	-0.187	-0.239
39	-0.277	-0.227	-0.340	-0.269	-0.215	-0.219
40	-0.219	-0.257	-0.295	-0.262	-0.181	-0.237
41	-0.206	-0.340	-0.254	-0.402	-0.228	-0.363
42	-0.168	-0.266	-0.284	-0.209	-0.164	-0.244
43	-0.222	-0.207	-0.288	-0.369	-0.182	-0.192
44	-0.203	-0.204	-0.248	-0.264	-0.159	-0.209
45	-0.168	-0.302	-0.321	-0.251	-0.225	-0.124
46	-0.255	-0.316	-0.342	-0.193	-0.203	-0.198
47	-0.205	-0.194	-0.362	-0.352	-0.307	-0.201
48	-0.187	-0.197	-0.331	-0.303	-0.276	-0.174
49	-0.258	-0.192	-0.181	-0.198	-0.151	-0.141
50	-0.166	-0.149	-0.268	-0.148	-0.172	-0.196
51	-0.257	-0.275	-0.231	-0.230	-0.201	-0.238

NBL IH 27 @ KEUKA ST.					
	CHLORIDE CONTENTS (ACID SOLUBLE)				
11-NB-B0	%	lb/cy	11-NB-A25	%	lb/cy
1A	0.21	8.2	8A	0.12	4.7
1B	0.06	2.3	8B	0.00	0.0
1C	0.06	2.3	8C	0.00	0.0
1D	0.10	3.9	8D	0.00	0.0
11-NB-E0			11-NB-F28		
2A	0.08	3.1	9A	0.10	3.9
2B	0.05	2.0	9B	0.10	3.9
2C	0.00	0.0	9C	0.02	0.7
2D	0.05	2.0	9D	0.08	3.1
11-NB-F3			11-NB-B32		
3A	0.13	5.1	10A	0.14	5.5
3B	0.02	0.8	10B	0.01	0.4
3C	0.01	0.5	10C	0.00	0.0
3D	0.01	0.4	10D	0.00	0.0
11-NB-D(6-7)			11-NB-A36		
4A	0.12	4.7	11A	0.14	5.5
4B	0.14	5.5	11B	0.04	1.6
4C	0.09	3.5	11C	0.02	0.9
4D	0.08	3.1	11D	0.01	0.4
11-NB-F14			11-NB-B(40-41)		
5A	0.00	0.0	12A	0.13	5.1
5B	0.08	3.1	12B	0.09	3.5
5C	0.00	0.0	12C	0.07	2.7
5D	0.01	0.4	12D	0.08	3.1
11-NB-D19			11-NB-D43		
6A	0.06	2.3	13A	0.06	2.3
6B	0.06	2.3	13B	0.04	1.4
6C	0.05	2.0	13C	0.06	2.3
6D	0.08	3.1	13D	0.05	2.0
11-NB-B23			11-NB-D48		
7A	0.10	3.9	14A	0.14	5.5
7B	0.00	0.0	14B	0.04	1.4
7C	0.00	0.0	14C	0.02	0.8
7D	0.00	0.0	14D	0.01	0.4
			11-NB-C51		
			15A	0.09	3.5
			15B	0.02	0.8
			15C	0.02	0.6
			15D	0.01	0.5

DEPTH: A=0"-1/2" B=1/2"-1" C=1"-11/2" D=1

SAMPLE	A	B	C	D
1	0.21	0.06	0.06	0.10
2	0.08	0.05	0.00	0.05
3	0.13	0.02	0.01	0.01
4	0.12	0.14	0.09	0.08
5	0.00	0.08	0.00	0.01
6	0.06	0.06	0.05	0.08
7	0.10	0.00	0.00	0.00
8	0.12	0.00	0.00	0.00
9	0.10	0.10	0.02	0.08
10	0.14	0.01	0.00	0.00
11	0.14	0.04	0.02	0.01
12	0.13	0.09	0.07	0.08
13	0.06	0.04	0.06	0.05
14	0.14	0.04	0.02	0.01
15	0.09	0.02	0.02	0.01
AVE. %	0.11	0.05	0.03	0.04
AVE.#/CY	4.23	1.93	1.10	1.49