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NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

NCHRP REPORT 558

Manual on Service Life of Corrosion-Damaged Reinforced Concrete Bridge Superstructure Elements

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Sterling, VA

Subject Areas
Bridges, Other Structures, and Hydraulics and Hydrology • Materials and Construction • Maintenance

Research sponsored by the American Association of State Highway and Transportation Officials
in cooperation with the Federal Highway Administration

TRANSPORTATION RESEARCH BOARD

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NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

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FOREWORD

By Amir N. Hanna
Staff Officer
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This report is a manual that provides step-by-step procedures for assessing the condition of corrosion-damaged bridge elements. It also includes procedures that can be used to estimate the expected remaining life of reinforced concrete bridge superstructure elements and to determine the effects of maintenance and repair options on their service life. This manual should be of interest to state engineers and others involved in the design, construction, and maintenance of highway bridges.

Corrosion-induced deterioration of reinforced concrete bridge superstructure elements is a common and costly problem in the United States. A rational decision regarding maintenance, repair, or replacement of such deteriorated elements must take into account the condition of the element, the extent of deterioration, the expected remaining service life, and the impact of alternative maintenance and repair options on the service life of such elements. However, available publications do not provide reliable procedures for evaluating the existing condition of corrosion-damaged elements or approaches for comparing the effectiveness of maintenance and repair alternatives. Without such information, the selection of the optimum repair strategy cannot be ensured, leading to the application of a less desirable and often more costly repair strategy. Thus, research was needed to identify suitable procedures for assessing the condition of corrosion-damaged bridge elements, estimating the expected remaining service life of such elements, and determining the effects of maintenance and repair options on their service life. Relevant background information was provided by Michigan State University under NCHRP Project 18-6; subsequent work and manual development was performed under NCHRP Project 18-6A.

Under NCHRP Project 18-6A, "Service Life of Corrosion-Damaged Reinforced Concrete Superstructure Elements," CONCORR, Inc., of Sterling, Virginia, was assigned the objective of developing a manual, for consideration and adoption by AASHTO, that provides step-by-step procedures for (1) assessing the condition of reinforced concrete bridge superstructure elements subjected to corrosion-induced deterioration, (2) predicting the remaining service life of such elements, and (3) quantifying the service life extension for such elements expected from alternative maintenance and repair options. The research was limited to concrete bridge superstructure elements reinforced only with epoxy-coated and/or "black" reinforcing steel and did not include prestressed concrete elements or concrete elements reinforced with other steel types.

To accomplish this objective, the researchers performed the following tasks:

1. Developed a protocol for condition assessment of reinforced concrete bridge superstructure elements;

2. Developed a model for corrosion initiation for use in projecting future damage;
3. Developed a model for estimating expected remaining service life that incorporated the concept of Susceptibility Index—an approach for interpreting the distribution of chloride ions in the concrete;
4. Presented the remaining service life estimation process as computational software in the form of a Visual Basic–driven Excel spreadsheet to facilitate use;
5. Conducted tests on three bridges and used the results in modeling the corrosion process and in validating the model's ability to project future deterioration;
6. Provided guidance on the service life that may be attainable with different corrosion control, repair, and rehabilitation techniques; and
7. Organized relevant information in the form of a manual to facilitate use by practitioners.

The manual presented herein provides a systematic approach for assessing the condition of corrosion-damaged reinforced concrete bridges and comparing the effects of alternative repair strategies on service life; it is recommended for consideration and adoption by AASHTO.

The research agency's final report contains the data used in the development and validation of the service life model described in this manual; it is not published herein. The report is available on the web as *NCHRP Web-Only Document 88* at http://trb.org/news/blurb_detail.asp?id=6150. Also, the computational software (Excel spreadsheet) for the service life estimation process is available on the web at http://trb.org/news/blurb_detail.asp?id=6149.

CONTENTS

PREFACE

Corrosion-induced deterioration of reinforced concrete bridge superstructure elements is a common and recurring problem in the United States. A rational decision regarding maintenance, repair, or replacement of such deteriorated elements must take into account the condition of the element, the extent of deterioration, the expected remaining service life, and the impact of alternative maintenance and repair options on service life of such elements. However, available publications do not provide reliable procedures for evaluating the existing condition of corrosion-damaged elements or approaches for comparing the effectiveness of maintenance and repair alternatives. Without such information, the process of selecting the optimum repair strategy becomes difficult.

Thus, a need was felt for the development of suitable procedures for assessing the condition of corrosion-damaged bridge elements, estimating the expected remaining service life of such elements, and determining the effects of maintenance and repair options on their service life. To meet this need, The National Cooperative Highway Research Program (NCHRP) initiated NCHRP Project 18-6A, "Service Life of Corrosion-Damaged Reinforced Concrete Superstructure Elements." The objective of the project was to develop a manual, for consideration and adoption by AASHTO, that provides step-by-step procedures for the following:

1. Assessing the condition of reinforced concrete bridge superstructure elements subjected to corrosion-induced deterioration.
2. Predicting the remaining service life of such elements.
3. Quantifying service life extension for such elements expected from alternative maintenance and repair options.

This manual was developed as part of this project. The scope of the manual is limited to conventionally reinforced bridge superstructure elements. The first five chapters of the manual discuss the state of the art and lay out the logic for the proposed protocols for assessing condition, predicting remaining service life, and, to some degree, quantifying the service life extension when certain alternatives are used in the repair and rehabilitation of bridge superstructure elements. The remaining three chapters of the manual provide step-by-step procedures for implementing the protocols developed in the first five chapters.

The protocol for condition assessment developed in this effort is integrated into the requirements of the National Bridge Inspection Standards (NBIS), thereby making the implementation of the protocol easier for local, state, and federal agencies. The requirements of condition assessment have been kept to a minimum, recognizing the scarcity of resources experienced by almost all governmental agencies. A well-defined procedure is proposed that would allow the owner agencies to perform minimal assessment to obtain sufficient information on their bridge superstructure elements and to plan the allocation of resources.

A mathematical model was developed for the initiation of corrosion on both black and epoxy-coated rebars. This model allows the users to estimate past progression of damage and project the development of future damage in terms of percent damage of the surface area under consideration. This approach allows each user to develop the criteria for end of service life that suits specific needs. A copy of the code (a macro in a spreadsheet) developed specifically to validate the model during the project is also made available for the users and can be accessed at http://trb.org/news/blurb_detail.asp?id=6149. This macro was not designed for public distribution and may not be as user friendly as desired; familiarity of using a spreadsheet program is required to use this macro. Chapter 8 deals with step-by-step procedures for using the macro.

In addition to the service life model, a new concept of Susceptibility Index was developed. In the past, many agencies have attempted to develop and use numerous complicated decision-making matrixes to identify applicable corrosion control systems. The Susceptibility Index developed in this project allows the use of a number to describe the distribution of chloride ions in the concrete element at the steel depth and to identify the applicable corrosion mitigation systems. The proposed correspondence between the Susceptibility Index and corrosion control systems should be considered as a suggestion and should be modified as required.

Based on a literature survey, an attempt has been made to provide some guidance with respect to additional service life that may be attainable using various corrosion control and repair and rehabilitation techniques. Sufficient information from independent sources was not available to provide a conclusive figure for additional service life for many of the technologies discussed in the manual. Additional service life attainable with any specific corrosion control or repair and rehabilitation technique is dependent on many factors. The most important of these factors is the applicability of that particular technique to the subject structure based on its corrosion condition, presence of other deterioration processes, and the exposure environment. In addition, the quality of the design and the application of the technique also significantly impact performance.

The mathematical model developed in this effort was validated against three bridge structures located in different environments in the United States. Two evaluations were conducted 2 years apart to ascertain the condition of the structure. The results of the first evaluation were used to model the corrosion process and to calibrate the model, and the results of the second evaluation were used to validate the ability of the model to project future deterioration. The results of the validation can be found in the appendix of the final report, which is available online as *NCHRP Web-Only Document 88* at http://trb.org/news/blurb_detail.asp?id=6150.

This manual is intended to be used by personnel with training and experience in repair and rehabilitation of reinforced concrete structures suffering from corrosion-induced deterioration. The application of the protocol in the manual will require the expertise and experience of both engineering and maintenance staff.

CHAPTER 1

Introduction

The National Bridge Inventory Database, maintained by the Federal Highway Administration, as of 2002 contained a total of 587,964 bridges. The average age of bridge structures in this database is 40 years, and 41 percent of the bridges are at least 40 years old. Over the past two decades significant attention has focused on the condition of the nation's aging highway bridge infrastructure. Several independent evaluations of the condition of the nation's infrastructure based on condition ratings contained in the database have been performed [1-3]. These studies ascertained that 14 percent of the bridges were rated structurally deficient and the primary cause of the deficiency was corrosion of reinforcing steel. The cost to maintain the nation's bridges during the 20-year period from 1999 to 2019 is estimated to be $5.8 billion per year, and the cost to improve and eliminate deficiencies over the same period is $10.6 billion [3]. A cost-of-corrosion study determined that the annual cost of corrosion to all bridges (including steel bridges) is $8.29 billion, and this estimate does not include indirect cost incurred by the traveling public due to bridge closures [2].

Considering the magnitude of the problem, a well-designed strategy needs to be developed to identify, maintain, repair, and/or replace the existing bridge infrastructure. The primary goal of this manual is to provide such a strategy. The scope of this manual is limited to managing the impact of corrosion of conventional reinforcing steel in bridge decks and other superstructure elements.

To address the ongoing corrosion deterioration, a bridge owner has to make decisions to maintain, repair, or replace the structure based on its present and future expected condition and to determine what alternative materials and methodologies should be used. Because of a lack of nationally accepted decision-making processes, most owners presently use local experience and expertise to make such complex decisions. Such decision-making processes have often resulted in inefficient, costly, nonstandard, and nonoptimal solutions. There is a strong demand for a protocol capable of determining the optimal course of action (maintenance, repair, or replacement) and assisting in the selection of materials and methodology.

To determine the optimal course of action, information on the present condition of the structure and the expected deterioration in the future is required. The present condition of the structure provides information on the quantity of repairs required and the type of repairs necessary. However, the expected future deterioration allows the owner to determine the efficacy of the repairs and assists in selecting a repair and prevention strategy that could minimize the life cycle cost of maintenance for the period of life desired from the structure. For example, if all delaminations and spalls were repaired on a concrete bridge deck that was uniformly contaminated with chloride ions, then, in the ear future, damage in areas that were not previously repaired can be expected. The repairs do not impact corrosion in areas not presently requiring repairs and may, under certain circumstances, accelerate it. If corrosion-induced damage continues to occur after the first repair is performed, then regular repair cycles will be required that will result in significant expense. The regular repair cycles can be avoided if the propensity for future corrosion-induced damage was known and appropriate measures were taken to control it. On the contrary, if it is known that the expected future damage was minimal, then that structure can be slated for maintenance at a later date and the present funds focused on structures with a higher level of urgency. The knowledge of future corrosion activity on the structure helps owners to identify cost-effective maintenance options.

The knowledge of future activity can be obtained in two formats, one as the remaining service life of the structure and the other as a function of concrete deterioration with time. To use remaining service life, the owner has to formulate a criterion that defines the end of service life. Numerous such criteria have been defined by researchers and owners. The service life approach provides the owners with the information as to when in the future, according to specific criteria, the structure will need to be maintained, repaired, or replaced. The other approach, function of concrete deterioration with time, provides the owners with information on the progression of damage in the future that can be used to decide what action is most appropriate and when. The owners can also use that information to determine the end of service life based on specific criteria. The difference in the two approaches is simply in the presentation of the output of the modeling and not in the modeling process. The process for predicting remaining service life can also be used to project future damage as a function of time. Because the second approach is more flexible, it is used in this manual. However, for simplicity and keeping in conformance with the naming convention in the literature, the modeling process is termed "service life modeling."

There are several approaches for estimating the structure's future damage. One such approach is to assume that the process that generated the present damage will continue to do so at the same rates in the future. In this approach, one would model the deterioration process and validate it against present damage on the structure and then use the model to project into the future. This validation can be performed for one or more data points from the past. However, it should be recognized that the rates and the processes that resulted in the present damage may not remain the same. For determining a cost-effective bridge maintenance strategy, the order of the magnitude of repair is more important than the exact amount of damage, and this approach is reasonable.

Although several mathematical models have been proposed to model the corrosion process based on the extent of deterioration and the presence or absence of deleterious agents, none have been verified in a scientific manner or have been standardized for use by the bridge community. In this study the model developed was statistically validated on three bridge structures located in Kentucky, Ohio, and Maryland, and a report documenting the results is presented in the appendix.

For any valid model to be successful, standardized input data must be available. Numerous destructive and nondestructive technologies are available for ascertaining the existing condition of the concrete element and quantifying the presence of deleterious agents. Several protocols for evaluating reinforced concrete bridge superstructures have been proposed. A functional protocol for evaluation must be complete, comprehensive, and specific to the model used for estimating remaining service life. Thus, such a protocol was developed to provide the necessary input for the model validated in this effort. Accuracy of any model is dependent on the sufficiency and quality of the input data. However, considering the limited resources of various state highway agencies, a practical compromise was devised to obtain reasonable accuracy with reduced data collection.

In addition to selecting the course of action, materials, and methodologies, an evaluation of the impact of various alternative materials and methodologies on the extension of remaining service life must be performed to allow for the selection of an optimal solution. Many materials and methodologies introduced to counteract corrosion have been evaluated on field structures, and field performance information is available in literature. This is provided to better ascertain the future performance and need for maintenance.

Objective and Audience

The primary objective of this project was to develop a manual, for consideration and adoption by AASHTO, that provides step-by-step procedures for the following:

1. Assessing the condition of reinforced concrete bridge superstructure elements subjected to corrosion-induced deterioration.
2. Predicting the remaining service life of such elements.
3. Quantifying the service life extension for such elements expected from alternative maintenance and repair options.

The scope of work for this project was limited to bridge superstructure elements reinforced with black reinforcing steel and, to some extent, epoxy-coated reinforcing steel. It did not include prestressed concrete elements and other modified steels.

This manual is targeted toward engineers and maintenance personnel charged with maintaining the bridge structures. This manual can be used by state and other highway agencies and adopted with or without modifications as guidelines for use within the organization.

Manual Organization

Chapters 1 to 5 provide information and explanation of the various facets of the protocol proposed in this manual, and Chapters 6 to 8 provide detailed step-by-step procedures for implementing the protocol.

CHAPTER 2

Condition Evaluation of Superstructure Elements

Introduction

Condition evaluation or bridge inspections are performed by every bridge owner as mandated by the National Bridge Inventory Program (NBIP). This program resulted in the development of the National Bridge Inspection Standards (NBIS), which prescribe minimum requirements for the inspection of highway bridges on public roads. The visual inspection method is by far the most common form of nondestructive evaluation (NDE) technique used to satisfy the NBIS requirements. The *AASHTO Manual for Condition Evaluation of Bridges* outlines five different types of bridge inspections:

1. **Initial Inspection—**Upon the completion of a new bridge structure, this inspection is performed to obtain all structure inventory and appraisal data and to determine the baseline structural conditions and identify current or potential problem areas.
2. **Routine Inspection—**This inspection is performed on a regular interval of time (usually every 2 years), in accordance with the requirements prescribed by the NBIS, to determine the physical and functional condition of the bridge and to identify changes since the last inspection.
3. **Damage Inspection—**Damage resulting from environmental or human actions triggers this inspection, whose primary goal is to identify the need for further action.
4. **In-Depth Inspection—**This inspection focuses on certain sections of the bridge structure to investigate deficiencies not detected during Routine Inspection.
5. **Special Inspection—**This inspection is conducted to monitor a single known defect or condition.

Routine Inspections are performed on a regular basis and provide a good history of the superstructure elements. The other inspections are performed only when triggered by a specific event. The Routine Inspection indicates the onset of or ongoing corrosion on a superstructure element based on the results of the visual survey; however, it does not provide sufficient information for planning purposes. In addition to collecting visual information, a few agencies collect concrete samples to analyze the distribution of chloride ions in concrete, measure the quantity of damage, or conduct a half-cell potential survey to determine the distribution of active sites where corrosion may be occurring. The results, in conjunction with the visual survey results, are used to determine the condition rating of the bridge element. In general practice, when the Routine Inspection identifies that the condition of the structure has degraded sufficiently to require a repair, a more detailed inspection is performed to prepare construction documents. The primary goal of this detailed inspection is to determine the quantity of repairs, the type of repairs, and the need for preventive measures. The scope of such inspections varies from state to state and agency to agency depending on local needs and available resources. Some agencies are more sophisticated than others and use one or more of the newer technologies available to do an inspection, model the damage to predict future progression, identify the repair, identify applicable corrosion mitigation technologies, and perform life cycle cost analysis.

This strategy puts a significant financial burden on the owner because it is does not allow the owner to manage the inventory cost-effectively. The owner is always responding to an urgent need for repairs, and the superstructure elements are often repaired when they have experienced significant damage. If a preventive strategy were to be used, the cost of repairs and the overall life cycle cost of the structure could be reduced. To implement such a strategy, a modeling tool is required that would allow the owner to estimate the future performance of the structure. An inspection protocol is proposed herein that uses such modeling and also minimizes the amount of survey work to be performed. In addition to modeling the expected future damage, this protocol proposes a method of quantifying the future propensity for corrosion by using an index that represents the distribution of chloride ions in the concrete. This index can be used to identify corrosion mitigation technologies that are most suited for arresting corrosion in the future.

Development of an Evaluation Protocol

An optimal approach to superstructure maintenance would be to modify the extent of the Routine Inspection to provide additional information that can be used for modeling and developing a Susceptibility Index (SI) with minimal cost and perform the In-Depth Inspection only when necessary. The additional information from the Routine Inspection can be used to plan and prioritize necessary prevention and repairs. The In-Depth Inspection can be used to collect data necessary to develop the construction documents.

Routine Inspections, as generally conducted by state and local agencies, do not provide sufficient information to correctly identify the corrosion status of the subject superstructure element. It mostly documents the progression of ongoing corrosion-induced damage based on the observation of telltale signs such as rust staining, cracking, delamination, and spalling of concrete on concrete surfaces with no surface treatment and sometimes the presence of chloride ions. On bridge decks that have an asphalt overlay, the initiation and progression of corrosion is not visible, and the Routine Inspection does not provide reliable information. If initiation of corrosion has been observed during Routine Inspection, but sufficient corrosion-induced damage has not yet occurred to necessitate a repair, then it is prudent for the owner to try and determine approximately when the structure would need major construction to restore or maintain its full functionality. Also, when a structure is scheduled for an expansion or an upgrade to meet present codes, it seems appropriate to identify the future performance of the existing sections. If the existing sections are expected to suffer from corrosion-induced damage in the future, a corrosion mitigation system may be installed during the expansion or the upgrade to obtain the desired service life.

To obtain a reasonable estimate of the future progression of corrosion-induced damage, at a minimum, the knowledge of chloride ion distribution, clear concrete cover, and quantity of damage in the form of delamination and spalling is necessary. With this information, a diffusion model can be used to estimate the increase in concrete damage as a function of time. A different approach would be to document the progression of damage with time and to fit a cumulative Weibull distribution curve to the data to predict future progression of damage. In addition to estimating future damage, the model could calculate SI based on the distribution of chloride ions in areas that are presently not damaged. This index would reflect the susceptibility of the undamaged areas to corrosion in the future and allow the owner to effect a corrosion prevention strategy.

The corrosion process occurs in two phases, corrosion initiation and propagation. The rate of corrosion during the initiation phase is low and can increase exponentially with time during the propagation phase depending on the various parameters that control its rate, such as temperature and its variation, availability of oxygen, conductivity of concrete, humidity, ingress of chloride ions, and formation of a macrocell. Therefore, if the corrosion process is allowed to continue to occur unabated, the rate of damage development will increase with time. This generally results in increase in cost as the quantity of repairs increases and often will require a major construction effort. With the increasing rate of corrosion and quantity of damage, the pool of suitable corrosion mitigation systems diminishes, more aggressive and costly corrosion systems become necessary, and replacement of the concrete element becomes a more viable option. During the early stages of corrosion, fewer repairs and less costly corrosion mitigation systems can be used, which likely reduces the life cycle cost of the structure. Use of a protocol with modeling damage and calculation of the SI would help identify options for implementing a prevention regime rather than the usual respond-to-urgency approach.

For bridge superstructures exposed to adverse environments, the question is not if corrosion will occur, but when it will occur and when a major repair will be required. Two types of corrosion condition surveys are proposed: the "Preliminary Corrosion Condition Evaluation" (PCCE) and the "In-Depth Corrosion Condition Evaluation" (In-Depth Inspection).

The PCCE should be performed in conjunction with Routine Inspection, but not at the 2-year frequency. It should be performed after the first signs of corrosion initiation are observed during a Routine Inspection, preferably during the subsequent inspection. The results of the first PCCE are then used to determine future action. Depending on the need and available resources and the results of the modeling performed with the data collected during the PCCE, several different actions can be taken. The results can be used to identify and install an appropriate corrosion mitigation system to arrest future corrosion-induced damage, or they can be used to just monitor the health of the structure and to optimally schedule the repair.

If the PCCE is used to monitor the health of the structure for the purpose of determining the optimal time to repair, more than one PCCE may be performed. For example, if the first PCCE suggests that corrosion initiation will not occur for 20 years, then another PCCE is not necessary for another 10 to 15 years. When the next PCCE should be performed is dependent on the type of structure, the wearing surface (bare concrete or asphalt overlay with or without membrane), and the presence of epoxy-coated rebar. The inclusion of the PCCE in the Routine Inspection should in no way impact the

Routine Inspection schedule, and the PCCE should be included in the Routine Inspection only as necessary.

The scope of work for the PCCE will depend on the structure, the owner's needs, and the available resources. The presence of an asphalt overlay with or without a membrane and the presence of epoxy-coated rebar will significantly impact the scope of work. A discussion of the recommended scope of work for each instance is presented later in this chapter.

When a bridge superstructure is already slated for repairs, based on either Routine Inspection or the PCCE, an In-Depth Inspection needs to be performed to obtain accurate repair quantities, determine the type of repairs (full depth, partial depth, or type of crack repair), ascertain the need for corrosion mitigation, and select a corrosion mitigation system for the structure. In planning the repair of a bridge superstructure element, it is important to examine the areas that have not yet deteriorated and will not be repaired. If the undamaged areas have a propensity for corrosion, some form of corrosion mitigation system will be required to ensure that corrosion in these areas does not reduce the service life of the concrete element. In addition, corrosion may be accelerated by the coupling of repaired areas and adjacent chloride contaminated areas. Under such circumstances, a corrosion control system will be necessary to stop or control corrosion in the undamaged areas to obtain the desired service life.

Additional tests are also performed in this evaluation to verify the existence of any other concrete deterioration processes—such as freeze-thaw damage, alkali-silica reactivity (ASR), and ettingrite formation—and to ascertain the compatibility of potential corrosion control systems. For example, if freeze-thaw damage or ASR is present, then the repair and control system must include a mechanism to control these deterioration processes in addition to corrosion. The knowledge of electrical continuity will be required if cathodic protection is to be considered for corrosion control. Data collected during previous PCCE can be used in the In-Depth Inspection, although some of the data will need to be augmented. The damage survey will need to be conducted with a higher level of accuracy to obtain reasonable quantities for construction. This inspection or evaluation should not be performed any earlier than 2 years from the actual construction because damage quantities can change, especially in the later stages of the corrosion process.

The following sections describe different test methods and techniques used in each type of evaluation.

Test Methods and Test Techniques

Numerous test methods and test techniques are available for use in evaluating bridge superstructures. Following is a discussion of these methods and techniques. Applicability and limitations of each test method or technique are identified.

Visual Survey

A visual survey is conducted to observe and document the overall condition of the structure under investigation. This survey is a vital part of the evaluation because the use of subsequent test procedures depends on the visual assessment of the structure. The inspection should follow an orderly progression over the structure so that no section of the deck or superstructure is overlooked. A visual survey should be conducted in accordance with ACI 201.1R-92, "Guide for Making a Condition Survey of Concrete in Service."

The following deteriorations are typically found on bridge superstructures and should be documented:

- Cracking,
- Spalling,
- Scaling,
- Rust staining,
- Efflorescence, and
- Patching or existing repairs.

The visual survey can be performed at various levels of accuracy. The highest level of accuracy would require documentation of each occurrence of deterioration and its exact dimensions. This would require significant time and labor resources. It is often not necessary to document deterioration to that level of accuracy. However, the following minimum documentation is recommended:

- Type and frequency of cracking. The type of cracking provides clues as to the cause of it. The frequency can be classified by using a rating scheme—such as very slight, slight, moderate, severe, and very severe—or can be documented as density (i.e., total length of cracks per unit of area). The density measurement of cracks can be limited to cracks exceeding certain width criteria, and the crack survey can be limited to a select section of the superstructure. It would be impractical to perform such a survey on the entire bridge deck or all beams and girders. This information becomes more relevant and results in a better estimate of quantities if crack repair in the form of routing or injection will be necessary.
- Location and dimension of spalls. Because all spalls, along with delaminated areas, will need to be repaired, accurate estimates of quantities are essential for preparing contract documents.
- Location of scaling. This should include an estimate of impacted area.
- General location of rust staining. This will give an idea of what parts are corroding.
- Locations and dimensions of all existing repairs or patches.

Photographic and video documentation of visual survey significantly improves the quality of the data and allows a review of the damage in more detail at a later date.

Delamination Survey

A typical form of deterioration induced by corrosion of reinforcing steel is cracking and delamination of the concrete. A delamination is a separation of concrete planes resulting from tensile failure. Depending on the ratio of cover to bar spacing, the fracture planes will either form cracks or cause a delamination parallel to the surface. The "Diagnosis of Deterioration in Concrete Structures" states that cracks are the likely results of a ratio of cover/bar diameter equal to 1, larger cracks and risk of delamination when cover/bar diameter equals 2, and delamination when cover/bar diameter equals or exceeds 3 [4]. The size of the delamination generally increases with time due to continuation of the corrosion process, freeze-thaw cycles, and impact of traffic. Upon attainment of critical size, a delamination will result in a spall. Upon reaching a critical quantity, delaminations can impact the structural integrity of the concrete element.

Several different techniques and types of equipment are presently available to detect delaminations. All of these techniques are based on the propagation or reflection of energy. Mechanical impact energy is used by techniques such as sounding and impact-echo. Acoustic energy is used by ultrasonic pulse velocity. Thermal and electromagnetic energy are used by infrared thermography and ground-penetrating radar, respectively.

Sounding

Perhaps the most commonly used and inexpensive method for determining the presence and extent of delaminations is sounding. Depending upon the orientation and accessibility of the concrete surface, sounding can be performed with a hammer, steel rod, or a chain. The concrete is struck with a hammer or rod, or a chain is dragged across a horizontal surface. Good concrete with no delaminations produces a sharp ringing sound; delaminated areas emit a dull, hollow sound. ASTM C 4580-86, "Standard Practice for Measuring Delaminations in Concrete Bridge Decks by Sounding," governs this test procedure. For bridge decks, a chain is dragged along the concrete surface to locate delaminated areas. Edges of delaminated areas are then defined using a steel rod or hammer. Delaminated areas are outlined on the concrete surface, measured, and recorded on drawings with reference to the survey grid coordinates. As delaminations are generally irregular in shape, the irregular shape is enclosed in a rectangular or a triangular shape, which is documented. During concrete repairs, regular geometric sections of concrete are removed and repaired because of the increase in efficiency and reduction of costs.

This technique depends on operator judgment and is prone to operator errors. Operator fatigue and background noise can reduce the accuracy and the speed of the survey. Often times, the sounding technique is combined with the use of sand and vibration of the concrete surface. On bridge decks when ambient noise is high, sand can be broadcast on the surface of the deck. The movement of the sand when a hammer is struck on the concrete surface is used to detect delamination. Sand particles on a delaminated surface will bounce up because of the rebound from the delaminated plane. If sand is not available, the operator can place his or her palm on the concrete surface and feel the rebound when the hammer is struck on the concrete surface.

Impact Echo

Impact echo is a process in which a mechanical impact produces a stress wave in a material. This stress wave travels through the material until a discontinuity is encountered. At this discontinuity, a portion of the stress waves are reflected. For any isotropic and elastic media, there are three modes of stress wave propagation: dilatational, distortional, and Rayleigh waves. Dilatational waves (P-waves, or compression waves) produce particle motion that is parallel to the direction of stress wave travel. Distortional (S-waves, or shear waves) produce particle motion that is perpendicular to the direction of stress wave travel. Rayleigh waves (R-waves) travel near the surface of the material in a retrograde elliptical motion.

Typically, a mechanical impact is produced by striking a small (diameter less than 2 inches) metal sphere on the surface of the material to be tested. This impact produces stress waves that propagate in all directions. As these stress waves propagate into a material, reflections occur at interfaces between two materials of differing acoustic impedance. A transducer, located at the surface near where the impact initiated, can receive these reflected stress waves. By measuring the time difference between the initial impact and the receipt of reflected stress waves from a boundary, the depth to that boundary can be estimated. It is important to have a basic understanding of the velocity of the stress wave traveling through the material. A typical P-wave velocity of 9,800 to 14,800 feet per second is assumed for concrete (depending on the proportions and specific gravities of the constituents). This value can also be measured by performing impact echo where the depth to a boundary or thickness is already known.

The resolution and depth of penetration are controlled by the impact duration or contact time between the metal impactor and the surface of the material. The impact duration is proportional to the diameter of the impactor;

shorter duration impacts are produced with impactors of smaller diameter. In addition, the frequency of the stress wave is proportional to the impact duration; an impact of shorter duration produces a stress wave of higher frequency. Thus, an impactor of small diameter will produce a higher-frequency stress wave. A higher-frequency stress wave will provide greater resolution but less depth of penetration.

Impact echo can be a useful technique given the proper scenario. The surface texture of the material to be tested is important to consider. Although impact echo can be used to perform analysis on hot-mix asphalt (HMA) pavements, because of the typically rough surface texture, the usefulness of this technique in this application may be limited. Impact echo, however, has been employed as a useful technique to monitor the condition of various concrete structures [5, 6].

Ultrasonic Pulse Velocity

Pulse velocity works in much the same manner as impact echo. The primary difference is in the stress wave that is applied. While impact echo employs a stress wave (resulting from a mechanical impact), ultrasonic pulse velocity uses a high-frequency (greater than 20 kHz) sound wave emitted from a piezoelectric transducer. The pulse velocity method employs two transducers, one to send and one to receive the sound wave. Using the two transducers, the travel time is calculated. If the thickness of the material is known, the travel time can be converted into velocity and compared with typical values (sound wave speed of approximately 12,100 feet per second for concrete). By comparing the calculated velocity with typical values, a determination can be made about the condition of the material under study.

Several factors may affect the accuracy of the ultrasonic pulse velocity method [7]. Because proper contact between the transducers and the surface of the material being tested is critical, a couplant is used to ensure that there is not an air gap between the surface of the material and the transducer. The temperature and the moisture content of the concrete may also play a small role in affecting the travel time of the ultrasonic pulse. In addition, the minimum spacing between the transducers (i.e., the thickness of material being tested) should be greater than approximately 4 to 6 inches. This value will depend upon the velocity of the ultrasonic pulse in the material and the frequency of vibration of the piezoelectric material in the transducer.

As with impact echo, there may be some difficulties in using ultrasonic pulse velocity on an HMA surface because of the rough surface texture typical of HMA layers and the difficulty of establishing a proper contact with the surface.

Infrared Thermography

Infrared thermography is based on the principle that defects within a material will alter the way heat flow is dissipated at the surface of that material [8, 9]. These changes in surface temperature can be measured to locate and possibly determine the quantity of subsurface defects. Heat flow will occur when the temperature of the material differs from the temperature of its surroundings.

A common practice when using infrared thermography on large structures is to perform the testing early in the morning while the sun is heating a structure or just after dark when the energy from the sun is being released into the environment. Since concrete structures typically involve large areas, natural sources of heat (such as the sun) can be useful in performing infrared thermography. However, this form of passive heating tends to reduce the ability to resolve the dimensions of specific defects since radiation from the sun is not very strong and thus it takes a significant amount of time to provide enough heat to flow through a large structure [10]. Several factors can influence the accuracy of infrared thermography outside the laboratory setting, including surface texture, wind speed, and surface moisture. Each of these factors can influence the way that heat is adsorbed or dissipated.

Ground-Penetrating Radar

Ground-penetrating radar (GPR) operates by directing electromagnetic waves toward an object of interest. In this technique, as the electromagnetic waves pass through the object, energy is reflected at boundaries between two materials possessing differing values of the dielectric constant (i.e., real part of the complex permittivity). While some energy is reflected at each boundary, the remainder continues propagating into the object. Typical dielectric constant values seen in civil engineering applications include air = 1, concrete = 8 – 12 (depending on material properties and proportions of constituents), and water = 81. More energy is reflected at an interface between two materials where the difference in the dielectric constant is greatest. Thus, it is easier to detect (more energy is reflected) a defect such as a water-filled delamination (water/concrete interface) rather than a defect such as an air-filled delamination (an air/concrete interface). In addition, less energy is reflected back from the air-concrete interface because of the similar dielectric constants, and more energy is available to travel inside the concrete.

Typical frequency ranges of operation for GPR are from 80 MHz to 2 GHz, with the higher-frequency antenna being more suited to bridge deck studies. An antenna of higher frequency will result in greater resolution (i.e., ability to differentiate between objects), but less depth of penetration. Research studies have shown that GPR can be used in certain instances to monitor subsurface condition of bridge decks. Some researchers used GPR to locate defects and reinforcement bars [11, 12].

The propagation of GPR through concrete is not completely understood, and many factors impact its propagation and reflection of the signal. At present, GPR is primarily used to measure slab thickness and location of embedded metals; all other applications for detecting cracks, delaminations, chlorides in concrete, etc., are qualitative comparisons only.

Cover Depth Measurements

The thickness of concrete cover over reinforcing steel has a significant influence on the time to initiation of corrosion when chloride ions are diffusing into the concrete element from the environment. Shallow cover on a structure will lead to more rapid accumulation of chloride ions at the steel depth in excess of the threshold required to initiate corrosion and subsequently results in faster development of concrete damage.

The location of a reinforcement bar and its depth of cover can be obtained nondestructively by using a pachometer or a covermeter. These devices measure variations in magnetic flux caused by the presence of reinforcement bars to locate their presence and depth. In general, detection of the reinforcement is very accurate given proper usage by the operator. Some pachometers or covermeters provide an estimation of the bar size; however, they may have an error of up to one bar size. Measurements of the cover depth are generally more accurate when the structure is lightly reinforced. As an alternative method, small holes may be drilled into the concrete to measure the cover. This method can be more accurate, but it also introduces defects into the structure.

Commercially available covermeters are usually compact with single-element, hand-held probes and allow easy access to structural elements. They are useful for locating and determining the cover over individual reinforcement bars. However, they can be time consuming when trying to determine location and cover depth over large areas.

Chloride Ion Content Analysis

Chloride ions are the primary cause of reinforcing steel corrosion. The primary sources of chloride ions are chloride-bearing admixtures used during construction, chloride-contaminated constituents (water or aggregate) used during construction, deicing salts applied to surface of structure, and air-borne chlorides and direct exposure to sea water in marine environments.

It is generally accepted that corrosion of reinforcing will only occur once a threshold value of chloride ion content adjacent to the bars is reached. It is generally given that this threshold value is approximately 0.025% to 0.033% by weight of concrete. Hence, it is important to determine the chloride ion distribution in a structure under investigation to be able to determine its susceptibility to corrosion. Chloride profiles (chloride concentration versus depth from the surface) provide valuable information on the source of the chloride ions and the apparent rate of diffusion of the chloride ions in the concrete. The rate of diffusion can be used to calculate when the chloride ion concentration at the steel/concrete interface will exceed the threshold required to initiate corrosion, if it has not already exceeded it.

The chloride content in concrete can be determined through analysis of powdered concrete samples. Samples can be collected on-site at different depths up to and beyond the depth of the reinforcing steel using a hammer drill. Extreme care should be exercised to avoid inadvertent contamination of the samples. Alternatively, cores can be collected and powdered samples can be obtained at different depths in the laboratory.

Chloride ions in concrete exist in two forms, chemically bound and soluble in the concrete pore water. The chloride ion content of concrete is usually measured in the laboratory using wet chemical analysis. The total chloride (or acid soluble) test method measures the sum total of all chemically bound and free chloride ions in the concrete. The water soluble test method measures only the free ions soluble in pore water. The water soluble chloride ions are linked to the initiation of corrosion. Because the water soluble test method is not very accurate or repeatable, the general practice is to use the acid soluble test method. Most researchers have used the acid soluble test method and reported varying threshold values for corrosion initiation depending on design of the concrete mix. It has been reported by several researchers that the chloride to hydroxide ratio is more important than the actual concentration of the chloride ions. However, in practice it is not easy to determine the chloride by hydroxide ratio.

Although laboratory testing is most accurate, it is also time consuming. It often takes weeks to produce usable results. As a result, field test kits have been developed that allow more rapid determination of chloride levels on-site. All field test kits use the acid method for analysis; they are typically not as accurate as laboratory analysis, but they do provide good correlation with the laboratory test method. Therefore, a correction factor must be applied depending on the type of field test kit used.

Results of the chloride content analysis are reported as either percentage chloride by weight of concrete, parts per million (ppm) of chloride ions, percentage chloride per weight of cement, or weight of chloride per volume of concrete. Expressing the percentage of chloride per weight of either cement or concrete or the weight of chloride per volume of concrete requires the knowledge of the cement content (typically 657 lb/yd^3) or a unit weight (typically 3,915 lb/yd^3).

Electrical Continuity Testing

Continuity testing is performed to determine if various metallic objects (usually reinforcing bars) within the concrete are in direct contact, or electrically continuous, with each other. This type of testing is needed for the following three reasons:

- Results of this test are needed prior to conducting the corrosion potential survey and rate of corrosion tests.
- Direct contact between reinforcing steel and other metals (e.g., aluminum or galvanized steel) can lead to corrosion due to dissimilar metals and the presence of electrical continuity supports the formation of macrocells.
- The state of electrical continuity of all embedded metals must be known when considering cathodic protection as a long-term protection option.

The electrical interaction of embedded reinforcing bars and external metallic components of bridge superstructures influence the results of all electrical tests run during a condition survey. The corrosion potential survey is particularly sensitive to continuity because all the reinforcing steel within a given area must be continuous. If a ground connection is made to a bar or other metallic component that is electrically isolated from the reinforcing bar in the survey area, the readings will essentially provide remote corrosion potential measurements of the isolated ground and are therefore meaningless.

Normally, steel chairs, direct contact at intersection points, and wire ties provide good electrical continuity throughout cast-in-place sections of a bridge. Precast concrete bridge members also typically exhibit good electrical continuity. However, electrical continuity should always be verified during a condition survey. Continuity across expansion joints, between scuppers and reinforcing bars, and between railings and reinforcing bars is always suspect and requires verification. Any metallic component can be used as the ground location for testing if it is electrically continuous to the reinforcing bars being tested. During the survey planning stage, proposed potential grid map locations should be laid out to avoid spanning obvious discontinuities.

Theoretically, when epoxy-coated reinforcing steel is encountered during evaluation of a structure, every bar should be electrically isolated (i.e., electrically discontinuous). However, previous experience in testing these structures has shown that the degree of electrical discontinuity can range from partial to complete depending on the structure and construction practice. Therefore, before conducting electrical tests on structures containing epoxy-coated reinforcing steel, electrical continuity testing should be performed.

Corrosion Potential Survey

Because corrosion is an electrochemical process, the electrical potential is a parameter that can be measured to indicate the state of the corrosion process. The corrosion potential of a reinforcement bar provides an indication of the status of corrosion at the measurement site at the time of measurement. A potential difference is measured between a half-cell that is placed on the surface of the concrete structure and a reinforcing bar acting as a ground. These values are compared with empirical values to determine the relative probability of corrosion activity. A surface map of potentials can be created by performing multiple measurements on the concrete surface following a grid pattern.

Corrosion potentials are most often performed using a copper-copper sulfate (Cu-$CuSO_4$) half-cell. In addition, silver-silver chloride (Ag-AgCl) or graphite cells can be embedded as permanent reference cells. The Cu-$CuSO_4$ half-cell is popular because copper is easily maintained at a standard potential over a wide range of operating conditions and because copper sulfate and distilled water are easily obtained.

Corrosion potential surveys are preferably carried out on a regular interval grid such that they can be plotted to create equipotential contour maps. Difficulties in half-cell placement, variations in chloride distribution, and changes in temperature and moisture content affect the half-cell potentials over a wide area. Therefore, evaluating a large number of closely spaced half-cell potentials is necessary.

Each test location to be surveyed must have a unique ground location if the underlying reinforcing steel is not electrically continuous. If continuity testing has verified the electrical continuity, a common ground location can be used for several potential measurements. The size and layout of the half-cell potential survey areas are dictated by the recording memory of the multimeter (if so equipped) and/or the physical layout of the structure. Typically, each span is mapped separately such that a map of each span can be individually printed. The ground location is established by exposing a portion of reinforcing bar and drilling a three-sixteenths inch diameter hole into it. A self-tapping screw can then be inserted and a test lead from the multimeter can be clipped to the screw for a secure connection.

The following guidelines have been developed for evaluating Cu-$CuSO_4$ half-cell potentials of uncoated reinforcing bars in concrete (see ASTM C-876, "Standard Test Method for Half-Cell Potentials of Uncoated Reinforcing Steel in Concrete"):

- A 90% probability of no corrosion activity on the reinforcing bar at the time of measurement exists if the half-cell potential are less negative than –0.200 V.

- An increasing probability of corrosion activity exists if the half-cell potential falls between –0.200 V and –0.350 V. This probability depends on factors such as chloride content at the reinforcing bar level, moisture content of the concrete, temperature, etc. Typically, values within this range are said to have an uncertain probability of corrosion activity.
- A 90% probability of corrosion activity on the reinforcing bar at the time of measurement exists if the half-cell potentials are more negative than –0.350 V.

Comparing differences in half-cell potentials across a structure or in an area of a particular concrete member is more indicative of the probability of corrosion activity. For example, a 5 ft^2 section of deck that has potentials that vary by 100 mV between readings is more active than a similar section with a 30-mV reading variation.

An important application of the corrosion potential survey is to develop a historical record of corrosion potentials for a given structure. If the potential survey is conducted at a regular time interval, variations in the potentials with respect to time can indicate if the corrosion activity of the steel is increasing or if the total area of steel showing active potentials is increasing.

It is important to point out that the corrosion potential is an indicator of the corrosion process and not a measurement of the corrosion rate. Corrosion rate is a function of many parameters, such as temperature, potential difference of an open circuit, concrete resistivity, ratio of anodic and cathodic areas, and rate of diffusion of oxygen to cathodic areas. Corrosion potentials can be measured using any high-quality multimeter (similar to that used for electrical continuity) and recorded manually. In addition, there are several commercial multimeters that have the ability to store readings for later analysis. There are also half-cell arrays that allow for testing at multiple points simultaneously using a single multimeter. These multiple array systems allow large areas (such as bridge decks) to be tested rapidly.

Corrosion Rate Measurement

Techniques for measuring the corrosion rate of reinforcement in concrete have been developed in recent years. These techniques provide information on the rate at which steel is being oxidized. The higher the rate, the sooner concrete cracking and spalling will appear at the surface. This information is very useful in estimating the time to additional damage and in selecting cost-effective repair and long-term protection systems. Although the techniques have been used in electrochemical laboratories for decades, field application of the equipment and methods are relatively recent developments.

Corrosion rate devices apply a small voltage or current perturbation to the reinforcement, and the corresponding current or voltage response is measured. The data are then manipulated using the Stern-Geary equation to derive the rate of corrosion. From a field test standpoint, one of the difficulties is in determining the area of reinforcement tested. This is important since corrosion rate is defined in terms of the corrosion current per unit area of reinforcement. The magnitude of the corrosion current measured is a direct indication of how fast corrosion is occurring on the reinforcement surface. High currents indicate a high corrosion rate, and vice versa.

The measurement of the corrosion rate is only valid for the particular conditions at the time of measurement. For example, ambient temperatures vary throughout the year and concrete moisture may also vary with time; these changes result in a corresponding change in the corrosion rate. Therefore, prediction of future corrosion activity must also include an evaluation of the dynamic environmental effects. The most active corrosion or highest corrosion rates may not be occurring at the time of the field study. Continuous or intermittent monitoring over a period of time provides a more accurate appraisal of the corrosion rate.

Corrosion rate testing (as described above) is only valid for conventional bare (i.e., uncoated) steel. No data interpretation procedures have been developed for epoxy-coated or galvanized reinforcement. Several commercial instruments have been developed to measure corrosion rate; each varies in the interpretation of the results from the respective instrument.

Petrographic Analysis

Several quality control issues and deterioration processes in concrete cannot be easily identified by nondestructive means. They, however, can be more easily discerned by collection of a core and examined petrographically. Petrographic analysis consists of inspection of a freshly fractured and polished concrete surface with the unaided eye and by microscopic examination. Petrographic examination is often supplemented with chemical analysis, X-ray diffraction analysis, and scanning electron microscopy.

Information obtained during a petrographic analysis may include

- Condition of material;
- Causes of inferior quality;
- Identification of distress or deterioration caused by chloride-induced corrosion, carbonation, ASR, freeze-thaw cycles, etc.;
- Probable future performance;
- Compliance with project specifications;
- Degree of cement hydration;

- Estimation of water-cement ratio and unit weight;
- Extent of paste carbonation;
- Presence of fly ash and estimation of amount of fly ash;
- Evidence of sulfate and other chemical attack;
- Identification of potentially reactive aggregates;
- Evidence of improper finishing;
- Estimation of air content and percentage of entrained versus entrapped air voids;
- Evidence of early freezing; and
- Assessment of the cause of cracking.

Selection of Tests for Corrosion Condition Evaluation

When resources permit, all tests listed previously should be performed during each evaluation. However, if it is necessary to reduce the scope of work, the selection of tests should be based on the primary goal of the survey. The primary goal of the PCCE is to predict the future progression of damage and calculate SI, whereas the primary goal of the In-Depth Inspection is to obtain sufficient data to prepare construction documents and calculate the SI. Following is a brief discussion on a methodology of selecting test procedures.

The visual and the delamination survey must be performed regardless of the goal of the corrosion condition survey; the degree of accuracy for the visual survey should remain the same. However, the extent of the delamination survey may be varied depending on the survey goals. For a PCCE survey, the delamination survey can be performed in representative test areas and not over the entire surface of the superstructure element. The test areas are selected so that they are representative of all variations in the structure because variations in the concrete condition, exposure, or construction quality may result in variation of durability of the concrete.

During an In-Depth Inspection, a high degree of detail for the visual and delamination surveys is required to derive repair quantities. Because the visual survey by itself is not sufficient to obtain total repair quantities, the delamination survey must also be performed. Most often, repair quantities are underestimated during the repair design process, and the project cost increases once construction is initiated. A general rule of thumb is to double the repair quantities identified during a visual and a delamination survey. During construction, the contractor will need to remove additional concrete adjacent to previously identified damaged areas. Good concrete repair practice requires that additional concrete adjacent to the damaged area be removed until no corrosion is observed on the reinforcing bars to account for the fact that the process of delamination has just been initiated adjacent to the damaged area but has not reached completion. Such areas are often hard to identify during the delamination survey and will need to be accounted for. Another cause for the increase in quantities is that during construction it is often more efficient to join one or more adjacent damaged areas, enlarge them, or regularize the geometry of the damage area. All of these factors increase the quantity of repair. Another option is for the contractor to estimate the exact repair quantities during the construction; the owner will have representatives on-site during construction to verify these quantities. This option has the advantage of closing the structure down only once during the construction, but it does not provide the owner with a good estimate on the total cost of the project because the cost is determined as the project progresses.

Care must be exercised if delamination testing is to be performed on treated bridge decks (i.e., decks with overlays, membranes, or some kind of surface treatment). Delamination surveys performed on concrete overlays identify not only the delaminations in the concrete slab but also the disbondment of the overlay from the concrete slab. In such cases, cores must be collected in areas identified to be delaminated to ascertain if the delamination is at the overlay-concrete interface or in the concrete slab. As sounding surveys cannot be performed on asphalt overlays, other nondestructive test methods have to be used. To date, none of the nondestructive test methods have proved efficient and/or accurate with asphalt overlays. Whenever asphalt overlays are to be removed for replacement, a delamination survey should be performed on the exposed deck. The same is true with all other overlays, such as thin concrete-epoxy, epoxy, and waterproofing membranes. Sounding surveys can be performed on concrete slabs treated with certain types of sealers and paints.

The chloride ion content analysis should also be performed during each PCCE and In-Depth Inspection. The most efficient way to perform chloride ion content analysis is to collect 2-inch-diameter cores during field evaluation and collect powdered concrete samples from the cores at various depths in the laboratory and analyze the powdered concrete samples for total chloride ion content. If the data from the cores are to be used as input for a diffusion model, chloride ion content analysis should be performed at least at six depths from the core. The core method affords full control on the quality of the sample and reduces contamination of the sample under field conditions. It is a lot easier and faster to collect a core than to collect powdered concrete sample from six depths from a given location. In addition to chloride ion content analysis, the core can be used for carbonation testing. There are two ways to collect powdered concrete samples from the core. One method uses a drill press to collect powdered samples from the side of the core from various depths. The other method uses a grinding wheel to powder the entire cross-section of the core at the subject depth. These methods are discussed in detail in Chapter 8.

The clear concrete cover information needs to be obtained only once from the structure. As clear concrete cover is not expected to change with time, it does not have to be collected during subsequent evaluations. The measurement of clear concrete cover is not laborious and can be efficiently collected using covermeters. The sampling size of the clear concrete cover should be as large as possible; at a minimum, 30 measurements must be obtained. Clear concrete cover may vary from one individual element to another due to construction practice. For example, it may vary from one span of the deck to another due to change in crew or construction practice. Concrete elements that are precast by a single manufacturer generally tend to have similar cover. Field data suggest that cover measurements generally satisfy the requirements of a normal distribution [13]. For beams and girders, it is necessary to conduct clear concrete cover measurement because of the possible low cover over the stirrups. When concrete cover is lower than 0.5 inch, the carbonation front reaches the steel-concrete interface quickly and steel starts to corrode. The thin cover over the reinforcing steel is easily delaminated and spalled.

On bridge structures, carbonation testing is most relevant on older structures, especially those built in the first half of the twentieth century. Carbonation testing can be performed on the cores collected for chloride ion content analysis, although not all cores need to be tested. Carbonation testing can also be performed in the field in small 0.5- to 0.75-inch-diameter holes. The holes can be drilled approximately 1 inch deep, and carbonation testing can be performed in the holes by spraying phenolphthalein solution. If conducted in the field, carbonation testing should be performed at three to five locations.

Electrical continuity testing must be performed if half-cell potential and/or corrosion rate testing is to be performed. It should also be performed if application of a cathodic protection system or electrochemical chloride extraction is a viable option for corrosion control on the subject superstructure element.

Half-cell potential and corrosion rate measurement are conducted to ascertain the state of corrosion in presently undamaged areas. Whether corrosion has initiated or not can be determined by evaluating the magnitude of the half-cell potential. Corrosion rates provide information on the rate at which corrosion is occurring at the time of the measurement at the location of the measurement. Both of these tests provide additional information on the propensity for corrosion in undamaged areas. They are often used to substantiate the conclusions reached based on results of visual, delamination, cover, and chloride surveys.

Petrographic analysis must be performed during an In-Depth Inspection and prior to selecting a corrosion control system. This test provides information on the quality and inherent deterioration mechanism in the concrete material. If the constituents of concrete make it susceptible to freeze-thaw, ASR, or other concrete deterioration processes, then these susceptibilities must be taken into account in the selection of the corrosion control system. The presence of one or more of these deterioration processes may impact the overall effectiveness of the corrosion control system.

When epoxy-coated rebars are used in the construction, it is necessary to know the condition of the epoxy. Epoxy-coated rebars should be collected in cores. In the laboratory, the epoxy-coated rebars are extracted from the concrete cores, and the condition (visual rating) of the epoxy-coated rebars must be documented along with adhesion of the coating and the number of defects or damages in the coating that expose the reinforcing steel to the environment.

CHAPTER 3

Service Life Modeling

Introduction

This chapter reviews the current state-of-the-art modeling of chloride-induced corrosion of steel in concrete and describes a diffusion model to be used for determining potential repair and rehabilitation alternatives for bridge superstructure elements. It concentrates on modeling chloride-induced corrosion of black steel and epoxy-coated rebar resulting from accumulation of chlorides in a bridge superstructure element due to exposure to chloride ions in the external environment, such as deicing salts. This chapter does not consider other corrosion mechanisms—such as carbonation or corrosion from chlorides cast into the concrete or other deterioration mechanisms such as alkali aggregate reactions, sulfate attack, or freeze-thaw damage.

The model both estimates accumulated damage as the function of age from completion of construction of the concrete element to age 100 years because of the assumption that no repair or corrosion mitigation is applied to the structure and calculates the SI discussed in Chapter 2. This index can then be used to identify appropriate corrosion prevention alternatives along with the repair of the concrete element.

Basis of a Model

Modeling the durability of reinforced concrete structures due to reinforcement corrosion requires a quantitative understanding of the environment, transport mechanisms through concrete, the corrosion process, and cracking and physical deterioration processes. Equations or statistics for each part of the process are readily available. Some of the models discussed in the literature have been developed for particular groups of structures, use particular software, and are difficult for others to replicate, while others are generic models designed to have wide application. This chapter summarizes the main research and literature in this area.

The process of chloride-induced corrosion of steel in concrete is described as follows:

1. Chlorides in the environment build up on the concrete surface.
2. Chlorides are transported through the concrete by a number of mechanisms, including diffusion and capillary action.
3. The chloride concentration builds up with time at the steel surface.
4. Once the chloride level achieves a critical threshold level, the passive oxide layer on the steel breaks down and corrosion starts.
5. Corrosion products have a higher volume than the steel-consumed products exerting tensile stresses on the concrete.
6. Concrete is weak in tension, so the concrete cracks either vertically to the surface or horizontally to form a delamination between reinforcing bars.
7. Cracks form pot holes or spalls, which lead to a degradation in the structure's appearance, function, and safety, leading to end of service life or time to repair.
8. The repairs may be made, and the cycle continues either in the previously undamaged areas or as the repair system degrades with time.

The process of modeling, therefore, requires the following:

- Calculating the chloride ion content at the surface of the concrete. This is termed the surface chloride ion concentration (C_0).
- Calculating the rate of transport from the surface to the steel. This is the rate of diffusion of the chloride ions into concrete and is designated by the use of a coefficient termed the diffusion coefficient (D).
- Determining the critical chloride concentration required to initiate corrosion. This is referred to as the corrosion threshold (C_T).

- Estimating the time to corrosion initiation. This is the age at which corrosion initiates on the reinforcing steel (T_i).
- Estimating the time from corrosion initiation to first cracking, followed by delamination and spalling. This is referred to as the propagation time (T_p).
- Estimating the time from first damage. This is referred to as the time to damage (T_d), which equals $T_i + T_p$.

Exposure to Chloride Ions

In the simplest terms, if there are no chlorides in the environment, there is no chloride-induced corrosion. Steel embedded in concrete develops a passive oxide layer that is highly protective and grows at a very slow rate. As long as the steel remains in good alkaline concrete, the passive layer will prevent corrosion initiation on the surface of the steel. This manual is primarily concerned with bridge superstructure elements and exposure to chloride ions from deicing salts.

As deicing salts are applied to the surface of the bridge deck, the chloride ions migrate into the deck concrete, and chloride-contaminated runoff through failed joints exposes caps, girders, and columns to chloride ions. Wetting and drying of the concrete increases the rate of accumulation of chlorides inside the concrete and can lead to chloride concentrations in concrete that are higher than in the external environment because evaporation increases the chloride concentration on the surface of the concrete. In literature, the chloride ion concentration at some depth just below the surface is often referred to as the surface concentration of chloride ions. It is this concentration of chloride ions that with time diffuses into the concrete element. Chloride profiles obtained from 210 cores collected from decks of six bridges indicated that the chloride concentration in the first 0.25 inch of concrete from the surface is very dependent on exposure conditions [13]. The accumulation of chloride ions occurs at about the depth of 0.5 inches because of the exposure of the surface of the concrete to moisture. Rain, snow, and water from other sources that flow over the bridge deck can wash away the chloride ions from the first 0.25 inch of concrete. However, the accumulation occurring a little deeper in the concrete is not affected by such exposure.

The areas not exposed to wetting and drying will normally be either too wet or too dry for corrosion. A resistivity below 50 kΩ-cm is needed to support a significant corrosion rate [14]; very dry concrete with high resistivity does not corrode even in the presence of chlorides. Also, if the concrete is too dry, there is no mechanism for transport of chloride into the concrete. Tuutti found that below a relative humidity of 60%, chloride-induced corrosion rates are negligible [15]. The wetting and drying environment will affect the transport of chlorides into concrete as discussed in the next section.

Diffusion of Chloride Ions

Chloride ions are transported in solution through the porous concrete cover in various processes. These include diffusion (driven by the concentration gradient between various sections of the concrete) and capillary action of water in a porous medium. Other mechanisms, such as wetting and drying, concentrate chloride ions more rapidly in concrete. Diffusion is affected by "sinks" within the concrete, such as binding with aluminate phases to form chloroaluminates, physical absorption of ions onto pore surfaces, and trapping in closed pores. One way of dealing with sinks is to use an "apparent" or "effective" diffusion coefficient, which is derived empirically from field or laboratory data. For example, if the diffusion coefficient is calculated from chloride profiles obtained from cores, this accounts for all effects and provides a reasonable estimate of effective rate of migration of chloride ions into the concrete.

Meijers developed a finite element analysis model that uses convection and conduction (i.e., diffusion) modeling of the chloride transport process [16]. However, most models assume that the dominant process is diffusion or that, for a reasonably well-constructed structure with reasonably good cover and concrete quality, the diffusion calculation is a reasonable approximation of the overall "real world" process.

The diffusion process is modeled by solving the one-dimensional equation for Fick's second law of diffusion:

$$\frac{\partial C}{\partial t} = D\left(\frac{\partial^2 C}{\partial C^2}\right) \tag{1}$$

where

C = chloride ion concentration
t = time
D = diffusion coefficient

This is usually solved by using the error function solution:

$$C_{(x,t)} = C_0\left[1 - erf\left(\frac{x}{2\sqrt{Dt}}\right)\right] \tag{2}$$

where $C_{(x,t)}$ = chloride concentration at depth x at time t
C_0 = surface concentration
erf = error function

Kranc et al. showed that the simple one-dimensional, semi-infinite solid approach is conservative because the reinforcing bar blocks the chloride transport, thus "backing up" chlorides at the steel-concrete interface because they cannot proceed through any further [17].

Corrosion Initiation

Corrosion initiation is generally deemed to occur once chloride concentration exceeds a given value or threshold. However, there is no fixed value of the corrosion threshold. Figure 1 shows data taken from a series of U.K. highway bridges by Vassie [18].

From Figure 1, it is clear that corrosion can initiate at a range of values from the low chloride levels of 0.1% to 1.8% by mass of cement. The observation of only 80% of bars corroding at 1.8% chloride by mass of cement can be explained by the presence of cathodic sites on the remaining 20% required for macrocell corrosion. With regards to corrosion initiation at 0.1% chloride by mass of cement, lowering of chloride threshold due to carbonation of the concrete and the effects of bound and unbound chlorides are suspected [14]. This issue, along with other variables, such as local exposure and variable permeabilities, led Bentz to use Monte Carlo calculations to look at the sensitivity of modeling to such variables [19]. There is extensive discussion of thresholds in the report on corrosion of metals in concrete prepared by ACI Committee 222 [20]. How to determine the effective chloride threshold for a concrete element is discussed later in this chapter.

Time to Damage

Time to damage is the sum total of the time required for chloride ions to diffuse down to the steel depth and accumulate in concentrations in excess of the corrosion threshold (T_i) plus the time required for sufficient corrosion to have occurred to generate the required amount of rust (or expansion) to produce cracking and/or delamination of the concrete (T_p).

The rate of accumulation of rust (i.e., rate of expansion) can be estimated by measuring the rate of corrosion of the steel. Several techniques and equipments are available to measure the in situ corrosion rate. Corrosion rate is a function of many factors, is not constant, and varies with time; it is very difficult to calculate the total accumulation of rust. In addition, the amount of expansion generated by the products of corrosion highly depends on the level of oxidation of the corrosion product. Several techniques are available to estimate the accumulation of rust or section loss of steel (section loss of steel is directly proportional to the amount of rust generated). Determining the time when cracking, delamination, or spalling occurs presents a series of significant problems because of the following factors:

- There is no easy way of converting section loss to cracking rate.

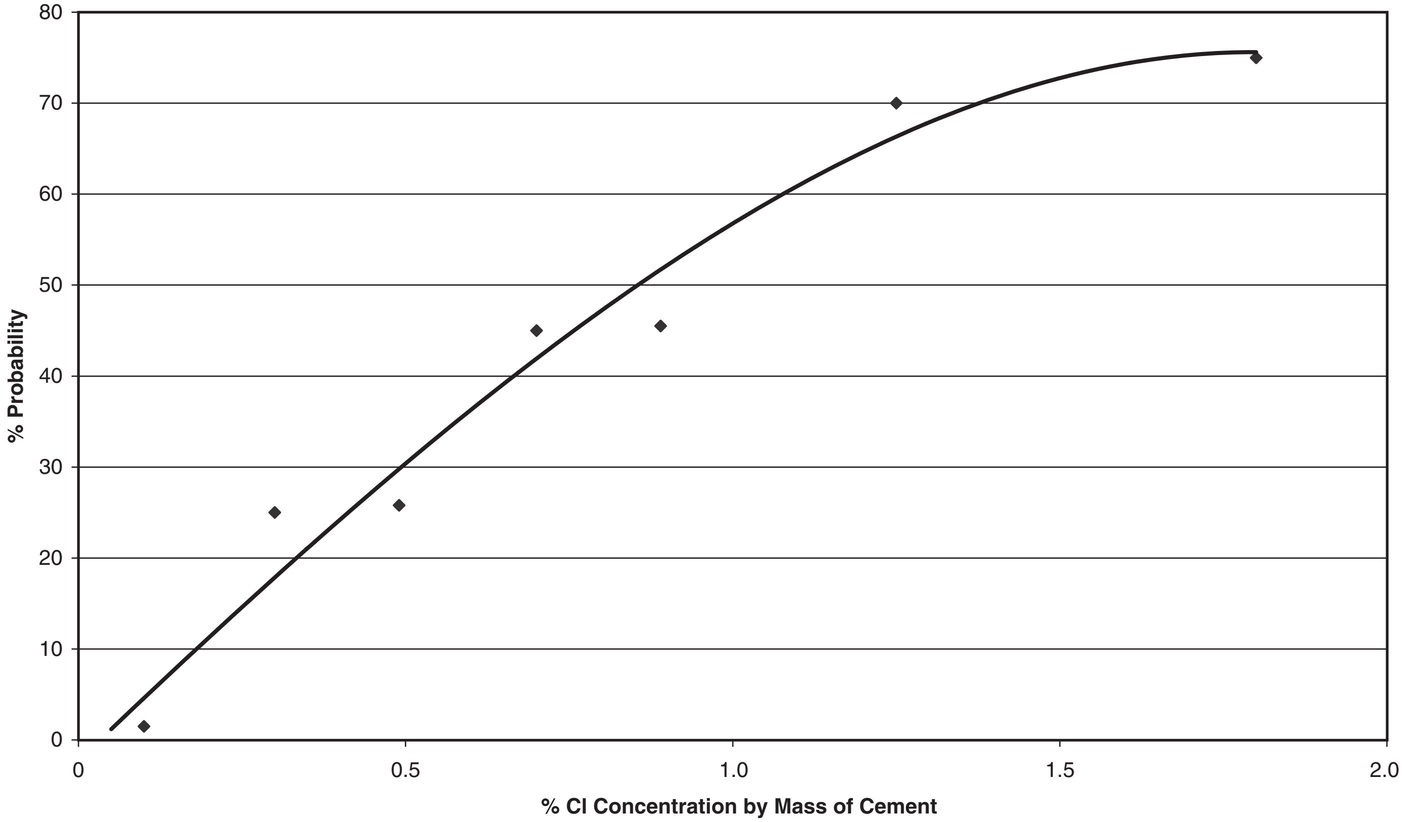


Figure 1. Corrosion initiation (% probability) vs. chloride concentration.

- Cracking patterns will be a function of steel layout and shape of the reinforced concrete member.
- Loading, especially live loading, will affect cracking rate.

One quantitative equation for converting section loss to cracking has been developed by Rodriguez et al. [21].

The crack width W at the concrete surface is:

$$W = 0.05 + \beta(X - X_0) \qquad \text{for } 0 < \text{W} < 1 \text{ mm} \tag{3}$$

where β = 0.01 for top cast steel and 0.0125 for bottom cast steel
X = bar radius decrease due to corrosion to produce crack width W
X_0 = bar radius decrease due to corrosion to induce crack initiation (at surface)
$X_0 = 83 + 7.4c/\phi - 22.6f_{c,sp}$

where c = cover
ϕ = bar diameter
$f_{c,sp}$ = tensile splitting strength of concrete

The tensile strength of concrete $f_{c,sp}$ can be derived from its compressive strength f_c [22].

$$f_{c,sp} = 0.12(f_c)^{0.7}$$

To calculate time to cracking, Equation 3 is used to determine the reduction in the bar radius for a given crack width. Corrosion rate measurements can then be used to determine when such a reduction will occur.

Liu proposed a different approach that is based on the mechanics of the generation of rust and the resulting stress in the concrete [23]. Liu idealized a rebar in concrete as a thick-walled cylinder and derived the following equation for the critical amount of rust required to produce a crack:

$$W_{\text{crit}} = \rho_{\text{rust}} \left(\pi \left[\frac{Cf_i}{E_{ef}} \left(\frac{a^2 + b^2}{b^2 - a^2} + \nu_c \right) + d_0 \right] D + \frac{W_{st}}{\rho_{st}} \right) \tag{4}$$

where W_{crit} = critical volume of corrosion product required to induce a crack
ρ_{rust} = density of rust
C = clear concrete cover
f_i = tensile strength of concrete
E_{ef} = effective elastic modulus of concrete
a = inner radius of the thick-walled cylinder (clear concrete cover – d_0)
b = outer radius of the thick-walled cylinder (clear concrete cover + $D/2$)
ν_c = Poisson's ratio for concrete
d_0 = thickness of the porous zone around the steel-concrete interface
D = diameter of the rebar
W_{st} = mass of steel corroded
ρ_{st} = density of steel

The rate of production of rust is given by:

$$W_{\text{crit}}^2 = 2\int_0^t 2.59 \times 10^{-6} \left(\frac{1}{\alpha} \right) \pi D i_{\text{corr}}(t)\, dt \tag{5}$$

where α = molecular weight of steel or corrosion products
$i_{\text{corr}}(t)$ = rate of corrosion as a function of time
t = time

The critical volume of rust required to generate a crack is calculated from Equation 4, and the time required to generate that volume of rust is then obtained by solving Equation 5. This model was validated against laboratory slabs. This model indicates that the time to cracking is dependent on physical properties of concrete (elastic modulus and Poisson's ratio), clear concrete cover, rebar diameter, and corrosion rate. The problem with using this model is that once corrosion initiates, the corrosion rate varies with time. Andrade and Gonzalez have suggested measuring the corrosion rate several times a year using the average of the measured corrosion rates [24].

Diffusion Models Proposed in Literature

As stated earlier, there are a number of models for corrosion of steel in concrete. Some are designed for use in new construction and are computed from data from construction details and environment, predicting time to first and subsequent repairs and life cycle costs for a new structure to compare durability options (such as changing cement types, adding corrosion inhibitors, changing the concrete cover, or changing bar types). Others are designed for projecting present data forward from existing structures to estimate time to damage.

Bazant developed a complete physical-mathematical model that describes the corrosion process in submerged concrete exposed to sea water [25]. A set of equations has been derived for the transport of oxygen and chloride ions through the concrete cover, the mass sources and sinks of oxygen and corrosion products, the cathodic and anodic electric potential, and the flow of ionic current through the concrete electrolyte. This model is largely based on theoretical assumptions and is completed by formulating the problem as an initial-boundary-value problem that can be solved by using the finite element method. In order to arrive at numerical solutions, the spatial distribution and geometry of anodic and cathodic areas on the reinforcing bars have to be assumed. This model has been applied to several illustrative numerical examples, and the results obtained show that for submerged concrete, diffusivities for chloride ions and oxygen—not only at the anodic (i.e., rusting) area, but also, and mainly, at cathodic areas—usually are the controlling factors. Although the model takes into account all relevant chemical and physical processes involved in reinforcement corrosion, several processes are not adequately addressed and the polarization behavior of the anodic and cathodic reactions has not been fully described. However, the model results in an improved understanding of the complex nature of the problem.

Similar models based on the same relationships, but expressed in more simplified forms, have been presented by Sagues and Kranc, Noeggerath, Naish et al., and Raupach and Gulikers [26–29]. These models have shown that these analytical electrochemical approaches lead to suitable results for specific corrosion problems.

In November 1998, a consortium was established under the Strategic Development Council (SDC) of ACI to develop an initial life cycle model based on the existing service life model developed at the University of Toronto. The life cycle model [30] provides insight into the durability of a given construction practice, thereby allowing designers to ascertain the impact of additives to concrete and corrosion prevention strategies applied to new construction on the long-term durability of the concrete element. In October 2000, Life-365 Version 1.0 software was released, and Version 1.1 followed in December 2001. In Life-365 software, the analyses are divided into four steps: predicting initiation period, predicting propagation period, determining repair schedule after the first repair, and estimating life cycle costs based on initial construction and future repair costs.

The Life-365 software predicts the initiation period assuming ionic diffusion to be the dominant mechanism. This software differs from other diffusion models in that it accounts for the variability of the diffusion coefficient with age and with temperature. It also attempts to model the impact of various additives, such as silica fume and fly ash (by reducing the diffusion coefficient to reflect the lower permeability) and corrosion inhibitors (by raising the chloride threshold required to initiate corrosion). The rate of accumulation and the maximum accumulation of surface chloride in this program are based on the type of structure, geographic location, and exposure.

A diffusion model developed by Sagues et al. to predict the future performance of existing structures has been validated on marine piles of several structures [31, 32]. Initially, Sagues et al. used specialized software to calculate the diffusion coefficient from field cores and the solution to the error function to evaluate marine piles of two structures [31]. Later, Sohanghpurwala and Diefenderfer developed a methodology to perform the modeling in a standard spreadsheet program and validated the model on marine piles of another structure [32]. The approach of Sagues et al. obtains probability distribution information on diffusion coefficient, clear concrete cover, and surface chloride ion concentration from the existing structure. It subdivides the concrete element into finite elements and determines the time to corrosion initiation for each finite element using the error solution to Fick's second law of diffusion. To calculate the time to corrosion initiation (T_i) for each finite element, values of diffusion coefficient, clear concrete cover, and surface chloride concentration are required. These are obtained from the probability distributions defined for each variable. The time for propagation (T_p) is assumed to be 3 to 6 years.

A similar software program has been developed in the United Kingdom. The diffusion coefficient is calculated from data taken from concrete blocks exposed to a marine tidal/splash zone environment and other laboratory data and is used to estimate time to initiation [33]. The time to failure is obtained from the Rodriguez formulas.

Similarly, Broomfield used the method of collecting chloride profiles from the structure under investigation to estimate the apparent diffusion coefficient [34]. The effective chloride diffusion coefficient was calculated using a parabolic approximation to the error function equation. Time to corrosion can be predicted from the individual sets of measurements based on the actual diffusion and exposure characteristics of the measurement location. The Rodriguez equations are then used in a separate module of the program to predict time to cracking. The chloride diffusion model was validated against field data in one case where surveys were conducted approximately 9 years apart. The earlier data set agreed with the later survey predictions of time to corrosion ± 2 years in five out of six cases where the initial prediction was greater than 10 years. In another seven out of nine cases, the earlier and later data sets agreed that corrosion was imminent, but at the point of measurement there was no damage.

Model Development

Two separate models were developed to deal with black steel and epoxy-coated rebar. The models were validated against field structures and/or data on field structures available in literature. As this manual is primarily concerned with chloride-induced corrosion, these models are chloride diffusion based. However, to assist in the case where no chloride information can be obtained, a damage-based model is also proposed. The damage-based model has not been validated.

Model for Black Steel

The proposed chloride diffusion model is based on developing statistical distribution functions of diffusion coefficient, surface chloride ion content, and clear concrete cover from the structure. It assumes that the past diffusion behavior and chloride accumulation will continue at the same rates in the future. However, it is clear that the diffusion process is affected by many factors, such as temperature, humidity, and concrete cure, and each of these factors varies with time and season. To model each factor and its variations would be a significantly more complex problem. In addition, knowledge of the exact processes and variations in these factors is not known. To overcome this complexity and lack of information, this model is based on measuring the structure's "effective or apparent diffusion" coefficient that represents the average diffusion that has resulted from the impact of all factors over the life of the structure, thereby eliminating the necessity to model each and every factor and its variation. Similarly, the chloride ion concentration just below the surface of the concrete can be used to represent the surface chloride ion concentration that represents the sum total of the chloride exposure that the concrete element has experienced. The clear concrete cover is the intrinsic property of a concrete element, which is the function of the design and construction practice and remains constant over time.

The diffusion coefficient, surface chloride ion concentration, and clear concrete cover all vary from one location to another. The measurement process has an error associated with it. Therefore, the distribution of each of these can be described by a probability function. The distribution of the clear concrete cover and the surface chloride ion concentrations are generally described by the normal distribution function. The diffusion coefficient is best described by the gamma distribution function. In general, the probability distribution function selected to represent the field data should be evaluated for good fit. However, the normal and the gamma functions have been found to best describe the field data and are used in this model. Once the probability distribution functions for diffusion coefficient, surface chloride concentration, and clear concrete cover have been developed, they are used in the model to represent field data.

The "apparent or effective diffusion" coefficient is estimated by measuring the profile of chloride ion concentrations as a function of depth ($C_{(x,t)}$– chloride ion concentration at depth x at age t at a specific location in the concrete element) in concrete cores collected from the subject concrete element. The profile is obtained by collecting powdered concrete samples from various depths from the concrete core and analyzing them for total chloride ion content using the total acid soluble analysis. The following function is then fitted to the chloride profile obtained from each core using nonlinear regression analysis:

$$C_{(x,t)} = C_0\left[1 - erf\left(\frac{x}{2\sqrt{Dt}}\right)\right] \qquad (6)$$

The nonlinear regression is performed by minimizing the sum-of-squares of vertical distances of the chloride profile data points from Equation 6. The best fit provides an estimate of diffusion coefficient D for that core. The fit is performed for t equal to the age of the structure at the time of collection of the core and C_0 equal to the concentration of chloride ions in the top layer of the core. If sufficient time has elapsed between the collection of the core and the analysis of the chloride profile, then the age at which the analysis is conducted should be used. Because the cores are not exposed to the environment after they have been collected from the concrete element, C_0 measured from the core may be lower than the actual in situ concrete because chloride ions will continue to diffuse, albeit at a different rate in the core.

The surface chloride ion concentration, C_0, is measured from core samples at the depth of 0.25 to 0.5 inch. Clear concrete cover information is obtained by nondestructively measuring the clear concrete cover at numerous locations using a covermeter or a pachometer.

The concrete element is subdivided into numerous finite elements. For each element, the diffusion coefficient (D), surface chloride ion concentration (C_0), and clear concrete cover (x) are selected from the statistical distribution functions such that the probability of selection of any value from the function is the same and can be applied to Equation 6.

It should be noted that this particular solution of the Fick's second law assumes that C_0 is constant during the diffusion process, although for an actual structure C_0 increases with time as chloride ions from the environment accumulate in the concrete. However, there is a limit beyond which the accumulation of chloride ions in the top layer of the concrete would cease to increase. This limit is presently not known. To account for the increasing C_0, it was assumed that C_0 increases linearly satisfying the following equation:

$$C_0 = mt \qquad (7)$$

where m is the rate of chloride accumulation and t is the age of the concrete element. The solution to Fick's second law is modified as follows to account for the variation of C_0:

$$C_{(x,T)} = \int_0^T m\left[1 - erf\left(\frac{x}{2\sqrt{Dt}}\right)\right]dt \qquad (8)$$

where t is the time that varies from age 0 years to T years, and T is the age at which the value of $C_{(x,T)}$ is calculated.

The goal is to determine the age at which $C_{(x,T)}$ will exceed the threshold required to initiate corrosion (i.e., the time to initiation, T_i). Time to propagation is assumed to be 5 years. Liu has found that, for highway concretes with average, 2-inch, clear concrete cover, the time to cracking is several orders of magnitude smaller than the time to initiation and ranges from 3 years to 5 years. The time to propagation, as modeled by Liu, depends on concrete properties, clear concrete cover, and corrosion rate. For much deeper clear concrete cover, or for concretes with higher tensile strength, the time to propagation would be longer than 3 to 5 years. Once time to corrosion initiation is known, time to damage, T_d, for each finite element at each age is calculated as follows:

$$T_d = T_i + t_p \tag{9}$$

The number of elements that meet the following requirement is calculated to produce a cumulative damage distribution:

$$T_d < T \tag{10}$$

The model is first evaluated at threshold levels as low as 300 ppm of chloride ion concentration. The cumulative damage results from the model at the age of the field evaluation are compared with the actual measured damage. If the two are not in agreement, the threshold value is adjusted upward until the model produces results comparable to the field measurements.

Analysis of modeling data obtained in this project indicates that the cumulative damage distribution can be best described by using the Weibull distribution, which is widely used for failure analysis and, therefore, is applicable to service life modeling. As diffusion modeling results in a Weibull distribution, it is reasonable to expect that if data on the quantity of concrete damage at several ages are available, then the Weibull distribution could be used to model the damage distribution of the concrete element. The modeling can be accomplished by obtaining concrete damage quantities at several ages and using the data to fit the Weibull curve. Once the Weibull parameters are estimated, the cumulative Weibull curve can be used to estimate future progression of damage.

Model for Epoxy-Coated Rebar

The epoxy-coated rebar diffusion model is very similar to the black reinforcing steel model discussed with one exception. The epoxy-coated rebar model also uses a probability distribution function to ascertain if the coating is damaged in the finite element. On black steel, corrosion initiates at all locations where the threshold concentration has been exceeded, whereas on epoxy-coated rebar, corrosion initiates only at the defects where the chloride ion concentration has exceeded the threshold.

Epoxy coating is a barrier system that is designed to keep chlorides and other chemical species that can initiate and sustain corrosion away from the reinforcing steel. It also provides a physical barrier between the steel and concrete interface. Epoxy is a very effective barrier because it does not allow deleterious species to permeate through it. However, the epoxy uptakes some amount of moisture, which results in temporary reduction in bond between the epoxy and the steel surface. The effectiveness of the epoxy as a barrier is not impacted by the reduction or loss of bond; it is impacted by the presence of coating damage or defect in the form of holidays, mashed areas, and bare areas. The defects in the coating are normally generated during application of the coating, storage and handling, transportation to site, placement in forms, and placement of the concrete. Corrosion on epoxy-coated rebars initiates at defects in the form of crevice corrosion and can spread by undercutting the coating. The rate of corrosion is controlled by the availability of cathodic sites and chloride ions. In addition, the coating may deteriorate with time, and more defects may appear on it. To account for corrosion spreading under the coating and deterioration of the coating, the amount of damage on the coating is varied with age. At age 0, the percentage of exposed surface area (i.e., damage or defect in the epoxy coating that exposes the steel surface) is assumed to be that allowed by the governing specifications or whatever the user believes it may have been. At the time of the field evaluation, cores that contain one or more epoxy-coated rebar sections are extracted, and the percentage of exposed surface area on each extracted section is documented. The average percentage of exposed steel observed on the extracted sections of epoxy-coated rebars is then used to determine the growth rate of deterioration. It is assumed that the rate of growth is linear, and this rate is used to determine when 100% of the surface of the epoxy-coated rebar will be exposed (i.e., no epoxy coating is left on the rebar). This rate of increase of deterioration is used by the model, and it is assumed that the rate will remain the same in the future. The model allows corrosion initiation on epoxy-coated rebars in the finite elements that have suffered epoxy coating damage. A probability distribution is used to determine if the epoxy coating in the finite element has suffered damaged or not.

Susceptibility Index (SI)

The distribution of chloride ions at the steel depth can be used to quantify both the susceptibility of the concrete element to corrosion in areas that are not presently damaged and the future susceptibility to corrosion-induced damage. If sufficient chloride ions are present to initiate corrosion, then corrosion-induced damage in the near future is expected, and only very aggressive corrosion mitigation techniques, such as cathodic protection and electrochemical chloride extraction, can be used to control the corrosion process. However, if the chloride ion concentration distribution at the steel depth is low and corrosion is not expected to initiate in the near future, less expensive corrosion control systems—such as sealers, membranes, and/or corrosion inhibitors—can be used to either control or stop the rate of corrosion. Therefore, an index that provides a good representation of the distribution of chloride ions at the steel depth would be very useful in selecting a corrosion control system.

The distribution of chloride ions can be obtained by collecting samples of the concrete and analyzing for chloride ion content at the steel depth. There are two methods of accomplishing this. One method requires locating reinforcing steel on the deck surface, drilling down, measuring the clear concrete cover over the reinforcing steel, and then collecting a powdered concrete sample from concrete adjacent to the reinforcing steel from the depth at which the reinforcing bar is located. Although this method has been and often is being used, it has many disadvantages, and the amount of time required for sampling is large, which means that the structure has to be closed down for a long period of time. The second method uses the field data collected for the service life model. The diffusion coefficients, the surface chloride concentrations, and the clear concrete cover can be used in conjunction with the diffusion model presented for black steel to determine the distribution of chloride ions at a given steel depth at a given age.

The following equation is proposed to represent the SI.

$$SI = \left(\left(\sum_{1}^{n}(Cl_{th} - X_i)\right) / (n \times Cl_{th}) \times 10\right) \qquad (11)$$

where Cl_{th} = chloride concentration threshold
X_i = chloride concentration in the ith finite element or location
n = number of finite elements used in the model or locations where measurements were made

This index is a ratio of the average moment from threshold over the threshold. It is scaled to 10 for ease of use. The value of this index is 0 if all the chloride ion concentration everywhere at the steel depth is at the threshold and is 10 if there are no chloride ions anywhere at the steel depth.

The threshold for corrosion initiation would be based on either the information available in the literature or the value used in the diffusion model that provides the best estimate of present damage. Because the threshold from literature may not be applicable to that particular structure, using the diffusion model to calculate the distribution of chloride ions and the apparent threshold seems to be more appropriate.

CHAPTER 4

Selection of Corrosion Mitigation Alternatives

Introduction

There are two steps to conducting a rehabilitation of a reinforced concrete element that has suffered corrosion-induced damage: (1) repair sections of the element that have suffered concrete failure in the form of cracking, delamination, and/or spalling and (2) provide a corrosion control system, if necessary, to prevent or minimize future deterioration. Performing a concrete repair, such as sealing a crack or removing delaminated concrete and patching, restores the concrete element to its original geometric form and most often does not have any impact on the ongoing corrosion process. Consequently, either the concrete repair can fail or concrete in areas not previously repaired can suffer from corrosion-induced damage. This requirement for periodic repair of failed sections of the concrete element can result in a significant increase in the resources required for maintenance. To minimize maintenance costs, a corrosion control system along with the repairs is necessary. The discussion in this chapter is limited to selection of a corrosion control system for reinforced concrete elements suffering corrosion-induced damage. However, some types of repairs can also provide corrosion control, and they are discussed as appropriate.

When a concrete patch is installed, fresh concrete is placed in the repair area. The new concrete's chemical makeup is different than that of the original concrete. This can, depending on the differences, result in startup or acceleration of ongoing corrosion. A corrosion cell is fueled by chemical or electrical imbalances. If, for example, a concrete patch is installed in a concrete element that has undergone exposure to chloride ions and/or had admixed chloride ions, the difference in chloride concentration between the original and the new concrete can result in the setup of a new corrosion cell or the acceleration of an existing cell. In a case such as this, corrosion on reinforcing bars traversing the old-new concrete boundary along the perimeter of the patch can initiate corrosion, or ongoing corrosion on those bars can accelerate, resulting in the failure of the patch along the repair perimeter. This failure mechanism, termed the "halo effect," can result from differences in pH of the two types of concrete. Because the areas not adjacent to the concrete patch are not in any way impacted by the repair, corrosion in these areas can continue unabated, and failure of concrete in those areas can occur after the concrete repairs have been completed.

Corrosion control systems fall into two broad categories: local (i.e., those applied in a repair area) and global (i.e., those applied to the entire concrete element). The primary goal of the local corrosion control systems is to mitigate the halo effect and improve the service life of the patch repair, whereas global corrosion control systems mitigate the halo effect and provide corrosion control over the entire concrete element. The local systems are only applicable when the concrete element has suffered local exposure to chloride ions, such as exposure to contaminated water runoff in certain sections of the element. Local systems can be used to avoid or reduce the halo effect that is due to contaminated concrete adjacent to the repair. They are completely ineffective at controlling corrosion in areas that are not adjacent to the repair.

Corrosion control systems can also be categorized by the mechanism by which they provide protection: mechanical (barrier) or electrochemical systems. The barrier systems control the transportation of chemical species such as chloride ions, oxygen, and moisture to sites where the corrosion reactions are occurring or may occur. The electrochemical systems control the progression of corrosion by interfering with the chemical or electrical aspects of the corrosion process. Under each of these categories, many different types of corrosion control systems are available, and each has its own advantages and disadvantages.

The effectiveness of a corrosion control system very much depends on the level of contamination of chloride ions in the concrete and the severity of chloride ion exposure. The barrier systems that are applied onto the surface of the concrete element are most effective when applied either to a repair or to existing concrete that is not sufficiently contaminated with chloride ions, thereby delaying or avoiding the accumulation of chloride ions in sufficient quantity at the steel depth and the consequent initiation of corrosion. If sufficient levels of chloride ions are already present in the concrete and can diffuse to accumulate in sufficient quantity at the steel depth, a barrier system is not as effective. The effectiveness of electrochemical systems such as cathodic protection and electrochemical chloride extraction generally do not depend on the quantity of chloride ions present in the concrete or on the severity of exposure because their level of protection can be adjusted to account for the corrosivity of the environment. The ability of other electrochemical systems such as corrosion inhibitors depends on the concentration of chloride ions and severity of exposure.

The first step in selecting a corrosion control system is to identify if local systems will suffice. If not, appropriate global systems will need to be identified. To determine whether a local or global system is appropriate, the distribution of chloride ions and the exposure conditions need to be determined. Subsequently, the selection of various alternatives from the local or global system type is also based on the distribution of chloride ions in areas that have not yet suffered damage. In addition, other information—such as half-cell potential data, corrosion rate information, clear concrete cover distribution, electrical continuity of the reinforcing steel, carbonation, and the presence or absence of other concrete deterioration such as freeze-thaw and ASR—is used in the decision making. To provide for a logical framework to analyze the chloride distribution and exposure conditions, the SI was proposed in Chapter 2 and defined in Chapter 3. The discussion of how the SI can be used follows.

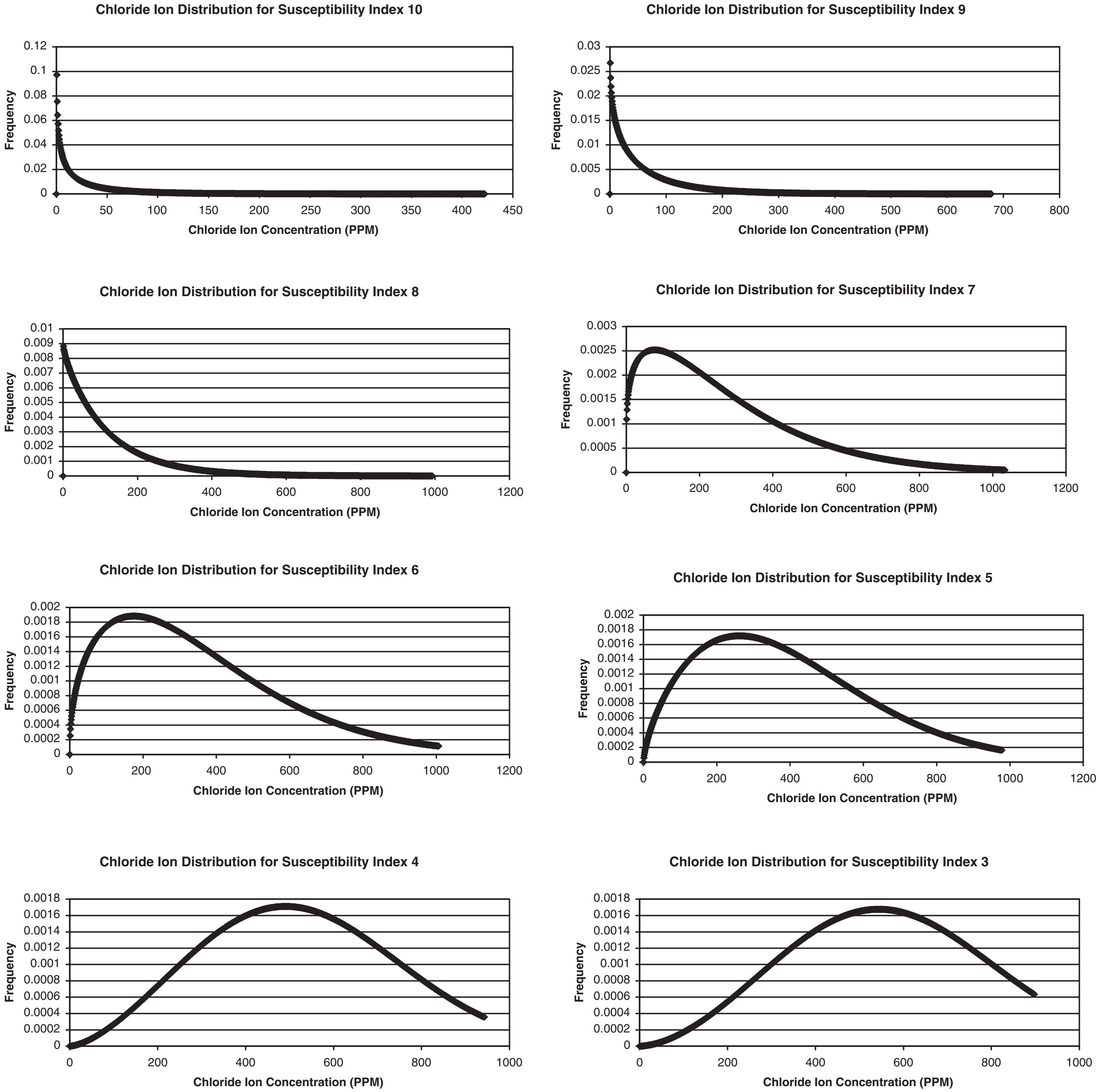
The two most important pieces of information normally obtained during condition evaluation are the quantity of damage and the distribution of chloride ions in sound (i.e., undamaged) areas. The quantity of damage is obtained during the visual and the delamination surveys. The distribution of chloride ions in sound areas is obtained by collecting powdered concrete samples or cores to determine the concentration of chloride ions at various depths, including the depth at which the reinforcing steel is located. The distribution of chloride ions reveals whether the chloride ions are admixed or whether they are from an external source. If they are uniformly distributed as a function of depth (in quantities in excess of the background chloride level), then most likely they were admixed. If the chloride ions show a diffusion distribution, it can be safely assumed that the chlorides are coming from the outside. It is also possible to have a combination of admixed chlorides and a diffusion of chlorides coming from the environment. In addition, an analysis of the distribution allows one to determine if the chloride exposure is localized or common to the entire concrete element. Standard practice would be to analyze the distribution of the chloride ions and determine the level of threat they pose in terms of corrosion-induced damage in the future. If the chloride ions are close to or above the threshold, a more aggressive corrosion control system such as cathodic protection or electrochemical chloride extraction may be required. However, if the distribution of chloride ions makes it insufficient to initiate corrosion at the time of the evaluation or to reach the threshold (at the steel depth) in the near future due to diffusion of the existing chloride ions in the concrete, a barrier system in the form of sealers, membranes, and overlays may be appropriate.

To assist in the process of analysis of the distribution of the chloride ions at the steel depth, the SI—a measure of the average distribution of the chloride ions with respect to the threshold—was developed. When the distribution of chloride ions is such that the concentration at all locations is below the threshold, the SI will vary from 0 to 10 (0 represents a condition in which the chloride concentration at all locations at the steel depth is equal to the threshold and 10 represents the condition in which the chloride concentration at all locations at the steel depth is close to 0). However, in practice there are situations in which the chloride concentration at the steel depth at some of the locations has exceeded the threshold and corrosion has initiated. As the number of locations increase where the chloride ion concentration at the steel depth exceeds the threshold, the SI will approach 0. When the numbers of locations with concentrations in excess of the threshold have a larger moment with respect to the threshold compared with the locations that have chloride concentration below the threshold, the SI will become negative. A negative SI would signify that corrosion has initiated at the majority of the locations and that corrosion-induced damage can be expected in the near future in the sound areas.

For the SI to be successful, the threshold value has to be properly ascertained. As discussed in Chapter 3, the threshold can vary over a significant range from one concrete mix to another and even in the same concrete. Therefore, the service life model can be used to select an appropriate threshold value. Service life modeling is performed at various threshold values, and the threshold value that provides the best estimate of damage, as compared with actual damage on the structure, is used as the threshold value for the SI. This threshold value is termed the "apparent threshold" because it represents the sum average of all variations in the threshold value required to initiate corrosion. The service life model can be used to determine the distribution of chloride ions at the steel depth at a given age. The apparent threshold value and the distribution of chloride ions can then be used to calculate the SI. For structures or concrete mixtures for which the threshold value is known with a reasonable degree of confidence, this threshold value can be used along with the actual chloride distributions without depending on the service life model to provide the apparent threshold value. Figure 2 presents the distribution of chloride ions at various SI levels based on an apparent chloride threshold value of 800 ppm. All corrosion control alternatives that may be most applicable for that particular distribution of chloride ions have been identified, and the types of corrosion control systems that are most likely to provide optimal protection for a particular SI are shown in Figure 3.

The following simplifications were made to prepare Figure 3 and apply primarily to global systems:

- All surface-applied coatings that are capable of controlling the flow of moisture into the concrete without hampering the outflow of moisture out of the concrete (i.e., they are breathable) have been lumped together as sealers. The term "sealers" is most often used to refer to silane/siloxane-based material that is applied to the surface of the concrete to reduce the inflow of water vapor but that is not as effective when water is ponded on the surface of the sealer. These


Chloride Ion Distribution for Susceptibility Index 10

Chloride Ion Distribution for Susceptibility Index 9

Chloride Ion Distribution for Susceptibility Index 8

Chloride Ion Distribution for Susceptibility Index 7

Chloride Ion Distribution for Susceptibility Index 6

Chloride Ion Distribution for Susceptibility Index 5

Chloride Ion Distribution for Susceptibility Index 4

Chloride Ion Distribution for Susceptibility Index 3

Figure 2. Chloride distribution at various values of SI.

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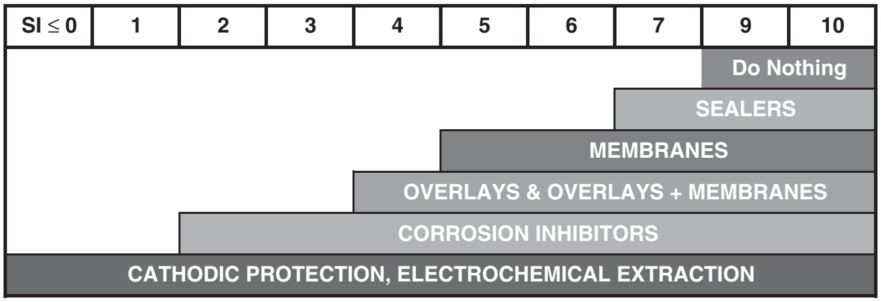

Figure 3. Optimal solutions for various values of Susceptibility Index.

sealers provide varying degrees of barrier to the transport of chloride ions into the concrete.

- The term “membrane” is used to denote surface-applied systems that do not allow transport of moisture in either direction and are considered waterproofing material. These systems, as long as their integrity is maintained, do not allow moisture or chloride ions to enter the concrete. In this category, the asphalt overlay with the waterproofing membrane is also included.
- Overlays include all kinds of concrete overlays, and they are installed to decrease the permeability of the top layer of concrete and to increase the depth that the chloride ions have to travel to reach the steel. In addition to providing corrosion benefits, the concrete overlays serve as a wearing surface on bridge decks.
- The corrosion inhibitor category includes all materials that have chemicals that can interfere with the corrosion process or parts of the corrosion process (such as the cathodic reaction or the anodic reaction) and reduce or stop the corrosion. The corrosion inhibitors can be surface applied or admixed with repair concrete. Some manufacturers market a combination of corrosion inhibitors, pore pluggers, and sealers. These materials are also included in the corrosion inhibitor category.
- The cathodic protection systems include galvanic systems, impressed current systems, and re-alkalization (which is considered a part of electrochemical chloride extraction).

Formulation of a Repair and Corrosion Control Strategy

Several different factors are considered in selecting the repair and the corrosion control strategy:

1. Type of superstructure element (i.e., deck, beams, and girders).
2. Type of reinforcement (conventional, prestressed, post-tensioned, black, or epoxy coated).
3. Quantity and distribution of damage over the concrete element.
4. Susceptibility of sound concrete to corrosion-induced damage in the future.
5. Severity of exposure.
6. Presence of other concrete deterioration processes, such as freeze-thaw and ASR.
7. Structural deficiencies.
8. Conformance with newer codes and the need to upgrade.
9. Structure close-down and phasing requirements.

Only Items 1 to 6 are discussed in this chapter; the other items and prestressed concrete elements are beyond the scope of this manual.

The type of superstructure element plays a significant role in the selection of the repair and corrosion control system. In the deicing salt environment, the deck generally experiences the most severe exposure condition. The other superstructure elements, such as the beams and the girders, are sheltered under the deck and are not directly exposed to deicing salts or the weather. They are most often only exposed to the corrosive environment in areas where the expansion joints in the deck have failed and chloride-contaminated water is running off onto the beams and girders. Therefore, these superstructure elements most often only require repairs and a local corrosion control system.

It is possible that the beams and girders may have admixed chloride ions. However, because they are generally precast, they are less likely to contain admixed chloride ions. The beams and the girders can suffer from other concrete deterioration processes, such as freeze-thaw and ASR, and may experience cracking, which would accelerate the diffusion of chloride ions into the concrete and require a less severe exposure to chloride ions to result in corrosion-induced damage. Both freeze-thaw and ASR require direct exposure to moisture to generate cracking. Because two exposed surfaces of the beams and the girders are vertical and the third one is the underside, the surfaces do not experience a severe enough exposure to moisture to suffer sufficient cracking necessary to accelerate the diffusion of chloride ions. In a marine environment, when the elevation of the beams and the girders are as low as, or lower than, 15 feet from the high water mark, then they are exposed to a sufficiently corrosive environment and

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can experience corrosion-induced damage that will require repair and a global corrosion control system.

The type of reinforcement plays a significant role in the selection of the repair type and corrosion control system. The majority of the corrosion control systems are targeted toward the black reinforcing steel type. If epoxy-coated rebars are present, a compatible repair and corrosion control system will be needed. In addition to repairing concrete, some form of protection will need to be provided to the damaged epoxy coating in the repair area. The report of the NCHRP 10-37C study documents the various repair and corrosion control strategies that are compatible with epoxy-coated rebars [35].

The quantity of damage plays a significant role in the selection of the repair type and the corrosion control system because it impacts both the structural integrity of the element and the cost of various options of repair.

If the element is in question or the element is considered to have failed, then repair is needed to restore the service capacity of the element. When the level of damage is not sufficient to impact the integrity of the element, then the cost of various repair options govern the selection of the repair. For example, on a bridge deck, the quantity of damage may be such that partial-depth replacement along with full-depth repair using a single concrete pour may be more cost-effective than patching each damaged area individually. This particular option of partial-depth replacement is not only a repair, but also a corrosion control system. In this approach, the contaminated top layer of concrete is removed by standard or hydrodemolition techniques and replaced with fresh concrete. If the depth of replacement has been selected to ensure that the majority of the chloride contamination has been removed, then the deck is as good as new. Otherwise, corrosion of the bottom mat of reinforcing steel will result from the formation of a large macrocell between the top mat, which is in fresh concrete, and the bottom mat, which is in contaminated concrete. The soffit repairs are more expensive to perform because they require access from the bottom of the structure. As concrete fails on the bottom surface, chunks of concrete can fall and cause injury to commuters below or cause damage to property. The diffusion model or the chloride profiles from concrete cores can be used to determine the depth to which the concrete has been contaminated with chloride ions and the depth of concrete removal. Another option is to place a monolithic layer of concrete to repair damaged areas and install an overlay to provide a new wearing surface and increase the clear concrete cover over reinforcing steel. This approach increases the time to corrosion initiation and can result in a significant extension in service life if chloride ions are not left in the original deck concrete in sufficient quantities to continue to initiate and support the corrosion reaction. Otherwise, corrosion may continue to occur and result in damage to the original concrete. The damage occurring in the original concrete may not be reflected immediately upon formation and may become noticeable only after significant damage has occurred under the overlay. The selection of an overlay as the sole corrosion control system depends on the SI. Figure 3 shows that such layers may be used only when the SI is equal to or greater than 4.

In conjunction with the selection of the repair type, the type of corrosion control system must also be identified. Two types of repairs that also served as the corrosion control system were discussed above. However, these types of repairs are effective only when the susceptibility of the sound concrete to corrosion is low and the SI is equal to or greater than 4. When the concentration of chloride ions in the concrete is sufficient to result in a SI of 4 or less, more aggressive corrosion control systems such as corrosion inhibitors, cathodic protection, and electrochemical chloride extraction must be considered. Numerous products and systems are available. Because each product has its own limitations and applications, and some are more effective than others in a given application, selection must be made with care. A brief discussion of some of the more popular corrosion control systems is provided in Chapter 5.

The quantity of damage and the SI provide a good definition of the condition of the concrete element and provide sufficient information on the types of repairs and the types of corrosion control systems that may be effective for that particular structure. The alternative strategies so identified must be further refined by considering the exposure condition. Although the SI may suggest a category of corrosion control system, some of the products in that category may not be as robust and reliable in the exposure condition that the structure experiences. For example, arc-sprayed zinc galvanic cathodic systems fall in the category of cathodic protection system, but they do not work well in dry conditions. Similarly, certain corrosion inhibitors may not be applicable to certain environments. A sealer system may be more appropriate on a beam or a girder but less appropriate on a bridge deck because of possible deterioration under traffic.

Finally, other deterioration processes have a significant impact on the selection of the repair and corrosion control system. When freeze-thaw damage is occurring on the concrete element, some form of waterproofing will be required to reduce the direct exposure of the concrete surface to moisture. Cathodic protection systems and electrochemical chloride extraction technologies can, under certain circumstances, accelerate the ASR and generate more cracking damage on the concrete.

When selecting the type of repair and corrosion control system, one must consider all of the factors discussed above. Sufficient information must be collected on the structure, including during the condition evaluation, to allow consideration of all the above factors.

CHAPTER 5

Extension of Service Life with Repair and Corrosion Mitigation Options

This chapter briefly discusses various corrosion control strategies that might be considered for rehabilitating a reinforced concrete structure experiencing deterioration due to chloride-induced corrosion. This chapter provides an overview and should be used as a starting point for obtaining more information on the actual systems being considered because new products, and new forms of old products, are introduced regularly. In each broad category are many products and systems offered by numerous vendors; all products and systems in a category are not equal, and each product and system has its own set of applications and limitations. Some, but not all, products and systems currently available have been evaluated in a controlled setting. The best source of information on many of the products and systems are the results of the research studies commissioned by various government and semi-government agencies. When considering performance history, it is necessary to consider the type and level of monitoring performed to report the performance. The fact that a product or system was installed on numerous structures does not vouch for its ability to perform.

The various categories of corrosion control systems are discussed under the appropriate subdivision of local or global systems. Corrosion inhibitors can be used locally and globally; therefore, they are discussed first in the local category, and their global application is discussed in the global category.

Local Corrosion Control Systems

Patching Materials

Patching materials are used to replace localized areas of deteriorated (i.e., spalled and delaminated) concrete. Because of the expansive forces involved in the corrosion process, deterioration of reinforced concrete structures caused by corrosion ultimately results in exposure of the reinforcing bars. Patches usually have a short service life because they do not address the cause of the problem (i.e., corrosion of the reinforcing bars). When concrete contaminated with chlorides in concentrations greater than the threshold level is left in place in the area surrounding the patches, inadvertent acceleration of the rate of the corrosion process occurs. The patched area acts as a large non-corroding site (i.e., cathodic area) adjacent to corroding sites (i.e., anodic areas), and thus corrosion cells are created.

Patching on bridge decks may be full-depth or partial-depth repairs. Deterioration of bridge decks due to corrosion of the reinforcing steel involves either the top layer of reinforcing steel only or both the top and bottom layers of reinforcement. If only the top layer of reinforcing steel is corroding, a partial-depth repair would be appropriate. For partial-depth deck repairs, deteriorated concrete is removed to the depth required to provide a minimum of 1.0 inch clearance below the top layer of reinforcing steel. The maximum depth of removal for a partial-depth repair should not exceed half the deck thickness. Corrosion of both the top and bottom layers of reinforcing bars often requires full-depth repairs. For full-depth repairs, deteriorated concrete is removed through the entire thickness of the concrete element.

Partial-depth deck patching materials include portland cement concrete (PCC) and mortar, quick-set hydraulic mortar and concrete, polymer mortar and concrete, and HMA. Because of their short service life (1 year or less), HMA patches should only be considered as temporary patches. PCC is typically used for full-depth deck patches and large partial-depth repairs.

The service life of deck patches somewhat depends on the type of patch (full- or partial-depth) and the patch material. The service life of deck patches ranges from 4 years to 10 years [36], although an FHWA TechBrief indicates that the service life of a patch ranges from 4 years to 7 years [37]. The service life of the patch depends largely on the corrosivity of the surrounding concrete and the development of the halo effect.

Patch rehabilitation involves removing and patching all damaged (i.e., spalled and delaminated), sound but corroding, sound but critically chloride-contaminated, and/or carbonated areas. In addition, the bridge element must be protected against future ingress of chloride ions by applying a barrier system on the surface or using an electrochemical system to control corrosion in the sound areas. With periodic application of an effective barrier system, the estimated service life of patch rehabilitation methods can be in excess of 50 years [36]. This estimate is based on the assumption that all sources of corrosion in the concrete element have been minimized. However, the bond between the patch and the concrete may deteriorate with time, resulting in a patch life significantly less than 50 years.

Reinforcing Bar Coatings

Corrosion of reinforcing bars in concrete (excluding corrosion attributable to carbonation) is initiated and sustained when sufficient amounts of chloride ions, oxygen, and moisture are present at the reinforcing bar surface. Therefore, minimizing or eliminating access of one or some of these elements is required to prevent, stop, or retard the corrosion process in concrete. One of the most commonly used mechanical means to protect the reinforcing bar against corrosion is to coat it with a material that acts as a physical barrier to harmful reactants.

Coatings on reinforcing bars are either applied to reinforcing bars prior to installation in a new concrete structure (i.e., fusion-bonded, epoxy-coated, and galvanized steel) or applied to reinforcing bars for the purpose of corrosion control in damaged concrete structures (polymer- and epoxy-modified coatings, epoxy zinc coating, etc.). The second type of application will be discussed further in this section.

The ideal requirements for a repair coating are that it (a) serve as a barrier to chemical species that take part in a corrosion reaction, (b) have no adverse effect on adjacent bars or surrounding patch material, and (c) can be applied easily in the field.

Surface preparation of the reinforcing bar prior to application of repair coatings is of prime importance. Proper surface preparation is essential to achieve maximum adhesion, which is the primary factor governing the performance of any protective coating [38].

Abrasive blast cleaning is typically used to clean reinforcing bars. Specifications clearly state that bars must be cleaned to a "near white" condition; this represents the approximate removal of 95% of rust, mill scale, and other contaminants from the bar surface [39-41]. Abrasive blast cleaning is used not only to clean the surface, but also to roughen the surface to facilitate bonding of the coating. The selection of specific abrasives determines the surface roughness, or anchor pattern (a function of the peak-to-valley depth); the number of peaks; and the shape of the peaks. For example, a grit blast media produces an angular shape that provides a better tooth for coating adhesion and more exposed steel surface area than the round profile of shot blast. In general, all grit, or a mixture of grit and shot, yields an optimum anchor pattern for epoxy coatings [42]. To ensure proper adhesion, any residue on the steel surface following the blast cleaning operation should be removed prior to application of the coating. Remaining residue can result in "backside contamination" (contamination at the coating-bar interface), which adversely affects adhesion.

There is a fundamental difference in the application of coatings on new versus corroded reinforcement bars. The former is performed in a factory where surface preparation, ambient conditions, and the coating process are under strict quality control. In the latter case, the coating is applied on-site, where it is much more difficult to control influential conditions. The first step in performing this work is to remove the defective concrete by a suitable method and expose the reinforcing bar both around its full circumference and for a short length beyond the area of corrosion before any surface preparation can be carried out. Unlike factory-applied coatings, which are performed using machinery, the field-applied repair coatings have to be applied manually.

Damaged concrete can be repaired by patching with plain cement mortar without the application of any reinforcing bar coating. Although some level of protection is generally achieved in the repaired area, corrosion of the reinforcement in surrounding concrete can actually be accelerated [43]. The effect is thought to be because the highly alkaline nature of the mortar patch makes the reinforcement in the repaired area more cathodic relative to the reinforcement in the surrounding concrete, which becomes anodic (i.e., active).

Repair coatings are not stand-alone items. Consideration must be given to the compatibility of the patching mortar material and the use of an appropriate bonding agent with a given coating system. Each type of coating will have compatible ancillary products. Specific guidelines are typically provided by each manufacturer.

There are several types of commercially available repair coatings, including polymer-modified cement slurry; nonpassivating epoxy coatings; passivating epoxy coatings; zinc-rich epoxy coatings (one or two part); and zinc-rich, water-based coatings.

Polymer-modified cement slurries, nonpassivating epoxies, and epoxies filled with passivating fillers (e.g., zinc phosphate and cement clinker) have been found to be prone to undercutting [44]. As each of these coating types isolates the reinforcement from the highly alkaline repair mortar, it keeps the reinforcement from being passivated. When passivating fillers are used, they appear to be fully bound in the polymer/epoxy film and offer little benefit. It has been reported that polymer-modified cement slurries containing rust inhibitors, silica fume, and sand can give better results and are not subject to undercutting. However, field experience with these particular repair coatings is limited.

Zinc-rich epoxy (one part) repair coatings have been reported to offer excellent protection to reinforcement in both the repaired area and the surrounding concrete [44]. Using this repair coating, the zinc provides electrical contact between the reinforcement and active zinc. The zinc then acts as an anode, thus protecting the reinforcement. However, it has been found that reinforcement bars with zinc-rich coating can suffer from accelerated corrosion damage. This can occur, with resulting delamination of the patch repair, when the chloride content in the concrete adjacent to the patch is very high and the surrounding reinforcing bar mat has a high corrosion rate. It is suggested that the zinc component of the repair coating undergoes anodic dissolution due to galvanic action (as intended) and that over a period of time the zinc is depleted from the coating. The problem is that the zinc protects both the reinforcement bars on which it is applied and nearby uncoated reinforcement that is electrically continuous to the coated reinforcement. The resulting dissolution rate of the zinc is much higher than desired. In addition, the underlying reinforcement tends to maintain its high anodic potential, and accelerated corrosion proceeds on the reinforcement because of a relatively large adjoining cathodic area.

Water-based, zinc-rich coatings are relatively new. No field data are available to substantiate laboratory data that show this type of coating to be very effective [45]. This type of coating is reported to have very good bonding characteristics. According to the manufacturers, the coating not only acts as a protective barrier to aggressive components, but also affords sacrificial protection and passivation action.

Repair of Epoxy-Coated Reinforcing Steel

When epoxy-coated rebars are present in a concrete element, consideration must be given to the repair of the epoxy coating in the damaged areas. The field-applied coating must be compatible with the original coating on the reinforcing steel. Sohanghpurwala et al. evaluated various repair strategies for epoxy-coated rebars and determined that a combination of a compatible epoxy coating and a high-resistance silica fume patch material was most effective in combating corrosion on epoxy-coated rebar concrete elements. The barrier provided by the field-applied epoxy is not as good as the factory-applied one; however, the high resistance of the silica fume patch material was able to minimize the corrosion current flow between the patch and the adjacent original concrete [35].

Corrosion Inhibitors

Corrosion inhibitors are chemical compounds or formulations that are used to control corrosion of metals in aggressive environments. They have been used extensively for many decades. Numerous different commercially available corrosion inhibitors have been developed for specific applications, in particular water treatment, petroleum refining, and other chemical process industries.

The use of inhibitors as a means of controlling corrosion of reinforcement in concrete is now receiving more attention. A number of inhibitors such as sodium nitrite, potassium dichromate, sodium benzoate, and stannous chloride were investigated in the laboratory in the late 1960s and early 1970s for potential application in concrete structures; however, they met with limited success [46–48]. In the late 1970s, calcium nitrite was identified as an effective inhibitor for reinforced concrete [49, 50]. In the early 1980s, organic amine-based inhibitors were introduced [51].

In the late 1980s, a Strategic Highway Research Program (SHRP) project evaluated 17 corrosion-inhibiting systems for reinforced concrete bridge components [52]. Based on the initial evaluation, four inhibitors were selected for additional testing [53]. A second evaluation showed that three of the four inhibitors were effective and that two methods could be used to deliver the inhibitor to the corroding reinforcement. One of the three inhibitors was calcium nitrite, and the other two were based on organic compounds.

The inorganic compounds (based on calcium nitrite) are marketed as a liquid admixture containing 30% calcium nitrite and a 15% solution of calcium nitrite. The organic products, which are proprietary formulations, include a water-based, organic amine and an oxygenated hydrocarbon. The amine-based inhibitors are currently marketed as a liquid admixture and a surface-applied inhibitor. Both of these organic amine products are also known as migratory corrosion inhibitors.

The method of operation for organic and inorganic corrosion inhibitors is vastly different. The calcium nitrite (inorganic)–based inhibitors provide corrosion protection by chemically oxidizing the surface of the reinforcement according to the following reaction [54]:

$$2\mathrm{Fe}^{++} + 2\mathrm{OH}^- + 2\mathrm{NO}_2 \rightarrow 2\mathrm{NO} + \mathrm{Fe}_2\mathrm{O}_3 + \mathrm{H}_2\mathrm{O}$$

In this process, ferrous, hydroxyl, and nitrite ions react to form nitric oxide, ferric oxide, and water. Generally, chloride ions react with ferrous ions to form a soluble complex that, upon reaction with hydroxyl ions, leads to the formation of ferrous hydroxide, $\mathrm{Fe(OH)}_2$. The chloride ions are then released back into solution for further reaction with additional ferrous ions. A typical reaction between ferrous ions and a chloride ion is given as follows [55]:

$$\mathrm{Fe}^{++} + 6\mathrm{Cl}^- \leftrightarrow \mathrm{FeCl}_6^{-4}$$

$$\mathrm{FeCl}_6^{-4} + 2\mathrm{OH}^- \leftrightarrow \mathrm{Fe(OH)}_2 + 6\mathrm{Cl}^-$$

The chloride and nitrite ions compete for the ferrous ions produced by the steel, and the relative amounts of the chloride and nitrite ions determine which of the above two chemical reactions occur. The effectiveness of the nitrite inhibitor therefore depends on the ratio of the nitrite to chloride ions in the concrete. Recommended dosage rates of the nitrite inhibitor are provided by the manufacturer. However, one investigation pointed out that the recommended dosage levels may be insufficient (by as much as 33 to 67%) to offer adequate protection against the quantities of chlorides indicated by the manufacturer [56].

The method of operation of the amine-based (i.e., organic) inhibitors is two-fold [57, 58]. First, the inhibitors block the permeation of chloride ions through the concrete matrix. The ingress of chloride (as well as other ions such as sulfates) is greatly reduced as a result of the hydrophobic properties imparted to the concrete by the organic inhibitors. Second, and more importantly, the inhibitors form a thin protective film or barrier on the surface of the reinforcement. This barrier prevents chlorides from coming in contact with the reinforcement. The chloride threshold to initiate corrosion is thus increased. It is claimed that these inhibitors migrate through the concrete for a considerable distance and seek out ferrous members; hence, they are called migratory inhibitors. Unlike nitrite inhibitors, the effectiveness of the amine-based inhibitors does not depend on the ratio of inhibitor to chloride in the concrete.

The effectiveness of nitrite as a corrosion inhibitor for reinforcement embedded in concrete when used as an admixture in new construction has been established through a number of independent studies and through field experience. However, there are advantages and disadvantages, as listed below.

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

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

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

- Accelerating properties of calcium nitrite can create difficulties in placing, finishing, and curing concrete under certain conditions (such as high ambient temperatures and use of cements with a low C_3A content).
- Nitrite may cause an increase in slump.
- Although an increase in strength may be seen, nitrite also has the detrimental effect of retempering due to stiffening of the treated concrete.
- Because the amount of nitrite required depends on the amount of chlorides in the concrete, careful estimates of these quantities are required (nitrite may lead to an over-dosage when used as an admixture).
- When nitrite is used in solution form, the concrete cover should not be more than 0.5 inch.

A recent study found that the nitrite inhibitor, when used in conjunction with patch repair on field structures, did not provide any benefit [59]. The inhibitors in this study were used as an admixture in the patch concrete and were applied either to the surface of the exposed reinforcing steel or to both the surface of the reinforcing steel and the surface of the patch repair. The test structures were exposed to a very corrosive environment, and the adjacent sound concrete was heavily contaminated with chloride ions.

As with nitrites, there are several advantages and disadvantages for the amine-based corrosion inhibitors, as listed below.

Advantages to using organic inhibitors include the following:

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

Disadvantages to using organic inhibitors include the following:

- Because of a more compact pore structure, organic inhibitors reportedly are not as effective when used with concretes having a low water-to-cement ratio.

- Organic inhibitors produce a slight decrease in concrete strength.
- When organic inhibitors are used as a coating, they cannot be applied below 36°F, nor should they be applied if the temperature is expected to fall below 36°F within 12 hours of application.
- The concrete surface should be as dry as possible before applying the organic inhibitor product.

The organic corrosion inhibitors evaluated along with the nitrite corrosion inhibitors in the SHRP study also did not exhibit any benefit when used as admixtures in patch concrete and applied either to the surface of the exposed reinforcement or to both the surface of the exposed reinforcement and the surface of the patch repair [59].

Corrosion Inhibitor Patching (Superstructure Elements)

There are two types of superstructure corrosion inhibitor rehabilitation patching methods. Type I is a standard patching method using a corrosion inhibitor-modified concrete patching material. Type II uses the same patching materials; however, it also includes four applications of a spray-on inhibitor on the exposed reinforcing bars and patch cavity prior to patching. For both methods, all damaged concrete, sound but actively corroding concrete, and critically chloride-contaminated concrete is removed.

For the Type I method, concrete removal areas are marked out and scored 1.0 inch deep along the patch perimeter with a dry concrete saw. The concrete is removed at least 1.0 inch below any reinforcing bars, the patch area and exposed reinforcement are properly cleaned, and the patch area is backfilled with a corrosion inhibitor modified concrete. A penetrating sealer is then applied to the entire structural element. For the Type II method, concrete is removed to the depth of reinforcement, the patch area surfaces and reinforcement receive four applications of spray-on corrosion inhibitor, and then the patch area is backfilled. Because of potential bond problems between the patch material and the original concrete due to spray application of some inhibitors, the patch area has to be sandblasted to remove any surface residue.

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Overlays

Overlays are used to restore the deck riding surface to original construction quality and to increase the effective cover over the reinforcing bars. Examples of overlays include latex-modified concrete (LMC), low-slump dense concrete (LSDC), and HMA with a preformed membrane (HMAM). All of these overlays increase the dead load on a structure and thus reduce the live load capacity. In addition, LMC and LSDC are not suitable for use on bridge decks where existing concrete may be susceptible to alkali-aggregate reactions (silica or carbonate) unless low-alkali cement is used or other preventative measures have been taken [60].

These overlays are considered repair methods because sound concrete (that may be chloride contaminated or carbonated) is left in place. The type of overlay has some influence on the service life of the repair; however, the amount and degree of contaminated concrete left in place remains the most important factor. Since the primary factor that influences the decision to overlay a bridge deck is the extent of surface damage, the amount of contaminated concrete left in place becomes a critical factor in determining the service life of the overlay system. The HMAM repair overlays tend to increase the moisture content of the concrete because they use an elastomeric membrane between the original deck surface and the overlay. Moisture enters through the damage in the elastomeric membrane and is trapped between the membrane and the concrete surface. This increase in moisture content can increase the average annual corrosion rate.

Rehabilitation of an existing corrosion-damaged deck consists of removing and patching all damaged (i.e., spalled and delaminated), sound but corroding, and sound but critically chloride-contaminated areas and overlaying the patched deck with LSDC, LMC, or HMAM. The economics of concrete removal must be considered when determining the extent of material to be removed. At some break-even point of removed surface area, the price for removing the entire concrete surface to a depth of at least 1.0 inch below the top layer of reinforcing bars will be less than the price for large areas of spot removal. The chloride content and carbonation of the concrete below the removal depth must also be determined. The chloride content of the concrete left in place must be low enough that existing chlorides do not diffuse into, and initiate corrosion of, the lower layer of reinforcing bars. In addition, carbonated concrete left in contact with the lower layer of reinforcing bars can initiate corrosion.

The service life of rehabilitation overlays is limited by the rate of diffusion of chloride ion through the LSDC and LMC and the leakage of chloride ions through the membrane of an HMAM. Thus, the overlays are significantly influenced by environmental exposure conditions (i.e., chloride concentration and temperature). An LSDC overlay applied on a contaminated concrete bridge deck was observed to fail by disbondment and delaminations in the original concrete in 20 years [61]. A study by the Virginia Transportation Research Council found that the service life of LMC overlays applied on several bridge structures in the state of Virginia was about 20 years [62].

The estimated cost for constructing a rehabilitation overlay on a prepared surface is approximately the same as the cost for constructing a repair overlay on a prepared surface plus the cost for additional concrete removal and replacement required for rehabilitation. A more complete discussion on the economics of concrete removal is presented by Vorster et al. [38]. The construction procedures, quality assurance/construction inspection measures, and material performance specifications are identical for repair and rehabilitation overlays.

Hydrodemolition—a process that involves a high-pressure water jet to break apart concrete by demolishing the concrete matrix—has been used to remove concrete to a specific depth in a cost-effective manner so that an overlay can be placed. This process is completed by several simultaneous mechanisms: cavitation, pressurization, and direct impact [63]. Cavitation, formed by rapidly changing pressure of flowing water, produces shock waves of such magnitude that the cement matrix is broken apart. Pressurization acts to break the concrete in tension along previously existing cracks and voids. Direct impact from the water spray nozzle removes loosened particles. Vorster et al. state that high production rates, because of significant automation, can be attained (once setup is complete) and selective areas of concrete can be removed to any desired depth [38].

Hydrodemolition has several advantages over typically used concrete removal techniques (e.g., hammering and rotary milling). The primary advantage is the ability to remove concrete between the reinforcement bars, which is usually inaccessible by rotary milling. Hammering may be used to remove selective areas of concrete between reinforcement bars; however, this technique is labor-intensive and may produce additional microcracking in previously sound areas.

Disadvantages of hydrodemolition include the necessity to clean up removed particles and flying debris. After the hydrodemolition equipment has passed over an area of concrete, aggregate, small pieces of concrete, and slurry are left behind. If this slurry is allowed to harden before removal, it may rebond to the newly scarified surface and present problems of adhesion when the new concrete is applied. In addition, a significant amount of runoff water (up to 31.7 gallons/minute/nozzle [38]) is created that must be collected and disposed of in an appropriate manner.

The equipment used with hydrodemolition is large and complex. It requires significant mobilization and cleanup times. In addition, this method of concrete removal is most productive, in terms of cost, when large areas of concrete are to be removed. Once the concrete is removed and the debris is cleaned from the deck, one of the aforementioned patch materials or overlay techniques can be applied.

Membranes

Membranes are elastic materials applied to bridge decks and are normally used in conjunction with HMA overlays. The main reason for employing a membrane is to provide a waterproof barrier to prevent the intrusion of chloride ions into the concrete deck. Proper application of an approved membrane and HMA wearing surface on bridge decks can greatly reduce the intrusion of chlorides into the concrete, thus serving as an effective bridge deck protection system. Currently, membranes and concrete overlays (latex-modified and low-slump dense concrete) are the most widely used corrosion protection systems for bridge decks. Twenty-two states in the United States use membranes as a standard bridge deck protective system; of the remaining states, 19 have used membranes in the past.

For existing, and in particular older, reinforced concrete structures, the performance of membranes depends on the initial chloride content in the concrete. If the chloride content is high near the surface, the chlorides will diffuse into the concrete after the membrane is applied and may critically contaminate the concrete at the reinforcement level over time.

As protective systems, membranes have several advantages and disadvantages. The advantages include the following:

- Membranes can be applied relatively rapidly, including application of the HMA wearing surface.
- Membranes can bridge and prevent reflection of most moving concrete cracks because of their elastic nature.
- The HMA wearing surface can provide a good riding surface.
- Membranes can be applicable to almost any deck geometry.

The disadvantages of membranes as protective systems include the following:

- The service life of the membrane may be limited by the wearing surface when exposed to heavy traffic.
- The HMA overlay is a nonstructural component of the deck slab, adding to the dead load without increasing structural capacity.
- The system is not suitable for grades in excess of 4% because the bond capacity is limited and shoving and debonding can occur under traffic.

Membranes are either preformed sheet systems or liquid, applied-in-place materials. The liquid-applied membranes can be applied as hot liquid and cold liquid. Preformed membranes are supplied in rolls of continuous sheets that are bonded to the bridge deck with an adhesive primer. They are placed in an overlapping configuration to provide a waterproofing layer. Hot-liquid membranes are placed on the bridge deck using brushes, spray, or rollers. The liquid must be preheated and applied above a minimum temperature (specified by the manufacturer) to ensure a waterproof layer. Cold-liquid membranes are placed similarly to the hot-liquid membranes, but without the need to preheat the material prior to placement.

The following performance criteria may be used to identify suitable membrane products for concrete bridge decks:

- **Water permeability:** Membranes should demonstrate effective waterproofing capabilities yet allow for vapor transmission to ensure long-term adhesion to the deck. If vapor transmission is not permitted, the membrane system may increase the moisture content of the bridge deck, thereby promoting corrosion.
- **Chloride ion permeability:** Protection of concrete from chloride ion intrusion is a major requirement for membranes. It is suggested that concrete that is waterproofed with a membrane be tested for permeability in accordance with the modified version of AASHTO T-277, "Rapid Chloride Permeability Test," and the charge passed should not exceed 100 coulombs.
- **Low-temperature flexibility:** Membranes should possess adequate flexibility to withstand the stresses caused by deck movements at low temperatures. No visible damage should occur when wrapping a sample of membrane around a 1-inch mandrel at 9°F.
- **Crack bridging:** Cracks already in existence on the bridge deck will grow with temperature and load changes; the membrane must have elastic properties to be able to accommodate changes in width. It is suggested that membranes be able to bridge a crack width of 0.06 inch at 32°F.
- **Bond strength:** A strong adhesive bond between the membrane and wearing surface reduces deformation of the HMA wearing surface layer by heavy wheel loading. The adequacy of the bond should be evaluated in both tension and shear, with minimum allowable values of 690 kPa and 172 kPa, respectively.
- **Resistance to indentation:** Because of the thermoplastic nature of some membranes, indentation and puncture by aggregates may occur during application and rolling of the HMA wearing surface. Testing for resistance to indentation shall result in no penetration at the expected maximum placement temperature.

Membrane deterioration may be expected because of repeated loading from traffic and age embrittlement. A SHRP project developed a nondestructive procedure for determining the effectiveness of a membrane [64]. It also concluded that the service life of bridge deck can be extended by 25 years by properly installing a membrane.

Another factor that determines the service life of the membrane is the service life of the overlying wearing surface. Membranes cannot resist damage during removal and replacement of a deteriorated HMA wearing surface. Therefore, the membranes must also be removed and replaced. Depending on the severity of the traffic and environment, some HMA overlays have required removal and replacement in 10 years or less. Because HMA does not prevent the intrusion of water, water accumulates above the membrane and weakens the bottom portion of the HMA layer. This leads to debonding and stripping of the HMA overlay. Other distresses associated with HMA are excessive wear of the surface in the wheel paths (especially in areas where use of studded tires is prevalent), lateral movement of the HMA layer, and rutting.

Sealers and Surface Coatings

Concrete sealers and surface coatings are used to prevent chloride ions from diffusing into the concrete. Measures that prevent water from entering the concrete will also minimize chloride intrusion. However, chloride ions that already exist in the concrete, especially near the surface, will diffuse into the concrete after the sealer or coating is applied and may critically contaminate the concrete at the reinforcement level over time.

A sealer is a solvent-based liquid applied to a prepared concrete surface. There are generally two categories of sealers: penetrating sealers and surface sealers. Penetrating sealers, such as silanes and siloxanes, react with the pore structure within hardened concrete to create a nonwettable (i.e., hydrophobic) surface. Surface sealers, such as linseed oil and epoxy (with a solids content of less than 50%), block the pores of concrete. For bridge deck applications, penetrating sealers are preferred because surface sealers can have an inadequate depth of penetration and may quickly wear away when exposed to abrasion from traffic. However, surface sealers would be adequate for application on beams and girders.

Penetrating sealers prevent liquid water from entering the concrete; however, they are very permeable to water vapor. Vapor permeability of sealers is desirable because it prevents moisture from being trapped inside the concrete element when the concrete is sealed. As the concrete dries out, the electrical resistance increases significantly, further inhibiting the corrosion process.

Several environmental exposure conditions may influence the service life of sealers applied to bridge decks. These include ultraviolet light, moisture, and surface wear due to traffic. The service life of penetrating sealers applied to bridge decks ranges from 5 to 7 years [36]. Typically, penetrating sealers should be reapplied every 6 years.

A coating is a one- or two-component organic liquid (such as epoxies, acrylics, methacrylate, and urethanes) that is applied in one or more coats to a prepared concrete surface. The primary purpose of the coating is to prevent the ingress of water into the concrete and, hence, the diffusion of chloride ions. However, unlike sealers, the vapor permeability of a coating is very low. Coating materials have high solids content, usually 100%, and the typical thickness of coatings after drying is in the range of 0.001 to 0.003 inch.

The selection of a coating material depends on individual site conditions. Epoxies are abrasion resistant and have a high adhesive strength; however, they are susceptible to degradation by UV light. Acrylics are brittle and normally have low impact strength. Urethanes have high impact strength and good weathering characteristics, but low abrasion resistance.

The following performance criteria should be used to identify suitable products for sealing concrete bridge decks:

- **Chloride screening:** Sealers should be able to reduce chloride ingress into concrete by at least 90% after 30 weeks of ponding with saltwater.
- **Penetration depth:** The initial depth of penetration should be 0.125 inch, and ideally 0.25 inch, to provide for protection from wear and UV light degradation.
- **Moisture vapor permeability:** The minimum vapor transmission should be 80% after sealing of the concrete. The percentage of vapor transmission is determined by comparing the vapor loss of sealed concrete to that of an unsealed concrete over a 14-day period. The concrete used in the test should be in a saturated, surface-dry condition.
- **Surface friction:** The surface should exhibit acceptable frictional characteristics after it is sealed.

Penetrating sealers, surface sealers, or coatings may be applied to protect superstructure elements. Environmental exposure conditions that influence the service life of sealers and coatings applied to superstructure components include UV light, moisture, and abrasive wave and ice action. Surface sealers not exposed to abrasive wave or ice action have a service life of 1 to 3 years. In the presence of abrasive wave or ice action, the service life of surface sealers may be less than 1 year. The service life of coatings depends on the type of coating material applied and the field exposure conditions. Coated bridge components subjected to sea spray may have a shorter life than those exposed to deicer salt runoff water [36].

Polymer surface treatments are primarily applied to bridge deck surfaces to reduce the infiltration of chloride ions and water. They are essentially a two-component, abrasive-resistant, organic coating applied in one or more coats to the concrete. The exception is that each layer is impregnated with aggregate to improve friction. Polymer binders that are commonly used include epoxy, acrylic, urethane, methacrylate, and high-molecular-weight methacrylate.

Polymer surface treatments cure quickly. As a result, traffic can be permitted on the bridge deck a few hours after treatment. Hence, polymer surface treatment is considered a rapid bridge deck treatment. Because polymer surface treatments are thin (0.25 inch to 0.50 inch thick) and tend to follow the contours of the deck, they do not provide the substantial improvement of ride quality or drainage of the deck obtained by overlays, but they result in much less increase in dead load.

Waterproofing with a polymer surface treatment or with a sealer/coating results in similar performance of an existing concrete. Chloride ions already present in the concrete, especially near the surface, will diffuse into the concrete after treatment and may eventually critically contaminate the concrete at the reinforcement level.

Corrosion Inhibitor Overlays (Bridge Decks)

The corrosion inhibitor overlay method was developed to limit the amount of sound chloride-contaminated concrete that had to be removed. Using this method, all sound but chloride-contaminated concrete surrounding the top layer of reinforcing bars is to remain in place.

The procedure for this method is as follows:

1. Dry mill the concrete cover;
2. Remove all damaged concrete, clean any exposed reinforcing bars, replace damaged reinforcing bars (if necessary), patch with a corrosion-inhibitor-modified concrete, and shotblast the entire deck surface;
3. Apply three spray applications of the corrosion inhibitor; and
4. Overlay with a corrosion-inhibitor-modified microsilica concrete (MSC), LSDC, or LMC.

The spray-on corrosion inhibitor may be either organic or inorganic. If the spray-on corrosion inhibitor is inorganic, then the patch and overlay material should be inorganic-inhibitor-modified concrete. Similarly, if the spray-on corrosion inhibitor is organic, the patch and overlay material should be organic-inhibitor-modified concrete.

In many cases, the thickness of the 2-inch overlay will be greater than the milling depth, thus adding an additional dead load to the entire structure. In these cases, the reduction in live load capacity must be checked against present and future requirements. In addition, the spray-on organic corrosion inhibitors leave a surface residue that significantly reduces the bond between existing concrete and the overly concrete. Hence, the entire deck surface must be lightly sandblasted or shotblasted to remove the bond-reducing surface.

During the inhibitor treatment process, all damaged (previously patched, spalled, and delaminated) and highly corroded areas are patched with a corrosion-inhibiting concrete. Also, highly chloride-contaminated concrete is removed by dry milling, and these areas are treated with a spray-on corrosion inhibitor. The spray-on inhibitor is intended to diffuse to the reinforcing steel and retard corrosion. The corrosion inhibitor in the overlay is intended to work the same way by diffusing into the concrete and providing protection throughout the life of the rehabilitated deck. However, chlorides may still diffuse through the overlay from future winter maintenance or exposure to seawater. Thus, the service life of the corrosion inhibitor deck rehabilitation method is influenced by the amount of inhibitor on the reinforcing steel surface, by chloride concentration, by temperature, and by type of overlay.

Cathodic Protection Systems

Cathodic protection technology has been used on ships and pipelines for many decades as a method to control corrosion. Stratfull offers one of the first examples of using cathodic protection on a highway structure [65]. As a result of extensive government and private industry research in the development of cathodic protection systems for reinforced concrete structures, the Federal Highway Administration has stated that cathodic protection is the only rehabilitation technique that has proven to stop corrosion in salt-contaminated bridge decks regardless of the chloride content in concrete [66]. Another publication describes the satisfactory operation of more than 350 systems in 37 states and 8 provinces in Canada [67]. A more recent evaluation of various types of cathodic protection systems suggests that cathodic protection is effective if designed, installed, and operated in accordance with recommended practice [68, 69]. Although cathodic protection can be applied during new construction, it is most often found in conjunction with rehabilitation of existing concrete structures. However, cathodic protection is not always needed, nor is it applicable on every structure [70].

Cathodic protection is achieved by supplying an external source of current to counteract the internal corrosion current. During the corrosion process, current flows from anodic areas to cathodic areas. The actual corrosion (i.e., reduction of cross section) occurs at these anodic areas. During cathodic protection, current flows from an auxiliary anode material through an electrolyte (i.e., concrete) to the surface of the reinforcing bars. Through this process, corrosion is greatly minimized. Cathodic protection can be grouped into two basic types of systems: impressed current and galvanic anode systems. In both cases, the reinforcing bars are forced to function as a cathode (hence, the name cathodic protection).

An impressed-current cathodic protection is achieved by driving a low-voltage direct current (generally less than 50 volts) from a relatively inert anode material, through the concrete, to the reinforcing bars. Direct current of sufficient magnitude and direction is applied to oppose the natural flow of current resulting from the electrochemical process. The direct current is supplied by an external power source, often a rectifier that converts alternating current to direct current. Recently, solar power and specially designed batteries have been successfully used as external power sources. The direct current is distributed to the reinforcing bars by an anodic material. The uniformity of the current distribution is critical; therefore, an anode is one of the most important components of a cathodic protection system. Current distribution is also a major consideration in the design of cathodic protection systems.

Galvanic cathodic protection is based on the principles of dissimilar metal corrosion and the relative position of specific metals in the galvanic series. No external power source is needed with this type of system, and much less maintenance is required. Because of the limited power provided by these systems, actual installations have primarily been on bridge substructure elements in marine environments where the concrete resistivity is generally much lower. An example of the use of galvanic cathodic protection systems is by the Florida Department of Transportation, where much success has been reported on bridge structures in marine environments [71, 72].

Prior to selecting a cathodic protection system for a given structure, several issues need to be considered:

- **Long-term rehabilitation:** Cathodic protection is most cost-effective when long-term rehabilitation (greater than 15 to 20 years) is desired.
- **Electrical continuity:** A closed electrical circuit (i.e., unbroken electrical path) between all reinforcing bars is required for a cathodic protection system to function properly.
- **Chloride concentration:** If chlorides at the reinforcement bar depth are in sufficient concentration to initiate corrosion, cathodic protection may be the only viable method of rehabilitation.
- **Alkali-silica reaction:** Because the application of cathodic protection system increases alkalinity at the steel-concrete surface, alkali-silica reaction can be accelerated.

The design of the specific cathodic protection system depends upon the type of surface to be protected (horizontal, vertical, soffit, etc.), geometry, reinforcement cover depth, environmental considerations (temperature and moisture), and structural considerations (additional dead load capacity). Several cathodic protection systems have been in operation for over 17 years [69]. The titanium-anode-based cathodic protection systems can be expected to last over 50 years under certain circumstances.

Electrochemical Chloride Extraction

Electrochemical chloride extraction is similar in principle to cathodic protection. However, the total amount of charge (i.e., current with time) is approximately 50 to 500 times that used for cathodic protection. Another important difference is that chloride extraction is a short-term treatment, whereas cathodic protection is normally intended to remain in operation for the life of the structure. Chloride extraction is particularly suited for structures in which active corrosion is occurring, but only minor concrete damage is present. In addition, the structure must be conventionally reinforced and have an expected remaining service life of 5 to 10 years.

Chloride extraction is accomplished by applying an anode and electrolyte to the structure's surface and passing direct current between the anode and the reinforcing bars, where the reinforcement acts as a cathode. Conduction of direct current through the concrete is accomplished by the movement of charged ions. Since anions (i.e., negatively charged ions) migrate toward the positively charge anode, it is possible to cause chloride ions to migrate away from the reinforcing bars. The speed at which this process is accomplished largely depends on the magnitude of the applied current.

The simple movement of ions through concrete does not appear to have any deleterious effects on the concrete. However, chemical changes occur at both the surface of the anode and the surface of the reinforcing bars. These changes are the result of electrochemical reactions that occur wherever current enters or exits the concrete.

Reduction reactions that result in an increase in alkalinity and possible evolution of hydrogen gas take place at the reinforcement according to the following reactions:

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

$$2H_2O + 2e^- \rightarrow H_2 + 2(OH)^-$$

The first of these reactions occurs slowly because the availability of oxygen in concrete is limited. Most of the current entering the reinforcement will therefore result in the production of hydrogen at the reinforcement surface. The evolution of hydrogen can cause hydrogen embrittlement on prestressed reinforcement. The production of hydroxyl ions in these reactions increases the alkalinity at the surface of the reinforcement. This tends to repassivate the reinforcement and thus helps to prevent the re-initiation of corrosion.

The electrochemical reactions that may take place at an inert anode include the following:

$$2H_2O \rightarrow O_2 + 4H^+$$

$$2Cl^- \rightarrow Cl_2 + 2e^-$$

The importance of these reactions depends on the conditions present within the concrete. If the electrolyte is very acidic ($pH < 4$), a significant amount of chlorine gas will evolve, resulting in safety and environmental concerns. If the electrolyte is alkaline ($pH > 7$), then the evolution of oxygen becomes the favored anodic reaction. The small amount of chlorine that evolves under such conditions is rapidly hydrolyzed to form hypochlorous acid and hypochlorite according to the following reactions:

$$Cl_2 + H_2O \rightarrow HClO + Cl^- + H^+$$

$$HClO \rightarrow ClO^- + H^+$$

Under these conditions, the evolution of oxygen and chlorine increases acidity (H^+). This outcome raises concern about etching of the concrete surface that is in contact with the electrolyte. These concerns were addressed by Morrison et al. in a project that identified conditions under which the chloride extraction process can be conducted safely and effectively [73]. Current density is maintained low enough to avoid damage to the concrete and reduction in bond. The electrolyte pH is controlled to avoid etching of the concrete surface and generation of gaseous chlorine.

A typical chloride extraction system uses an anode/blanket composite and a contained borate-buffered electrolyte. The borate buffer maintains the electrolyte at a high pH despite acid generation at the anode. The anode used is an inert catalyzed titanium mesh, which resists corrosive anodic reactions. The system is installed by fastening the anode/blanket to the concrete surface. A sump tank is provided at the base of the structure to act as a reservoir for the electrolyte. The electrolyte is continuously pumped from the tank to the top of the structure and flows by gravity back into the tank.

The recommended current used for chloride extraction is in the range of 100 to 500 mA/ft^2 of treated surface. Current levels below 500 mA/ft^2 have proven to be harmless to the concrete structure. Current on field structures will usually be limited because of the 50-VDC maximum operating voltage. The National Electrical Code (NEC) requires exposed live electrical parts operating at 50 V or greater to be guarded; this is not possible with most chloride extraction installations. The electrical resistance of the concrete in most structures is such that the process will usually operate at maximum voltage; in addition, the current will be well below 500 mA/ft^2.

The duration of the treatment process is usually expressed in terms of total charge (i.e., current with time). Effective chloride extraction is typically accomplished by applying a total charge of 600 to 1,500 A-hr/ft^2. Too little charge will not remove sufficient chlorides or allow enough alkalinity to build up at the reinforcement to effectively prevent further corrosion, and too much treatment involves unnecessary expense. The recommended amount of total charge is usually reached in 10 to 50 days. A higher charge and longer treatment time is recommended for structures in which the chloride content at the level of reinforcement is high.

The treatment process described above will remove approximately 20% to 50% of the chloride present in the concrete, depending on the amount of chloride present, the distribution of chloride within the structure, and the design of the reinforcement. Typically, after treatment is complete, sufficient chloride will remain in the structure to initiate corrosion. However, the remaining chloride is usually distributed well away from the reinforcement, and much time is required for the chlorides to redistribute in sufficient quantities at the reinforcement to initiate corrosion. The return to corrosive conditions is further delayed by the buildup of alkalinity that occurs at the surface of the reinforcement. This chloride extraction process was successfully demonstrated on a bridge deck in Ohio and substructures in Florida, New York, and Ontario [74]. Clemena and Jackson showed this technique to be effective on two structures in Virginia [75], and Broomfield and Buenfeld demonstrated the effectiveness of chloride extraction on a bridge structure in England [76]. Tritthart stated that the effectiveness of the extraction technique is limited if significant chlorides have moved beyond the depth of reinforcement [77]. Sohanghpurwala verified the effectiveness of electrochemical chloride extraction on bridge superstructure and substructure elements and on slabs in the laboratory [78]. He reported that slabs treated with electrochemical chloride extraction had not exhibited any signs of corrosion 10 years after the treatment and concluded that protection for another 10 years could be expected.

CHAPTER 6

Procedure for Design of Repair and Corrosion Control System

The procedure for designing a repair and corrosion control system for a bridge superstructure concrete element includes the following steps:

1. Observe symptoms indicating corrosion of reinforcing steel embedded in concrete during the Routine Bridge Inspection.
2. Perform the Preliminary Corrosion Condition Evaluation (PCCE).
3. Use the service life model developed for this project to schedule repair and rehabilitation of the structure. The service life model is used to determine the remaining service life.
4. Perform the In-Depth Corrosion Condition Evaluation within 2 years prior to construction. This evaluation is used to determine quantity of damage that will need to be repaired and to select a corrosion control system to extend the service life of the superstructure element.
5. Use the service life modeling and Susceptibility Index (SI) to develop a repair and rehabilitation strategy. Data collected in the PCCE and the In-Depth Evaluation are used to calculate SI. The quantity of damage and the SI are used to develop a repair and corrosion control strategy.
6. Develop construction documents.

A flow chart depicting the decision-making process for the above steps is presented in Figure 4.

Overview of Procedures for Designing Repair and Corrosion Control System

All bridge structure owners perform regular maintenance and NBIS-prescribed Routine Inspections. During a Routine Inspection, if signs of corrosion are observed, then a decision with regards to performance of the PCCE needs to be taken. If a PCCE has not been previously performed for the structure, one should be performed during the next Routine Inspection. After the completion of the PCCE, the service life model should be used to generate the damage-versus-age plot. The plot is then used to determine the remaining service life of the concrete element in question.

To determine the remaining service life, a criterion must be defined in the form of percentage of surface damage that can be sustained by the concrete element prior to requiring major repair and rehabilitation. Structural and functional aspects of the concrete element will govern the criterion for scheduling the repair of the concrete element. This criterion will vary from one type of element to another. For example, the criterion for a bridge deck most likely will differ from that for a beam or a girder. The time required to reach the criterion from the time of the analysis is termed “remaining service life.” For example, if the criterion for a bridge deck is *x*% damage, and the service life model results indicate that it will take another 8 years to reach that damage level (from the time of the analysis), then the remaining service life is 8 years.

The preferred repair option will depend on the remaining service life. For example, if the remaining service life is greater than 20 years, the optimal solution most likely will be to continue with the regular maintenance, whereas if the remaining service life is between 10 years and 20 years, a corrosion control system may be an optimal choice because it provides an inexpensive barrier corrosion control system that controls the ingress of chloride ions and increases the remaining service life. Generally, a major repair would be scheduled if the remaining service life is less than 10 years. Owners should identify the optimal options based on their needs and experience. Once a structure is scheduled for repair, the In-Depth Evaluation is performed 2 years prior to construction. This would be followed by modeling of service life, design of a repair and

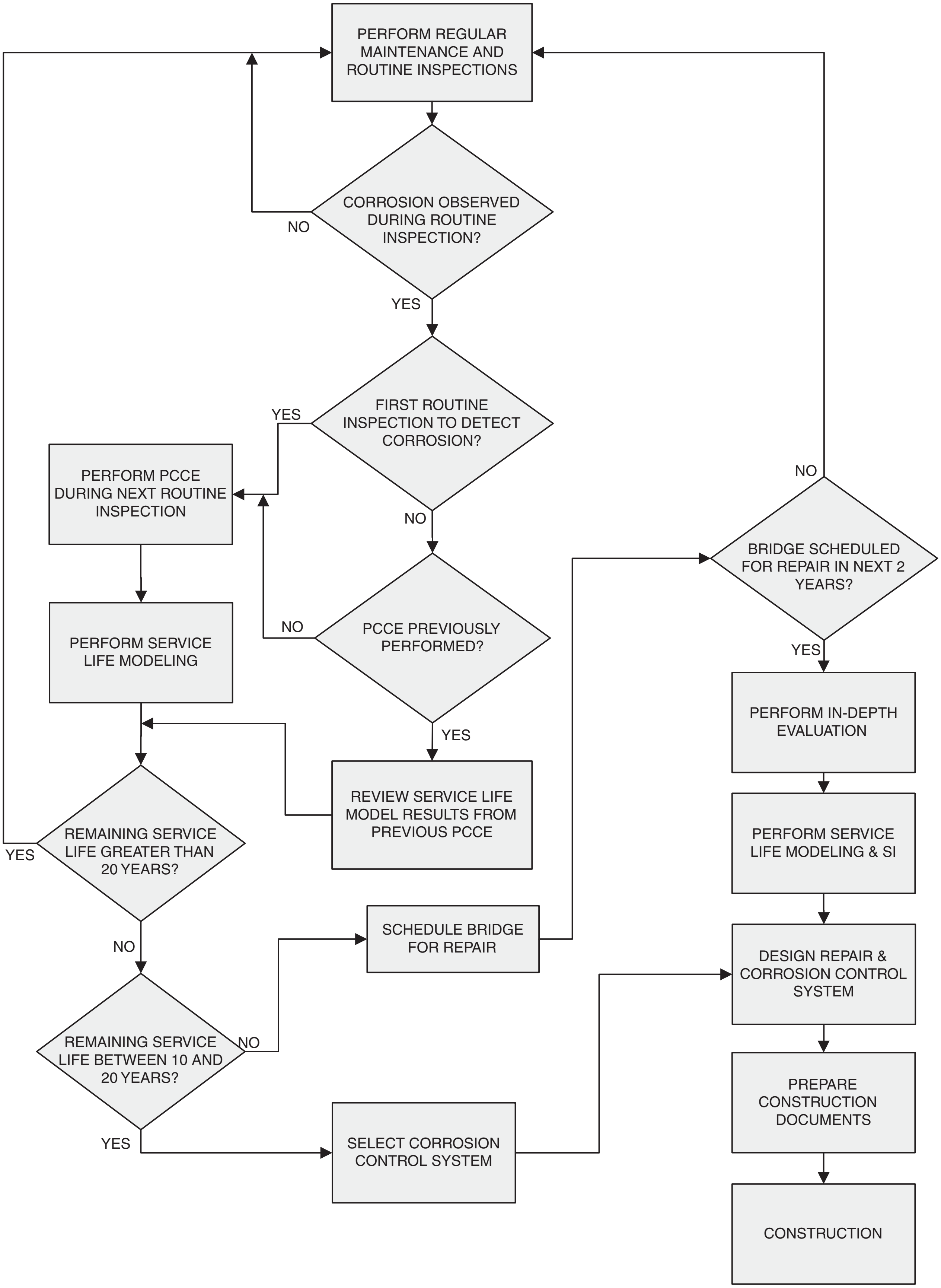

Figure 4. Methodology for designing repair and corrosion control system.

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corrosion control system, preparation of construction documents, and construction.

The methodology should be followed after each Routine Inspection. If signs of corrosion have been detected, then the performance of a PCCE in the past must be verified. If a PCCE has been performed in the past, then the previous service life model results should be used to recalculate the remaining service life. The next step in the methodology should be selected on the basis of the recalculated remaining service life.

Preliminary Corrosion Condition Evaluation (PCCE)

Recommendations for selection of test methods and techniques for PCCE are presented in Table 1. Clear concrete cover, visual and delamination surveys, and chloride profile analysis are essential and must be conducted during the PCCE. If desired, other tests may also be conducted. Minimum sampling size for each test method is presented in Table 2. If variations in exposure or other factors that may impact the corrosion process are noted, sampling must be performed in at least one area representing each variation.

In-Depth Corrosion Condition Evaluation

Recommendations for test methods and techniques for the In-Depth Evaluation are presented in Table 3. In addition to the data collected in a PCCE (clear concrete cover, visual and delamination surveys, and chloride profile analysis), electrical continuity testing and petrographic analysis are required. Minimum sampling size requirements are listed in Table 4.

If clear concrete cover data are available from a previous PCCE, the data need not be collected again in the In-Depth Evaluation. All other tests must be conducted, and data from a previous PCCE and the In-Depth Evaluation shall be used in the service life modeling.

Sampling Size

Sampling size of the various test methods is based on the sensitivity of the model to the input parameters. Because it is inexpensive to collect clear concrete cover data, 30 measurements per span are recommended. However, the cost of collecting chloride profile and epoxy-coated rebar cores is much higher, and so fewer samples are recommended. An increase in the

Table 1. Recommendations for testing during PCCE.

Type of Element/ Type of Steel/ Surface Treatment	Cover	Visual	Delamination	Chloride Profile	Epoxy-Coated Rebar Cores	Continuity	Carbonation	Petrographic	Half-Cell	Corrosion Rate
Deck										
Black Steel										
Bare	✓	✓	✓	✓	N/A	×	×	×	†	†
Concrete Overlay	✓	✓	✓	✓	N/A	×	×	×	†	†
Asphalt Overlay	✓	N/A	N/A	✓	N/A	×	×	×	N/A	N/A
Epoxy Overlay	✓	N/A	N/A	✓	N/A	×	×	×	N/A	N/A
Membranes	✓	✓	N/A	✓	N/A	×	×	×	N/A	N/A
Sealers	✓	✓	✓	✓	N/A	×	×	×	N/A	N/A
Epoxy-Coated Rebar										
Bare	✓	✓	✓	✓	✓	×	×	×	†	†
Concrete Overlay	✓	✓	✓	✓	✓	×	×	×	†	†
Asphalt Overlay	✓	N/A	N/A	✓	✓	×	×	×	N/A	N/A
Epoxy Overlay	✓	N/A	N/A	✓	✓	×	×	×	N/A	N/A
Membranes	✓	✓	N/A	✓	✓	×	×	×	N/A	N/A
Sealers	✓	✓	✓	✓	✓	×	×	×	N/A	N/A
Beams & Girders										
Black Steel										
Bare	✓	✓	✓	✓	N/A	✓	†	†	†	†
Paints	✓	✓	✓	✓	N/A	✓	†	†	†	†
Epoxy-Coated Rebar										
Bare	✓	✓	✓	✓	✓	✓	†	†	†	†
Paints	✓	✓	✓	✓	✓	✓	†	†	†	†

Notes: ✓ - mandatory, × - recommended, † - optional, N/A - not applicable

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Table 2. Minimum sampling size for PCCE.

Test Method	Minimum Sampling Size
Clear Concrete Cover (CCC) (using nondestructive testing methods). Several actual CCC measurements should be collected to calibrate nondestructive testing equipment.	30 measurements per span.
Visual Survey	Entire surface of the concrete element.
Delamination Survey	10% of the total surface area. If exposure is variable, use several test areas. The test areas should be selected to represent all variations.
Chloride Profile Analysis	1 location per 3,000 square feet or a minimum of 5, whichever is higher.
Epoxy-Coated Rebar Cores	Minimum of 5.

number of samples collected will improve the accuracy of the service life model. Sampling size for electrical continuity testing and petrographic analysis is based on general field practice.

Service Life Modeling and Susceptibility Index

The clear concrete cover, visual and delamination surveys, and chloride profile data are used to model the service life of the concrete element. The model may need to be run for several chloride threshold values until there is a good agreement between the model-predicted damage and the total damage observed for all condition evaluations conducted on the element. The chloride threshold value for which the model best predicts actual damage is used to calculate the SI and subsequently select a corrosion control system. When dealing with large structures, if a variation in exposure conditions is suspected across the length of the structure, then the structure should be subdivided into sections, service life modeling should be performed, and the SI should be calculated for each section.

Selection of Repair and Corrosion Control System

Guidelines are provided to identify (a) all corrosion control alternatives that may be most applicable for a particular distribution of chloride ions and (b) the types of corrosion control systems that are most likely to provide optimal protection for a particular SI. Table 5 presents various combinations of repairs and corrosion control systems that can be used in conjunction for a range of SI. The following simplifications were made to identify various types of corrosion mitigation systems:

- All surface-applied coatings that are capable of controlling the flow of moisture into the concrete without hampering

Table 3. Recommendations for testing during In-Depth Evaluation.

Type of Element/ Type of Steel/ Surface Treatment	Cover	Visual	Delamination	Chloride Profile	Epoxy-Coated Rebar Cores	Continuity	Carbonation	Petrographic	Half-Cell	Corrosion Rate
Deck										
Black Steel										
Bare	✓	✓	✓	✓	N/A	✓	×	✓	†	†
Concrete Overlay	✓	✓	✓	✓	N/A	✓	×	✓	†	†
Asphalt Overlay	✓	N/A	N/A	✓	N/A	✓	×	✓	N/A	N/A
Epoxy Overlay	✓	N/A	N/A	✓	N/A	✓	×	✓	N/A	N/A
Membranes	✓	✓	N/A	✓	N/A	✓	×	✓	N/A	N/A
Sealers	✓	✓	✓	✓	N/A	✓	×	✓	N/A	N/A
Epoxy-Coated Rebar										
Bare	✓	✓	✓	✓	✓	✓	×	✓	†	†
Concrete Overlay	✓	✓	✓	✓	✓	✓	×	✓	†	†
Asphalt Overlay	✓	N/A	N/A	✓	✓	✓	×	✓	N/A	N/A
Epoxy Overlay	✓	N/A	N/A	✓	✓	✓	×	✓	N/A	N/A
Membranes	✓	✓	N/A	✓	✓	✓	×	✓	N/A	N/A
Sealers	✓	✓	✓	✓	✓	✓	×	✓	N/A	N/A
Beams & Girders										
Black Steel										
Bare	✓	✓	✓	✓	N/A	✓	†	†	†	†
Paints	✓	✓	✓	✓	N/A	✓	†	†	†	†
Epoxy-Coated Rebar										
Bare	✓	✓	✓	✓	✓	✓	†	†	†	†
Paints	✓	✓	✓	✓	✓	✓	†	†	†	†

Notes: ✓ - mandatory, × - recommended, † - optional, N/A - not applicable

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Table 4. Minimum sampling size for In-Depth Evaluation.

Test Method	Minimum Sampling Size
Clear Concrete Cover (using nondestructive test methods). Several actual CCC measurements should be collected to calibrate nondestructive test methods equipment.	30 measurements per span. If cover measurements from a previous PCCE are available, they can be used instead of collecting the data again in the in-depth evaluation.
Visual Survey	Entire surface of the concrete element.
Delamination Survey	Entire surface of the concrete element.
Chloride Profile Analysis	1 location per 1,000 square feet.
Electrical Continuity Testing	5 reinforcing steel bars in each span. Must include both transverse and longitudinal bars.
Epoxy-Coated Rebar Cores	Minimum of 5.
Pertrographic Analysis	1 location per 3,000 square feet or a minimum of 5, whichever is higher.

the outflow of moisture (i.e., they are breathable) have been lumped together as "sealers." The term sealers is most often used to refer to silane/siloxane-based material that is applied to the surface of the concrete to reduce the inflow of vapor but are not as effective when water is ponded on the surface of the sealer. All materials in this category provide varying degrees of barrier to the transport of chloride ions into the concrete.

- The term "membrane" is used to denote surface-applied systems that do not allow transport of moisture in either direction and that are considered waterproofing materials. These systems, as long as their integrity is maintained, do not allow moisture and chloride ions to enter concrete. The asphalt overlay with the waterproofing membrane is included in this category.
- The term "overlays" refers to cementitious and noncementitious wearing surfaces installed on the deck surface. Asphalt overlays are not included because they do not serve as barriers to moisture and chloride ions. Overlays are installed to reduce the rate of or stop the ingress of moisture and chloride ions into the original concrete element and to increase the depth to which the chlorides have to diffuse to reach the steel, thereby increasing the time to corrosion initiation. In addition to corrosion benefits, these overlays serve as a wearing surface on bridge decks.
- The term "corrosion inhibitors" refers to all materials that have chemicals that can interfere with the corrosion process or parts of the corrosion process, such as the cathodic or the anodic reactions, or that combine with one of the reactants to reduce their availability and consequently reduce or stop corrosion. The corrosion inhibitors can be surface applied or admixed with repair concrete.
- The term "cathodic protection systems" refers to galvanic and impressed-current systems, and re-alkalization is considered a part of electrochemical chloride extraction.

Table 5. Selection of corrosion control system.

REPAIR TYPES	SI < 2	SI ≥ 2	SI ≥ 4	SI ≥ 5	SI ≥ 7	SI ≥ 8
WHEN REPLACEMENT IS MORE COST-EFFECTIVE	NEW CONCRETE ELEMENT (CORROSION CONTROL CAN BE INCORPORATED INTO IT)					
TOP LAYER CONCRETE REPLACEMENT	SEALERS, MEMBRANES, OVERLAYS			SI TOO HIGH FOR TOP CONCRETE LAYER REPLACEMENT		
PATCH REPAIR + OVERLAY	CATHODIC PROTECTION EPOXY-COATED REBARS		OVERLAY SERVES AS A CORROSION CONTROL SYSTEM			
PATCH REPAIR						DO NOTHING
					SEALERS	
				MEMBRANES		
			OVERLAYS & OVERLAYS PLUS MEMBRANE			
		CORROSION INHIBITORS				
	CATHODIC PROTECTION & ELECTROCHEMICAL CHLORIDE EXTRACTION					

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Table 6 summarizes the service life extensions that have been reported in literature for various corrosion control systems. It should be noted that the summary is not exhaustive and many of the categories are broad. There are numerous products in each category, and the service life provided by each product highly depends on the applicability of the product to the structure's corrosion condition. For example, a corrosion control system applied to a structure exposed to a very mild environment (for that particular corrosion control system) can provide a significantly longer service life than if it had been applied to a structure in a very aggressive environment. Similarly, when a corrosion control system that is inappropriate for the corrosion condition of the structure is used, early failure can occur. Service life extension by a corrosion control system depends on the corrosion condition of the structure, the exposure conditions, the applicability of that particular system to the condition of the structure, the quality of design and application of the system onto the structure, and the maintenance of the system after its application.

Table 6 should be used only as a guideline; further information on the system of interest should be obtained prior to determining the obtainable service.

Planning for Corrosion Condition Evaluation

Proper planning for condition evaluation can help reduce the time required in the field and acquire better data. The goal of the condition evaluation must be clearly defined, and the testing planned should be in line with the goals of the survey. Although the same test methods and techniques may be used during various different condition evaluations, the selection of sampling sites, the number of sampling sites, and the level of detail obtained will vary from one condition survey to

Table 6. Extensions in service life for various corrosion control systems.

Corrosion Control System	Service Life (years)	Comments
Patching	4 to 10, 4 to 7	Patching with Portland concrete cement and mortar.
Reinforcing Bar Coating		No information available in literature.
Repair of Epoxy-Coated Rebar	> 3	Study did not monitor the repair procedure beyond 3 years; therefore, it is difficult to predict its service life.
Corrosion Inhibitor Surface Application	4 to 6	Service life is based on application of the inhibitor in the test patches in highly contaminated concrete.
Corrosion Inhibitor Plus Patching	4 to 6	Service life is based on application of the inhibitor in the test patches in highly contaminated concrete.
LMC Overlay	20	Based on study of several bridges in the state of Virginia.
LSDC Overlay	20	Numerous studies corroborate the findings of this study.
HMAM Overlay	< 10, 25	Less than 10 years is based on the failure of the HMA overlay, which would also mean the end of service life of the waterproofing membrane.
Penetrating Sealers	5 to 7	The service of 7 years for penetrating sealers is generally accepted.
Surface Coatings		There are numerous kinds of coatings, and sufficient information is not available to define this category.
Corrosion Inhibitor Overlays		No information available in literature.
CP	5 to > 25	There are numerous kinds of CP systems, and service life varies from one type to another.
ECE	10 to 20	Service life of ECE-treated concrete element is governed by ingress of chloride ions after the treatment. The service life quoted herein is based on no chlorides migrating into the concrete element.

LMC = latex-modified concrete
LSDC = low-slump dense concrete
HMAM = hot mix asphalt with a preformed membrane
CP = cathodic protection
ECE = electrochemical extraction

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another. However, certain information on the structure must be collected and reviewed prior to the survey, including the following:

- Size and type of structure,
- Unusual features in the design,
- Structure location and topography,
- Environmental exposure (temperature variations, marine environment, etc.),
- Reinforcing steel layout,
- Type of reinforcement (epoxy coated, bare steel, or galvanized),
- Repair and maintenance history,
- Year of construction,
- Number of spans,
- Presence of protective systems (cathodic protection, overlay, membranes, etc.),
- Average daily traffic (ADT),
- Limitations of traffic control (if any), and
- Annual application rate and type of deicing chemical (if used).

In addition to collecting information about the structure, the following logistical provisions must be ensured:

- Power and water must be available for operating equipment and conducting certain tests.
- Sensitivity of the surrounding environment to debris generated during the survey must be considered. At certain sites, especially in some marine environments, discarding debris may be prohibited in order to protect local biological species.
- There must be access to the concrete elements that have to be evaluated (e.g., specialized equipment may be required for access to girders and beams).
- A safety plan needs to be prepared to ensure that all work is performed in a safe manner.
- A traffic control plan needs to be prepared, and sequence of testing needs to conform to the traffic control plan. Restrictions need to be taken into account when planning the testing schedule (e.g., in some urban areas, traffic control is only allowed during certain hours of the day).
- An emergency plan must be in place to deal with any accidents that may occur.

Based on the type of survey, it may be necessary to document the location of the sampling site using a grid. If a grid is required, planning for the orientation of the X-Y grid must be prepared prior to going on-site. Either the grid can be marked on the surface of the concrete element or some form of grid markers can be placed on the surface, such as a measuring tape or a rope or string marked at equal intervals.

Most often during a condition evaluation, concrete excavation or collection of cores is required. The excavated areas must be patched with an appropriate and approved material. It is necessary to ensure that the patch is installed using proper construction practices so as not to increase the susceptibility of the concrete element to corrosion. If epoxy-coated rebars are involved, any damage inflicted on rebars exposed during the condition survey will need to be repaired (e.g., by using an epoxy material to patch the excavation, thus avoiding any exposure of the damaged sections of the epoxy-coated rebars to the environment).

Proper marking and handling of the samples is very important, and a proper marking and handling plan needs to be developed. Using more than one marking scheme on each sample will make it easier to identify the samples when some parts or one of the markings is erased or becomes unreadable.

All equipment must be tested prior to use, and, when possible, backup should be provided. Some pieces of electronic equipment most commonly used in condition evaluation can be affected by exposure to certain environments. In marine environments, certain electronic equipment will fail after several days of use because of exposure to chloride ions in the air.

For greater productivity, an optimal crew size should be selected for each condition, evaluation, and scope of work. If a sequence of all sampling and testing work is prepared, the optimal crew size becomes obvious. Only certain tests and sampling can be conducted simultaneously. Often it is beneficial to conduct the visual and the delamination survey first because much of the testing and sampling work is not performed in the delaminated areas. This survey is often followed by locating reinforcing steel for coring, chloride sampling, electrical continuity testing, and cover measurement. Half-cell potential measurements, corrosion rate measurement, and core collection can occur simultaneously at different parts of the concrete element if equipment used in one test is not interfering with equipment used in another test.

CHAPTER 7

Field Evaluation Procedures

This chapter presents detailed procedures for field testing and sampling methodologies. The procedures have been prepared primarily for bridge deck application, but they can easily be applied to other superstructure members.

Grid Stationing

1. For safety and coordination with traffic control patterns, mark grids in the direction of traffic flow. In the field, it is often more convenient to have a separate grid for each set of lane closures. If only one lane can be closed at a time, then a separate grid is marked for each lane. Sometimes only one span is closed and the grid can be established only for that span. All grids can later be combined in the laboratory to provide a single point of reference for the entire structure.
2. Identify the span numbering scheme from the bridge drawings and start at Span 1 in the direction of traffic.
3. Number the lanes from right to left in each direction. If only one lane exists in each direction, then mark it as Lane 1 in each direction.
4. Locate the origin (0, 0), which is the starting point for the grid at the intersection of the rightmost edge of the rightmost lane (facing in the direction of traffic) of the closed section.
5. Mark the X-axis in the longitudinal direction and the Y-axis in the transverse direction. Generally, a grid spacing of either 2 feet or 5 feet is used.
6. Suggested Equipment (Figure 5):
   a. 100-foot, reel-type tape measure or land wheel. A rope or a string marked at regular intervals can also be used.
   b. Lumber crayon (keel) or spray paint.

Visual Survey

1. Conduct the visual survey in accordance with ACI 201.1R-92, "Guide for Making a Condition Survey of Concrete in Service."
2. Sketch the surface view of the concrete element to be surveyed.
3. Record dimensions of areas.
4. Sketch location of observed typical deterioration (spalls, patches, cracks, scaling, rust staining, and efflorescence) and note severity of each.
5. Photograph the structure such that the overall condition and any items of interest are documented.
6. Suggested Equipment:
   a. Tape measure.
   b. Camera.

Delamination Survey

1. Perform the delamination survey in accordance with ASTM D-4580-86, "Standard Practice for Measuring Delaminations in Concrete Bridge Decks by Sounding."
2. Using a hammer, chain drag (Figure 6), or chain, perform a delamination survey in the subject area (Figure 7). Other nondestructive equipment can also be used (see Chapter 3).
3. Mark delaminated areas with a lumber crayon (keel) using simple geometric shapes such as a square, a rectangle, and a triangle. Because it is very time consuming to identify the exact shape of the delamination and transfer it to a drawing, simple shapes are used. However, the use of the shapes must be consistent throughout the survey.
4. Record dimensions of delaminated areas and any areas that have been patched by either concrete or asphalt.
5. Determine the percentage of delaminated area as follows:

$$Delam\% = \frac{X}{A}100$$

where
X = total delaminated area
A = total surface area of the concrete element

6. Suggested Equipment:
   a. Hammer/chain/chain drag.

Figure 5. Equipment for grid marking.

b. Lumber crayon (keel).
c. Tape measure.

Cover Depth Measurements

1. With a covermeter, determine and mark on the surface of the deck the location and direction of the reinforcing steel in the areas to be tested.
2. A minimum of 30 measurements per span are generally required. Of the 30 measurements, 5 measurements need to be actual depths measured (see Steps 3, 4, and 5), and the balance of 25 can be obtained directly from the covermeter and then correlated.
3. Using a hammer drill and a 0.75-inch drill bit, drill a hole to expose the reinforcing steel. A core drill can be used in the place of a hammer drill. Use a 1- or 2-inch core bit to expose the reinforcing steel.
4. Clean out all dust and debris (including water if a core drill was used) from the excavation using a wet-dry vacuum.
5. Measure the cover depth as shown in Figure 8.
6. Patch the excavation upon completing all testing on the exposed steel.
7. Suggested Equipment:
   a. Covermeter.
   b. Lumber crayon (keel).
   c. Generator.
   d. Hammer drill or a core drill.
   e. 0.75-inch hammer bit or a 1- or 2-inch core bit.
   f. Wet-dry vacuum.
   g. Tape measure.

Continuity Testing—Direct Current Method

1. Using a covermeter, locate and mark the required number of transverse and longitudinal reinforcing bars on

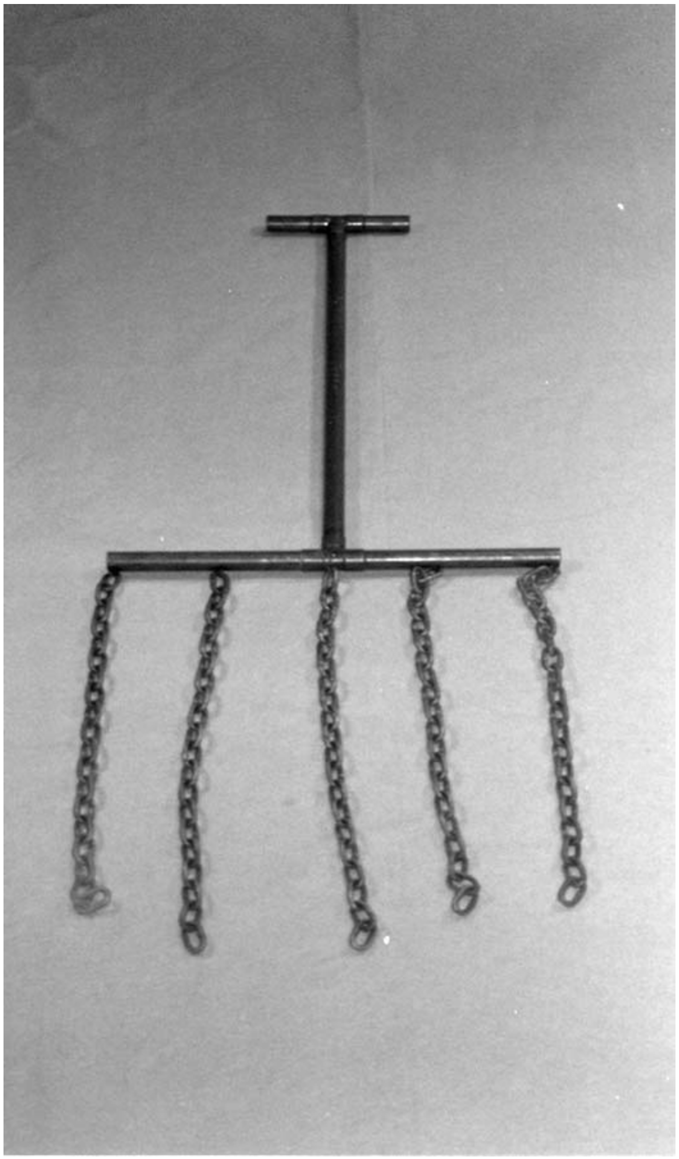

Figure 6. Chain Drag.

Figure 7. Chain dragging in progress.

each span of the deck. Of each reinforcing bar type, at least five bars should be tested in each span. If there is rebar already exposed from other tests or in spalled areas, this rebar could also be used instead of excavating new areas.

2. To expose a bar, use a hammer drill with a 0.75-inch drill bit. Drill down until the rebar is exposed.
3. Select one exposed rebar to be the ground location.
4. At the ground location, drill into the rebar and set a self-tapping screw, making sure that there is a tight fit. Use an alligator clip attached to the test lead to make connection with the screw in the rebar.
5. At the other exposed rebars, clean the surface of the bar to a bright metal with a wire brush, emery cloth, or file. Cleaning may not be necessary if a hammer drill was used to expose the bar.
6. Set the multimeter to read resistance. Ensure that the range automatically selected by the meter produces a resolution of 0.1 Ω or less.
7. Use one short test lead to connect to the rebar for testing and one long test lead to connect to the rebar designated as the ground.
8. Zero the multimeter and check the battery according to the manufacturer's instructions. Check and record the resistance of all test leads, including any length of wire necessary to reach all test locations.
9. Attach the long test lead from the positive terminal of the multimeter to the ground location.
10. Attach the short test lead from the negative (i.e., common) terminal of the multimeter to the rebar to be tested.
11. Read and record the resistance in ohms. Reverse the leads at the meter, and read and record the resistance again.
12. Set the multimeter to read voltage. Ensure that the range automatically selected by the meter produces a resolution of 0.001 V. Read and record the voltage difference between the two points.
13. Subtract the lead resistance obtained in Step 8 from both resistance values obtained in Step 11. If the resultant resistance values are both less than 1 Ω (or less than 1 Ω apart) and the voltage difference is no more than 0.001 V, the two points are electrically continuous.
14. Repeat this process for each exposed rebar using the same ground. When continuity of rails and other embedded metals in the concrete element have to be ascertained, the same procedure can be used. Use the rebar designated as the ground and the rail or the embedded metal as the test point.
15. If the direct current (DC) method shows different results when the leads are reversed or estimates negative resistance, the measurement may have been impacted by corrosion or stray current in the concrete element. Disconnect the multimeter and replace it with a soil resistance meter that reads alternating current (AC) (Figure 9). Connect the two leads to the soil resistance meter, and balance the bridge to determine the resistance between the ground and the test rebar. If the AC resistance is less than 1 Ω, then the rebars are considered to be continuous.

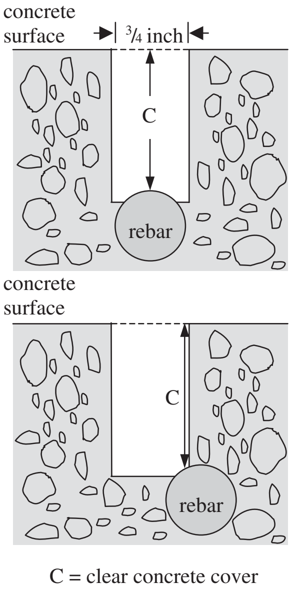

Figure 8. Cover depth.

Figure 9. Multimeter and AC soil meter.

16. When epoxy-coated reinforcing steel is encountered during evaluation of a structure, expect discontinuity between reinforcing steel.
17. Patch all test locations (unless the area will be used in another test).
18. Suggested Equipment:
    a. Multimeter.
    b. Soil resistance meter.
    c. Lead wires.
    d. Probe wires.
    e. Alligator clips.
    f. Hammer.
    g. Chisel.
    h. Wire reel.
    i. Rechargeable drill.
    j. Steel-tapping bits.
    k. Self-tapping screws (to match the size of the tapping bit).
    l. Screwdriver.
    m. Covermeter.
    n. Lumber crayon (keel).
    o. Hammer drill.
    p. 0.75-inch hammer bit.
    q. Extension cord.
    r. Generator.
    s. Wet-dry vacuum.

Core Sampling—Chloride Ion Distribution

1. Perform core extraction in accordance with ASTM C42/C42M-99, "Standard Test Method for Obtaining and Testing Drilled Cores and Sawed Beams of Concrete."
2. The area(s) to be sampled should be free from any defects, such as spalls, delaminations, and patches (unless testing patch material). Sample areas may be placed adjacent to defective areas, but not in them.
3. Using a covermeter, locate and mark all reinforcing steel in the area where the core(s) will be collected.
4. Mark the depth of the core to be taken plus 1 inch on the outside of the core bit (2-inch diameter).
5. Locate the core bit so as to avoid any reinforcing steel in the core.
6. Drill down to the desired depth (Figure 10).
7. Place the tip of the chisel in the gap between the core and the deck concrete. Gently hammer on the head of the chisel until the core breaks at the bottom. It helps to tap on the chisel a couple of times and then move the chisel to another location around the core.
8. If a delamination is observed in the core, take the core at another location. Delamination in the core can be verified by checking for a crack in the original concrete at the level of the break in the core.
9. Immediately upon extraction, label each core with Bridge #, Lane #, and Span # and X, Y coordinate of the sample.
10. Pack the core in a sealable plastic bag, and record all of the information from Step 9 on the outside of it.
11. Clean out the slurry/water from the excavation using a wet-dry vacuum.
12. Patch all test locations.
13. Suggested Equipment:
    a. Core drill.
    b. 2-inch core bit.
    c. Generator.
    d. Covermeter.
    e. Lumber crayon (keel).
    f. Hammer.
    g. Chisel.
    h. Wet-dry vacuum.
    i. Sealable plastic bags.

Figure 10. Core drilling.

Core Sampling—Epoxy-Coated Rebar

1. Perform core extraction in accordance with ASTM C42/C42M-99, "Standard Test Method for Obtaining and Testing Drilled Cores and Sawed Beams of Concrete."
2. The area(s) to be sampled should be free from any defects, such as spalls, delaminations, and patches (unless testing patch material). Sample areas may be placed adjacent to defective areas, but not in them.
3. Using a covermeter, locate and mark all reinforcing steel in the area where the core(s) will be taken. The sample will be taken at the intersection of two rebars in the top mat of reinforcement.
4. Mark the depth of the core to be taken on the outside of the core bit (4-inch diameter).
5. Drill down to the desired depth.
6. Place the chisel in the cut grove, and gently hammer while moving the chisel along the grove until the core breaks loose.
7. If a delamination is observed in the core, take the core at another location. Delamination in the core can be verified by checking for a crack in the original concrete at the level of the break in the core.
8. Immediately upon extraction, label each core with Bridge #, Lane #, and Span # and X, Y coordinate of the sample.
9. Wrap the core in wet burlap, pack it in a sealable plastic bag, and record all of the information from Step 8 on the outside of it.
10. If the core is not to be tested within 24 hours of receipt by the laboratory, it shall be stored in a freezer below 14°F.
11. Clean out the slurry/water from the excavation using a wet-dry vacuum.
12. Patch all test locations.
13. Suggested Equipment:
    a. Core drill.
    b. 4-inch core bit.
    c. Generator.
    d. Covermeter.
    e. Lumber crayon (keel).
    f. Hammer.
    g. Chisel.
    h. Wet-dry vacuum.
    i. Sealable plastic bags.
    j. Burlap.

Core Sampling—Petrographic Core Extraction

1. Perform core extraction in accordance with ASTM C42/C42M-99, "Standard Test Method for Obtaining and Testing Drilled Cores and Sawed Beams of Concrete."
2. The area(s) to be sampled should be free from any defects, such as spalls, delaminations, and patches (unless testing patch material). Sample areas may be placed adjacent to defective areas, but not in them.
3. Using a covermeter, locate and mark all reinforcing steel in the area where the core(s) will be taken.
4. Mark 5 inches on the outside of a 4-inch diameter core bit.
5. Locate the core drill such as to avoid any reinforcing steel.
6. Drill down to the desired depth.
7. Place the chisel in the cut grove, and gently hammer while moving the chisel along the grove until the core breaks loose.
8. If a delamination is observed in the core, take the core at another location. Delamination in the core can be verified by checking for a crack in the original concrete at the level of the break in the core.
9. Immediately upon extraction, label each core with Bridge #, Lane #, and Span # and X, Y coordinate of the sample.
10. Store the core in a sealable plastic bag, and record all of the information from Step 9 on the outside of it.
11. Clean out the slurry/water from the excavation using a wet-dry vacuum.
12. Patch all test locations.
13. Suggested Equipment:
    a. Core drill.
    b. 4-inch core bit.
    c. Generator.
    d. Covermeter.
    e. Lumber crayon (keel).
    f. Hammer.
    g. Chisel.
    h. Wet-dry vacuum.
    i. Sealable plastic bags.
    j. Pen.
    k. Tape measure.

Corrosion Potential Survey

1. Perform the corrosion potential survey in accordance with ASTM C-876, "Standard Test Method for Half-Cell Potentials of Uncoated Reinforcing Steel in Concrete."
2. A corrosion potential survey should be performed only after an area has been tested for continuity and found to be continuous, after a delamination survey has been conducted, and after the delaminations have been marked on the surface.
3. Either mark a 2-foot × 2-foot grid on the surface or use tape measure, rope, or string with marks on a 2-foot interval as a guide in both the longitudinal and the transverse directions.

4. Fill a copper-copper sulfate half-cell with copper sulfate solution in accordance with the manufacturer's instructions.
5. Select one exposed rebar that is ground installed for another test or expose a rebar to be the ground location (see Continuity Testing—Direct Current Method).
6. At the ground location, drill into the rebar and set a self-tapping screw, making sure that it is a tight fit. Use an alligator clip attached to the test lead to make connection with the screw in the rebar.
7. Connect the half-cell by a test lead to the negative terminal of your multimeter.
8. Connect the positive terminal on the multimeter by a test lead to the ground connection on the rebar. Please note that test leads should not exceed 500 linear feet for a test lead wire thinner than American Wire Gage (AWG) 24.
9. Spray with water all locations where half-cell potential measurement is to be made. Let the water soak in for a few minutes prior to making the measurement.
10. Set the multimeter to read in volts.
11. Begin your survey and record all readings to the nearest 0.001 V. Measurements should not be made in delaminated or spalled areas.
12. Suggested Equipment:
    a. Multimeter (with a 10-MΩ input impedance).
    b. Copper-copper sulfate half-cell.
    c. Copper sulfate solution.
    d. Test leads.
    e. Alligator clips.
    f. Wire reel.
    g. Rechargeable drill.
    h. Steel-tapping bits.
    i. Self-tapping screws.
    j. Screwdriver.
    k. Covermeter.
    l. Lumber crayon (keel).
    m. Measuring tape.
    n. Hammer drill.
    o. 0.75-inch hammer bit.
    p. Extension cord.
    q. Generator.

Corrosion Rate Measurement Survey

1. Select the rebar on which the corrosion rate measurement will be performed.
2. If all the steel is continuous, use the ground used for continuity testing or the half-cell potential measurement.
3. If steel is not continuous, install a ground on the bar to be tested.
4. If a ground has to be installed on the rebar to be tested, expose the rebar 18 inches away from the test location by drilling with a 0.75-inch drill bit.
5. At the ground location, drill into the rebar and set a self-tapping screw, making sure that it is a tight fit. Use an alligator clip attached to the test lead to make connection with the screw in the rebar.
6. Follow the manufacturer's instructions to use the corrosion rate measurement device. The procedure for conducting the corrosion rate measurement depends on the device.

Carbonation Testing

1. Choose a test for measuring carbonation depth. All available tests require 0.15% solution of Phenolphthalein in ethanol to be sprayed onto dry, freshly cut concrete.
2. In the field, find an area where the reinforcing steel has just been exposed for clear concrete cover measurement or electrical continuity testing.
3. In the laboratory, slice a core into two parts along its longitudinal axis.
4. Remove any dust from the surface of the area to be tested with a lint-free, salt-free, dry tissue paper.
5. Spray Phenolphthalein solution onto the cut surface. The indicator will exhibit a pink color on uncarbonated concrete and no color on carbonated concrete.
6. Measure and record the depth of carbonation in inches.
7. Suggested Equipment:
    a. 0.15% solution of Phenolphthalein in ethanol.
    b. Measuring tape or ruler.
    c. Drill with a drill bit to expose fresh concrete in the field.
    d. Concrete saw to slice the core in the laboratory.
    e. Lint-free, salt-free tissue paper.

Patching Excavated Areas

1. Patch all excavated areas with high-quality, dry, bagged concrete mix having a maximum aggregate size of 0.5 inch to 1 inch.
2. Suggested Equipment:
    a. Dry, bagged concrete mix.
    b. Mixing bucket.
    c. Trowel.
    d. Water.

CHAPTER 8

Laboratory Evaluation Procedures

Evaluation, Storage, and Preparation of Epoxy-Coated Rebar Cores

Pre-Extraction Procedures

1. If the cores are not to be tested within 24 hours of receipt by the laboratory, store them in a freezer below 14°F.
2. When cores are removed from the freezer, thaw them for a minimum of 24 hours or to room temperature, whichever is earlier.
3. Document the core identification number and date of initial testing on data sheets.
4. Measure the temperature of each core. If the temperature is less than 77°F, allow further thawing until a minimum temperature of 77°F is attained.
5. Remove all moisture from the surface of the cores with lint-free, salt-free, absorbent tissues to ensure that the surface is dry.
6. Document core information (Figure 11), including diameter, height[1] (h_t), cover over the topmost rebar (C_1 and C_2),[2] and height of concrete below the bottommost rebar (h_b). All lengths to be measured to the nearest one-eighth inch.
7. Visually observe the core to identify concrete damage (such as cracking), honeycombing, and evidence of deterioration (such as rust staining and delamination).
8. Label the core in multiple locations with the core identification number.
9. If the core will be used for chloride analysis after the extraction of the rebar, mark the horizons to be powdered every 0.25 inch from the top surface. Also mark the horizon of 5 to 5.25 inches for background chloride information.

Extraction of Epoxy-Coated Rebar from the Core

1. Dry saw cut a 0.75-inch-deep groove in two diagonally opposing quadrants (Figure 12).
2. Carefully break the core into two pieces along the sawcuts using a chisel and hammer so as not to damage the epoxy-coated rebars.
3. Carefully remove the epoxy-coated bars from their traces.

Bar Selection and Preparation

1. Evaluate no more than two bars from each core.
2. Select the bars to be tested using the following criteria:
   - If more than two layers of bars are found in a core, only the top two layers shall be selected for evaluation.
   - If a layer contains more than one bar, one bar shall be randomly selected for evaluation.
   - Any bars that are not completely extracted (entire cross-section) shall not be tested.
3. With a black permanent marker, paint the top half of one end of each selected bar to designate its orientation in the core.
4. Drill and tap the other end (i.e., the bottom end) of each selected bar. Install a screw and attach a prelabeled tag. When there is more than one bar extracted from the core, the top bar in the mat and the bottom bar should be identified as such.

Visual Inspection of Epoxy-Coated Bars

1. Dry the epoxy-coated rebar if necessary by wiping with lint-free, salt-free, absorbent tissues.
2. Record bar size, bar length, deformation pattern, and coating color for each epoxy-coated rebar.
3. Visually observe the condition of the coating and the reinforcing steel, and rate according to Table 7. If the rating of

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[1] The height of a core may vary from one location to another around the perimeter of the core if both the upper and lower surfaces are not smooth. The maximum core height should be recorded as the maximum height that contains the entire cross-section of the core.

[2] The concrete cover depth is to be measured from both ends of the topmost bar.

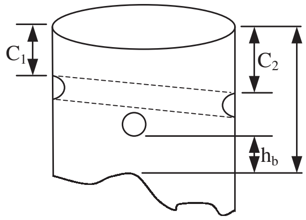

Figure 11. Core information.

the bar is equal to or greater than 4, no further testing needs to be performed.

4. Record the number of mashed areas, bare areas, and blisters for each rebar. Include comments for each rebar as necessary.

Holiday Detection

1. Attach the ground wire of the holiday detector to the screw that was previously installed on the rebar.
2. Perform the holiday detection test on all surfaces of each bar in accordance with ASTM 62, “Standard Test Method for Holiday Detection in Pipeline Coatings”—Method A.
3. When recording the number of holidays, distinguish between holidays located in bar ID areas from those not located in bar ID areas.[3]

Coating Adhesion

1. Measure coating adhesion in accordance with NACE TM0185—Section 5.3.2, “Knife Adhesion Testing,” with three modifications:
   a. The sample will not be heated in an autoclave.
   b. Use an “X” shaped groove in place of the “V” groove described.
   c. Use a modified rating scale. (See Step 5.)
2. Conduct wet adhesion test in three undamaged areas.[4]
3. If damaged coating areas are identified on the bar, conduct up to three additional wet adhesion tests adjacent to the damaged areas.
4. Record visual observations of the steel beneath the coating.
5. Rate all adhesion test areas according to Table 8.

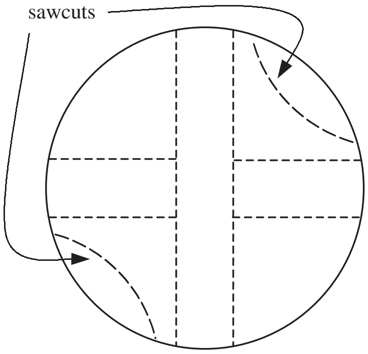

Figure 12. Sawcut for extraction of bars.

Preparation of Chloride Cores

1. Wear latex gloves to prevent contaminating the core and samples while handling them.
2. Remove the core from protective bag and dry brush the core to remove any dried coring residue attached to the surface.
3. Starting at the top surface, mark 0.25-inch horizons along the length of the core for as many samples as required. A minimum of six horizons must be collected. Mark another horizon between 5 and 5.25 inches for use in determining the background chloride content.

Table 7. Ratings for epoxy-coated rebars.

Rating	Description
1	No evidence of corrosion. 
2	A number of small, countable corrosion spots. 
3	Corrosion area less than 20% of total epoxy-coated rebar surface area.
4	Corrosion area between 20% and 60% of total epoxy-coated rebar surface area. 
5	Corrosion area greater than 60% of total epoxy-coated rebar surface area.

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[3] During holiday detection, bare areas will also produce a beep. Areas previously categorized as bare areas will not be counted as holidays. If a holiday is detected over a mashed area, the mashed area will be recategorized as a bare area.

If a continuous beep is detected along a bar, then the number of holidays will be considered equal to the length of bar along the test trace divided by the width of sponge.

[4] The knife adhesion test conducted without drying in a desiccator is termed the “wet adhesion test.” This test measures coating adhesion in an as-is condition.

The wet adhesion test must be completed within 24 hours from the time the rebars are extracted from the core.

Table 8. Ratings for coating adhesion.

Rating	Description
1	Well-adhered coating that cannot be peeled or lifted from the substrate steel. 
3	Coating that can be pried from the substrate steel in small pieces, but cannot be peeled off easily. 
5	Coating that can be peeled from the substrate steel easily, without residue. 

Setting the Chloride Core

1. There are two ways in which powdered concrete samples from the identified horizons can be obtained.
   - If a grinding wheel is used (Figure 13), place the core vertically into the vise attached to the drill press. To prevent the core from breaking while grinding, it is recommended that the core be placed inside a 3-inch-long PVC pipe that has been split down the sides before the core is inserted into the vise.
   - If a drill bit is used (Figure 14), place the core horizontally into the vise attached to the drill press. To help to prevent the core from breaking while grinding, it is recommended that the core be placed inside a 3-inch-long PVC pipe that has been split down the sides before the core is inserted into the vise.

Figure 13. Grinding wheel setup.

Figure 14. Drill bit setup.

2. Label a single sample bag with the core number, the location of the core, and the horizon to prevent accidental use of a bag for a different horizon.
3. Wrap a section of aluminum foil around the core below the area to be sampled to catch the powder as it falls.
4. If the drill press used to powder the cores is a variable-speed model, set it at the slowest speed to help contain the powder being created.

Powdering the Chloride Core

1. Wear a dust mask to avoid breathing in the dust generated by the powdering process.
2. There are two ways in which chloride cores can be drilled:
   - If a grinding wheel is used (Figure 15), powder the core down to the desired horizon. Depending on the width of the cutting surface of the grinding disk, several passes along the surface of the core may be required to powder the entire horizon.
   - If a drill bit is used (Figure 16), set the core so that the drill bit is centered on the median point of the horizon to be powdered. Drill about halfway into the core, then back the bit out and rotate the core so you can make another pass into it. Leave approximately one-eighth inch between bore holes to help prevent the core from breaking. Continue this process until the core has been rotated 360 degrees. Caution: when using this method to powder cores, it is not uncommon for the core to break into many pieces. When marking the horizons, mark the entire circumference and label the horizon

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Figure 15. Grinding wheel operation.

number (or nominal depth) in several areas so that the core can reassembled if necessary.

3. Carefully remove the foil containing the powdered sample and gently pour the sample into the prelabeled bag. Discard the foil to prevent contaminating the next sample.
4. Vacuum the drill press or grinding wheel to remove any dust to prevent contamination of the next sample.
5. Test the chloride samples collected according to AASHTO T 260-94, "Sampling and Testing for Total Chloride Ion in Concrete Raw Materials," Procedure A—Total Ion or ASTM C 1152, "Standard Test Method for Acid-Soluble Chloride in Mortar and Concrete."
6. Suggested Equipment:
   a. Drill press with an attached vise.
   b. Grinding wheel or 0.25-inch carbide drill bit.
   c. PVC pipe of appropriate diameter.
   d. Aluminum foil.
   e. Dust mask.
   f. Sealable plastic bags.
   g. Pen.
   h. Brush.
   i. Vacuum.
   j. Latex gloves.

Figure 16. Drill bit operation.

Service Life Model Procedure

Preparing Macros for Use in Microsoft Excel

1. The name of the file containing the macros for service life modeling and SI is "Service Life Model 2.0A."
2. The macros in this file require Microsoft Excel 2003 or a later version. If the file is opened in a version of Excel lower than 2003, one of the macros that controls both the processing of core data and the calculation of diffusion coefficient will not work.
3. Open the file and on the Tools menu click Add-Ins.
4. Make sure that a checkmark is present next to Analysis ToolPak and Analysis ToolPak – VBA. These add-ins are provided with Excel. If they are not checked, click on the box to their left and click OK.

Bridge Information Page

1. Click on the Bridge tab at the bottom and open the Bridge sheet.
2. Enter all information on the bridge page in the appropriate fields (Figure 17). Enter dates in mm/dd/yyyy format, including the year of construction.

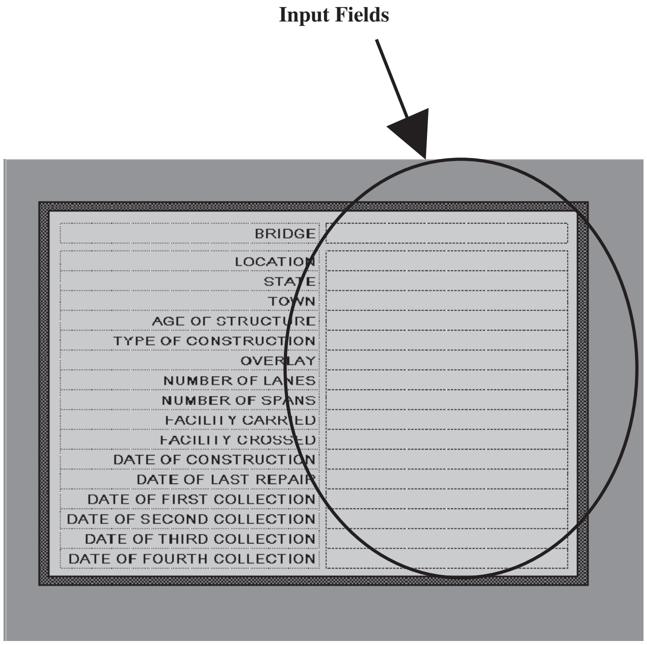

Figure 17. Bridge information page.

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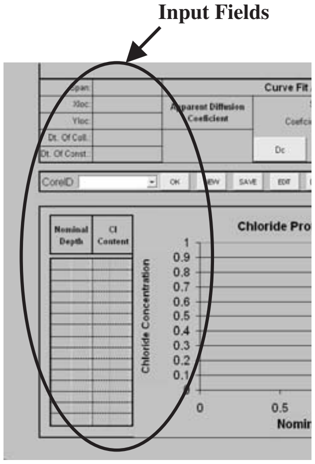

Figure 18. Core processing page.

3. If chloride profile samples were collected from the structure at various times (more than 6 months apart), include up to four dates of collection.
4. After all information is entered, click on the Core Processing tab and open that page.

Core Processing Page

1. To enter data for each chloride profile location, click on the New button. This will clear all information from the sheet.
2. Enter the core ID number or location label in the box following Core ID (Figure 18).
3. Click on OK. You will be asked if you want to save this core.
4. Enter Span #, X Location, and Y Location in upper left side of sheet (Figure 19).
5. Move the cursor over to the cell next to Dt. Of Coll: a drop-down menu will appear. Click on the drop-down menu arrow and select the date when the subject core was collected or the location was sampled from the deck. The Date of Construction has already been brought over from the Bridge Information page. The age of the sample at collection will show on the right top of the page after Age.
6. Enter the appropriate data under Nominal Depth and Cl Content (Figure 20). Then entered data will appear in the Chloride Profile – Dc Curve Fit graph to the right of the page. Nominal Depth and Cl Content can be entered in any units. The calculated Dc will be in the same units as the Nominal Depth data. The units of Dc are (unit of Nominal Depth)^2/year.
7. Click on Dc in the upper middle of the page (Figure 21). The Apparent Diffusion Coefficient, Sum of Squares (SS), and Coefficient of Correlation (R^2) will be calculated and will appear in the appropriate locations. The Best Fit Line will also be calculated and will appear in the Chloride Profile – Dc Curve Fit graph.
8. Click on Save to save all entered and calculated data.
9. Click on New to clear the page and enter the data for the next core or location.
10. To delete a core and its information, simply select the CoreID from the drop-down menu, click OK, and then click Delete.

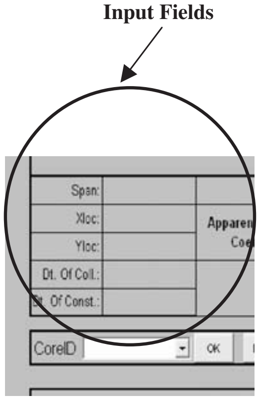

Figure 19. Span, location, and date of collection.

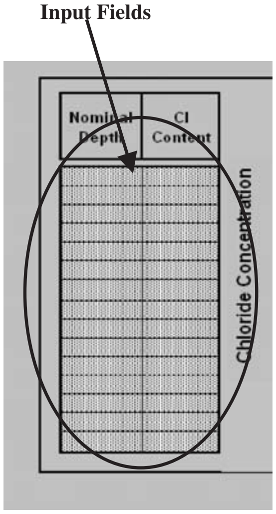

Figure 20. Nominal depth and Cl content.

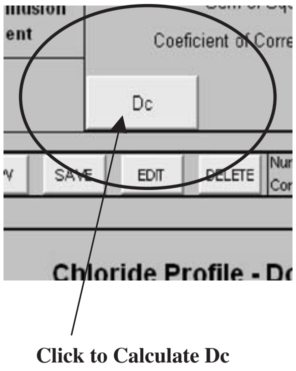

Figure 21. Chloride profile—Dc.

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Click to import Data.

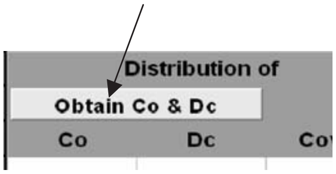


Figure 22. Distributions page.

11. To edit a core label, select the CoreID from the drop-down list, then click Edit. A form will open. Type in the new CoreID and click OK. Then click Save.
12. Repeat Steps 1 through 11 until all cores or sampling locations and their data have been entered.
13. Open the Distributions sheet by clicking on the Distributions tab.

Distributions Page

1. Click on Obtain Co & Dc (Figure 22). This will import that data from the Core Data Sheet. The Core Data Sheet is hidden.
2. Enter, in any order, all cover depth measurements in the Cover column (Figure 23).
3. Place the cursor into any cell that does not have a button. Click on Co (Figure 24). The statistics for Co will be calculated, and a plot showing the distribution of the data and a normal distribution curve for the data will be plotted.
4. Repeat Step 3 for Dc and Cover.
5. Open the Model Results page by clicking on the Model Results tab.

Input Cover

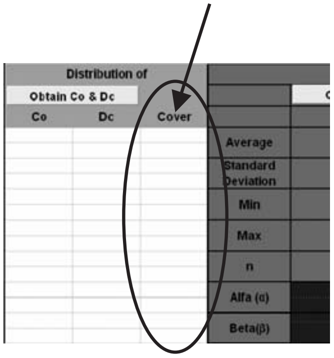


Figure 23. Cover depth measurements.

Click to calculate Co Statistics

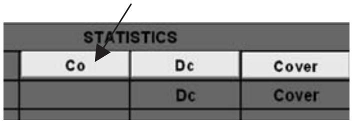
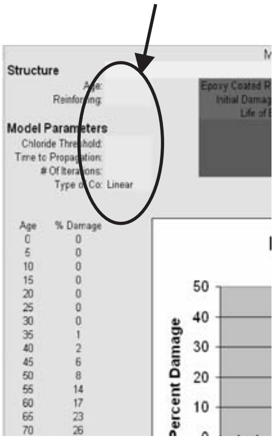


Figure 24. Statistics of Co.

Input Fields



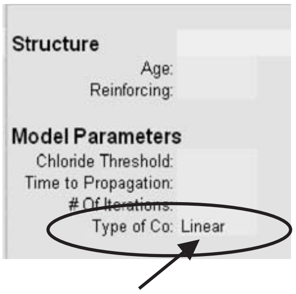
Type of Co: Linear

Age	% Damage
0	0
5	0
10	0
15	0
20	0
25	0
30	0
35	1
40	2
45	6
50	8
55	14
60	17
65	23
70	26

Figure 25. Model Results page.

Model Results Page

1. Enter the age of the samples in the Age field (Figure 25). If the chloride profiles were collected on different dates, use the weighted average age.
2. In the Reinforcing field, using the drop-down menu, select either Black or Epoxy.
3. Enter the required information in the Model Parameters section (Figure 26).
   a. Chloride Threshold: Start with 300 ppm or 1.2 pcy.
   b. Time to propagation: Use 3 to 6 years. Time to propagation can be adjusted based on average corrosion rate. Use corrosion rate device manufacturer's guidelines for time to damage.
   c. # Of Iterations: Use between 10,000 and 15,000.
   d. Type of Co: Use the drop-down menu to select either Constant or Linear. If constant is selected, the model will assume that the surface chloride ion concentration remained constant throughout the life of the structure and is equal to that measured at the time of sampling. If Linear is selected, the model assumes that at time = 0, Co = 0, and Co increased to the value at the time of sampling linearly.



Type of Co: Linear

Select Linear or Constant

Figure 26. Model Parameters section.

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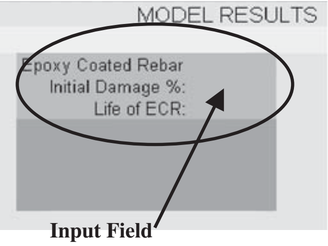


MODEL RESULTS

Epoxy Coated Rebar

Initial Damage %:

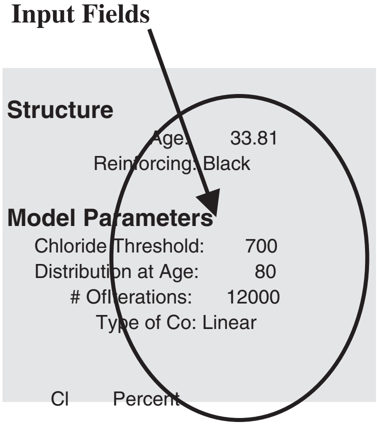
Life of ECR:

Figure 27. Epoxy-Coated Rebar section.

4. If Epoxy was selected in Reinforcing, then in the blue section labeled Epoxy Coated Rebar (Figure 27) fill in the following information:
   a. Initial Damage %: Estimated initial damage at the time of the installation of epoxy-coated rebar into concrete. This can be based on allowable damage by the specifications governing during the time of installation.
   b. Life of ECR: Based on the quantity of damage observed on the epoxy-coated rebars extracted in the cores, estimate the life of the epoxy coating assuming that the rate of degradation is linear. This can be calculated as follows:

$$\text{Life} = \frac{100 \times \text{Age}}{(\text{Present Damage} - \text{Initial Damage})}$$

5. Click on the Model Damage button to obtain the Percent Damage to Age chart and graph.
6. Check to see how the model prediction compares with the actual damage measured during the previous evaluations. If the predictions of the model are not close, vary the Chloride Threshold value and rerun the model. Repeat this step until the predicted values match reasonably well with the actual values.
7. Open the Cl Distribution page (Figure 28) by clicking on the Cl Distribution tab.



Structure

Age: 33.81

Reinforcing: Black

Model Parameters

Chloride Threshold: 700

Distribution at Age: 80

Of Iterations: 12000

Type of Co: Linear

Cl Percent

Figure 28. Cl Distribution page.

Cl Distribution Page

1. Under the Structure heading, adjacent to Age, fill in the average age that was used in Model Results.
2. The reinforcing type will not matter for this macro. Leave the default value to Black.
3. In the Model Parameters block, fill in the following:
   a. Chloride Threshold: Use the Chloride Threshold value from the Model Results page. (The predicted damage value had been determined to agree reasonably well with actual damage.)
   b. Distribution at Age: Enter the age at which the SI is desired.
   c. # Of Iterations: Use a value between 10,000 to 15,000.
   d. Type of Co: Use Constant or Linear, whichever was used in the Model Results.
4. Click on the Chloride Distribution button. Adjacent to SI, the calculated index is provided.
5. Save the File with an appropriate name.
6. If a message appears stating that the program cannot close the file because the file is linked to another file, click OK and continue closing the file.

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Abbreviations and acronyms used without definitions in TRB publications:

AASHO	American Association of State Highway Officials
AASHTO	American Association of State Highway and Transportation Officials
ACRP	Airport Cooperative Research Program
ADA	Americans with Disabilities Act
APTA	American Public Transportation Association
ASCE	American Society of Civil Engineers
ASME	American Society of Mechanical Engineers
ASTM	American Society for Testing and Materials
ATA	American Trucking Associations
CTAA	Community Transportation Association of America
CTBSSP	Commercial Truck and Bus Safety Synthesis Program
DHS	Department of Homeland Security
DOE	Department of Energy
EPA	Environmental Protection Agency
FAA	Federal Aviation Administration
FHWA	Federal Highway Administration
FMCSA	Federal Motor Carrier Safety Administration
FRA	Federal Railroad Administration
FTA	Federal Transit Administration
IEEE	Institute of Electrical and Electronics Engineers
ISTEA	Intermodal Surface Transportation Efficiency Act of 1991
ITE	Institute of Transportation Engineers
NASA	National Aeronautics and Space Administration
NCFRP	National Cooperative Freight Research Program
NCHRP	National Cooperative Highway Research Program
NHTSA	National Highway Traffic Safety Administration
NTSB	National Transportation Safety Board
SAE	Society of Automotive Engineers
SAFETEA-LU	Safe, Accountable, Flexible, Efficient Transportation Equity Act: A Legacy for Users (2005)
TCRP	Transit Cooperative Research Program
TEA-21	Transportation Equity Act for the 21st Century (1998)
TRB	Transportation Research Board
TSA	Transportation Security Administration
U.S.DOT	United States Department of Transportation

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