RESEARCH REPORT 1405-1

STATE-OF-THE-ART REPORT ABOUT DURABILITY OF POST-TENSIONED BRIDGE SUBSTRUCTURES

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16. Abstract

Durability design requires an understanding of the factors influencing durability and the measures necessary to improve durability of concrete structures. The objectives of this report are to:

1. Survey the condition of bridge substructures in Texas;
2. Provide background material on bridge substructure durability; and
3. Review durability research and field experience for post-tensioned bridges.

A condition survey of existing bridges in Texas was used to identify trends in exposure conditions and common durability problems. The forms of attack on durability for bridge substructures in Texas are reviewed. Basic theory for corrosion of steel in concrete is presented, including the effect of cracking. Corrosion protection measures for post-tensioned concrete are presented. Literature on sulfate attack, freeze-thaw damage, and alkali-aggregate reaction is summarized. Literature on the field performance of prestressed concrete bridges and relevant experimental studies of corrosion in prestressed concrete is included. Crack prediction methods for prestressed concrete members are presented.

This report is part of Project 0-1405, "Durability Design of Post-Tensioned Bridge Substructure Elements." The information in this report was used to develop the experimental programs described in Research Reports 1405-2 and 1405-3 and in the preparation of durability design guidelines in Report 1405-5.

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Research Report 1405-1

Research Project 0-1405

DURABILITY DESIGN OF POST-TENSIONED BRIDGE SUBSTRUCTURE ELEMENTS

conducted for the
Texas Department of Transportation

In cooperation with the
U.S. Department of Transportation
Federal Highway Administration

by the
CENTER FOR TRANSPORTATION RESEARCH
BUREAU OF ENGINEERING RESEARCH
THE UNIVERSITY OF TEXAS AT AUSTIN

October 1999

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

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DISCLAIMER

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Research Supervisors

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SUMMARY

The durability of structural concrete is a very broad subject area, about which many structural engineers have a limited knowledge. A lack of attention to durability has contributed to the poor condition of the civil infrastructure throughout the world. It is important to understand the factors influencing durability and the measures necessary to improve durability of concrete structures. The objectives of this report are:

1. To survey the condition of bridge substructure in Texas;
2. To provide background material concerning the subject of concrete bridge substructure durability; and
3. To review and summarize research and field experience related to the subject of post-tensioned bridge substructures.

The report begins with a condition survey of existing bridges in Texas. This survey was used to identify trends in exposure conditions and common durability problems throughout the state. The literature review portion of the report begins with a discussion of exposure conditions and the forms of attack on durability for bridge substructures in Texas. Basic theory for corrosion of steel in concrete is presented, and an in-depth review on the effect of concrete cracking on corrosion is included. The effect of cracking is of great interest to this project since post-tensioning may be used to control cracking, and the effect on corrosion could influence mixed reinforcement designs. A large summary of corrosion protection measures for post-tensioned concrete structures is presented. Relevant literature on the subjects of sulfate attack, freeze-thaw damage, and alkali-aggregate reaction is reviewed and presented. Literature about the field performance of prestressed concrete bridges is reviewed to provide insight on past and current problems experienced by post-tensioned bridges in service. A selected review of relevant experimental studies of corrosion in prestressed concrete is included. Lastly, crack prediction methods for structural concrete members are presented.

This report was prepared as part of Research Project 0-1405, “Durability Design of Post-Tensioned Bridge Substructure Elements.” The information contained in this report was used to develop the testing programs described in Research Reports 1405-2 and 1405-3. A substantial portion of the reviewed literature was also used in the preparation of durability design guidelines in Report 1405-5.

Chapter 1: Introduction

1.1 BACKGROUND

1.1.1 Bridge Substructure Durability

Durability is the ability of a structure to withstand various forms of attack from the environment. For bridge substructures, the most common concerns are corrosion of steel reinforcement, sulfate attack, freeze-thaw damage, and alkali-aggregate reactions. The last three are forms of attack on the concrete itself. Much research has been devoted to these subjects, and, for the most part, these problems have been solved for new structures. The aspect of most concern for post-tensioned substructures is reinforcement corrosion. The potential for corrosion of steel reinforcement in bridges is high in some areas of Texas. In the northern regions, bridges may be subjected to deicing chemicals leading to the severe corrosion damage shown in Figure 1.1(a). Along the Gulf Coast, the hot, humid saltwater environment can also produce severe corrosion damage, as shown in Figure 1.1(b).

(a) Deicing Chemical Exposure
"Attack from Above"

(b) Coastal Saltwater Exposure
"Attack from Below"

Figure 1.1 - Typical Corrosion Damage in Texas Bridge Substructures

In 1998, the American Society of Civil Engineers (ASCE) produced a "report card" for America's infrastructure, as shown in Figure 1.2. Bridges faired better than most other areas of the infrastructure, receiving a grade of C-minus. However, a grade of C-minus is on the verge of being poor, and the ASCE comments that accompanied the grade indicated that nearly one third of all bridges are structurally deficient or functionally obsolete. What these statistics mean is that there are many bridges that need to be either repaired or replaced. These statistics also mean that more attention should be given to durability in the design process, since a lack of durability is one of the biggest contributors to the poor condition of the infrastructure.

American Society of Civil Engineers

1998 Report Card for America's Infrastructure

Subject	Grade
Roads	D-
Bridges	C-
Mass Transit	C
Aviation	C-
Schools	F
Drinking Water	D
Wastewater	D+
Solid Waste	C-

"Nearly 1 of every 3 (31.4%) bridges is rated structurally deficient or functionally obsolete.

It will require $80 billion to eliminate the current backlog of deficiencies and maintain repair levels."

Figure 1.2 - ASCE Evaluation of Infrastructure Condition

Larosche[1] performed an analysis of bridge substructure condition in Texas using the TxDOT Bridge Inventory, Inspection and Appraisal System (BRINSAP) as part of Project 0-1405. The BRINSAP system contains bridge condition rating information in a computer database of more than 30,000 bridges. The analysis of BRINSAP data indicated that more than ten percent of bridges in some districts of Texas had deficient substructures. The data also indicated that the substructure condition is controlling the service life of the bridge in many cases. The overall conclusion of the BRINSAP data analysis is that more attention should be given to the durability of bridge substructures. The analysis of BRINSAP data is the focus of Chapter 2 in this report.

1.1.2 Post-Tensioning in Bridge Substructures

1.1.2.1 <u>Benefits of Post-Tensioning</u>

Post-tensioning has been widely used in bridge superstructures, but has seen only limited applications in bridge substructures. There are many possible situations where post-tensioning can be used in bridge substructures to provide structural and economical benefits. Some possible benefits of post-tensioning are listed in Table 1.1.

Although pretensioning or post-tensioning is normally chosen for structural or construction reasons, many of the same factors can improve durability. For example, reduced cracking and crack widths offers the potential for improving the corrosion protection provided by the concrete. Reduced reinforcement congestion and continuity of reinforcement means that it is easier to place and compact the concrete with less opportunity for voids in the concrete. Post-tensioning is often used in conjunction with precasting. Precast concrete offers improved quality control, concrete quality and curing conditions, all leading to improved corrosion protection. Bonded post-tensioning also provides the opportunity for multiple levels of corrosion protection for the prestressing tendon, as shown in Figure 1.3. Protection measures include surface treatments on the concrete, the concrete itself, the duct, the grout and strand or bar coatings such as epoxy or galvanizing. Post-tensioning also provides the opportunity to electrically isolate the prestressing system from the rest of the structure.

Table 1.1 - Possible Benefits of Post-Tensioning

Benefit	Structural Behavior	Construction	Durability
Control of Deflections	✓		
Increased Stiffness	✓		
Improved Crack Control (higher cracking moment, fewer cracks, smaller crack widths)	✓		✓
Reduced Reinforcement Congestion	✓	✓	✓
Continuity of Reinforcement	✓		✓
Efficient utilization of high strength steel and concrete	✓		✓
Quick, efficient joining of precast elements	✓	✓	✓
Continuity between existing components and additions	✓	✓	✓

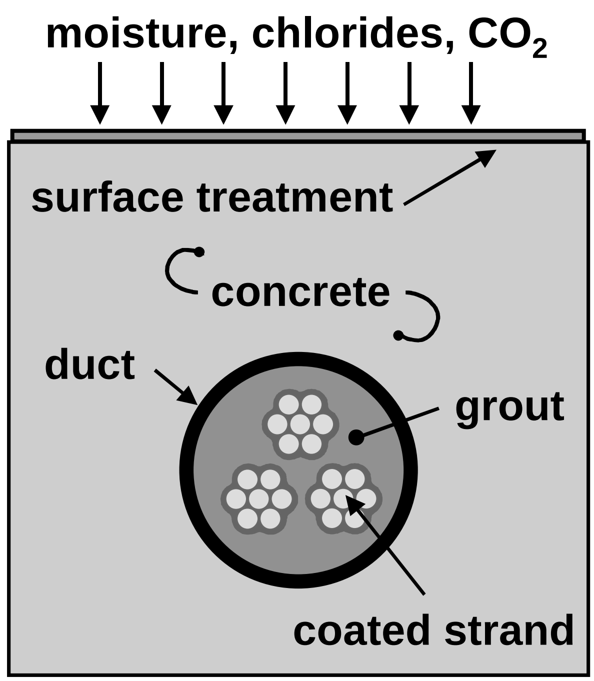
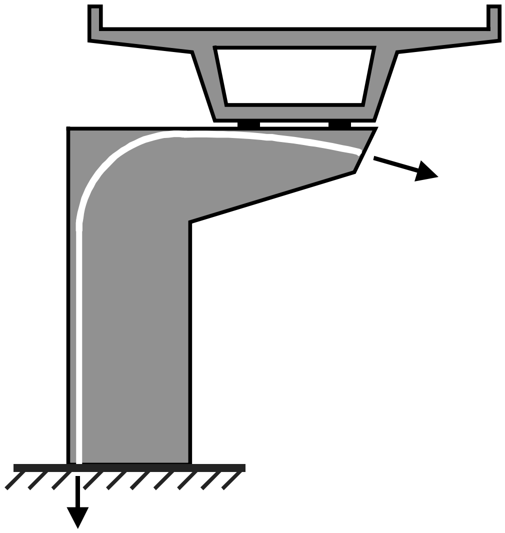
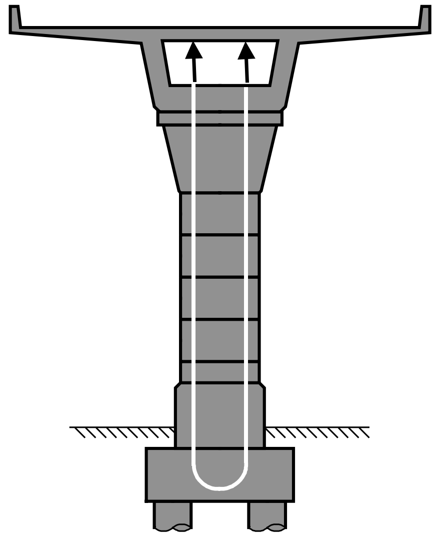


Figure 1.3 - Multilevel Corrosion Protection for Bonded Post-Tensioning Tendons

1.1.2.2 Bridge Substructure Post-Tensioning Applications

Post-tensioning has been used successfully in many bridge substructures. The possible applications for post-tensioning are only limited by the imagination of the designer. Several substructure post-tensioning applications are shown in Figure 1.4(a) through (h).



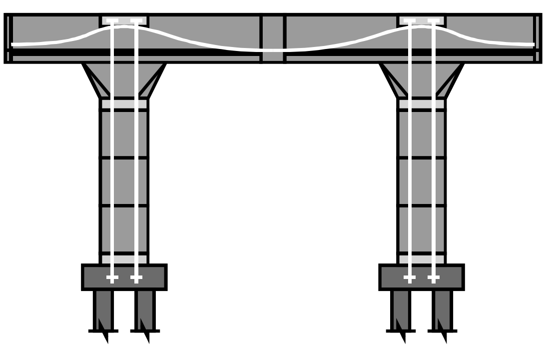


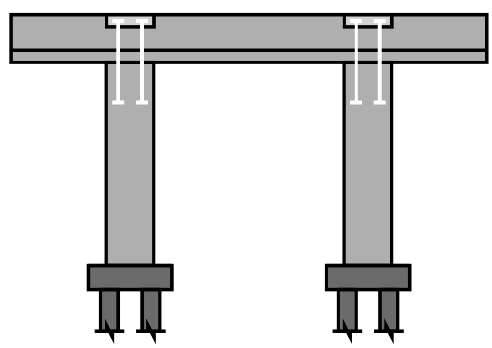
(a) Cantilever Substructure

Post-tensioning provides continuous reinforcement from the cantilever to the foundation. Deflection control and crack control are improved. Heavy reinforcement congestion in the joint region of the column is reduced.

(b) Precast Segmental Hollow Pier

Post-tensioning provides continuous reinforcement in the substructure. Temporary post-tensioning is used during construction for structural integrity.





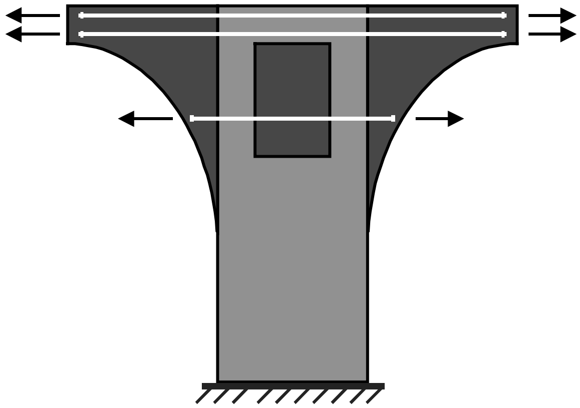
(c) Precast Frame Bent

Post-tensioning provides continuity of reinforcement and structural integrity for this entirely precast substructure. Construction proceeds rapidly, minimizing traffic interruption.

(d) Precast Bent Cap Post-Tensioned to Cast-in-Place Columns

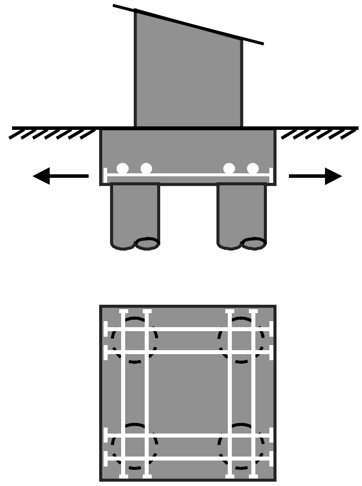
Post-tensioning provides continuity between precast and cast-in-place components. Erection is rapid, minimizing traffic interruption.

Figure 1.4 - Applications of Post-Tensioning in Bridge Substructures



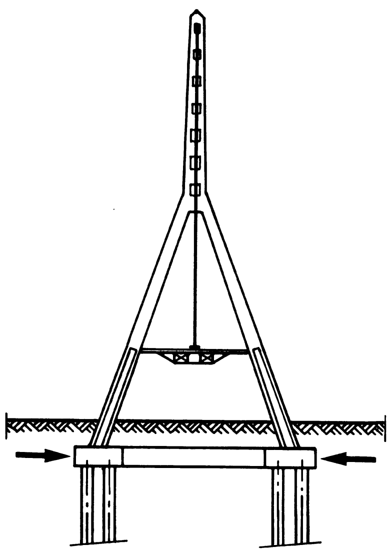
(e) Widening of Existing Substructure

Cantilever overhangs are added to allow widening of the bridge. Post-tensioning is used to provide continuous reinforcement and to improve shear transfer between the overhangs and existing substructure.



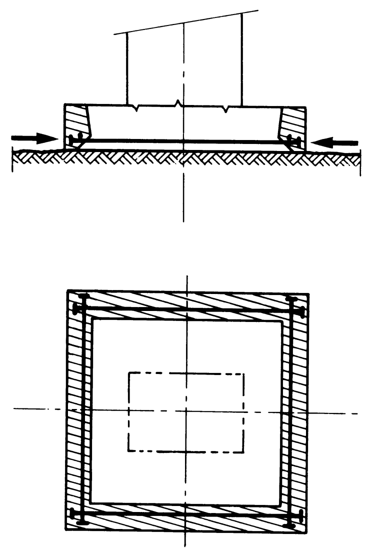
(f) Pile Cap

Post-tensioning is used to reduce the necessary size of the pile cap and the required steel area. The concentrated application of the post-tensioning anchorage forces is well suited to strut and tie methods of design for this element.



(g) Tie Beam[2]

High strength prestressing steel used for post-tensioning provides the necessary reinforcement for the large tension forces in the tie beam.



(h) Strengthening of Existing Footing[2]

Post-tensioning improves force transfer between existing and added concrete.

Figure 1.4 - Applications of Post-Tensioning in Bridge Substructures – Continued

1.1.3 Mixed Reinforcement in Structural Concrete

The recent development of the *AASHTO LRFD (Load and Resistance Factor Design) Bridge Design Specifications*[3] explicitly recognized the use of mixed reinforcement for the first time in American bridge and building codes. Mixed reinforcement, sometimes referred to as partial prestressing, describes structural concrete members with a combination of high strength prestressing steel and nonprestressed mild steel reinforcement. The relative amounts of prestressing steel and reinforcing bars may vary, and the level of prestress in the prestressing steel may be altered to suit specific design requirements. In most cases, members with mixed reinforcement are expected to crack under service load conditions (flexural cracks due to applied loading).

In the past, prestressed concrete elements have always been required to meet the classic definition of full prestressing where concrete stresses are kept within allowable limits and members are generally assumed to be uncracked at service load levels (no flexural cracks due to applied loading). The design requirements for prestressed concrete were distinctly separate from those for reinforced concrete (nonprestressed) members, and were located in different chapters or sections of the codes. The fully prestressed condition may not always lead to an optimum design. The limitation of concrete tensile stresses to below cracking can lead to large prestress requirements, resulting in very conservative designs, excessive creep deflections (camber), and the requirement for staged prestressing as construction progresses.

The use of varied amounts of prestressing in mixed reinforcement designs can offer several advantages over the traditional definitions of reinforced concrete and fully prestressed concrete:[4,5]

- Mixed reinforcement designs can be based on the strength limit state or nominal capacity of the member, leading to more efficient designs than allowable stress methods.
- The amount of prestressed reinforcement can be tailored for each design situation. Examples include determining the necessary amount of prestress to:
  - balance any desired load combination to zero deflections
  - increase the cracking moment to a desired value
  - control the number and width of cracks
- The reduced level of prestress (in comparison to full prestressing) leads to fewer creep and excessive camber problems.
- Reduced volume of steel in comparison to reinforced concrete designs.
- Reduced reinforcement congestion, better detailing, fewer reinforcement splices in comparison to reinforced concrete designs.
- Increased ductility in comparison to fully prestressed designs.

Mixed reinforcement can provide a desirable design alternative to reinforced concrete and fully prestressed designs in many types of structures, including bridge substructures. Recent research[6] at The University of Texas at Austin has illustrated the structural benefits of mixed reinforcement in large cantilever bridge substructures.

The opposition to mixed reinforcement designs and the reluctance to recognize mixed reinforcement in design codes has primarily been related to concerns for increased cracking and its effect on corrosion. Mixed reinforcement design will generally have more cracks than comparable fully prestressed designs. It has been proposed that the increased presence of cracking will lead to more severe corrosion related deterioration in a shorter period of time. Due to the widely accepted notion that prestressing steel is more susceptible to corrosion, and that the consequences of corrosion in prestressed elements are more severe than in reinforced concrete (see Section 4.3), many engineers have felt that the benefits of mixed reinforcement are outweighed by the increased corrosion risk. Little or no research has been performed to assess the effect of mixed reinforcement designs on corrosion in comparison to conventional reinforced concrete and fully prestressed designs.

1.2 Research Project 0-1405

The issues described in the preceding sections prompted the development of Project 0-1405, “Durability Design of Post-Tensioned Bridge Substructure Elements,” at the Center for Transportation Research at The University of Texas at Austin. The research is sponsored by the Texas Department of Transportation and the Federal Highway Administration, and was performed at the Phil M. Ferguson Structural Engineering Laboratory. The title of Project 0-1405 implies two main components to the research:

1. Durability of Bridge Substructures, and
2. Post-Tensioned Bridge Substructures.

The durability aspect is in response to the deteriorating condition of bridge substructures in some areas of Texas. Considerable research and design effort has been given to bridge deck design to prevent corrosion damage, while substructures have been largely overlooked. In some districts of the state, more than ten percent of the substructures are deficient, and the substructure condition is limiting the service life of the bridges.

The second aspect of the research is post-tensioned substructures. As described above, there are many possible applications in bridge substructures where post-tensioning can provide structural and economical benefits, and can possibly improve durability. Post-tensioning is now being used in Texas bridge substructures, and it is reasonable to expect the use of post-tensioning to increase in the future as precasting of substructure components becomes more prevalent and as foundation sizes increase.

Problem:

The problem that bridge engineers are faced with is that there are no durability design guidelines for post-tensioned concrete structures. Durability design guidelines should provide information on how to identify possible durability problems, how to improve durability using post-tensioning, and how to ensure that the post-tensioning system does not introduce new durability problems.

1.3 Research Objectives and Project Scope

1.3.1 Project Objectives

The overall research objectives for Project 0-1405 are as follows:

1. To examine the use of post-tensioning in bridge substructures,
2. To identify durability concerns for bridge substructures in Texas,
3. To identify existing technology to ensure durability or improve durability,
4. To develop experimental testing programs to evaluate protection measures for improving the durability of post-tensioned bridge substructures, and
5. To develop durability design guidelines and recommendations for post-tensioned bridge substructures.

A review of literature early in the project indicated that post-tensioning was being successfully used in past and present bridge substructure designs, and that suitable post-tensioning hardware was readily available. It was decided not to develop possible post-tensioned bridge substructure designs as part of the first objective for two reasons. First, other research[6,7,8] on post-tensioned substructures was already underway, and second, the durability issues warranted the full attention of Project 0-1405. The third objective was added after the project had begun. The initial literature review identified a substantial amount of relevant information that could be applied to the durability of post-tensioned bridge substructures. This thorough evaluation of existing literature allowed the scope of the experimental portion of the project to be narrowed. The final objective represents the culmination of the project. All of the research findings are to be compiled into the practical format of durability design guidelines.

1.3.2 Project Scope

The subject of durability is extremely broad, and as a result, so is the scope of Project 0-1405. Based on the project proposal and an initial review of relevant literature, the project scope and necessary work plan were defined. The scope of the research flows from the overall objective of developing durability design guidelines. The design guidelines must address two questions:

1. When is durability a concern?
2. How can durability be improved?

The project tasks related to these questions are illustrated in Figure 1.5 and Figure 1.6. The experimental work in the project involves the tasks listed in Figure 1.6.

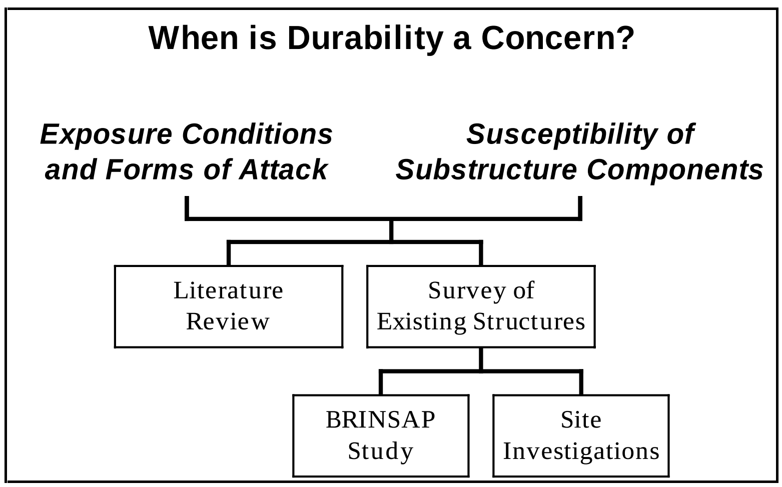


Figure 1.5 - Project Work Plan: Identifying Durability Concerns

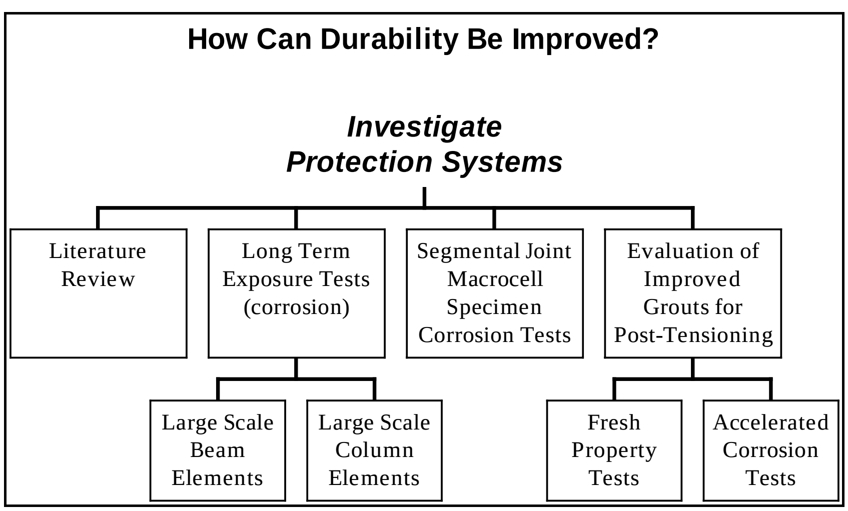


Figure 1.6 - Project Work Plan: Identifying Durability Protection Measures

In order to identify situations when durability is a concern for a bridge substructure, the exposure conditions and forms of attack at a particular bridge location must be known. Another important factor is the susceptibility of the various components of the substructure to attack. For example, certain forms of attack may be more of a concern to columns than bent caps, and vice versa. The research tasks in this portion of the project include a review of literature and a survey of existing structures. By examining the condition of existing structures, we can learn from past problems and successes. This portion of the research used bridge condition rating information (BRINSAP data) and site visits to identify trends in substructure durability problems throughout Texas.

The largest portion of Project 0-1405 is focused on the question of how can durability be improved for post-tensioned bridge substructures. This question is addressed by investigating protection systems using literature and experimental testing programs. The main research components include large-scale, long-term corrosion tests with beam and column elements, a small testing program investigating corrosion protection at the joints in precast segmental bridges, and the development of improved grouts for post-tensioning. A large amount of literature was found on the subject of concrete durability early in the project. Detailed information was available for sulfate attack, freeze-thaw damage and alkali-aggregate reaction. For this reason, it was decided to focus the experimental portion of the project on corrosion of reinforcement in post-tensioned concrete, as evident in Figure 1.6. The detailed literature on concrete durability could be used to develop durability design guidelines on those aspects.

1.4 PROJECT REPORTING

The project tasks described in the preceding section were performed by graduate research assistants B.D. Koester,[9] C.J. Larosche,[1] A.J. Schokker[10] and J.S. West,[11] under the supervision of Dr. J.E. Breen and Dr. M.E. Kreger. The segmental joint macrocell specimens were developed and constructed by R. P. Vignos[12] under TxDOT Project 0-1264. This testing program was transferred to Project 0-1405 in 1995 for long-term testing. Project 0-1405 is not complete, with the long-term beam and column exposure tests and the macrocell corrosion tests currently ongoing. The major tasks to be completed in the future include continued exposure testing and data collection, final autopsy of all beam, column and macrocell specimens and preparation of the final durability design guidelines.

The research performed during the first six years of Project 0-1405 is reported in a series of five reports. In all, nine reports are planned for Project 0-1405, with report numbers and titles as listed in Table 1.2. A brief description of the Reports 1405-1 through 1405-5 is provided below.

Table 1.2 - Project 0-1405 Report Titles and Expected Completion Dates

Number	Title	Estimated Completion
1405-1	State of the Art on Durability of Post-Tensioned Bridge Substructures	1999
1405-2	Development of High Performance Grouts for Bonded Post-Tensioned Structures	1999
1405-3	Long-term Post-Tensioned Beam and Column Exposure Test Specimens: Experimental Program	1999
1405-4	Corrosion Protection for Bonded Internal Tendons in Precast Segmental Construction	1999
1405-5	Interim Conclusions, Recommendations and Design Guidelines for Durability of Post-Tensioned Bridge Substructures	1999
1405-6	Final Evaluation of Corrosion Protection for Bonded Internal Tendons in Precast Segmental Construction	2002
1405-7	Design Guidelines for Corrosion Protection for Bonded Internal Tendons in Precast Segmental Construction	2002
1405-8	Long-term Post-Tensioned Beam and Column Exposure Test Specimens: Final Evaluation	2003
1405-9	Conclusions, Recommendations and Design Guidelines for Durability of Post-Tensioned Bridge Substructures	2003

Report 1405-1 (this document) provides a detailed background to the topic of durability design of post-tensioned bridge substructures. The report contains an extensive literature review on various aspects of the durability of post-tensioned bridge substructures and a detailed analysis of bridge substructure condition rating data in the State of Texas.

Report 1405-2 presents a detailed study of improved and high performance grouts for bonded post-tensioned structures. Three testing phases were employed in the testing program: fresh property tests, accelerated corrosion tests and large-scale pumping tests. The testing process followed a progression of the three phases. A large number of variables were first investigated for fresh properties. Suitable mixtures then proceeded to accelerated corrosion tests. Finally, the most promising mixtures from the first two phases were tested in the large-scale pumping tests. The variables investigated included water-cement ratio, superplasticizer, antibleed admixture, expanding admixture, corrosion inhibitor, silica fume and fly ash. Two optimized grouts were recommended depending on the particular post-tensioning application.

Report 1405-3 describes the development of two long-term, large-scale exposure testing programs, one with beam elements, and one with columns. A detailed discussion of the design of the test specimens and selection of variables is presented. Preliminary experimental data is presented and analyzed, including cracking behavior, chloride penetration, half-cell potential measurements and corrosion rate measurements. Preliminary conclusions are presented.

Report 1405-4 describes a series of macrocell corrosion specimens developed to examine corrosion protection for internal prestressing tendons in precast segmental bridges. The report briefly describes the test specimens and variables, and presents and discusses four and a half years of exposure test data. One-half (nineteen of thirty-eight) of the macrocell specimens were subjected to a forensic examination after four and a half years of testing. A detailed description of the autopsy process and findings is included. Conclusions based on the exposure testing and forensic examination are presented.

Report 1405-5 contains a summary of the conclusions and recommendations from the first four reports from Project 0-1405. The findings of the literature review and experimental work were used to develop preliminary durability design guidelines for post-tensioned bridge substructures. The durability design process is described, and guidance is provided for assessing the durability risk and for ensuring protection against freeze-thaw damage, sulfate attack and corrosion of steel reinforcement. These guidelines will be refined and expanded in the future under Project 0-1405 as more experimental data becomes available.

1.5 REPORT 1405-1 - STATE-OF-THE-ART REPORT ON THE DURABILITY OF POST-TENSIONED BRIDGE SUBSTRUCTURES

The durability of structural concrete is a very broad subject area. Many different issues are involved, and a tremendous amount of research has been performed on many of these issues. Durability is also a subject about which many structural engineers have a limited knowledge since it is rarely addressed in structural engineering education. A lack of attention to structural durability has contributed to the poor condition of much of the civil infrastructure throughout the world. It is important to understand the factors influencing durability, and the measures necessary to improve durability of concrete structures. The purpose of this report is twofold:

1. To provide background material on the subject of concrete bridge substructure durability, and;
2. To review and summarize research and field experience related to the subject of post-tensioned bridge substructures.

The information contained in this report was used to develop the testing programs described in Research Reports 1405-2 and 1405-3. A substantial portion of the reviewed literature was also used in the preparation of durability design guidelines in Report 1405-5.

This report is not all inclusive on the subject of bridge substructure durability, choosing instead to focus on corrosion of steel reinforcement and concrete durability in terms of sulfate attack, freeze-thaw damage and alkali-aggregate activity. Because the subject of Project 1405 is the durability of post-tensioned bridge substructures, corrosion of steel reinforcement is emphasized since post-tensioning has the largest influence on this aspect of durability. The report begins with a condition survey of existing bridges in Texas. This survey was used to identify trends in bridge substructure durability throughout the state. The literature review portion of the report begins with a discussion of exposure conditions and the forms of attack on durability for bridge substructures in Texas. Basic theory for corrosion of steel in concrete is presented, and an in-depth review on the effect of concrete cracking on corrosion is included. The effect of cracking is of great interest to this project since post-tensioning may be used to control cracking, and the effect on corrosion could influence mixed reinforcement designs. A summary of corrosion protection measures for post-tensioned concrete structures is presented. Relevant literature on the subjects of sulfate attack, freeze-thaw damage and alkali-aggregate reaction was reviewed and presented in terms of exposure conditions, mechanism of attack, influencing factors and protection methods. Literature on the field performance of prestressed concrete bridges was reviewed to provide insight on the types of past and current problems experienced by post-tensioned bridges in service. A selected review of relevant experimental studies of corrosion in prestressed concrete is included. Lastly, crack prediction methods for structural concrete members are presented. The crack prediction methods were used in the design of the beam exposure test program and analysis of experimental results. The development of the experimental programs relied heavily on the reviewed literature. In particular, the effects of cracking on corrosion, field performance of prestressed bridges, and past prestressed concrete corrosion research were used to shape the beam exposure testing program.

This report is supplemented by two appendices:

- Appendix A - Crack Widths and Corrosion: Literature Review
- Appendix B – Field Performance of Prestressed Concrete Bridges: Literature Review

Chapter 2:
Condition Survey of Existing Bridges in Texas

The performance of existing bridges can provide valuable information on the durability of concrete structures. This chapter describes a survey of the condition of bridge substructures in Texas. The goal of the survey was to examine trends in substructure condition throughout the state. This information was used to aid in identifying sources of durability concerns as a function of geographical location and exposure. The survey was also used to identify aspects of substructure design that may be prone to durability problems, with the intention of learning from past problems and successes.

The Bridge Inspection and Appraisal Program (BRINSAP) in Texas is the current method by which TxDOT routinely inspects, manages and maintains each of the state's "on-system" and "off-system" bridges. As part of this ongoing inspection a complete database of all of the state's 33,640 "on-system" bridges are kept on computer files. This data can be reduced to pertinent aspects of the structure, such as substructures, and further reduced to determine the material composition of the substructure. Through this database, initial determinations regarding the condition of Texas bridges concrete substructures were made.

In this chapter, the number of distressed substructure conditions in Texas will be presented. These structures will be categorized by geographical areas in which the primary factor is corrosive attack. In addition, the factors which contribute to a high replacement value will be discussed.

2.1 THE APPRAISAL SYSTEM

Evidence for corrosive attack in Texas Bridge substructure components can be found in BRINSAP data. In 1978, the Federal Government "Code of Federal Regulations, 23 CFR 650 C" required that "each highway department shall include a bridge inspection organization capable of performing inspections, preparing reports, and determining ratings in accordance with the provisions of the *American Association of State Highway and Transportation Officials (AASHTO) Manual and Bridge Standards*." Of primary importance to this research are the BRINSAP program objectives of:

- Maintaining an up-to-date inventory that indicates condition of all bridges on public roadways.
- Determining the extent of minor deterioration requiring routine maintenance and repair work as the basis for planning bridge maintenance programs.
- Determining the extent of major deterioration requiring rehabilitation or replacement as the basis for planning bridge replacement and rehabilitation programs.

These program objectives and the database associated with these objectives are the cornerstone for assessing the current substructure performance in each of TxDOT's 25 districts.

The rating system that BRINSAP uses for appraising each bridge substructure is given in Table 2.1. The individual deficiencies in the various features are evaluated as to how they effect the safety and serviceability of the bridge as a whole. The intent of the appraisal rating is to compare the existing bridge to a newly built one that would meet the current standards for the particular highway system of which the bridge is a part.

Table 2.1 - The Rating Guide for BRINSAP Appraisal[13]

Rating	Description
9	Excellent condition
8	Very good condition-no problems noted
7	Good condition-some minor problems
6	Satisfactory condition-minor deterioration of structural elements (limited)
5	Fair condition-minor deterioration of structural elements (extensive)
4	Poor condition-deterioration significantly affects structural capacity
3	Serious condition-deterioration seriously affects structural capacity
2	Critical condition-bridge should be closed until repair
1	Failing condition-bridge closed but repairable
0	Failed condition-bridge closed and beyond repair
N	Not applicable

A rating of 5 is used to determine bridges which may be considered for repair or replacement. Of interest to this study are bridges which have a rating of 5 or below, where the prevailing factor in the deteriorated condition is suspected to be corrosion (the BRINSAP database does not distinguish between corrosion and other forms of deterioration). The appraisal rating of 5 was used as a baseline measure to establish the number of substructures in Texas with significant deterioration. This data was acquired through the BRINSAP database. The sample chosen is all of the on-system bridges in Texas. On-system is defined as any bridge on the State and Federal Highway System, State and/or Federal Systems including the following:

- Interstate Highways
- US Highways
- State Highways
- State Loops or Spurs
- Farm or Ranch to Market Roads
- Park Roads
- Recreation roads
- Metropolitan Highways (Federal-Aid Urban Systems)

2.2 OVERALL BRINSAP FINDINGS

Texas is an extremely large state with significant changes in geography, topography and more significantly, climate. To assess Texas bridge substructures with the aid of the BRINSAP data, a “sample” of on-system bridges with a substructure rating of 5 was selected from all of the on-system bridges in Texas. This data is presented in Figure 2.1, where the incidence of bridges with a substructure condition rating of 5 or lower is shown by TxDOT district.

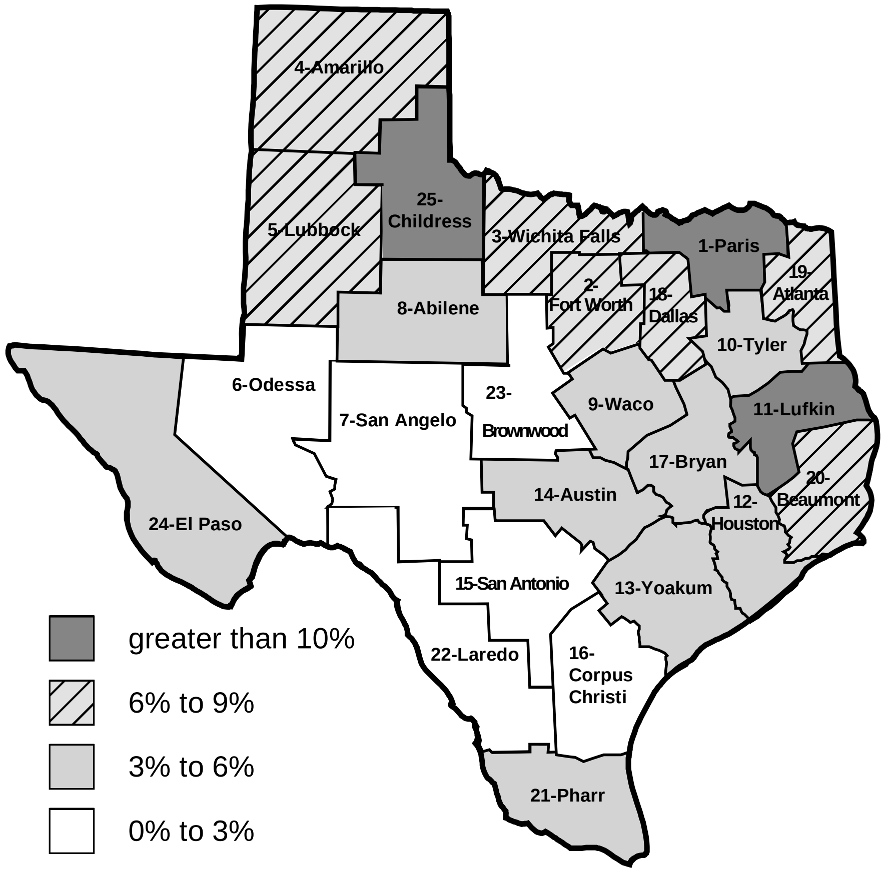


Figure 2.1 - Incidence of Deficient On-System Bridge Substructures in Texas

One aspect of interest in the BRINSAP data sample is the age of the structure. This statistic is reflective of the durability impact on the longevity of the structure. The age of the structure was also used to map the areas of low longevity. These areas were grouped in the study by district. The significance of the district areas with more or less longevity will be addressed further in the Field Study portion of this report. Table 2.2 lists other significant variables from this sample. The values shown are indicative of the state on-system Bridges with a substructure rating of 5 or below for the entire state.

Table 2.2 - Pertinent Variables for On-System Bridges with a Substructure Rating of 5 or Below

Variable	Statewide
Mean Age	42 years old
Median Age	41 years old
Mean ADT (average daily traffic)	11,000
Median ADT	3,100
Mean Number of Spans	8.7
% of Bridges where Substructure Controls Longevity	70%

The most revealing statistic from Table 2.2 above is the percentage of bridges where the longevity is controlled by the substructure. The fact that 70% of the bridges are deficient because of substructure problems shows the key importance of substructure durability. The incidence of bridges where the substructure limits the bridge service life is shown by district in Figure 2.2. If the substructure deteriorates to this replacement rating of 5, the entire structure must be replaced. The condition of the entire superstructure is put at risk because of substructure deterioration. In terms of replacement value the cost of infrastructure has now significantly increased. In rehabilitation work, several reinforced concrete decks have been replaced with the rest of the original structure intact. In fact, the bridge is generally widened at this point to accommodate an increase in traffic flow. However, a deteriorated substructure leaves the bridge designer no options except for complete replacement. As one would suspect the actual replacement versus rehabilitation cost for the State of Texas is very difficult to quantify and beyond the scope of this research. However, the fact remains that substructure deterioration in Texas among the structures in this sample is prevalent and the rehabilitation cost is very significant.

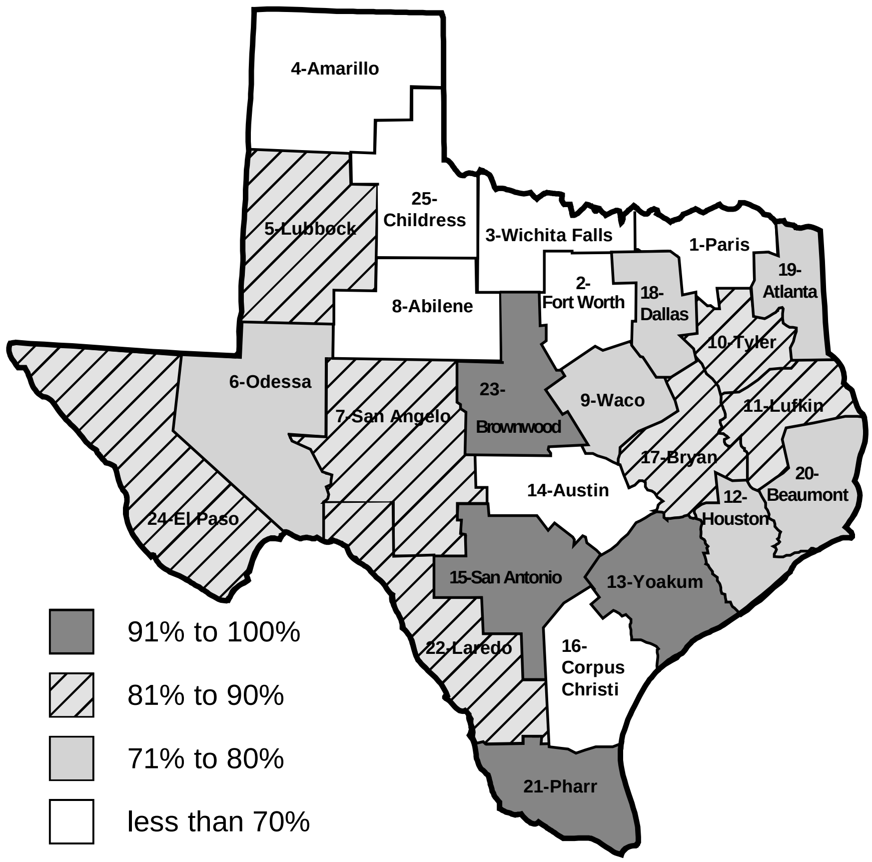


Figure 2.2 - Incidence of Bridges Where Substructure is Deficient but Superstructure Condtion is Satisfactory or Better

The mean age of bridges with deficient substructures is shown graphically in Figure 2.3. Figure 2.3 illustrates distinct regions and shows clear trends. The San Angelo district may be the exception as far as mean age. It is interesting to note the low median age of the bridges in the Texas Panhandle. Lubbock District has a mean age of 27.7 years and the Amarillo District is 30.6 years old. Currently, TxDOT is designing bridges for an expected service life of 75 years.[14] The FHWA deicing line is significant from the standpoint of dividing the geographic regions. There are 10 districts above the deicing line. Of those ten districts, eight have a mean age of less than 37.5 years or half of the required design service life.

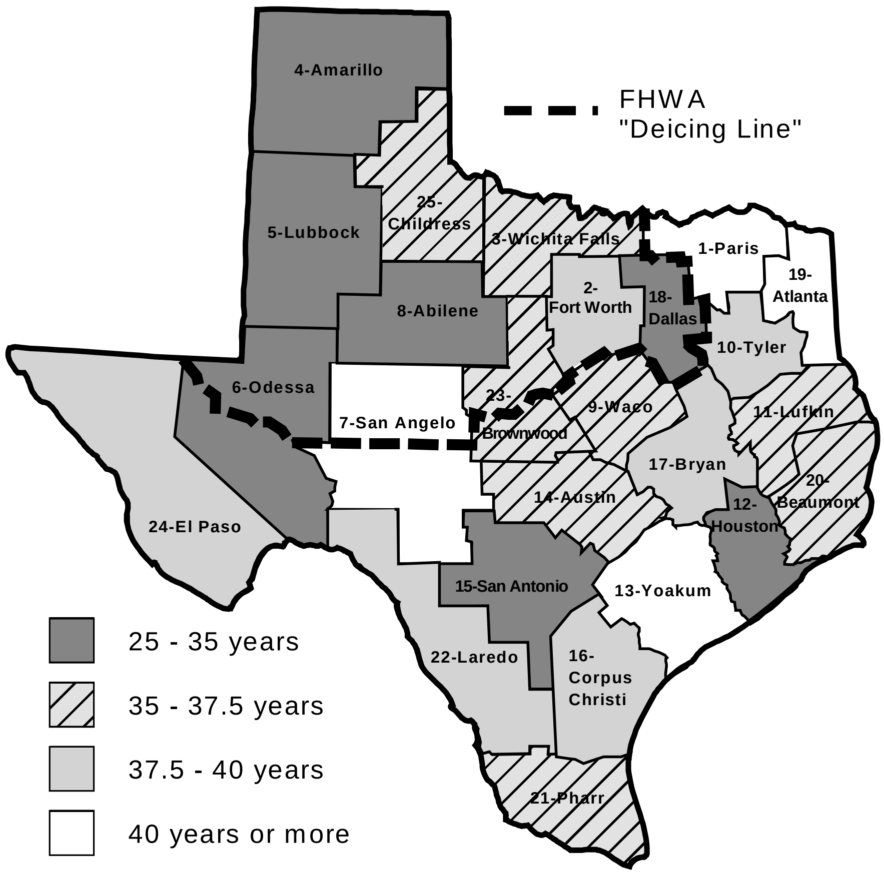


Figure 2.3 - The State of Texas, by District Depicting Mean Age of Deficient Bridge Structures

The sample size for the urban districts are large enough to be a representative sample. Houston, for instance, has 85 bridges with a substructure rating of 5 or below. The mean age of those 85 bridges is 30 years and this is considered an accurate number. Concern arises in some of the rural districts whose sample sizes are much lower. Lubbock is an example of a low sample size with a sample of 28 bridges representing the district.

A second problem arises in the actual data. The BRINSAP data does not distinguish between a durability problem versus some other type of substructure defect. An example of this effect could be a foundation problem. The bridge could experience settling problems where the damage is extensive and throughout. This type of structural defect was found to be quite rare but possible. A telephone survey of all 25 districts was conducted and the BRINSAP coordinators contacted. In general, these coordinators agreed that the possibility of a 5 rating with regard to substructures was in most cases a durability attack.

To further cloud the data, the initial data set collected from BRINSAP did not exclude bridges with a timber or steel substructure. This problem was corrected by excluding steel and timber substructures to insure only concrete substructures in the sample of bridges. Steel substructures which have deteriorated to the condition rating of 5 will have to be replaced. Therefore, the initial BRINSAP data runs still have some significance as this data suggests a corrosive environment and an appropriate substructure design should be considered when TxDOT replaces these structures.

2.3 THE GEOGRAPHIC REGIONS

The corrosive environments in The State of Texas are distinctive from region to region because of its vast geographic size. To facilitate the effectiveness of this report and ongoing research the state was divided into 4 regions, each with a similar environment. The previous section illustrates this similarity in west Texas. The mean age figure from the sample of distressed substructures depicts several districts above the deicing line with bridge longevity of approximately half of the design service life. The higher incidence of deterioration and shorter service life is in all likelihood due in large part to the chlorides introduced from the deicing salts. Because of this common link, the deicing line is used to define the geographic region West Texas. To the east of the West Texas region is an area of Texas that includes the districts Paris, Atlanta, Lufkin and Tyler. These districts comprise the region named the Northeast Region. This particular region has a higher durability risk from sulfate attack. The Central Texas Region is comprised of Bryan, Waco, Austin, San Antonio and El Paso. This region is named because these districts form a central band through Texas. The Central region has a low probability of corrosion and Figure 2.3 bears this fact out. The exception is the San Antonio District where the mean age is in the 25-35 year range. These districts results are discounted due to the data's small population. The actual number of deficient substructures in San Antonio is 25 bridges or 1% of the total number of on-system bridges. The Coastal Region, named because all of the districts have a gulf coast line, is the fourth region. The districts included are Beaumont, Houston, Yoakum, Corpus Christi and Pharr. Similar to West Texas, the Coastal Region suffers from severe chloride attack. Table 2.3 summarizes the respective districts in the associated regions.

Table 2.3 - Durability Regions and Their Respective Districts

Region	District	Region	District
West		**Northeast**	
	Odessa		Paris
	Lubbock		Atlanta
	Amarillo		Tyler
	Childress		Lufkin
	Abilene	**Central**	
	Wichita Falls		Bryan
	Fort Worth		Waco
	Dallas		Austin
Coastal			San Antonio
	Beaumont		San Angelo
	Houston		El Paso
	Yoakum		Brownwood
	Corpus Christi		Laredo
	Pharr		

One of the goals of this research project in defining the Texas Bridge durability problem was to establish trends. The BRINSAP data suggests a pattern of corrosion among several districts. The number of on-system bridges in Texas with a substructure rating of 5 or less is significant. TxDOT currently has 1,775 on-system bridges with a concrete substructure with a rating of 5 or less. The total number of on-system bridges in the state of Texas is 33,640 bridges. Thus, the sample of deteriorated concrete substructures represents 5.3 % of all the on-system bridges in Texas.

In Section 2.2 there was some concern expressed as to the relative sample size on a per district basis. Table 2.4 gives a complete numerical breakdown for the on-system structures in Texas. This table illustrates the relative percentages of bridges with a deteriorated concrete substructure. The average is 5.3% and the median value is 4%. These numbers are significant. Even the central Texas region districts have an average sample size of 28 deficient bridges per district, Brownwood and Laredo have a sample size below 25 bridges.

Table 2.4 - A Summary of BRINSAP Data

District		On-System Bridges	Deficient Substruct. (%)	Average Age	Average Daily Traffic	Average No. Spans	Substruct. Controls (%)
1	Paris	1317	17%	41	2,045	7	63%
2	Fort Worth	2088	7 %	39	17,693	7	64%
3	Wichita Falls	1034	8%	37	3,496	6	53%
4	Amarillo	724	9%	31	12,390	9	65%
5	Lubbock	439	6%	28	4,574	4	82%
6	Odessa	1039	2%	34	5,133	12	75%
7	San Angelo	1188	1%	45	1,626	10	80%
8	Abilene	1371	4%	35	3,672	7	57%
9	Waco	1608	3%	36	8,409	6	70%
10	Tyler	1152	4%	40	3,248	8	87%
11	Lufkin	779	14%	36	2,757	8	83%
12	Houston	2861	3%	31	32,773	11	74%
13	Yoakum	1630	3%	41	2,334	14	91%
14	Austin	1592	3%	37	6,437	8	63%
15	San Antonio	2542	1%	30	18,132	8	96%
16	Corpus Christi	1208	2%	38	2,222	11	69%
17	Bryan	1115	4%	39	3,472	10	82%
18	Dallas	3821	9%	34	24,406	10	73%
19	Atlanta	1087	7%	46	1,758	7	71%
20	Beaumont	1086	8%	37	14,671	11	74%
21	Laredo	782	1%	39	2,310	4	80%
22	Pharr	583	4%	36	5,834	11	96%
23	Brownwood	908	0.4%	35	1,193	5	100%
24	El Paso	981	3	38	7,756	6	84%
25	Childress	705	13%	37	742	9	37%

2.3.1 Replacement Cost

The replacement cost for bridges with substantial substructure deterioration can be very significant. For illustration purposes, an approximate replacement cost of $81,000.00 per span is assumed based on an average 2-lane bridge with an average span length of 80 feet.[15] Consider the Corpus Christi district where 2% of the on-system bridges have deficient substructures. The substructure condition is limiting the service life for 69% of these bridges. The cost to replace these bridges could be estimated as follows:

Number of bridges to replace because of substructure deterioration:

= 1208 x 0.02 x 0.69
= 17 bridges

The cost would be:

17 x 11(number of spans) x $ 81,000 / span = **<u>$15,147,000</u>**

This cost is based on replacing those bridges having a deteriorated substructure where the substructure is controlling the longevity of the bridge. In fact, the actual number of bridges to replace is the sample size of 24 bridges and this cost is approximately 21 million dollars. Furthermore, the structures have an average life of 38.4 years, which means the structure will have to be replaced 1.9 times in a 75-year design service life.

Figure 2.4 illustrates the average number of spans for bridges with a substructure rating of 5 or below. The Texas coast has several bridges with a significant number of spans due to the large number of saltwater bays and the inter-coastal waterway. This high number of spans, which adversely affects the replacement cost, illustrates the severity of the substructure durability problem in Texas.

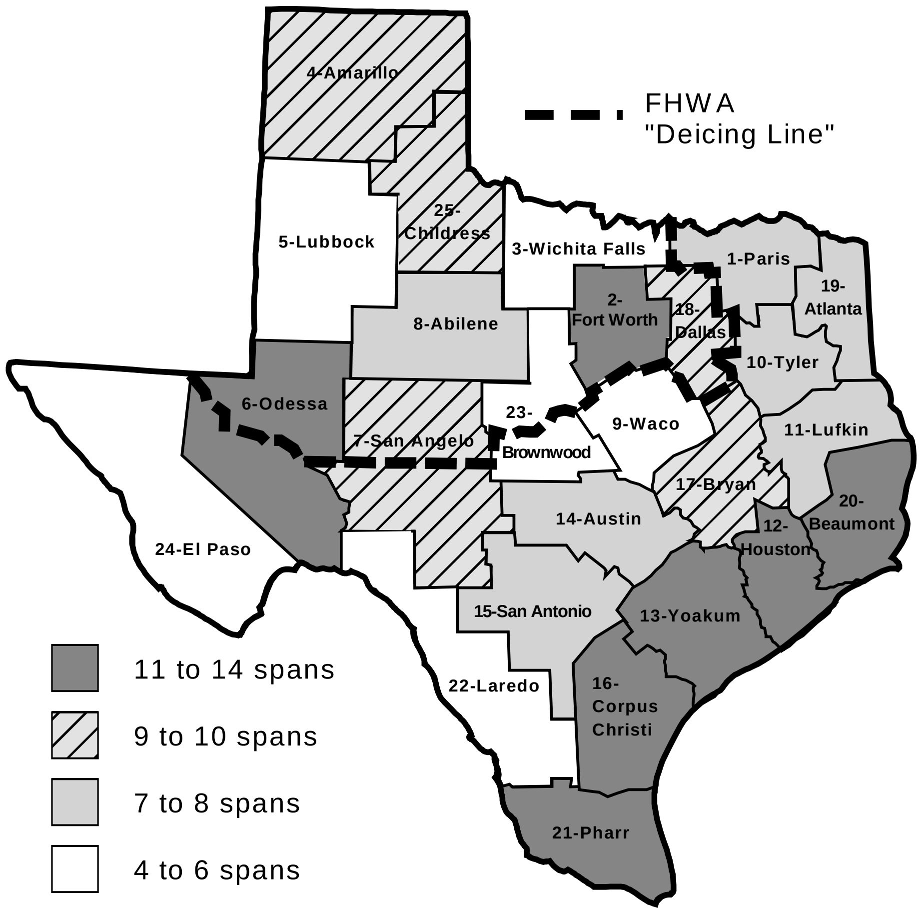


Figure 2.4 - Average Number of Spans/Bridge

To further illustrate the magnitude of the problem, consider the value of a highway by the number of vehicles that a particular highway or structure serves per day. This daily average of vehicles is referred to as "average daily traffic" or ADT. Significant traffic volume increases the replacement cost of a structure when the cost of traffic disruption is considered. Average daily traffic volumes for bridges with a substructure rating of 5 or below are listed in Table 2.4. This information is presented graphically in Figure 2.5. This figure illustrates the cost and complexity of bridge replacement along the Texas coast in terms of traffic concerns. In each of the coastal districts the average ADT exceeds 2000 vehicles per day. In three of the five coastal districts the ADT exceeds 4000 vehicles per day. The Houston district has the most difficult conditions for bridge replacement due to three significant statistics:

Low mean age of structures: 30.5 years

High average number of spans per bridge: 11

High average daily traffic volume: 32,773

The replacement cost for the deteriorated substructures in the Houston district using a 10% increase per 10,000 ADT to reflect traffic control costs and the assumptions given earlier is calculated as follows:

Cost = 1.3 x 11 spans x 85 bridges x $81,000 / span

= **$98,455,500**

Thus, approximately 100 million dollars is required to replace bridges with deteriorated substructures in the Houston district. In addition to this cost is the fact that the bridges in the sample have a average life of 30 years and therefore would have to be replaced twice in a normal 75 year design life. Table 2.5 is a sample of additional districts where the adverse combinations will certainly lead to high replacement costs. To further understand the economic impact, the entire State of Texas has a current bridge replacement cost due to substructure deterioration of $2,193,493,000. This figure is based on all of the multipliers and equations given above and the data in Table 2.4.

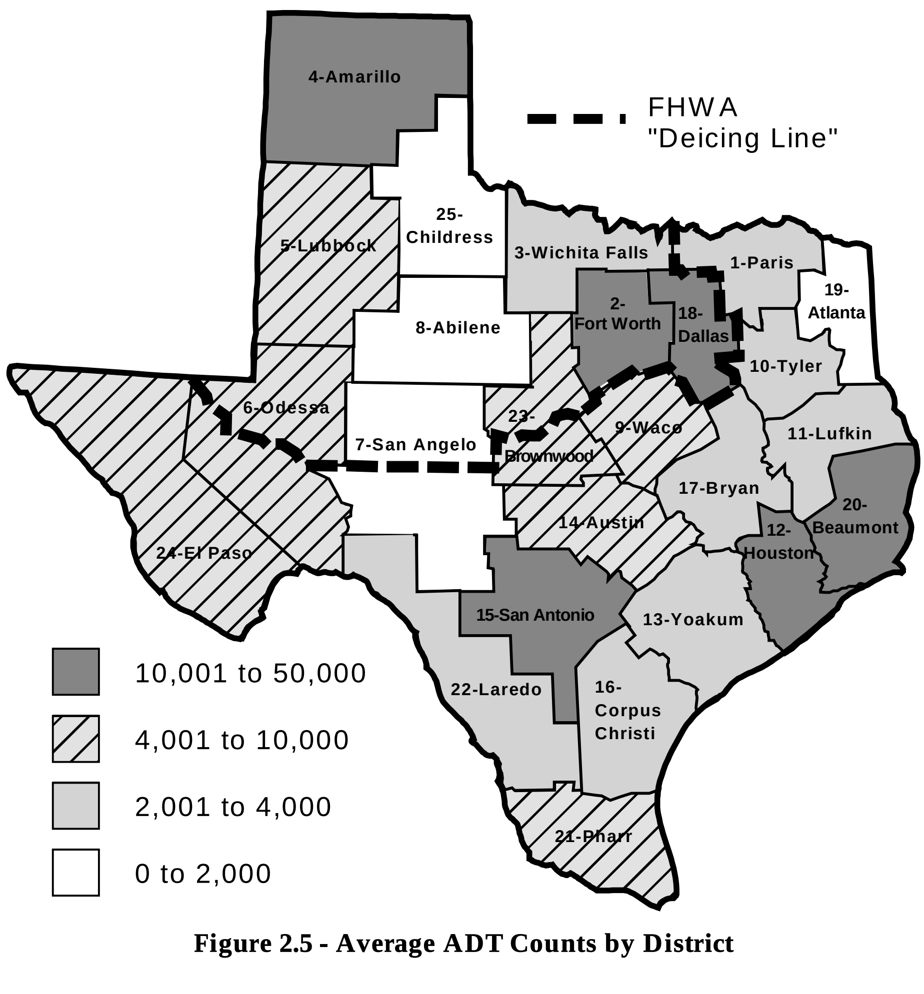


Figure 2.5 - Average ADT Counts by District

Table 2.5 - Additional Districts and Their Adverse Conditions

District	Detrimental Aspects
Amarillo (4)	Low mean age of structure (30.6) High ADT (12,390) High average number of spans (9)
Beaumont (20)	Low mean age of structure (36.9) High ADT (14,671) High average number of spans (11)
Dallas (18)	Low mean age of structure (34.4) High ADT (24,406) High average number of spans (10)
Odessa (6)	Low mean age of structure (34.4) Moderate ADT (5,133) High average number of spans (12)

2.4 FIELD TRIP INVESTIGATIONS

The primary purpose of dividing the state into four geographic regions was to reduce the amount of work that is required to adequately field review the districts. Once the four geographic regions were established, a representative district was selected from each region. The four geographic regions along with their respective districts are given in Table 2.6. The Amarillo, Corpus Christi and Austin districts were visited and a significant amount of data was acquired. Due to the time constraints of the project, the Paris district was not visited.

Table 2.6 – Representative Districts and Their Respective Regions

District	Geographic Region the District Represents
Amarillo	West Texas
Austin	Central Texas
Corpus Christi	Coastal Area of Texas
Paris	Northeast Texas

2.4.1 The Amarillo District

The Amarillo district represents the West Texas region. Bridges in this region can experience severe corrosive attacks from deicing salts. The district currently has the responsibility for 724 on-system bridges. Of the 724 on-system bridges in Amarillo, 114 of these structures have a substructure rating of 6, 32 structures have a substructure of 5 and 14 have a rating of 4 or below. A rating of 4 is a substructure in "poor condition where deterioration has significantly affected the structural capacity." The longevity of bridges with deficient substructures in the Amarillo district is a significant statistic, as illustrated by the data listed in Table 2.7.

Table 2.7 - Approximate Year Built for Bridges with Deficient Substructures in the Amarillo District

Number of Bridges	Age of Structure
8	built after 1980
9	built after 1970
81	built after 1960
62	earlier than 1960

The sample from the Amarillo district is very representative of the West Texas region. To take a micro view of the corrosion phenomena in this region a field visit was conducted for several structures. These structures were either visited or discussed in detail with the BRINSAP personnel of this district. A listing of the structures reviewed is given in Table 2.8.

Table 2.8 - Individual Projects Reviewed in the Amarillo District

Structure Number	Year Built	Substructure Rating	Bridge Type	Superstructure Rating	ADT
149	1984	5	Prestressed Conc. Girder	8	4750
1	1971	5	Precast Conc.	5	3900
26	1962	4	Pan	5	3650
2	1962	4	Steel I-Beam	6	5800
10	1932 / 1948*	4	Conc. Girder	4	3650
7	1957	4	Pan Girder	5	170
5	1977	6	Prestressed Conc. Girder.	8	4000
39	1966	4	Prestressed Conc. Girder	6	30700

The trends in the Amarillo district, which is representative of the western region, are significant. The portion of the substructure that is impacted the most is the area directly under an open joint. This area is exposed to chloride-laden moisture and is prone to several freeze thaw cycles over the course of a single winter. Generally, the top and sides of the cap will exhibit cracking and splitting along the top reinforcement, clearly indicative of corrosion. This form of damage is shown in Figure 2.6 for a bridge constructed in 1984. This photograph shows a bent cap with cracking parallel to the main reinforcement. The leaching of calcium carbonate above the bent cap on the bottom of the slab is indicative of a construction joint.

Figure 2.6 - Top and Side Splitting around Upper Reinforcement in Bent Cap (Amarillo)

As the structure begins to age, chlorides migrate to the lower portion of the cap and the entire cap becomes cracked and in need of replacement. Figure 2.7 shows a structure where the corrosion has become more pronounced. This structure was built in 1962, and was 34 years old at the time of the photo. Note the significant amount of damage from the chloride penetration in this structure and the resulting corrosion.

Figure 2.7 - Severe Deterioration of an Amarillo Bent Cap

Another item of interest is the substructure column deterioration observed in the Amarillo District. Chloride migration into the columns can occur from three principle methods:

- The water from the bridge deck runs through the open joint down onto the cap and then travels the length of the column as the water runs to the ground.
- Accumulation of plowed snow from the roadway is piled against the column below the structure.
- Several bridges have been constructed with the beams bearing directly on the columns. This detail leads to the same corrosive attack that was illustrated in the bent caps with the attack occurring on the top of the column.

Figure 2.8 illustrates the corrosive attack of single columns directly supporting a beam and under an open joint.

Figure 2.8 - Single Column Directly under a Construction Joint in Amarillo.

The adverse affects of snow accumulations pushed onto the bridge columns can be seen in Figure 2.9. The field studies in Amarillo offered significant insight to the transportation of chloride ions into the structural member. The structure shown in Figure 2.9 supports two overpass spans on Interstate 40. Interstate 40 has a high volume of traffic with an ADT count of 1850 cars per day. Table 2.9 illustrates the chloride samples taken from this substructure by a concrete powder test performed by TxDOT.

Figure 2.9 - Deterioration of Columns Due to Salt Laden Snow Piled against the Column

Table 2.9 - Chloride powder test on columns, Project 275-1-38 Amarillo (IH 40)

Sample	Depth	%Cl	PPM	Lbs./Cu. Yd.
1	0"-1"	.08	755	3.0
2	1"-2"	.11	1,058	4.2
3	0"-1"	.66	6,581	26.3
4	1"-2"	.33	3,270	13.1
5	0"-1"	.14	1,381	5.5
6	1"-2"	.14	1,440	5.8

Sample	Location	Span	Distance
1 & 2	Bent #2	not available	not available
3 & 4	Bent #3	Col. #3	2' from Top
5 & 6	Bent #3	Col. #3	4' from Ground

The most significant aspect from these tests is the concentration of the chloride ions some distance from the top of the column and the bottom of the column. Consider samples 5 & 6, located four feet from the ground. In this area the concentration is higher closer to the reinforcement and is concentrated above the height where salt laden snow would accumulate. This example can be found repeatedly in Amarillo. These observations suggest that there is water movement within the pore structure of the concrete or capillary action.

The durability attacks in the substructure members in several Amarillo bridges are significant. The most prevalent attacks occur under open joints between spans. However, there are also corrosion indications under and near construction joints. Finally, there is enough evidence to suggest capillary rise or "wicking" in bridge columns where salt laden snow has accumulated against the bridge columns.

2.4.2 *The Corpus Christi District*

The Corpus Christi District was selected as a representative sample of the coastal area. The primary area of concern is structures located immediately adjacent to the Gulf of Mexico. There are 25 on-system bridges in this district with a substructure rating of 5 or below. This sample of deficient bridges is approximately 2% of the total of on-system bridges in this district. This 2% is primarily on the coast.

The durability problems are similar in nature to the Amarillo District. The source of chlorides is primarily from wave action and ocean spray onto the bent caps. The horizontal splitting of the upper and lower reinforcement is evident in Figure 2.10.

Figure 2.10 - Horizontal Splitting of the Upper and Lower Reinforcement in a Typical Bent Cap

The coastal substructures suffer from column and piling durability problems as well. The columns have the same type of attack as the columns in the West Texas region, displaying similar forms of deterioration although the exposure conditions and source of chlorides are different. The columns typically exhibit concrete splitting in the longitudinal direction, as illustrated in Figure 2.11. The most severe corrosion damage in the coastal bridge substructures occurs above the water line where a fresh supply of oxygen is readily available. A question has been raised as to how much of the chloride penetration comes from tidal action and spray versus how much results from capillary rise in the pore structure of the concrete. Misra and Uomoto have documented the question of capillary rise in the pore structure of the concrete.[16] Misra and Uotomo state that "...in this case by capillary action, plays a vital role in the transport of chlorides within the concrete." This capillary action is a significant aspect considered in this research project in the development of the long-term column corrosion tests (see Report 1405-3).

Figure 2.11 - Face Splitting of a Bridge Column in Corpus Christi

2.4.3 The Austin District

The Austin District was selected as the representative district of the central region. Due in large part to the weather and the environment, very little corrosion occurs in these districts.

The Central Texas Area has very little corrosion among its substructures. The BRINSAP report shows 48 structures with a substructure rating of 5 or below. The actual number of structures with significant corrosive attack was 7. This number was determined through field investigation and subsequent meetings with Mr. Jeff Howell, P.E., the BRINSAP coordinator for this district. Of the seven bridges with significant substructure corrosion, only one structure was built after 1950. The low ratings for the remaining bridges stem from foundation settlement, impact damage and bridge scour.

2.5 CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE BRINSAP STUDIES

The rudimentary statistics required to examine basic trends in substructure condition in TxDOT bridges are presented in this chapter. This data indicates that the durability of bridge substructures in Texas has been and can continue to be a significant problem. The analysis of BRINSAP data suggests that more attention should be given to the durability of bridge substructures in the design process. However, the statistics presented in this chapter are primarily for bridges that have been in existence for some time. The poor condition of many bridges may result from the lack of attention to durability at the time of construction. Advances have been made in recent years to improve substructure durability, including increased concrete cover, enhanced concrete quality, and the use of epoxy-coated reinforcement. However, the effectiveness of these changes is uncertain due to the short length of service, and a potential for further improvement exists.

A more complete indication of trends in Texas bridge substructure condition could be obtained by adding information on the extent, severity and cause of deterioration to the BRINSAP database. Other desirable information would be a category to subdivide concrete substructures into cast-in-place, precast, pretensioned or post-tensioned. Another factor, alluded to in the preceding paragraph, is that throughout the state different districts have used various corrosion prevention techniques. These techniques could be incorporated into the BRINSAP database to provide a comparison between structures built in the same geographical region with a similar environment.

Certainly, the ability of the current BRINSAP program is such that all of these recommendations could be adopted with little effort. As technology in bridge and bridge substructure construction changes, the tools for recording the condition and deterioration of bridges needs to be refined to reflect and track the performance of this technology. This effort will help close the gap between the design service life and the actual service life of the substructures in Texas.

Chapter 3:
Bridge Substructure Durability Exposure Conditions

Durability is the ability of a structure to withstand various forms of attack during its service life, including weathering, chemical attack, corrosion, abrasion or any other form of deterioration. In concrete bridge substructures, the most common forms of environmental attack are corrosion of reinforcement, sulfate attack on concrete, freezing and thawing damage and alkali-aggregate reactions. In order for a deterioration process to occur, interactions must occur between the materials of the structure and the environment. These interactions depend on many factors, including the material properties, structural form, design details, construction quality, concrete curing and type and severity of the environment. The environment refers to both the general atmospheric climate around the structure and the localized conditions around different elements of the structure.

The design requirements for the durability limit state are primarily dictated by the environmental conditions. In general, the severity of a given environment is influenced by the availability of moisture, the presence of aggressive agents in the moisture and temperature. There are three general environments where substructure durability may be a concern: coastal exposure, freezing exposure and aggressive soils. These exposures may occur singly or in combination.

3.1 Coastal Exposure

Coastal exposures are one of the most severe environments for concrete structures. This danger is particularly true for structural components located directly in the seawater, as in the case of bridge substructures. Seawater contains dissolved salts that affect the durability of concrete. The most prevalent salts in order of quantity are sodium, magnesium and potassium chlorides and magnesium, calcium and potassium sulfates.[17] These salts provide sources of chlorides and sulfates, which can lead to corrosion of reinforcement and sulfate attack on concrete. To a lesser extent, these salts also provide a source of alkalis which may lead to expansive alkali-aggregate reactions if reactive aggregates are present. There are four main exposure zones for a structure in a coastal exposure. Each zone and the associated forms of attack are described below and shown in Figure 3.1.

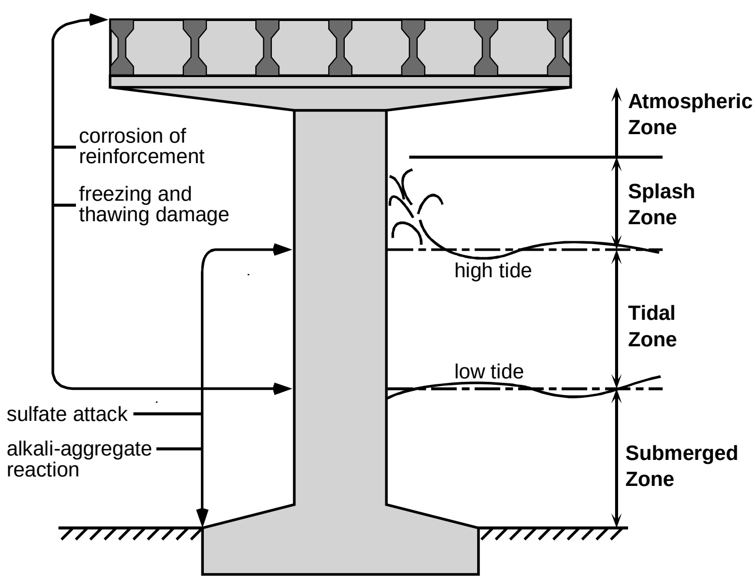


Figure 3.1 – Substructure Exposure Zones and Forms of Deterioration in Coastal Seawater Exposures

Atmospheric Zone

Description: Concrete in this zone is never in direct contact with the seawater, but is exposed to moisture and salts from spray and salt laden mist.

Possible Attack:
- Corrosion of reinforcement may occur due to chlorides.
- Freezing and thawing damage may occur (dependent on climate).

Splash Zone

Description: Concrete in this zone is above high tide but is subjected to direct wetting due to wave action.

Possible Attack:
- Corrosion of reinforcement may occur due to chlorides.
- Freezing and thawing damage may occur (dependent on climate).

Tidal Zone

Description: Concrete in this zone lies between low and high tide, and is subjected to continual cycles of submerging and exposure to air.

Possible Attack:
- Corrosion of reinforcement may occur due to chlorides.
- Freezing and thawing damage may occur (dependent on climate).
- Sulfate attack may occur due to sulfates in seawater.
- Alkali-aggregate reaction may occur due to alkalis in seawater (if reactive aggregates are present).

Submerged Zone

Description: Concrete in this zone is below low tide and is continually submerged.

Possible Attack:
- Sulfate attack may occur due to sulfates in seawater.
- Alkali-aggregate reaction may occur due to alkalis in seawater (if reactive aggregates are present).

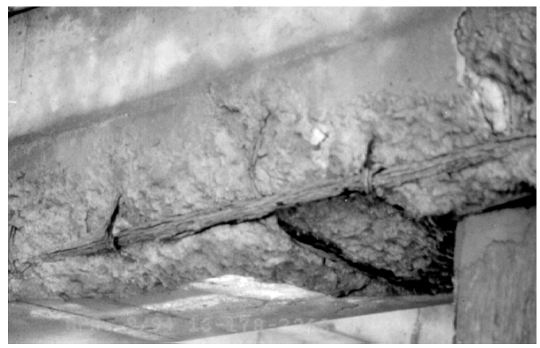
Corrosion of steel reinforcement requires oxygen and thus generally occurs only in zones which experience some amount of drying. Also, corrosion rates are highest when humidity is in the 90-95% range.[17] The greatest risk of corrosion occurs in the splash and atmospheric zones for these reasons. Corrosion in the tidal zone is normally limited due to the shorter drying periods and slower rate of oxygen diffusion through saturated concrete. The submerged zone of concrete has a low risk of corrosion due to lack of oxygen. Several examples of typical corrosion damage in coastal exposures are shown in Figure 3.2. These photos are all of Texas bridges located along the Gulf of Mexico. Photo (a) shows the underside of a pan-joist bridge. Holes had been cored through the deck to improve drainage. These holes allowed salt-laden moisture to drain onto the joists where insufficient concrete cover was provided and corrosion damage occurred at the base of the joists. Photos (c) and (d) show typical corrosion of columns or trestle piles. In (c), large vertical cracks have developed along the line of the longitudinal reinforcement. In (d), severe corrosion has caused complete spalling of the cover concrete, exposing the longitudinal and transverse reinforcement. The corrosion damage occurs above the high tide line within the splash zone of the structure. Moisture and chlorides have entered the concrete from spray or splashing, or possibly through capillary rise.

Frost damage is most severe in concrete that is saturated, and therefore concrete within the tidal zone or immediately above the high tide level may experience the most significant damage. Freeze-thaw damage rarely occurs below the low tide level since the seawater would also have to freeze.

Sulfate attack occurs primarily in zones where the concrete is submerged for some period, allowing greater sulfate concentrations. The greatest risk of sulfate attack normally occurs in the tidal zone and submerged zone. The same holds true for alkali-aggregate reactions due to alkalis in the seawater.

The temperature range to which a structure is subjected also affects durability. Increases in temperature have an accelerating effect on many chemical reactions, including corrosion. The general rule of thumb is that a temperature increase of 10 degrees Celsius doubles the rate of reaction.[17] Traditionally, seawater environments in cold climates, such as the North Sea, were viewed as the most severe exposure for structures. More recently, the accelerating effects of high temperatures have been recognized as equally or possibly more severe than the combination of freezing temperatures and corrosive environments.

(a) Atmospheric Zone - Girder



(b) Atmospheric Zone – Bent Cap

(c) Splash Zone - Column

(d) Splash Zone – Column

Figure 3.2 – Coastal Exposure Corrosion Damage in Bridges

3.2 FREEZING EXPOSURE

Environments where structures may be exposed to freezing temperatures may lead to freeze-thaw damage of concrete. A secondary effect is that the use of deicing chemicals in freezing exposures can exacerbate freeze-thaw damage and may lead to corrosion of steel reinforcement if the deicing agents contain chlorides.

The severity of freeze-thaw damage of concrete is a function of the presence of moisture in the concrete and the number of times the moisture freezes and thaws. Frost damage worsens when repeated cycles of freezing and thawing occur. Thus a moderate winter climate which experiences many freeze-thaw cycles can cause more frost damage than a severe winter climate that remains below freezing for long periods. Detailed discussion is provided in Section 6.2.

Although deicing chemicals would not normally be applied directly to substructure components, moisture and chlorides may come into contact with the substructure through leaking joints in the bridge deck, inadequate drainage or from splashing due to traffic movement. Corrosion of steel reinforcement from this type of exposure can be severe due to its localized nature and often very high concentration of chlorides as discussed in more detail in Section 5.1. Photos of substructure corrosion resulting from drainage of deicing chemicals are shown in Figure 3.3. Typical damage locations include columns and the underside of bent caps located near leaking deck joints. Very severe corrosion damage results due to the localized nature of the attack.

(a) Corrosion at Leaking Deck Joint

(b) Column Corrosion

(c) Bent Cap Corrosion

(d) Column Corrosion

Figure 3.3 - Corrosion Due to Deicing Chemicals in Freezing Exposure

3.3 AGGRESSIVE SOILS

Chemical attack on concrete in the form of sulfate attack or alkali-aggregate reactions may occur in soils containing sulfates or alkalis. The presence of these aggressive agents must be accompanied by moisture for attack to occur (assuming the concrete is susceptible to either form of attack). Moisture provides the transportation mechanism for sulfates and alkalis to penetrate the concrete, and is also necessary for the deleterious reactions to occur.

3.4 SUBSTRUCTURE EXPOSURE CONDITIONS IN TEXAS

The geography, geology, and climate of Texas are such that all three forms of environmental exposure described above may be encountered, as shown in Figure 3.4. This map is a compilation of information from Ref. 18 and 19. The map is subdivided to indicate the TxDOT Districts. Different regions of the state present different bridge substructure durability concerns that must be addressed in the design process. Reinforcement corrosion, sulfate attack and freeze-thaw damage can all occur depending on the location of the bridge. Alkali-aggregate reactions are not necessarily exposure related, since the source of alkalis may be internal in the form of alkalis in the cement. Therefore, alkali-aggregate reactions may occur anywhere within the state if reactive aggregates are used in combination with normal (other than low alkali) cement.

The coastal environment along the Gulf of Mexico provides a severe environment for bridge substructures. The high average annual temperature and humidity combined with the seawater exposure results in severe conditions for corrosion of steel reinforcement and sulfate attack. The occurrence of freezing temperatures is extremely rare along the Gulf of Mexico, and therefore freeze-thaw damage of concrete is not a concern.

The northern regions of the state may experience freezing temperatures during the winter months. When combined with precipitation and the use of deicing agents, these regions may experience freeze-thaw damage and corrosion of steel reinforcement. The FHWA "Deicing Line" is indicated in Figure 3.4. Districts north of this line are required to use deicing chemicals to keep interstate highways and bridges free of ice during winter months.

The northwestern regions of the state have an increased probability of sulfate attack due to aggressive soils. The risk of sulfate attack in these regions is highest where moisture content of the soil is highest, such as bodies of water, rivers, or areas where drainage around the structure is poor.

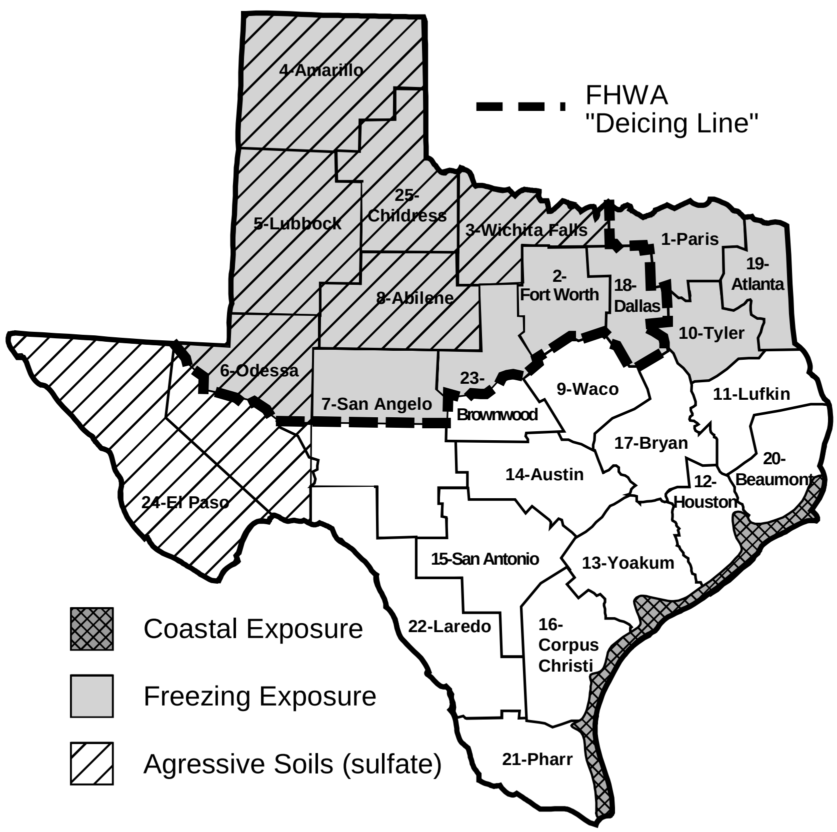


Figure 3.4 - Substructure Exposure Conditions for the State of Texas

Chapter 4:
Corrosion of Steel Reinforcement in Concrete

Corrosion is an electrochemical process in which steel is affected by a particular environment, resulting in a measurable loss of metal. The deleterious effects of corrosion are illustrated in Figure 4.1. The corrosion of steel reinforcement means that iron is being removed from the steel. The liberated ferrous ions are then free to complex with hydroxyl ions to form various corrosion products depending on the availability of oxygen. The corrosion products can occupy up to six times as much volume as steel,[17] leading to cracking and disruption of the concrete. Corrosion of steel reinforcement in concrete may significantly affect structural integrity through reduced capacity or fracture of the steel reinforcement, due to loss of bond between the steel and concrete or through cracking and spalling of the concrete.

The following sections discuss the fundamentals of corrosion of steel in concrete and some specific aspects of the corrosion of prestressing steel. These subjects are all described in detail in many different sources. The intention of the following sections is to provide a brief introduction only, and references are included if more detail is desired.

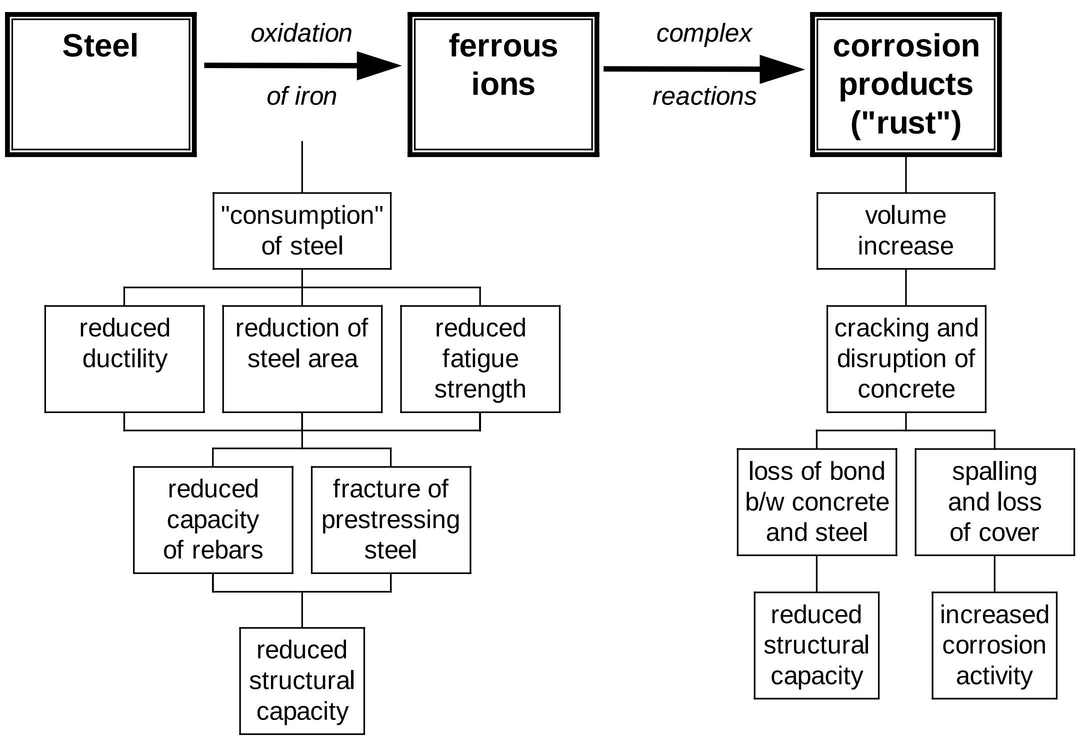


Figure 4.1 - Deterioration Mechanism for Corrosion of Steel in Concrete

4.1 CORROSION FUNDAMENTALS

The fundamentals of corrosion are similar for prestressed and nonprestressed reinforcement. Steel in concrete is normally protected from corrosion by a passive film of iron oxides on the steel resulting from the alkaline environment of the concrete. Passivation of the steel may be destroyed by carbonation (CO_2)

or by the presence of chloride ions. In the absence of a passive film, corrosion may occur. The electrochemical process of corrosion requires several key elements, all of which must be present for corrosion to occur. The rate of corrosion is affected by many complex factors.

4.2 BASIC CORROSION CELL IN CONCRETE

The electrochemical corrosion cell consists of three key elements:

1. Anode: The location that loses metal (anodic reaction).
2. Cathode: The location of oxygen reduction (cathodic reaction).
3. Electrolyte: The conductive medium in which the corrosion takes place.

Chemical reactions occur on the steel at the anodic and cathodic regions. The concrete acts as the electrolyte, and electrical charge transfer occurs between the anodic and cathodic areas. The anode and cathode must be connected electrically for charge transfer to occur. The concrete (electrolyte) conducts current through ion transfer to complete the corrosion cell. The basic anodic and cathodic reactions are as follows:

Anodic Reaction: $Fe \rightarrow Fe^{2+} + 2e^-$

Cathodic Reaction: $\frac{1}{2}O_2 + H_2O + 2e^- \rightarrow 2(OH)^-$

Iron is oxidized at the anode (i.e., iron is removed from the steel, and two electrons are liberated). At the cathode, oxygen is reduced by the electrons liberated at the anode, and hydroxyl ions are produced. In order to conserve charge, the reaction rates for the anodic and cathodic reactions must be equal. The charge flow in the corrosion cell is completed by the movement of the hydroxyl ions from the cathode to the anode by diffusion. The ionic diffusion process can by modeled by Ohm's Law, indicating the charge flow between the anode and cathode is inversely proportional to the resistivity of the concrete (electrolyte).[22] Thus, in order for corrosion to occur, the passive film on the steel must be breached to permit oxidation of the iron, water and oxygen must be available at the cathode, and the resistivity of the concrete must be low enough to permit ionic diffusion between the anode and the cathode.

The corrosion cell may exist as a macrocell or a microcell. The corrosion microcell consists of very small anodes and cathodes separated by a distance of as small as a micron. The corrosion macrocell consists of anodic and cathodic regions separated by a finite distance of millimeters or meters. The macrocell may develop on a single bar, or it may occur between different layers of steel or different areas of the structure. A common example of a corrosion macrocell is a bridge deck where the top layer of steel acts as the anode and the bottom layer as the cathode, as shown in Figure 4.2. The top layer is closer to the source of moisture and chlorides, and therefore becomes depassivated before the bottom layer. Oxygen is readily available near the bottom layer to facilitate the cathodic reaction. Electrical continuity between the layers is provided by stirrups or other reinforcement or ties. Another form of macrocell corrosion can occur at crack locations.[20] The anodic reaction occurs in the vicinity of the crack, while the cathodic reaction takes place on the same bar within the distance between cracks, as shown in Figure 4.3. The steel near the crack becomes depassivated due to chloride or carbon dioxide penetration at the crack. This form of corrosion cell can lead to very high corrosion rates and pitting corrosion due to the large cathode area in comparison to anode area. In most situations of corrosion in concrete structures, a combination of microcell and macrocell corrosion occurs.[22]

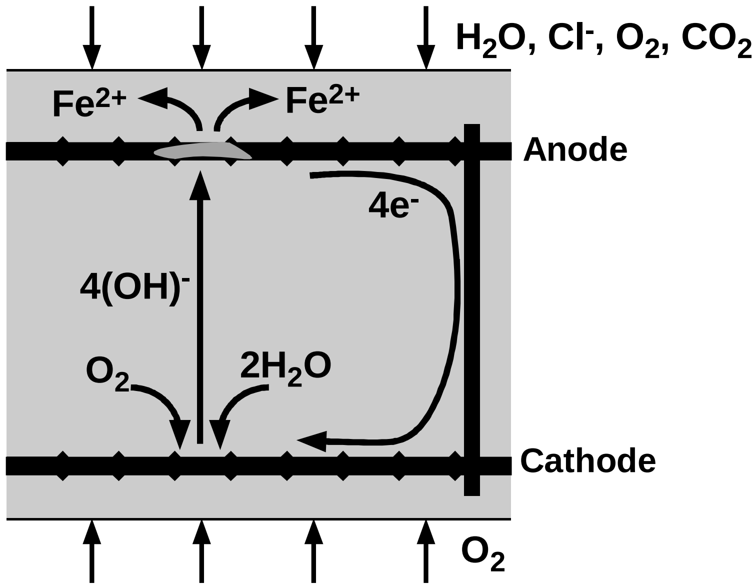


Figure 4.2 - Idealized Macrocell Corrosion

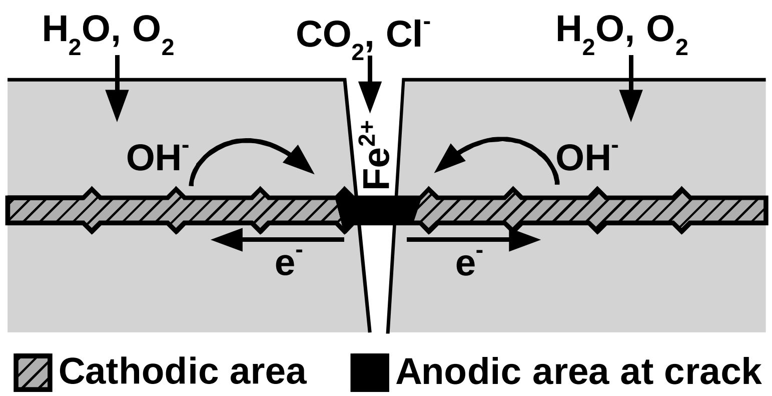


Figure 4.3 - Macrocell Corrosion at a Crack

If chlorides are present in the concrete, they may act as a catalyst by introducing an additional anodic reaction:[21,22]

	$Fe + 3Cl^-$	➔	$FeCl_3^- + 2e^-$
followed by	$FeCl_3^- + 2OH^-$	➔	$Fe(OH)_2 + 3Cl^-$

or

	$Fe + 4Cl^-$	➔	$FeCl_4^{2-} + 2e^-$
followed by	$FeCl_4^- + 2OH^-$	➔	$Fe(OH)_2 + 4Cl^-$

or

	$Fe + 6Cl^-$	➔	$FeCl_6^{3-} + 3e^-$
followed by	$FeCl_6^{3-} + 2OH^-$	➔	$Fe(OH)_2 + 6Cl^-$

These reactions remove ferrous or ferric ions from the steel by forming complex ions with the chlorides. The ferrous or ferric ions are then deposited near the anode where they join with hydroxyl ions to form various corrosion products. The chloride ions are released to repeat the process. Since the chloride ions are not consumed, the process can become autocatalytic. The electrons released during these reactions flow through the steel to the cathodic reaction.

The anodic and cathodic reactions are followed by a variety of secondary reactions that form the expansive products of corrosion. Although the Fe^{2+} and OH^- ions both diffuse into the concrete (from the anode and cathode respectively), the corrosion products form near the anode because the OH^- ions are smaller and more diffuse through the concrete more readily.[23] If the supply of oxygen is restricted, ferrous oxides and hydroxides form:

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

or $$Fe^{2+} + 2(OH)^- \rightarrow FeO + H_2O$$

If oxygen is available, ferric oxides and hydroxides form:[21,24,25]

	$2Fe(OH)_2 + ½O_2$	➔	$Fe_2O_3 \bullet H_2O + H_2O$	(red rust)
or	$2Fe(OH)_2 + ½O_2 + H_2O$	➔	$2Fe(OH)_3$	(red rust)
or	$3Fe(OH)_2 + ½O_2$	➔	$Fe_3O_4 + 3H_2O$	(black rust)
or	$2FeO + ½O_2 + H_2O$	➔	$Fe_2O_3 \bullet H_2O$	(red rust)
or	$2FeO + ½O_2 + 3H_2O$	➔	$2Fe(OH)_3$	(red rust)
or	$3FeO + ½O_2$	➔	Fe_3O_4	(black rust)

4.2.1 Passivation

The environment of concrete normally provides good corrosion protection for steel. One of the most widely accepted mechanisms for corrosion protection is the formation of a passive film on the surface of steel embedded in concrete. The passive film is a thin, tightly adherent, non-conducting layer of iron oxide that slows or stops the corrosion process by preventing ferrous ions (Fe^{2+}) from entering solution in the electrolyte.[22] The passive film only affects the anodic reaction, and the cathode reaction may occur on surfaces where the film is intact.

Initially, the steel must corrode to produce the passive film. The process begins soon after construction as hydration of the cement raises the pH of the concrete. Recalling the anodic, cathodic and secondary reactions listed in the preceding section, ferrous hydroxide ($Fe(OH)_2$) is produced by the corrosion process. If oxygen and moisture are present, the ferrous hydroxide is converted to gamma iron oxide (γ-Fe_2O_3) and a thin passive film is formed on the surface of the steel. The passive film remains stable when the alkalinity of the concrete exceeds about pH 12,[22,26] and the steel is protected from further corrosion. The preceding discussion is only one theory of the passivation of steel in concrete. The actual composition of the passive film is not certain, and for this reason it is commonly referred to as simply an oxide film.[27]

4.2.2 Stages of Corrosion in Concrete Structures

The process of corrosion is often separated into the stages of initiation and propagation,[28] as illustrated in Figure 4.4. Depassivation of the steel defines the transition from the initiation stage to propagation.

This simple model can be used to illustrate the effects of time to corrosion initiation and corrosion rate on the service life of the structure. Two different situations are illustrated in Figure 4.5. In the first situation, structure A has a lower corrosion rate, but its service life is shorter since corrosion initiation occurred earlier than structure B. In the second situation, structure B has a shorter service life although the corrosion rates are the same as the first situation. Thus, it is important to consider both the time to corrosion and corrosion rate, as a single parameter is not sufficient to determine the service life of the structure.

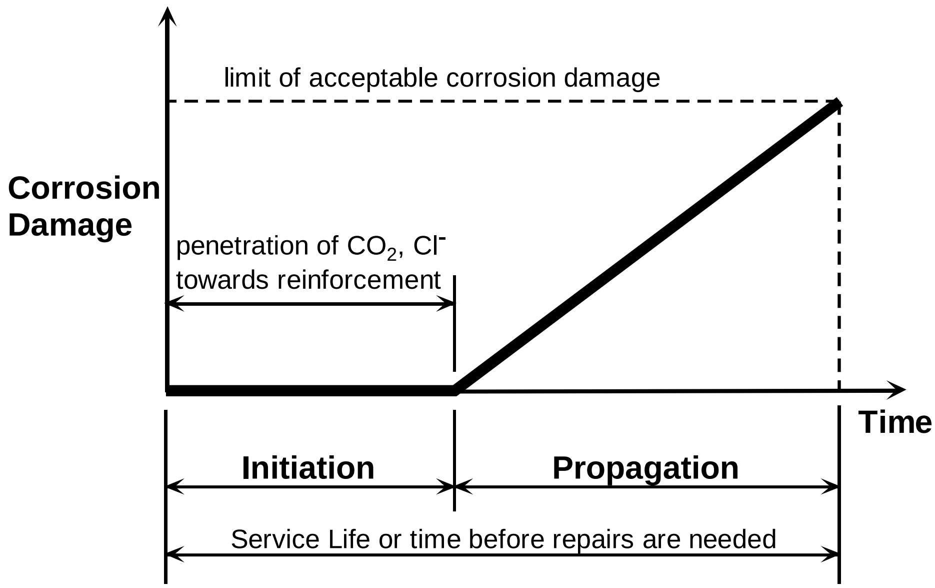


Figure 4.4 - Stages of Corrosion of Steel in Concrete (adapted from Ref. 28)

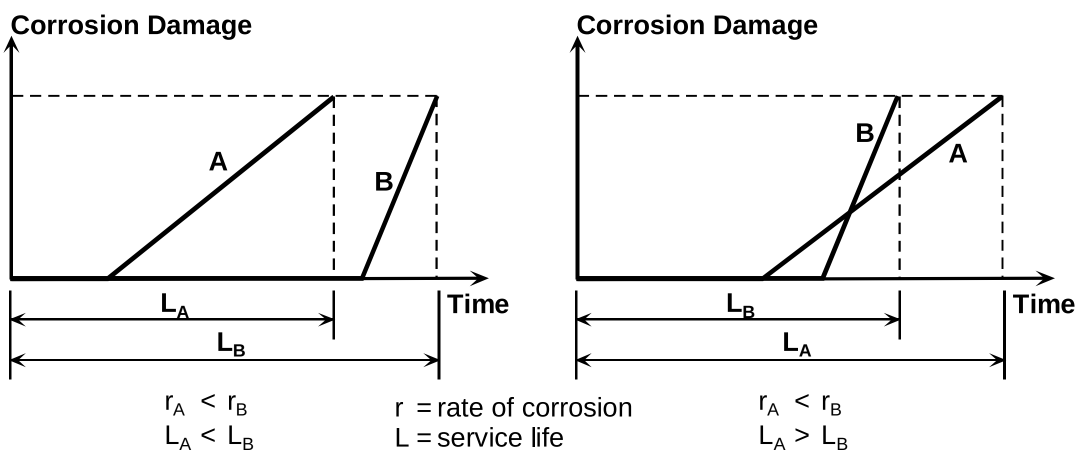


Figure 4.5 - Effect of Time to Corrosion and Corrosion Rate on Service Life (adapted from Ref. 28)

4.2.2.1 <u>Stage 1: Initiation – Factors Affecting the Time to Corrosion</u>

As described previously, steel in concrete is normally protected from corrosion by a passive oxide film on the surface of the steel that prevents or limits corrosion. Destruction of the film, or depassivation, can allow corrosion to begin. The first stage of the corrosion process concerns the portion of the service life where various actions may lead to depassivation of the steel. The most common causes of depassivation are carbonation of the concrete and the presence of certain substances, and in particular, chlorides.

Carbonation

Carbonation of concrete occurs when atmospheric carbon dioxide penetrates the concrete. In the presence of moisture, carbon dioxide will react with calcium hydroxide in the concrete to produce calcium carbonate, as shown below.

$$CO_2 + Ca(OH)_2 \xrightarrow[H_2O]{} CaCO_3 + H_2O$$

The formation of calcium carbonate reduces the pH of the concrete to as low as 8, where the passive film is no longer stable allowing corrosion to begin.[29]

The process of carbonation is slow in good quality concrete. The rate of carbon dioxide penetration is a function of the square root of the exposure time.[30] Factors affecting the rate of carbonation include the concrete permeability, cracking, the moisture content of the concrete and relative humidity. The presence of cracks will allow the carbonation front to reach the steel rapidly (on a local scale). This carbonation may contribute to the formation of macrocell corrosion at the crack (see Figure 4.3). Carbonation will not occur in concrete that is saturated or very dry. The rate of carbonation is highest for relative humidity of 50% to 70%.[29]

Carbonation may be slowed by specifying concrete with low permeability. Options include the use of low water-cement ratios, use of superplasticizer to reduce water demand, and the use of mineral admixtures. Compaction and proper curing are needed to ensure low permeability. Concrete surface treatment or sealers may slow penetration of carbon dioxide.

Chlorides

The role of chlorides in depassivation has been much debated. The general consensus is that once the level of chlorides in the concrete at the steel exceeds a certain limit, the passive film either breaks down or is no longer able to protect the steel from corrosion. Several theories have been proposed, and some have been summarized in Ref. 22, 24 and 27. In general terms, the role of chlorides in depassivation may take one or a combination of the following forms:

- Chloride ions may disperse the passive film.
- Chloride ions may make the film permeable to Fe^{2+} ions allowing the anodic reaction to occur even when the passive film is present.
- Chloride ions may penetrate the passive film and anodic reactions with Cl^- acting as a catalyst may occur (see Section 4.2).
- Chloride ions may reduce the pH, making the passive film unstable.

Chlorides may be present in the concrete from any of the concrete constituents. Most design and construction specifications or durability guidelines limit the amount of permissible chlorides in concrete constituents. Chlorides may also penetrate the concrete from external sources, most commonly deicing chemicals and seawater.

The factors affecting the penetration of chlorides are the same as those for carbonation. The best protection against chloride penetration is sound, good quality concrete. Thicker concrete cover increases the exposure duration required for chlorides according to the square root time relationship.[17] Doubling the cover will increase the exposure duration before chlorides reach the steel by approximately four times. More information is provided in Section 4.2.3.

4.2.2.2 Stage 2: Propagation – Factors Affecting the Rate of Corrosion

Once the passive film on the steel has been destroyed, the corrosion process may begin provided that the essential components of the corrosion cell are present (see Section 4.2). The rate of corrosion, or the rate at which iron is removed from the steel, is a function of many factors[27] including the presence of moisture and oxygen, concrete permeability and resistivity, gradients in chloride ion concentration, heterogeneity's in the concrete and steel, pH of the concrete pore water, carbonation, cracking and stray currents.

Corrosion Cell Components

In general, the anodic and cathodic reactions must be able to occur for the corrosion cell to develop. For the anodic reaction, the passive film must be absent to permit oxidation of iron. A supply of oxygen and moisture is necessary for the cathodic reaction to occur.

The ratio of the cathode area to anode area has a significant effect on corrosion rate. Conservation of charge requires that there is no accumulation of charge (i.e., electrons liberated at the anode flow through the steel to the cathode where they are used in oxygen reduction). The corrosion rate or severity in terms of metal loss is related to the current density at the anode (i.e., the corrosion current or number of electrons flowing through the anode area). For a given current in the corrosion cell, the corrosion current density increases as the electrode area decreases. Therefore, as the ratio of cathode area to anode area increases, the anodic current density increases. Situations where large cathode/anode area ratios exist can lead to very high corrosion rates and severe metal loss at the anode. Examples include macrocell corrosion at a crack in the concrete, or at a defect in an inert coating on the steel. Conditions where the steel is only depassivated over a small area will also lead to high corrosion rates.

The corrosion cell also requires charge transfer between the anode and cathode. Electrons move between the anode and cathode through the steel. If electrical contact is interrupted in a corrosion macrocell, corrosion will cease. Charge transfer also occurs through ion diffusion between the anode and cathode. If the resistivity of the concrete is high, diffusion of the ferrous and hydroxyl ions will be slow and corrosion will be limited as discussed in more detail under polarization effects.

Corrosion Potential (Driving Force)

The electrochemical corrosion cell works in the same manner as an electrical circuit. The current flowing in the circuit is related to the electromotive force in the corrosion cell.[22] The electromotive force is measured as a potential (voltage) difference between the anode and cathode. The electromotive force indicates the potential energy in the system, but does not directly indicate the corrosion rate for reasons that will be discussed under polarization effects.

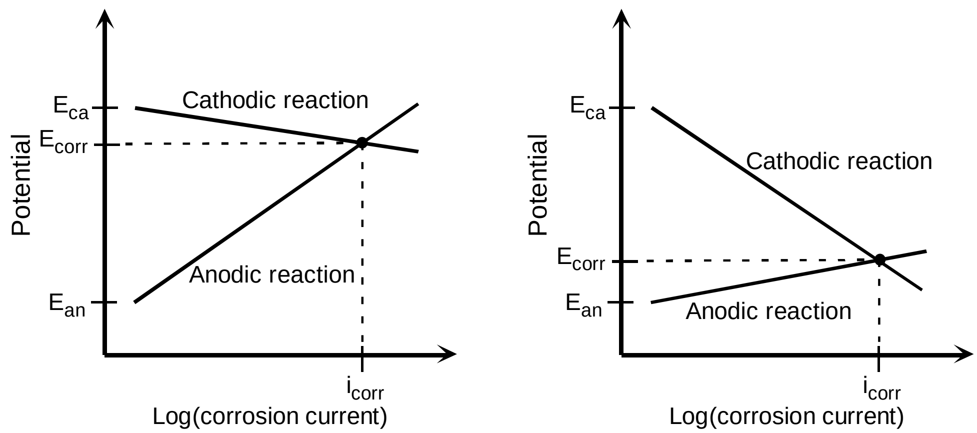
The electromotive force or potential difference in the corrosion cell can result from many sources. In general, any non-uniformity between the anode and cathode or non-uniformity within the electrolyte (concrete) will produce a potential difference.[22] Examples include dissimilar metals and concentration cells. The latter term refers to a potential difference resulting from non-uniform concentrations of oxygen, moisture, chloride ions or metallic ions in the concrete. Temperature gradients or variations in pH may also produce concentration cells. Concentration cells commonly lead to macrocell corrosion. Examples include higher chloride levels for steel near an exposed surface, non-uniform concrete moisture contents along the length of a bar and the presence of cracks in the concrete.

Polarization Effects

Several factors may limit the rate of corrosion in spite of a large electromotive force. These conditions are referred to as polarization effects. The discussion of polarization effects is best illustrated using mixed-potential corrosion theory. Modern corrosion theory is described in most corrosion textbooks, including Ref. 31 and 32, or information can be found in Ref. 33 and 34. Readers unfamiliar with modern corrosion theory are referred to these sources.

Polarization is defined as a shift in the half-cell potential of an electrode away from its free or reversible potential due to current flow.[22,27] Typical mixed electrode plots are shown in Figure 4.6. E_{ca} and E_{an} indicate the reversible half-cell potentials of the cathode and anode, respectively. When the anode and cathode are coupled together, current begins to flow and an equilibrium is reached at E_{corr} (open circuit potential of the corroding element) and i_{corr} (corrosion current). The curves (lines) in the figure indicate the polarized anodic and cathodic half-cell reactions.

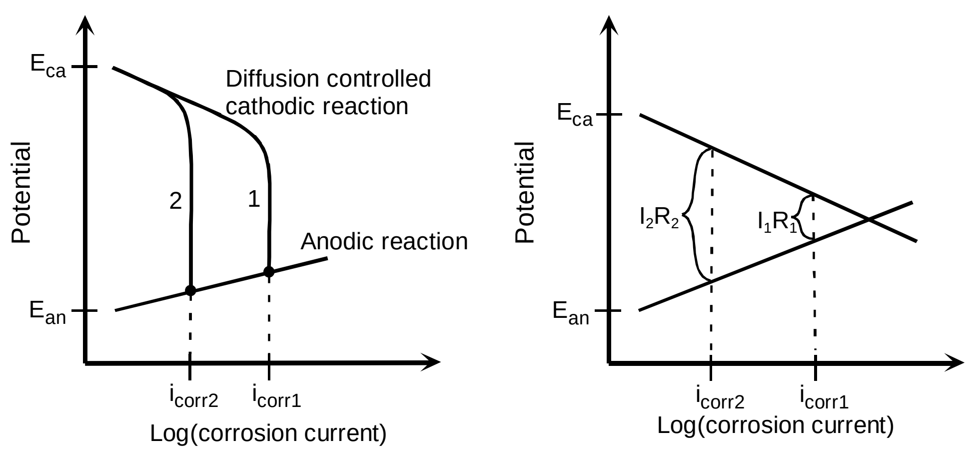
The corrosion rate of the system may be controlled by three different kinds of polarization: Activation Polarization, Concentration Polarization and Ohmic Polarization.[27] Activation polarization refers to situations where the corrosion rate is controlled by reactions at the metal-electrolyte interface. An example would be the passive film on steel that limits oxidation of iron, slowing the anodic reaction as is illustrated in Figure 4.6(a). If the cathodic reaction is slower than the anodic reaction, the process is said to be cathodically controlled as shown in Figure 4.6(b).



(a) Anodic Control **(b) Cathodic Control**

Figure 4.6 – Electrochemical Processes Under Activation Polarization[27]

The most common corrosion rate controlling mechanisms in concrete structures are concentration polarization and ohmic polarization. Concentration polarization occurs when the conditions change in the electrolyte near the anode or cathode, slowing the reaction rate. An example of concentration polarization in concrete is depletion of oxygen near the cathode, as illustrated in Figure 4.7(a). This condition is referred to as cathodic diffusion control. The slope of the polarized cathodic reaction curve approaches infinity, resulting in very negative half-cell potentials but low corrosion rates. The oxygen diffusion rate for case 2 is lower than case 1 in Figure 4.7(a). Ohmic polarization occurs when the resistivity of the concrete is high. The electromotive force (potential) available for corrosion is reduced by the resistance of the concrete, sometimes referred to as the "IR effect" or "IR drop." As a result, the corrosion rate is limited as shown in Figure 4.7(b). The corrosion rate decreases as the IR drop increases, as indicated in the figure.



(a) Concentration Polarization **(b) Ohmic Polarization**

Figure 4.7 – Common Polarization Effects in Concrete Structures[27]

4.2.3 *Role of Chlorides*

Chlorides are one of the most significant influencing factors for corrosion of steel in concrete. Therefore, it is prudent to summarize the effects of chlorides that have been discussed already, and introduce some issues not addressed thus far. The role of chlorides in corrosion includes:[21,22,24,27]

1. **Catalyst for oxidation of iron (anodic reaction):** Chlorides may accelerate corrosion by providing additional anodic reactions. Several mechanisms were discussed in Section 4.2.
2. **Depassivation of the steel:** The presence of chlorides may result in destruction of the protective passive film on the steel. Several mechanisms were discussed in Section 4.2.2.
3. **Concentration Cells:** Concentrations of chlorides in the concrete will cause a shift in the half-cell potential of the steel. The resulting potential difference provides the corrosion cell with the electromotive force for corrosion.
4. **Concrete Resistivity:** The presence of chloride ions in the concrete reduces the resistivity of the concrete. Also, the chlorides increase the saturation of the concrete, further decreasing resistivity.
5. **Reduced pH:** The presence of chlorides may reduce the pH, affecting the stability of the passive film and increasing the possibility of pitting corrosion.
6. **Freezing damage:** Freeze-thaw damage of concrete is generally more severe in the presence of chlorides. Cracking and spalling from freeze-thaw damage allows faster penetration of moisture and chlorides and leads to more severe corrosion damage.

4.2.3.1 <u>Chloride Corrosion Threshold</u>

The risk of corrosion increases when the concentration of chlorides in the concrete increases. The concept of the chloride corrosion threshold is used to indicate the chloride content above which corrosion will occur. This concept assumes that moisture and oxygen and the other conditions necessary for corrosion are present. Although this concept is very appealing, it is very difficult to define since many factors influence the necessary chloride content for corrosion. Some of the issues include:

- **How the chloride level is determined:** total chloride, acid-soluble chloride or water-soluble chloride.[27] Chlorides may be bound into various compounds in the concrete, making them unavailable for corrosion.[35,36,37,38] This binding suggests water-soluble chlorides should be used. However, the chlorides may later become available for corrosion,[35,39,40,41,42] suggesting that total or acid-soluble chloride content is more conservative.
- **Whether chlorides were admixed or penetrated from external sources:** More chlorides will be bound if admixed into the concrete.[27,35]
- **Cement composition:** C_3A content of the cement affects chloride binding.[36,37,38,43]
- **Exposure conditions:** Moisture and oxygen concentrations influence corrosion activity, and thus affect the threshold. The presence of sulfates may cause chlorides to become “unbound.”[35,39,40,41,42]
- **pH of concrete pore solution:** Since the pH influences the passive film on the steel, some suggest that instead of a chloride threshold, the corrosion threshold should be in terms of the ratio of chloride ions to hydroxyl ions (Cl^- / OH^-).[25,39,43,44]

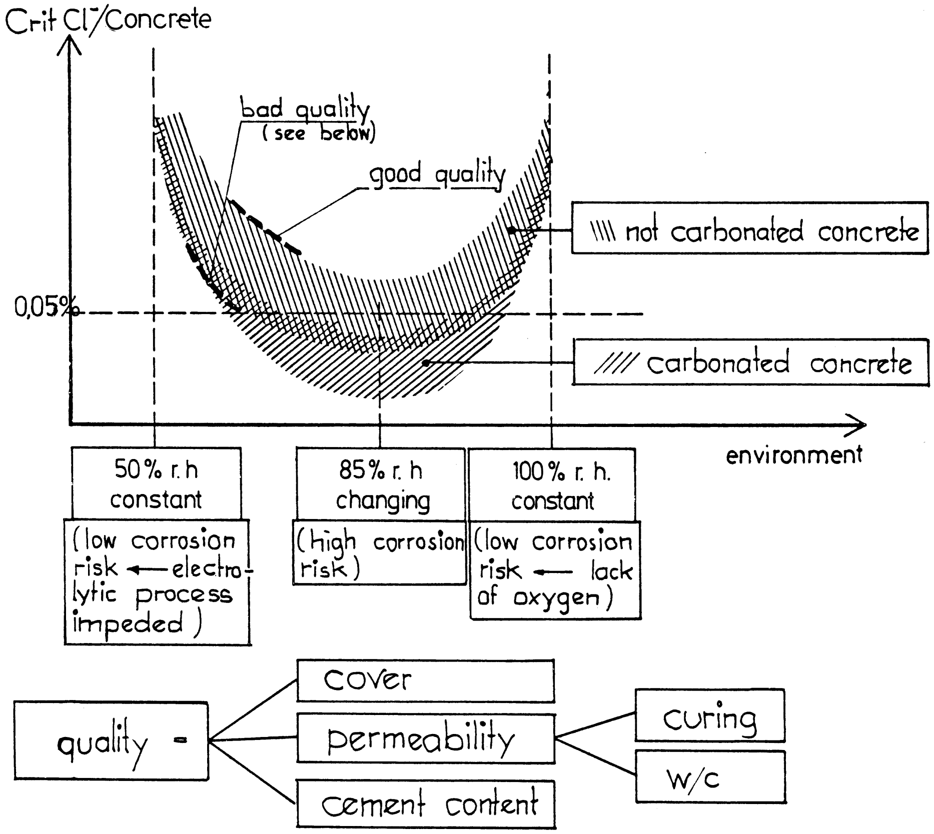


Figure 4.8 - CEB Critical Chloride Ion Content for Corrosion[17]

Many researchers have proposed chloride corrosion threshold values. Kahhaleh[45] performed a thorough literature search on the subject and reported threshold values of 0.14% to 0.35% acid-soluble chlorides by weight of cement. A value of 0.2% acid-soluble chlorides by cement weight is often reported and recommended.[27,46] The Comité Euro-International du Béton[17] (CEB) provides the chart shown in Figure 4.8. This figure shows the influence of environment, concrete quality and carbonation on the critical chloride content. The CEB suggests that the chloride corrosion threshold in concrete (not carbonated) is approximately 0.05% chloride by weight of concrete, or about 0.4% by weight of cement.[17] It is not clear whether these values are in terms of total, acid-soluble or water-soluble chlorides. Additional discussion on the issues surrounding chlorides and corrosion in concrete is provided in Ref. 27 and Ref. 46. Due to the significance of chlorides in the corrosion process, design for corrosion protection is primarily concerned with the preventing chlorides from entering the concrete and reaching the steel as discussed in detail in Chapter 5.

4.3 Corrosion of Prestressing Steel

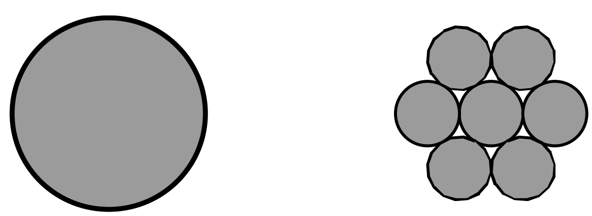
The consequences of prestressing steel corrosion are potentially more severe than corrosion of mild steel reinforcement. This sensitivity is primarily due to the higher strength of the prestressing steels, and the high level of stress in the steel. Prestressing steels normally experience stress levels in service on the order of 70% to 80% of their ultimate strength. This percentage is much lower in mild steel reinforcement. For example, working stress design methods for reinforced concrete (ACI 318-95,[47] Appendix A) limit the tensile stress in Grade 60 (400 MPa) reinforcement to 40% of the yield strength. The stress levels will likely be higher for strength design methods, but will remain lower than prestressing steels. Due to the lower stress levels, the loss of cross-sectional area due to corrosion is less likely to lead to tensile failure of mild steel reinforcing bars. The higher strength of prestressing steel also means that there is less steel area in the member cross-section. As a result, the loss of one prestressing strand or bar will have a more significant effect on the capacity of the member than the loss of an equivalent sized mild steel bar. This effect is illustrated in Table 4.1 for two of the beam sections described in Report 1405-3 (Long-term Beam Exposure Tests). The Non-PS (nonprestressed, reinforced concrete) and 100%U PS (prestressed based on nominal strength requirements) beam types were designed for the same load requirements. If one #4 bar (12.7 mm dia.) is assumed to fail in the Non-PS section, the nominal flexural capacity is reduced by 4.7%. If one 12.7 mm dia. prestressing strand is assumed to fail in the 100%U PS section, the nominal strength is reduced by 13.5%.

Table 4.1 - Effect of Corrosion (Loss of Flexural Reinforcement) on Nonprestressed and Prestressed Members Designed for Equivalent Loading

Bream Section Type	Design Flexural Strength, M_n	Reduced Flexural Strength**	% Reduction
Non-PS	529 kN-m (4680 k-in.)	504 kN-m (4460 k-in.)	4.7%
100%U PS	529 kN-m (4680 k-in.)	457 kN-m (4046 k-in.)	13.5%

** Non-PS: one #4 bar assumed failed
100%U PS: one 12.7 mm prestressing strand assumed failed

Prestressing steels are generally believed to be more susceptible to corrosion than mild steel reinforcement for several reasons. In the case of 7-wire prestressing strands, the surface area to volume ratio is larger than for the equivalent diameter bar, as illustrated in Figure 4.9. This larger ratio means that more surface area is available for corrosion, and the cross-sectional area of the strand may be reduced at a faster rate. The configuration of the 7-wire prestressing strand also makes the strand more susceptible to crevice corrosion. Crevice corrosion is a type of severe corrosion occurring in small spaces or crevices, such as the interstices between wires. The geometric constraints of the crevice enhance the formation of chloride ion concentration cells. Once corrosion has initiated, it progresses similar to pitting corrosion. Due to the geometry of the crevice, Fe^{2+} ions cannot disperse easily, and chloride ions are drawn into the crevice by the positive charge accumulation. The process becomes autocatalytic as the presence of chloride ions leads to formation of hydrochloric acid (HCl) and higher corrosion rates ensue.



12.7 mm dia. bar | **12.7 mm dia. strand**

Surface area per 100 mm length:

Bar: 3990 mm^2
Strand: 9310 mm^2

Figure 4.9 - Surface Area of Bars and Strands

High strength prestressing steel is also more prone to other forms of corrosion related deterioration that do not occur in mild steel reinforcement. These forms include stress corrosion cracking, hydrogen embrittlement, fretting fatigue and corrosion fatigue. These types of deterioration mechanisms are very difficult to detect, and can lead to brittle failure with little or no sign of warning. The reader is referred to the following references for additional information on these forms of deterioration: Ref. 17, 31, 32, 33, 48, and 49. Many additional references can be found in Ref. 33 and 49. ACI Committee 222, Corrosion of Metals in Concrete, is currently in the process of developing a state-of-the-art report on corrosion of prestressing steels that will discuss these issues in depth. Major contributors to this report include H.R. Hamilton (see Ref. 33) and the senior author of this report.

4.4 EFFECT OF CONCRETE CRACKING ON CORROSION

The influence of cracking and crack widths on the corrosion of steel reinforcement in concrete members is a subject that has received much debate. In the past, considerable research and discussion has been devoted to this topic without arriving at a general consensus. In general, two points of view exist:[27]

1.) Cracks reduce the service life of the structure by permitting a more rapid means of access for moisture, chloride ions and oxygen to reach the reinforcement, thus accelerating the onset of corrosion. (i.e., cracking has a significant effect on corrosion)

2.) Cracks may accelerate the onset of corrosion, but such corrosion will be localized to the region of the crack. It is suggested that over time, chloride ions will eventually penetrate even uncracked concrete, initiating more widespread corrosion. Thus, after a long service life, the difference between the amount of corrosion in cracked and uncracked concrete will be minor. (i.e., cracking does not have a significant effect on corrosion)

The two points of view are illustrated in Figure 4.10 and Figure 4.11. Both points of view indicate that the presence of cracks will accelerate the onset of corrosion. The first point of view suggests that the accelerated onset of corrosion will lead to more corrosion damage in a shorter period, and thus reduce the service life of the structure. The second point of view suggests that the corrosion rate in uncracked concrete will reach the corrosion rate at the crack locations after some duration. This point of view implies that the length of time between corrosion initiation at a crack, t_i, and corrosion initiation in uncracked concrete, t_{icr}, is not significant. This latter point of view means that the two curves in Figure 4.11 are close together when the entire service life is considered, and thus the early onset of corrosion at cracks has little effect on the service life in comparison to the case where the concrete was entirely uncracked.

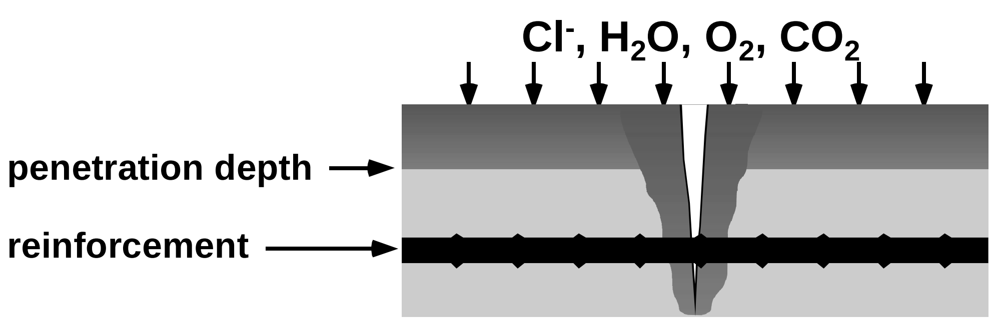


Figure 4.10 – Point of View 1: Increased Penetration of Moisture and Chlorides at Crack Location Accelerates the Onset and Severity of Corrosion

The opinions presented in the preceding paragraph are primarily based on past research and experience related to corrosion in reinforced concrete before the development of modern high performance, low permeability concrete. Much of this research and experience has focused on relating crack width to corrosion, and trying to determine whether there is a critical crack width (i.e., a crack width below which corrosion will not occur or is negligible) for corrosion.

Either or both of these opinions are normally reflected in design code provisions for crack control. While there are other important reasons for crack control in concrete structures, the concern for corrosion is often prevalent. Code provisions and design recommendations normally recognize different exposure conditions and assign crack control requirements that become more strict as the exposure becomes more severe.

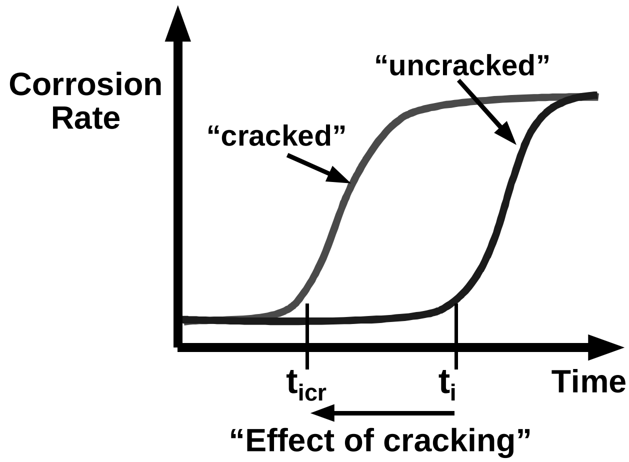


Figure 4.11 – Point of View 2: Cracking Accelerates Onset of Corrosion, But Over Time Corrosion is Similar in Cracked and Uncracked Concrete

In order to gain further insight into the issue of how significant is cracking to corrosion protection from the perspective of the research objectives described in this report, an in-depth literature review was performed on the subject. The review is separated into two sections: 1) technical publications, and 2) related research. In the technical publications section, relevant code provisions and technical committee publications are presented and examined. In the research section, a extensive summary of historical and recent research in the area of reinforcement corrosion and the influence of cracks is reviewed. Due to the tremendous amount of research and discussion in this area, the literature review is not all-inclusive. However, a considerable portion of the available literature has been considered, with emphasis placed on the most relevant information.

The literature review is presented in Appendix A. Portions of the review were contributed to by Brad Koester, who was a Master of Science graduate student on the project at the time. Conclusions and observations from the two main areas of the literature review are presented in Sections 4.4.1 and 4.4.2.

4.4.1 Design Codes and Technical Committees: Cracking and Corrosion

Section A.1 of the cracking and corrosion literature review (Appendix A) investigated how various building and bridge design codes and various technical societies and committees addressed the issue of cracking and corrosion. Emphasis was placed on whether crack control provisions and guidelines were related to crack widths and exposure conditions, and whether prestressed concrete was addressed. Design codes from several countries were reviewed. Technical committee documents from the American Concrete Institute (ACI) and the Comité Euro-International du Béton (CEB) were also considered. The reader is referred to Appendix A for more detail and references for all reviewed documents.

For comparison purposes, the maximum allowable crack widths specified or implied by the reviewed design codes and technical committees are presented in Figure 4.12 and Figure 4.13 for the most mild and most severe environment specified by each document. In some cases, intermediate environmental conditions were also specified but have not been shown here. The majority of the crack widths presented in the figure are specified as surface crack widths, with the exception of the Ontario Highway Bridge Design Code (OHBDC)[A.6] and the U.S. Bureau of Public Roads (USBPR)[A.19] which use maximum allowable crack widths at the level of the reinforcement. These values will be less than crack widths at the concrete surface.

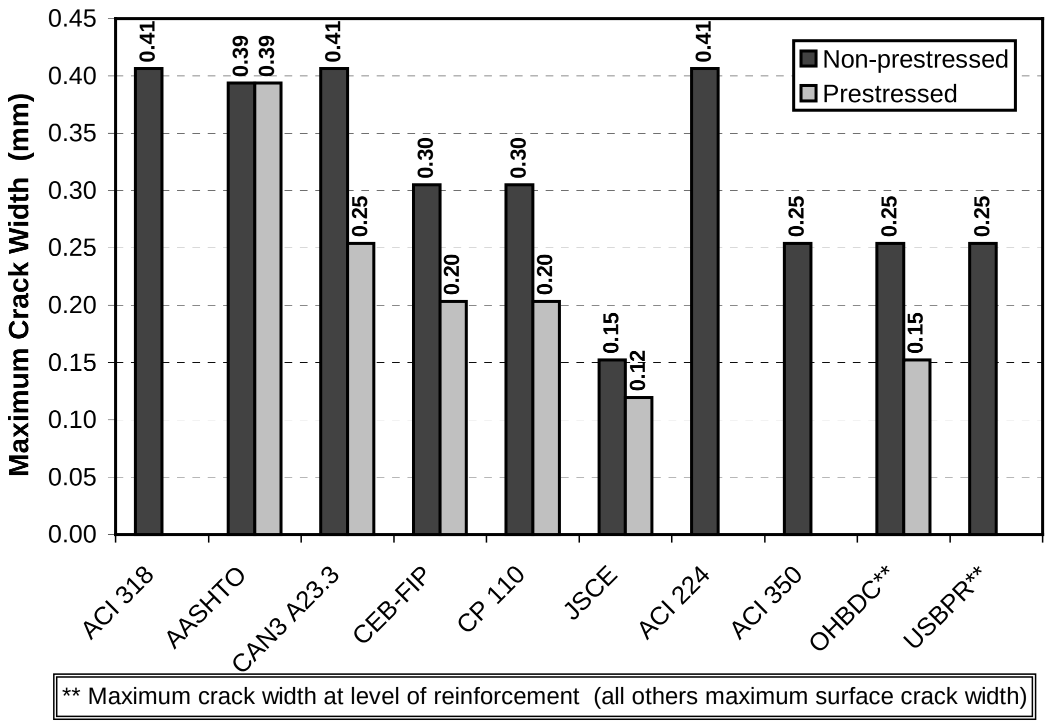


Figure 4.12 - Comparison of Allowable Crack Widths: Mild Exposure

For reinforced concrete elements, the range of recommended allowable crack widths is fairly consistent. The highest value of 0.41 mm (0.016 in.) (surface crack width), recommended by ACI 318-95[A.2] and ACI 224R-90[A.16] for mild environments, is intended for interior exposures or dry air and thus represents a condition not likely to be encountered in bridge applications. The most strict requirements for allowable crack widths in reinforced concrete members are provided by the JSCE standards for concrete structures[A.11] and ACI 224R-90.[A.16] The JSCE surface crack width requirements are specified in terms of the amount of concrete cover. Because larger cover is required by the JSCE for severe exposures, the allowable surface crack width is larger than for mild conditions. The severe environments defined by both the JSCE standard and ACI 224R-90 are for marine structures, and thus would reasonably represent the most severe exposure conditions to be encountered for bridge structures.

The recommended allowable crack widths for prestressed concrete elements are, in general, more strict than for nonprestressed elements with the exception of the *AASHTO LRFD Specifications*[A.3] where they are equal. In several cases, the commentary sections accompanying the recommendations indicated that stricter crack width requirements were necessary for prestressed concrete elements due to the higher susceptibility of prestressing steels to more severe forms of corrosion such as pitting and stress corrosion, and the increased uncertainty in computing crack widths for prestressed members. The most strict requirements are again specified by the JSCE standard.[A.11] For severe corrosive environments such as marine exposures subjected to tides and splashing, no cracks are allowed by the JSCE. A similar requirement is specified by the CEB-FIP Model Code[A.8] for severe environments. However, the CEB-FIP Model Code allows this requirement to be relaxed to a surface crack width of 0.2 mm (0.008 in.) for post-tensioned members if positive protection methods are employed. Recommended protection methods include epoxy coating of the strands or use of an impermeable duct.

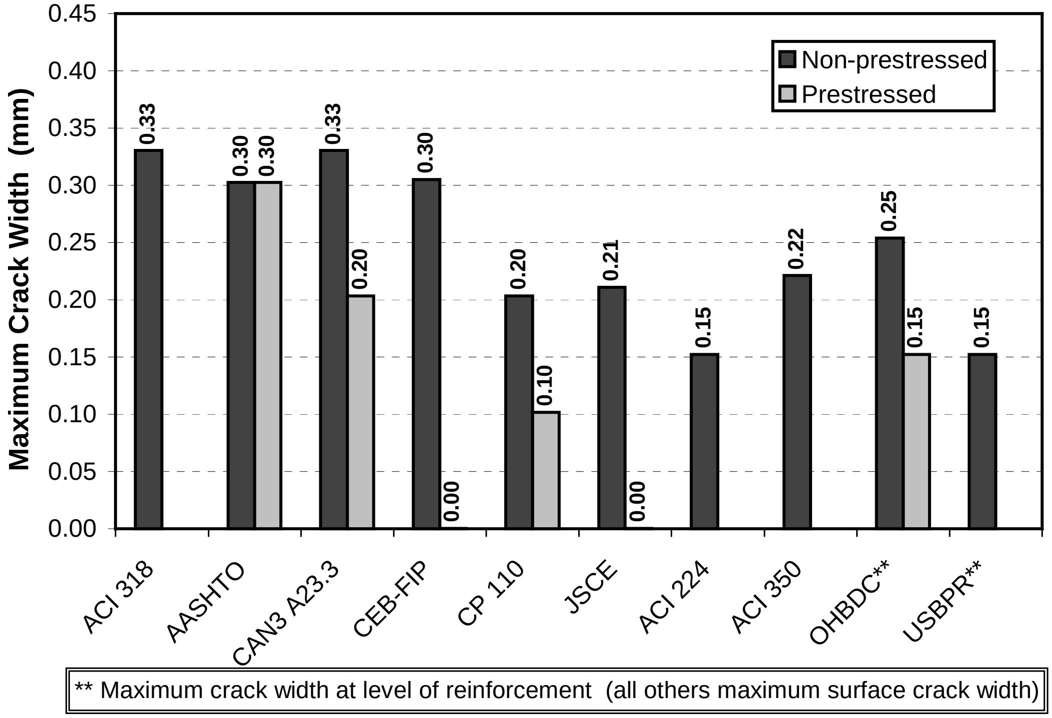


Figure 4.13 - Comparison of Allowable Crack Widths: Severe Exposure

Often it is argued that crack width criteria should not be based on surface crack widths. The foundation of this argument is that most surface crack width calculation formulas are a direct function of concrete cover. Thus, the calculated surface crack width could be reduced by reducing concrete cover, producing conditions less desirable from a durability viewpoint. For example, a member with a large cover may have surface crack widths exceeding the recommended limits but still perform satisfactorily in a corrosive environment while a member with crack widths within recommended limits but with only minimal cover may corrode severely. The *AASHTO LRFD Specifications* recognize the benefit of increased cover by applying an upper limit on the value of cover thickness to be used in the crack control calculations (the actual cover may of course exceed this value). In this manner, the increased cover is not penalized by the crack control provisions. Considering the conflicting interests of surface crack width and concrete cover, it appears that the approaches specified by the Ontario Highway Bridge Design Code,[A.6] British Standard CP 110,[A.9] JSCE Standard[A.11] and the U.S. Bureau of Public Roads[A.18] provide a better approach to controlling crack widths by specifying crack widths at the level of the reinforcement or by specifying surface crack widths limits as a function of concrete cover thickness.

Several of the codes and specifications reported in this summary treat cracking explicitly as a separate limit state. These include the CEB-FIP Model Code, British Standard CP110, Swiss Standard SIA 162[A.10] and the JSCE Standard. In each case, the provisions for crack control, whether through detailing or through crack width limits, have been placed in a separate section or chapter. Similarly, the Ontario Highway Bridge Design Code lists cracking as a serviceability limit state and provides a section on control of cracking. It is assumed that the writers of each of these specifications have deemed cracking and crack control significant enough to warrant a separate limit state. This approach is in contrast to the major North American codes (with the exception of the OHBDC) where crack control is addressed implicitly in the sections on flexural design. The reasons for this contrast are uncertain. Possibilities include differences in design philosophy, varying opinions on the importance of cracking or simply a lack of attention.

The majority of code documents and technical committee recommendations regarding corrosion protection of steel reinforcement attempt to de-emphasize the importance of crack widths. Reasons for this de-emphasis on crack width are normally given as uncertainty regarding critical crack widths and uncertainty in the calculation of crack widths. Instead, emphasis is placed on good quality concrete, increased concrete cover, careful placement and construction and good detailing. In most cases however, only limited information is given related to concrete quality and placement, and the only quantitative requirements are related to reinforcement detailing (number, size and spacing of bars, concrete cover, etc.). Investigation into the basis of the detailing requirements frequently reveals allowable crack widths as a source. Thus, the state-of-the practice related to corrosion protection appears to be historically founded on allowable crack widths.

4.4.2 Experimental Studies: Cracking and Corrosion

Section A.2 of the cracking and corrosion literature review (Appendix A) summarized a considerable number of corrosion research studies where the influence of cracking was specifically addressed. The literature review focused on the conclusions regarding cracking and corrosion, and in particular if a critical crack width for corrosion was reported. Any factors contributing to the reported conclusions were summarized. The review was split into three areas: short-term corrosion studies of reinforced concrete, long-term reinforced concrete studies and studies investigating prestressed concrete. The prestressed concrete studies were not separated into short and long term since the few available references were all considered short-term studies. The reader is referred to Appendix A for more detail and for all references to reviewed documents.

The exposure duration and numbers of specimens for all reviewed research are summarized in Figure 4.14. What is immediately obvious in this figure is that a large portion of the experimental work on the influence of cracking on corrosion has considered relatively short exposure periods. In addition, the amount of research on prestressed concrete is very limited in comparison to nonprestressed concrete.

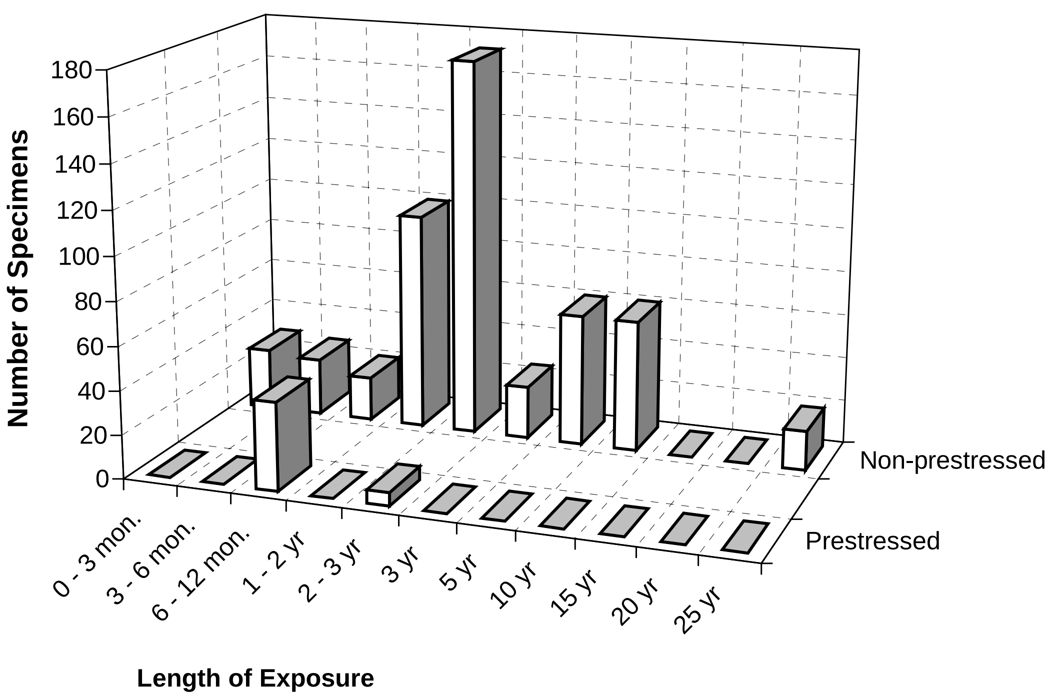


Figure 4.14 - Summary of Corrosion Studies Considering Crack Width

A summary of results from the reviewed crack width studies for reinforced (nonprestressed) concrete is provided in Table 4.2 and Table 4.3. In general, results from the various studies do not draw the same conclusions regarding the influence of crack widths on corrosion of reinforcing steels. Many researchers did not report a critical crack width below which corrosion will not occur. For the researchers who did report a critical crack width, the range of compiled results was 0.1 mm to 0.5 mm (0.004 in. to 0.020 in.), as shown in the two tables. It is surmised that the broad range of critical crack widths could be attributed to the nature of cracking in reinforced concrete and the wide variation in test methods employed by the various researchers. However, it is interesting to note that the compiled range of critical crack widths is approximately equal to the range of allowable crack widths recommended by the various code bodies and technical committees reported in Section A.1 and shown in Figure 4.12 and Figure 4.13.

Table 4.2 - Summary of Short-term Crack Width Studies – Reinforced Concrete

Investigator	Crack Width	Conclusions
Kahhaleh, K.Z.[A.24]	n/a	no specific information on critical crack widths
Houston, Atimtay, & Ferguson[A.25]	0.13 mm (0.005 in.)	Critical crack width for normal cover thickness
Vennesland & Gjorv[A.26]	0.4 to 0.5 mm (0.016 in. to 0.020 in.)	critical crack width (under sea water)
Lin, C.Y.[A.27]	n/a	no conclusive remarks about critical crack widths
Makita, Mori, & Katawaki[A.28]	n/a	no conclusions about critical crack widths
Misra & Uomoto[A.29]	0.5 mm (0.020 in.)	crack width limit beyond which crack propagation may occur
Okada & Miyagawa[A.30]	0.1 to 0.2 mm (0.004 in. to 0.008 in.)	critical crack width
Swamy, R.N.[A.31]	0.10 to 0.15 mm (0.004 in. to 0.006 in.)	critical crack width
Berke, Dalliare, Hicks & Hoopes[A.32]	0.2 mm (0.008 in.)	critical crack width
Schiessl and Raupach[A.33]	n/a	crack width has little influence on corrosion

When the duration of the testing program is considered by comparing the results in Table 4.2 and Table 4.3, an interesting trend is apparent. In general, short-term tests seem to indicate that some relationship does exist between crack width and corrosion rates while longer-term tests seem to discount the role of crack widths on the overall durability of specimens subjected to aggressive environments. However, in both short and long-term exposure tests, concrete cover seems to be a very significant parameter. In some studies it is argued that smaller covers cannot absorb aggressive agents quite as well as larger covers, and thus chlorides and other aggressive media can more easily permeate the cover and attack the reinforcing steel. Others argue that the cover is primarily important in the prevention of concrete spalling.

The reported research is almost exclusively for nonprestressed concrete members. A summary of the reviewed research on prestressed concrete is provided in Table 4.4. All of these studies are considered short term. Of the reviewed studies of prestressed concrete, only Poston[A.44] did a comparison between the performance of nonprestressed and prestressed members. Poston showed that in short-term exposure tests, prestressing to reduce crack widths showed a marked improvement in prevention of corrosion. However, only two crack widths and two levels of prestressing (zero and 100%) were considered. The other two reviewed studies dealt with pretensioned concrete members. From a corrosion standpoint, post-tensioned concrete has traditionally been deemed more desirable due to the added levels of protection provided by the duct and grout. However, the studies by Moore et al[A.45] and Perenchio et al[A.46] both indicated that corrosion was significantly limited when the pretensioned members were uncracked. These results suggest that good performance may be obtained when pretensioning is used to produce "crack-free" concrete. However, since companion nonprestressed members were not used, it is difficult to quantify this benefit. Also, the exposure duration was short in both studies, and it is possible that the findings of these studies may be less significant if the exposure period was longer. The lack of previous research in this area and the potential for improved durability reported by Poston suggest that this topic is worthy of further research.

Table 4.3 - Summary of Long-term Crack Width Studies — Reinforced Concrete

Investigator	Crack Width	Conclusions
Tremper, B.[A.34]	n/a	cracks within range of study did not promote serious corrosion
Ohta, T.[A.35]	n/a	no conclusive crack width value
Francois & Arliguie[A.36]	n/a	no conclusive crack width value
O'Neil, E.F.[A.37]	0.4 mm (0.016 in.)	critical crack width
Schiessl, P.[A.38,A.39]	n/a	no critical crack width given – does not support the idea of a critical crack width
Tuutti, K.[A.40]	n/a	cracks have only a local effect and do not change corrosion mechanisms
Beeby, A.W.[A.41,A.42,A.43]	n/a	crack widths have little long-term influence – current guidelines unnecessary

Table 4.4 - Summary of Crack Width Studies — Prestressed Concrete

Investigator	Crack Width	Conclusions
Poston, R.W.[A.45]	0.05 mm (0.002 in.)	prestressed specimens: no corrosion of nonprestressed reinforcement
	0.38 mm (0.015 in.)	corrosion present in both prestressed and nonprestressed specimens
Moore, Klodt & Hansen[A.46]	0.1 mm (0.004 in.)	smallest cracks for which pitting corrosion was observed
Perenchio, Fraczek and Pfiefer[A.47]	n/a	severe corrosion was found in both cracked and uncracked concrete

Only one of the reviewed research studies investigated the effect of crack spacing and number of cracks. Schiessl and Raupach[A.33] concluded that increased crack spacing would increase corrosion rates at the crack locations, resulting in more metal loss. The use of prestressing may increase crack spacing in comparison to reinforced concrete elements designed for the same loading. The consequence of this increased spacing needs to be investigated.

In most cases, the research reported in this literature summary may have a major shortcoming that could influence the general results and conclusions of the research. Much of the work was performed prior to the development and use of high performance concrete and other corrosion protection measures such as reinforcement coatings and corrosion inhibitors. In many cases, the concrete used in the reported research would be considered poor by current criteria, and far from high performance. It is possible that the effect of cracking on corrosion for the conditions of the reported research could be very different from the results obtained if high performance concrete or a corrosion inhibitor was used.

4.4.3 *Discussion: Cracking and Corrosion Literature Review*

Looking at the findings of the literature review in the context of the research described in this report, several important observations can be made. Most of these observations identify areas where research is needed.

4.4.3.1 Design Codes

- The crack control provisions for almost all design codes were developed for reinforced concrete members. In some cases these provisions have been applied to prestressed concrete members designed to be cracked at service load levels (sometimes referred to as partial prestressing). The applicability of these provisions to prestressed concrete needs to be determined.
- The basis for crack control provisions (or distribution of reinforcement provisions) in most design codes is allowable surface crack widths. The discussion in Section 4.4.1 suggests that this approach may not be the best.
- The design code crack control provisions do not consider overall cracking behavior in terms of crack width and crack spacing. Both factors, and in particular crack spacing, could be significantly different for prestressed concrete components, and both could influence corrosion protection.

4.4.3.2 Research

- Considering the findings of the reviewed research, it is apparent that emphasis should not be placed on the concept of a critical surface crack width for corrosion. The wide range of reported critical crack widths and the inherent variability in cracking behavior make the critical crack width concept impractical. A more general approach considering other factors including cover and crack spacing needs to be investigated.
- All but one of the reviewed research programs focused on crack widths, and did not consider crack numbers and spacing. When crack spacing was addressed, it was found to have a significant influence on corrosion rates at crack locations. The influence of crack spacing needs further investigation.
- The effect of prestressing on cracking, in comparison to reinforced concrete members, was not addressed sufficiently. Issues including the effect of prestressing on crack widths, crack spacing and corrosion for prestressed concrete and reinforced concrete members designed for the same loading have not been investigated.
- The concrete used in the reviewed research would be considered adequate or often sub-standard by modern standards. The research did not address the influence of high strength and/or low permeability concrete on the issue of cracking and corrosion. High strength concrete could affect crack patterns and widths, while low permeability could have a significant influence on corrosion mechanisms and corrosion related deterioration in cracked concrete.

The observations listed above reveal several aspects of cracking and corrosion that need investigation in the context of modern prestressed concrete structures with high performance concrete. Many of these aspects will certainly have an effect on the significance of cracking on corrosion. Returning to the two points of view described in the beginning of this section, it is possible that prestressing and high performance concrete could change these opinions. If the time to corrosion initiation in uncracked concrete is increased by various measures, then the early onset of corrosion at crack locations becomes increasingly significant in controlling the service life of the structure (i.e., if the time between t_{icr} and t_i becomes large). This hypothesis would imply that the effect of cracking could indeed be significant.

4.4.4 *Final Thoughts on Cracking and Corrosion*

In some ways, the two opinions presented at the beginning of Section 4.4 and shown in Figure 4.10 and Figure 4.11 have been based on an oversimplified or limited view of the effects of cracking on corrosion. Additional factors can influence the significance of cracking on corrosion. Some of these factors are discussed in this section.

4.4.4.1 Effect of Cracking on Stages of Corrosion Process

Returning to the concept of the stages of corrosion (Section 4.2.2), the total service life of a structure, or the time until repairs are needed, is determined by the length of time to corrosion initiation, L_i, and the length of the corrosion propagation period, L_p (time from corrosion initiation to the point at which unacceptable damage has occurred). It is generally accepted that cracking will affect the time to corrosion, L_i, at least in the vicinity of the crack. What is less clear is the effect of cracking on the propagation period, L_p. The length of the propagation period is a function of the corrosion rate and the level of unacceptable damage. The effect of cracking on corrosion rate has not been well defined, largely due to the large number of factors involved and the inherent variability of cracking. Analytical work, such as that reported by Schiessl and Raupach,[20] has shown that corrosion rate at a crack is a function of the crack spacing. However, this work was not confirmed experimentally, and it is difficult to make any generalizations regarding the rate of corrosion as a function of crack width and spacing.

Defining the level of unacceptable damage is another aspect that is often oversimplified. Many times the results of simple experimental studies are extrapolated to structures, with debatable results. Beeby[24] defines two criteria for assigning the level of unacceptable corrosion damage: loss of section and disruption of concrete. Loss of section refers to conditions where corrosion has reduced the reinforcement cross-section to a level at which the capacity of the structure is affected. Disruption of concrete refers to the condition of more general corrosion leading to extensive cracking and spalling of the concrete cover. This form of damage is unsightly and can present hazardous conditions for pedestrians and vehicles. It can also lead to increased corrosion rates (up to ten times[24]) as the steel protection from the concrete diminishes. Disruption of concrete is a function of the length of steel over which corrosion is occurring, bar diameter, cover and concrete strength. It is generally assumed that disruption of concrete will be the controlling factor in reinforced concrete structures.[24] However, corrosion models are frequently based on loss of section since they have been calibrated from small-scale test specimens.[28] In some cases, loss of section will be the controlling factor in a structure. Conditions of highly localized corrosion, particularly in prestressed structures (see Section 4.3), could make loss of section the controlling criteria. The collapse of the Ynys-y-Gwas bridge in Wales is an example of highly localized corrosion leading to failure.[50]

4.4.4.2 Number of Cracks

One final factor that has been largely overlooked is the extent of cracking in terms of the number of cracks and crack spacing. This issue has not been fully investigated experimentally or theoretically, and has not been incorporated into any corrosion models. The reason for this omission is that most corrosion in concrete research has considered reinforced concrete where cracks will be frequent and well distributed. The extent of cracking could be significantly different for prestressed structures, or for structures with high performance concrete.

4.4.4.3 Summary

The effects of cracking can be summarized in general terms as follows:

- Cracks may significantly reduce the time to corrosion initiation in comparison to sound, good quality concrete. This effect is relatively insensitive to crack width.
- Cracks may affect the corrosion rate. However, no general conclusions can be made on the rate of corrosion as a function of crack width. One research program suggested corrosion rate is a function

of crack spacing, not width. It is generally assumed corrosion rates will be higher in cracked concrete in comparison to sound, good quality concrete.

- The level of unacceptable damage in a structure can be considered at a local scale (loss of section) or a global scale (disruption of concrete). Different situations may exist where one or the other form of corrosion damage may control.

In order to fully assess the effect of cracking, both the local and global effects of cracking on corrosion should be considered. This approach is also suited to the two definitions of unacceptable damage. The effect of cracking can be illustrated by subdividing the structure to consider local and global effects, and by using the two stages of corrosion concept. For example, consider the two concrete beams shown in Figure 4.15. The beams have been subdivided into five sections for consideration. It is assumed that the details and loading are such that Beam 1 has three cracks, while Beam 2 has one crack. Based on the literature review described in the preceding sections, the simplifying assumption that crack width has little effect on the time to corrosion and corrosion rate is made. The length of time to corrosion is assumed to be $L_{i,cr}$ and $L_{i,uncr}$ for cracked and uncracked concrete, respectively, with $L_{i,uncr}$ greater than $L_{i,cr}$. The corrosion rates after initiation are assumed to be R_{cr} and R_{uncr}, with R_{cr} higher than R_{uncr}. It is assumed the corrosion rate is constant over time.

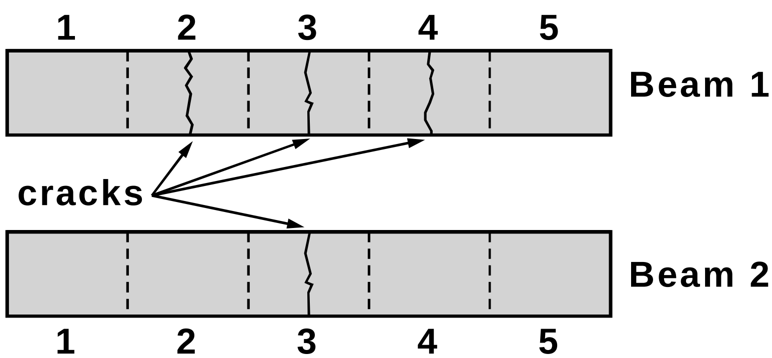


Figure 4.15 - Beams for Effect of Cracking Illustration

A plot of corrosion damage over time is made for each of the five subdivisions on the beams, as shown in Figure 4.16 and Figure 4.17. For Beam 1, the curves for Locations 1 and 5 are the same, and indicate corrosion in uncracked concrete. The curves for Locations 2, 3 and 4 are identical, and represent corrosion in cracked concrete. For Beam 2, Locations 1, 2, 4, and 5 are uncracked, and Location 3 is cracked. Arbitrary units have been used in the plots, and both figures are plotted to the same scale. The total corrosion damage in a beam is assessed as the cumulative plot of the five curves for each beam. The limit of unacceptable damage is defined in two ways:

1. **Criteria 1:** The corrosion at a given location in the member exceeds a critical value (i.e., loss of section controls).
2. **Criteria 2:** The cumulative corrosion damage for the member/structure exceeds a critical value (i.e., disruption of concrete controls).

Values have been assumed for $L_{i,cr}$, $L_{i,uncr}$, R_{cr}, R_{uncr} and the two damage criteria, considering the assumptions made earlier. It is not intended to quantify or recommend values for the variables in this demonstration, although the relative relationships between cracked and uncracked concrete (i.e., $L_{i,cr} < L_{i,uncr}$, etc.) have been assumed based on reported observations.

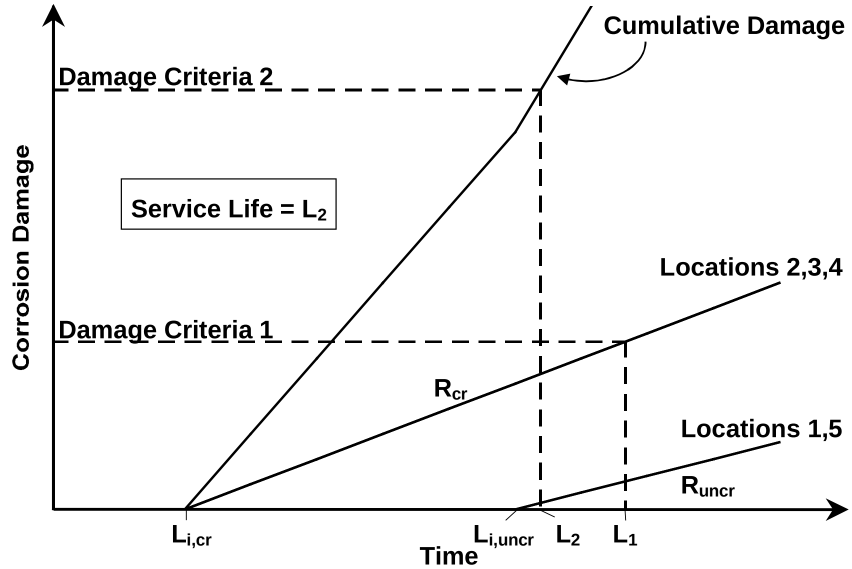


Figure 4.16 - Corrosion Damage Plot for Beam 1

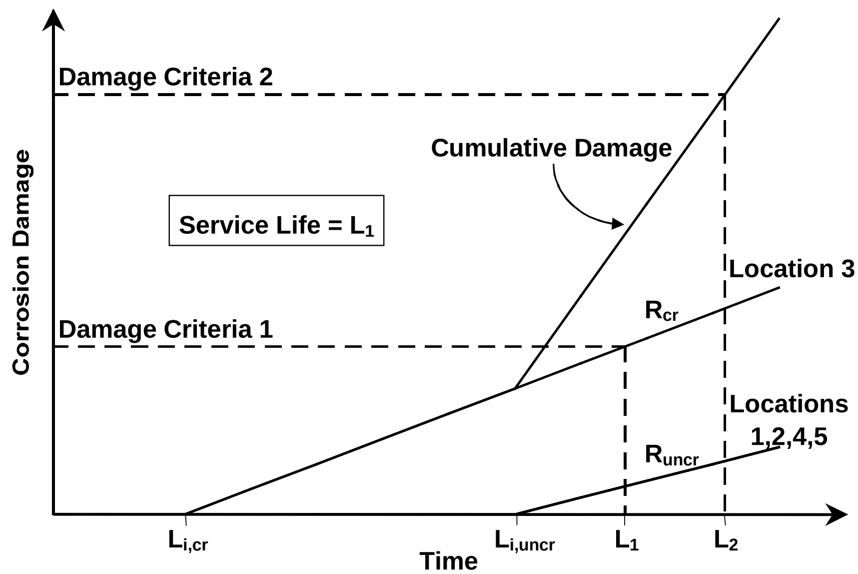


Figure 4.17 - Corrosion Damage Plot for Beam 2

Several observations can be made from the plots for Beam 1 and Beam 2:

- The service life for Beam 1 is dictated by Criteria 2, disruption of concrete. Criteria 1, loss of section, controls for Beam 2.
- The service life for Beam 1 is shorter than the service life for Beam 2, suggesting more cracks could lead to more corrosion damage in a shorter duration.
- When cracking is limited, as in the case of Beam 2, the controlling damage criteria may be loss of section.

- Using prestressing to limit the number of cracks and increase crack spacing could increase the service life. It may also change the corrosion damage criteria from overall disruption of concrete to localized loss of reinforcement section.

- If the concrete quality is improved in terms of lowered permeability and increased resistivity, or if the cover thickness is increased, it would be logical to assume the time to corrosion initiation in uncracked concrete ($L_{i,uncr}$) could be increased significantly, and the corrosion rate (R_{uncr}) would be decreased. Looking at Figure 4.16, it is clear that this combined effect could increase the service life of the structure. However, in the case of limited cracking in Beam 2 (Figure 4.17), these improvements would not increase the service life since it is limited by a localized loss of section at the crack locations.

The preceding discussion is a very simplified look at the effect of cracking on corrosion, yet it addresses important factors that have frequently been overlooked in the past. Many assumptions have been made for the two specific cases shown in Figure 4.15. Many other situations and the effect of other variables could be considered/illustrated in the same manner. The intention of this discussion is not to draw conclusions on corrosion rates, times to corrosion and the effect of cracking. Rather, the purpose of this exercise is to illustrate that the effect of cracking could be important for some situations, and general conclusions stating that the effect of cracking on corrosion is significant or is not significant should not be made since many factors are involved.

Chapter 5:
Corrosion Protection for Post-Tensioned Concrete Structures

Many aspects of corrosion protection for post-tensioned concrete structures are similar to corrosion protection for conventional reinforced concrete structures. The basics of corrosion protection for reinforced concrete structures are provided by many sources. Several excellent "primers" on the subject include "Corrosion of Metals in Concrete" (ACI 222R-96),[27] "Guide to Durable Concrete" (ACI 201.2R-92)[51] and "Durable Concrete Structures – CEB Design Guide."[17] Post-tensioned concrete introduces several extra variables in corrosion protection, including additional hardware and grouting of the tendon ducts. These aspects are the focus of this section. Some general aspects of corrosion protection are briefly reviewed first.

Corrosion of steel reinforcement or prestressing steel in concrete is a complex phenomenon influenced by many factors. For this reason, it is best to approach corrosion protection by providing several measures to guard against different influences and breakdowns or limitations in any single protective measure. This strategy is often referred to as providing multilevel corrosion protection. Corrosion protection in post-tensioned concrete can take many forms. Four general categories of protection mechanisms are listed in Table 5.1.

Table 5.1 - Corrosion Protection Mechanisms and Methods

Mechanism of Protection	Protection Method	
Prevent chlorides from entering concrete	**1.) reduce permeability:** - low w/c ratio - use fly ash/silica fume/slag - eliminate voids (thorough compaction, reinforcement detailing) - moist or steam curing - reduce/control cracking	**2.) waterproof membranes** **3.) surface polymer impregnation** **4.) structural form** **5.) structural detailing (drainage, anchorage locations)** **6.) crack width limitations**
Prevent chlorides from reaching steel (in the event chlorides penetrate the concrete)	**1.) increased cover** **2.) coated reinforcement & tendons (epoxy-coated, copper clad, etc.)** **3.) encapsulated post-tensioning systems**	**4.) plastic post-tensioning ducts** **5.) cement grout (bonded post-tensioning)** **6.) greases and waxes (unbonded post-tensioning)**
Control corrosion reactions	**1.) galvanized reinforcement/tendons** **2.) electrical isolation**	**3.) corrosion inhibitors** **4.) cathodic protection**
Remove reactive substance (steel)	**1.) FRP reinforcement & tendons**	**2.) stainless steel reinforcement/tendons**

Corrosion protection requires attention during design and construction of the structure, and through the life of the structure with proper maintenance. This report is primarily concerned with the design aspects of corrosion protection for post-tensioned structures. The durability design aspects of corrosion protection can be grouped as follows:

- Overall structural form
- Structural design details

- Corrosion protection provided by concrete
- Post-tensioning system details

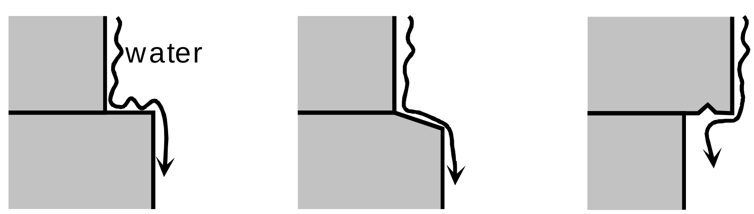
The first three categories are not unique to post-tensioned structures, and the concepts are similar to measures for conventional reinforced concrete structures. The final category of corrosion protection measures involves aspects unique to post-tensioned concrete and thus is the focus of this report. A brief discussion of the first three categories is followed by a more in-depth look at corrosion related aspects of the post-tensioning system. Both bonded and unbonded post-tensioning systems are considered, although emphasis is placed on bonded post-tensioning.

5.1 STRUCTURAL FORM

Corrosion must be considered at the conceptual design stage and may influence selection of the structural form. The layout and geometry of the structure determines which portions of the structure will be exposed to aggressive environmental conditions. In some cases, such as marine substructures, exposure to an aggressive environment is unavoidable. However, in other cases the structural form can have a large impact on durability. Because the most common cause of reinforcement corrosion is moisture-borne chlorides, whether as seawater or deicing chemicals, adequate drainage is a critical factor in the selection of structural form. Many design aspects that may influence corrosion and durability are revealed by examining the field performance of structures (see Chapter 7).

5.1.1 Drainage

Drainage is always considered in bridge superstructure design, and it should also be considered for the substructure components. Adequate superstructure drainage must be provided and must ensure chloride-laden water does not come in contact with the substructure. If drains are provided, they must be located such that they can readily be inspected, maintained and repaired. Horizontal surfaces in the substructure should be avoided by sloping wherever possible, as shown in Figure 5.1. Typical examples would include the top surface of bent caps and pile caps, and flanges of girders and inverted T-beams. Drainage and ventilation are important for box girders and hollow columns. Although the interior of these elements are not intended to be exposed to moisture, water accumulation can occur and lead to corrosion damage.



Poor **Better** **Better**

Figure 5.1 - Avoiding Horizontal Surfaces (adapted from Ref. 17)

5.1.2 Joints

A very common source of substructure corrosion problems is moisture and chlorides dripping onto substructure components through leaking deck joints, as shown in Figure 5.2. Corrosion damage can occur on bent caps and columns or piers. Sloping of the top surface of the bent cap as shown in Figure 5.3 could reduce the severity of corrosion damage. However, proper joint design and maintenance are better solutions when the severe conditions shown in Figure 5.2 are encountered. Another option is to reduce the number of expansion joints by making the bridge deck continuous for several spans.[52]

Figure 5.2 – Severe Substructure Corrosion Damage Due to Defective Expansion Joint

5.1.3 Splashing

Substructure components adjacent to roadways where deicing chemicals are used can be prone to corrosion damage due to splashing. Figure 5.4 shows a close-up of the bases of a column located very close to traffic. Splashing of chloride-laden moisture has caused extensive corrosion damage. Increasing the distance between the roadway and substructure may increase initial construction costs, but may reduce long-term costs by avoiding this type of corrosion damage.

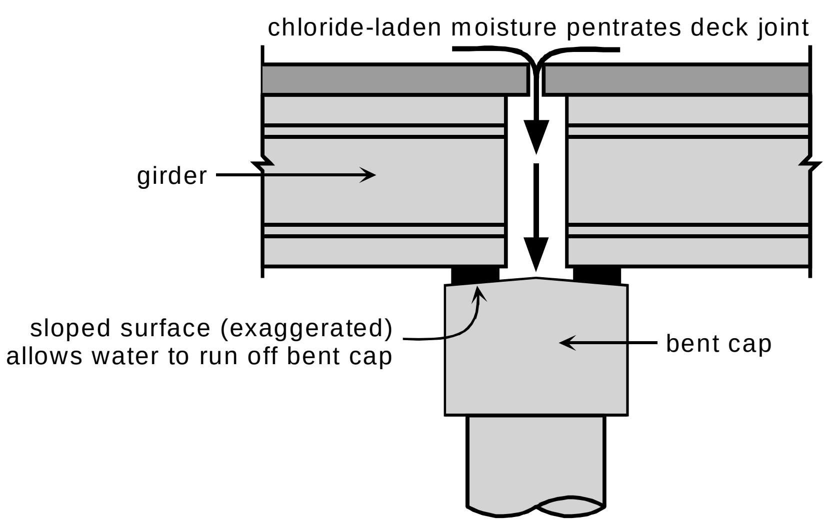


Figure 5.3 - Sloped Bent Cap to Promote Run-Off (adapted from Ref. 17)

Figure 5.4 - Column Corrosion Resulting from Splashing Adjacent to Roadway

5.1.4 Geometry

Other aspects of the structural geometry can influence durability. For example, consider the two substructures shown in Figure 5.5. Assuming the substructures are located in a marine saltwater environment, the single column/pier substructure (A) offers a more durable alternative than the multi-column or trestle pile substructure (B). The surface area to volume ratio is decreased in the large single column. In addition, increased concrete cover is more easily accommodated and the potential for spalling is reduced.

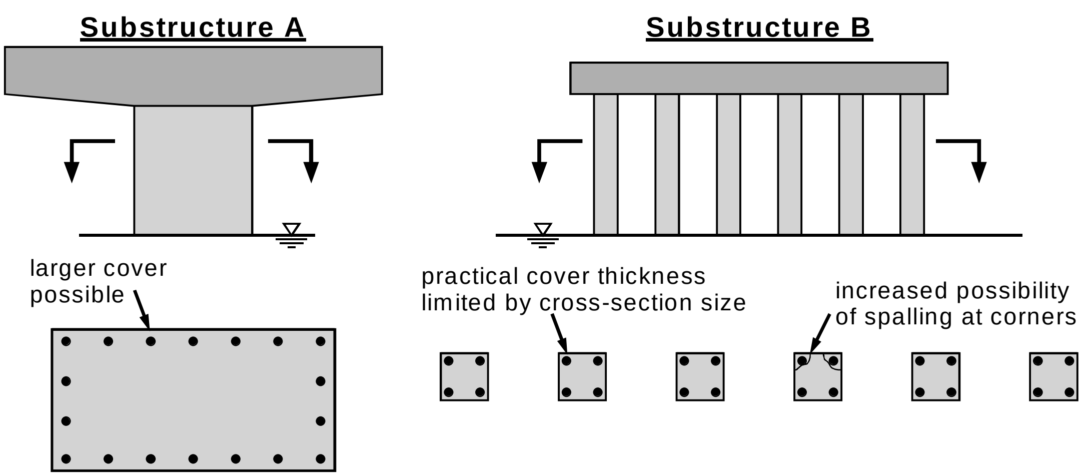


Figure 5.5 - Geometry Effects on Durability for Alternate Substructure Designs

5.2 STRUCTURAL DESIGN DETAILS

Many structural design details can influence the durability of the element and structure. Many of these design aspects do not necessarily influence the structural behavior of the member, and therefore should not have a negative effect on the design. Most design details that affect durability are related to the constructibility of the structure. In general, the quality of construction and durability will increase if the structure is easier to build.

5.2.1 Cracking

Concrete cracking provides easy access for moisture and chlorides to reach the reinforcement. Although the significance of cracking on corrosion is often debated (see Section 4.4), cracking should be avoided or minimized where possible. Attention must be paid to both intended cracking due to loading, and unintended cracking due to plastic shrinkage and settlement, drying shrinkage, thermal effects and differential settlement. Crack control under design loading is covered by most design codes. Guidance on control of other forms of cracking is provided in many references, including Ref. 17, 53 and 54.

5.2.2 Reinforcement Detailing

Reinforcement detailing not only affects cracking in the structure, it also can influence construction. Congested reinforcement details make concrete placement difficult and can lead to poor compaction and voids. Complicated details also can lead to construction errors in the reinforcement and insufficient cover.

5.2.3 Post-Tensioning Details

The location of post-tensioning anchorages plays a large role in corrosion protection for the anchorages. The common location at member ends or edges often exposes the anchorage region to moisture and chlorides as discussed in more detail in Section 5.4.5.

Details of the post-tensioning tendon profile must facilitate proper grouting. Provision must be made for appropriate vent locations. More detail is given in Section 5.4.4.

5.3 CONCRETE AS CORROSION PROTECTION

Concrete is the single most important factor in ensuring structural longevity since it affects all aspects of durability, including corrosion of reinforcement. Concrete acts as a physical barrier to moisture and chlorides, and provides the alkaline environment necessary for formation of the passive film on the surface of the steel. The effectiveness of concrete as corrosion protection for steel reinforcement is dependent on the concrete material properties and on design and construction practices. Many excellent sources of information on concrete as corrosion protection are available, including ACI 222R-96,[27] ACI 201.2R-92,[51] CEB Durable Concrete Structures Design Guide[17] and Neville.[35] These sources provide a solid background and many references on concrete as corrosion protection, addressing requirements for suitable material properties and design and construction related issues. In addition to the references listed above, a tremendous amount of research has been performed on this subject matter.

The corrosion protection provided by the concrete can be improved in many ways, including reduced permeability of concrete, increased clear cover, corrosion inhibitors and concrete surface treatments. Each is described below.

5.3.1 Concrete Permeability

The permeability of the concrete is the single most important factor for providing durable concrete, including the corrosion protection. The permeability of concrete controls the rate at which moisture, oxygen and carbon dioxide penetrates the concrete. Because the penetration of moisture provides the transport mechanism for chlorides and other aggressive substances, lowering the concrete permeability increases the length of time before aggressive agents reach the steel, and thus improves corrosion protection. The permeability of concrete is affected by four general factors:[35,55]

1. Pore structure of the cement paste
2. Aggregate
3. Voids in the concrete
4. Cracking of the concrete

5.3.1.1 Pore Structure of the Cement Paste

The transportation of water and gases through concrete is significantly affected by the size, distribution and continuity of pores within the cement paste. The cement paste is assumed to consist of C-S-H gel, void spaces or capillary pores and various products of the hydration process. The C-S-H gel is defined as the cohesive mass of hydrated cement, and consists primarily of calcium silicate hydrates (C-S-H). In addition to the capillary pores within the cement paste, the C-S-H gel also contains voids referred to as gel pores. These pores are the interstitial spaces between the gel particles.

Due to their nature, the gel pores are much smaller than the capillary pores. The size of the gel pores is in the range of approximately 15 to 20 Angstroms in diameter, or only about one order of magnitude larger than a molecule of water.[35] Because the gel pores constitute the space between gel particles, the size and distribution of the gel pores is characteristic of the type of cement and is relatively independent of the water-cement ratio (w/c ratio) and the extent of hydration. Thus, the porosity of the C-S-H gel is affected by the properties of the cement itself. In general terms, finer cements will produce gel with a lower porosity than coarse cements. In mature concrete, gel pores constitute approximately 28% of the volume of the C-S-H gel.[35] However, due to the small size of the gel pores and the fact that the pores are generally well distributed and discontinuous throughout the gel, the overall effect of the gel pores on the permeability of the concrete is minimal.

Capillary pores are the remnants of the water filled space that exists between the partially hydrated cement grains. The typical size range for capillary pores is 500 to 500,000 Angstroms.[17] Due to their larger size in comparison to gel pores, the permeability of the cement paste is primarily controlled by the capillary pores. The size and distribution of capillary pores is a function of the hydration of the cement. As the process of hydration proceeds, the water within the pores is replaced by solid hydration products (C-S-H gel) and the volume of capillary pores is reduced. In addition, the formation of C-S-H gel also tends to obstruct the capillary pores, causing them to become discontinuous and further reducing the permeability of the cement paste. Thus, the permeability of the cement paste due to capillary porosity is influenced by the factors affecting the hydration of the cement.

The capillary porosity is also significantly affected by the water content of the concrete. For lower water-cement ratios, the permeability of the cement paste may be considerably reduced due to the greater extent of C-S-H gel formation which reduces the volume and continuity of the capillary pores. In general, for water-cement ratios less than 0.38, the volume of C-S-H gel formed during hydration of the cement is enough to completely fill the capillary pores. For water-cement ratios between 0.38 and 0.6, the amount of C-S-H gel formation is usually significant enough to disrupt the continuity of the capillary pores, provided that complete hydration of the cement is allowed to occur. For water-cement ratios higher than 0.7, gel formation is insufficient to block the capillary pores even with complete hydration. Tests have shown that concrete permeability decreases by up to four orders of magnitude as the water cement ratio is reduced from 0.75 to 0.26.[56] The effect of water-cement ratio on chloride ion penetration was demonstrated by Clear,[57] as shown in Figure 5.6. ACI Committee 201[51] reports chloride ion permeability for concrete with water-cement ratios of 0.40 and 0.50 to be 400 to 600 percent higher than concrete with a water-cement ratio of 0.32. For concrete exposed to aggressive environments, Committee 201 recommends that the water-cement ratio should be as low as possible, and preferably below 0.40.

The use of mineral admixtures with pozzolanic characteristics influences both the gel and capillary porosity of the cement paste, and hence affects the permeability. The most commonly used mineral admixtures in North America are fly ash and silica fume. Considerable research[37,58,59,60,61,62,63,64,(and others)] has shown lower permeability and reduced penetration of chlorides for concrete containing fly ash and silica fume in comparison to ordinary Portland cement concrete.

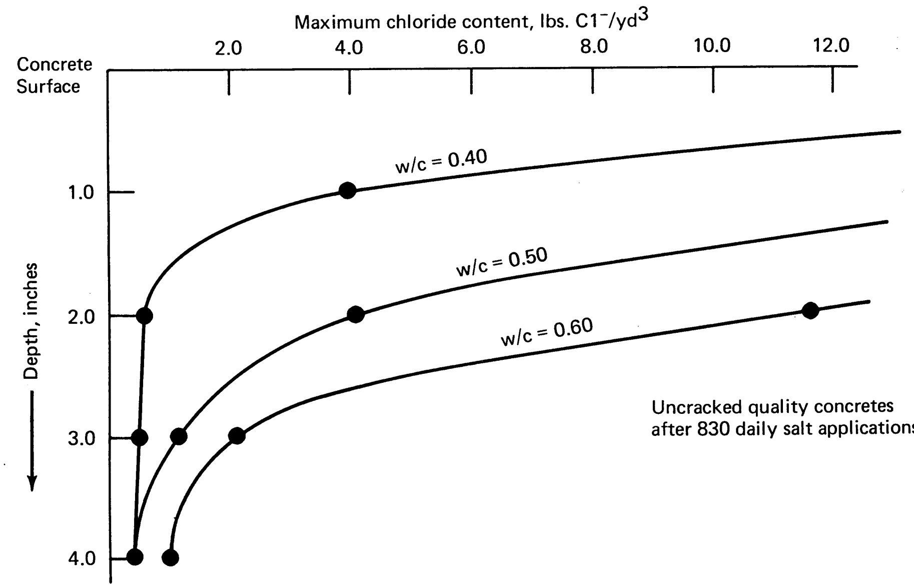


Figure 5.6 - Effect of Water-Cement Ratio on Chloride Ion Penetration[57]

Fly ash is normally used to replace a portion of the cement in concrete. Typical replacement amounts range from 20 to 35 percent by mass. Partial cement replacement with fly ash may reduce concrete permeability in three ways:[35,58]

- The hydration of a material with pozzolanic properties (i.e., the pozzolanic reaction) consumes calcium hydroxide ($Ca(OH)_2$) and produces additional or secondary calcium-silicate-hydrate (C-S-H) gel. *(The primary C-S-H formed by hydration of Portland cement comprises the largest portion of the hardened cement paste, and is primarily responsible for the strength of the paste. Calcium hydroxide is a by-product of cement hydration. It is soluble, and may be leached out of the concrete, leaving a network of pores that increases permeability.)* The pozzolanic reaction increases the strength and reduces the permeability of the paste by removing soluble $Ca(OH)_2$ and filling capillary pores with C-S-H.
- Fly ash particles have a spherical shape, and thus have lower interparticle friction than the angular cement particles. Therefore, less water is needed to attain a given slump, leading to fewer capillary pores. Typical water reduction ranges from 5 to 15 percent in comparison to concrete with Portland cement only (similar total amount of cementitious material).[35]
- The lower interparticle friction of fly ash particles also reduces permeability through better consolidation or "packing" of the fly ash and cement particles. This improved consolidation tends to reduce entrapped air and the volume of large capillary pores.

Silica fume may be used to replace a portion of the cement, or it may simply be added to the concrete to increase the total cementitious content. Silica fume also possesses pozzolanic properties, reducing permeability through production of additional C-S-H gel and removal of $Ca(OH)_2$. Silica fume particles are spherical, but are ten to one thousand times smaller than fly ash and cement particles.[35] Due to their small size and large surface area, silica fume normally increases water demand and superplasticizer is needed to maintain low water-cement ratios. The small particle size is beneficial though, contributing to lower permeability in three ways:[35,58]

- The small particle size and increased surface area limits bleedwater. Permeability is reduced by eliminating voids resulting from bleedwater trapped under aggregate.
- The hydration of cement is enhanced as the very small silica fume particles provide nucleation sites for calcium hydroxide. $Ca(OH)_2$ forms in numerous small crystals rather than large crystals.
- The very small spherical particles improve “packing” of silica fume and cement particles, filling the voids in the transition zone between cement particles and aggregate.

The importance of complete cement hydration on the permeability of the concrete is evident from the preceding paragraphs. In general, steam or moist cured concrete will have a lower permeability due to the potential for thorough hydration. Concrete that is allowed to dry prematurely may have a significantly higher permeability due to the lesser extent of hydration. In addition, drying may produce shrinkage cracks through the C-S-H gel separating capillary pores, thus rendering them once again continuous. Proper curing is particularly important when mineral admixtures such as fly ash and silica fume are used.[65,66] Guidelines for curing are provided by the ACI Committee 308 report “Standard Practice for Curing Concrete,”[67] and Ref. 17.

5.3.1.2 Aggregate

The permeability of concrete is also affected by the properties of the aggregate, although to a lesser extent than water-cement ratio.[35] The influence of the aggregate is usually small in comparison to the cement paste since the aggregate is completely surrounded by the paste. For aggregates with very low permeability, the permeability of the concrete may be reduced due to the longer flow path required for moisture to circumvent the aggregate.

5.3.1.3 Voids in the Concrete

Consolidation or compaction of concrete is also necessary for low permeability. Voids or excessive entrapped air resulting from poor placing practices, lack of vibration or congested reinforcement will increase permeability. The effect of inadequate consolidation on chloride penetration is illustrated in Figure 5.7. In some situations, slightly larger sections may be required to relieve reinforcement congestion and facilitate thorough placement of the concrete.

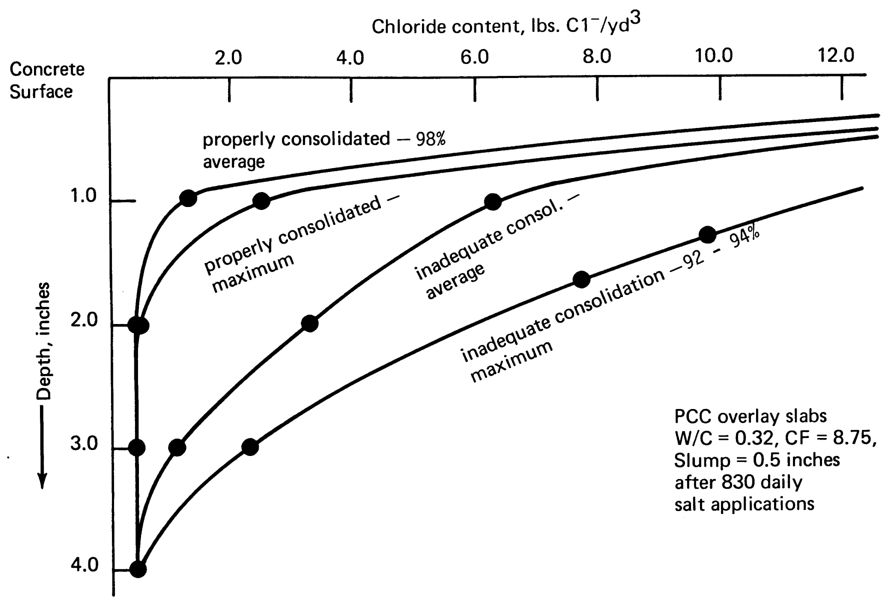


Figure 5.7 – Effect of Consolidation on Chloride Ion Penetration[57]

5.3.1.4 Cracking of Concrete

Cracks within concrete will have the obvious effect of increasing the permeability of concrete by providing direct routes for the movement of moisture. In order to ensure low permeability, special attention must be given to the prevention of cracks due to sources such as creep, shrinkage, temperature effects, plastic settlement during curing, load effects, either due to design loads or overloads, and imposed deformations due to support settlement. In severe exposure conditions, the use of prestressing to reduce the cracking resulting from normal reinforced concrete behavior is a possible approach for maintaining low permeability of concrete. Crack control was discussed previously in Section 4.4 and 5.2.

5.3.2 Concrete Cover Thickness

The thickness of concrete cover over the reinforcement plays a significant role in corrosion protection. Increased clear cover provides improved protection for the steel, particularly if low permeability concrete is used. The penetration of chlorides over time can be approximated by a square-root time law.[17] For example, if the concrete cover thickness is doubled, it will take approximately four times as long for chlorides to penetrate to the depth of the reinforcement.

5.3.3 Corrosion Inhibitors

Corrosion inhibitors are admixtures that may be added to the concrete to slow or interfere with the electrochemical reactions of corrosion. Many different corrosion inhibitors have been developed and investigated with mixed results. A brief summary is provided in Ref. 27. Other references include 68, 69, 70 and 71. Most corrosion inhibitors used in concrete are described as barrier layer formers.[68] These inhibitors tend to deposit on or around the steel surface to inhibit the corrosion reactions. Some work by forming an impermeable barrier on the surface of the steel. Others work to passivate the steel, preventing corrosion.

One of the more commonly used inhibitors in concrete is calcium nitrite ($Ca(NO_2)_2$). This compound reacts with ferrous ions produced during corrosion to stabilize the passive layer on the steel and prevent further corrosion. This corrosion inhibitor does not affect the time to corrosion initiation, but rather limits the corrosion rate after corrosion has started by re-passivating the steel. The calcium nitrite is consumed during the process, and therefore the corrosion protection is not indefinite. Neville[35] states “inhibitors are no substitute for concrete of low penetrability: they are merely an additional safeguard.”

5.3.4 Concrete Surface Treatments

Concrete surface treatments work to improve corrosion protection by preventing moisture and chlorides from entering the concrete. Surface treatments include waterproof membranes and surface polymer impregnation. In the latter, the exposed surface of the concrete is impregnated with polymer that fills the voids and cracks in the concrete, providing a barrier with very low permeability. Various overlays may be used to provide a low permeability barrier over existing concrete. Options include polymer concrete overlays, latex-modified concrete overlays and overlays with low permeability Portland cement concrete. Overlays are most practical on horizontal surfaces such as bridge decks. Additional information is provided in Ref. 27.

5.4 BONDED POST-TENSIONING SYSTEM DETAILS

The most common forms of bonded post-tensioned construction are multiple strand tendons and bonded prestressing bars. Corrosion protection for the bonded post-tensioning system itself consists of several components including the duct, grout and anchorage protection. Coatings for the prestressing steel can also provide an additional layer of corrosion protection. Epoxy-coated and galvanized prestressing steels are both options in bonded post-tensioned systems. Aspects of the corrosion protection system for bonded post-tensioning are described below.

5.4.1 Post-Tensioning Tendon Materials Selection

The selection of the type of material for prestressing tendons can be dictated by both structural and durability concerns. This section describes some of the choices for prestressing tendon materials when corrosion is a concern.

The common forms of prestressing steel used in North America are high strength prestressing wire, seven-wire prestressing strand and high strength prestressing bars. The selection of prestressing wire, strand or bar for a particular application is dependent primarily on structural requirements and construction considerations rather than durability.

The permissible materials and manufacturing processes for prestressing steels used in North American structures are dictated by standard specifications and code requirements such as *AASHTO LRFD Design*[3] *and Construction*[72] *Specifications,* ACI 301[73] and ACI 318.[47] These specifications require prestressing strand, wire and bars to conform to ASTM standards A416,[74] A421[75] and A722,[76] respectively. In some countries, quenched and tempered steels have been used for prestressing. This manufacturing process can leave the steel more susceptible to stress corrosion cracking than cold drawn prestressing steels. The material requirements of AASHTO and ACI preclude the use of quenched and tempered steel for prestressing, and thus only cold drawn prestressing wire or hot-rolled bar are used in North America.

Metallic and non-metallic coatings have been investigated as protection methods for mild steel reinforcement.[27,77] The most common and widely used are epoxy coating and zinc galvanizing. Other coatings that have shown good results in laboratory testing include stainless steel-clad bars, copper-clad bars and zinc alloy-clad bars.[77] Some coatings have been considered for prestressing steels.[48,68] Suitable coatings for prestressing steel must possess several important properties that often make technology applied to mild steel reinforcement unusable.[48] For example, the coating must not have an adverse effect on the strength or ductility of the steel, must have sufficient flexibility and ductility to withstand stranding during manufacture and elongation during stressing without cracking or spalling. Coatings also should not have a detrimental effect on bond between the steel and concrete and should be able to withstand handling and placement without damage. Finally, the improvement in corrosion protection provided by the coating should not be at a prohibitive cost. At present, only epoxy coatings and zinc galvanizing have been successfully applied to prestressing steels. Each is discussed in this section.

The development of advanced composite materials or fiber reinforced plastics has produced an additional choice in the selection of prestressing tendons. These materials are non-corroding but require special design considerations. They are also discussed in this section.

5.4.1.1 Epoxy Coatings for Prestressing Steel

Epoxy coating is a widely used organic coating for corrosion protection that isolates the steel from contact with oxygen, moisture and chlorides. Epoxy-coated seven-wire prestressing strand and threaded prestressing bars are widely available in the U.S.

Epoxy-Coated Strand

Epoxy-coated strand is available in two configurations: coated, and coated and filled. The two configurations are shown in Figure 5.8. In the coated configuration, a thick epoxy coating is provided around the exterior circumference of the seven-wire strand. In the coated and filled version, the interstices between the individual wires are filled with epoxy in addition to the external coating. By filling the interstices with epoxy, migration of moisture and chlorides along the strand interstices is prevented. Both configurations of epoxy-coated strand are available either with a smooth surface or with grit particles embedded on the surface to improve bond transfer characteristics. The smooth surface epoxy-coated strand is intended for use in applications where bond is not critical, such as unbonded post-tensioning systems, external post-tensioning systems and stay cables. When used in unbonded systems, the strand must still be encased in a duct as the smooth epoxy coating is not a replacement for the sheathing used in monostrand post-tensioning systems. The grit impregnated epoxy-coated strand is intended for used in bonded post-tensioning systems and in pretensioned applications. Epoxy-coated strand is available in conventional sizes of 10 to 16 mm (3/8 in. to 0.6 in.) diameter at Grade 1860 (270 ksi). Details of installation and stressing procedures are provided in a PCI report on the use of epoxy-coated strand.[78]

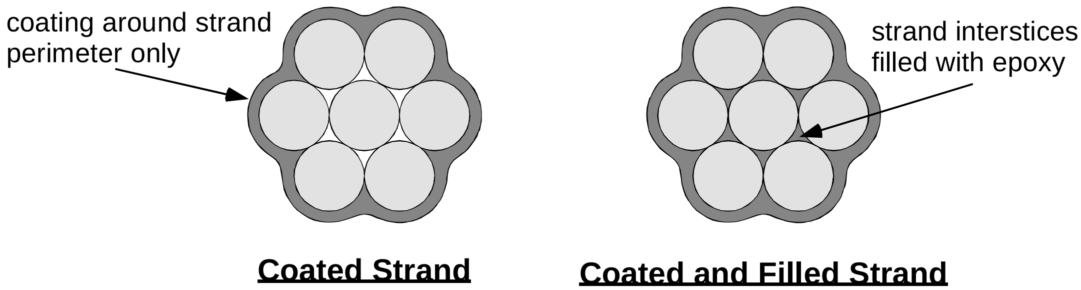


Figure 5.8 - Epoxy Coated Strand Types

Epoxy-coated strand is manufactured to meet the requirements of ASTM A882-92, "Standard Specification for Epoxy-coated Seven-Wire Prestressing Steel Strand"[79] and ASTM A416, "Standard Specification for Seven-Wire Prestressing Steel Strand."[74] The physical properties of the epoxy coating used for prestressing strand are significantly different from those used to coat mild steel reinforcement. The epoxy coating developed for prestressing strand is very tough and ductile with good bond to the steel to withstand the elongation during stressing. The coating is also durable and abrasion resistant to minimize damage during handling, placement and stressing. The design final coating thickness for the strand is usually 0.76 mm (0.03 in.),[80] although the thickness can range from 0.63 to 1.14 mm (0.025 to 0.045 in.) according to ASTM A882.[79] This strand coating thickness is considerably thicker than the coating thickness for mild steel reinforcement (0.18 to 0.30 mm (0.007 to 0.012 in.)[81]).

The coating is a thermo-setting, fusion-bonded epoxy applied in a continuous process to the bare strand.[48] The manufacturing process starts with strand that meets ASTM A416. The strand is mechanically cleaned and then preheated to 300°C prior to application of the coating. The strand is then run continuously through a fluidized bed of electrostatically charged epoxy particles. As the electrically grounded strand passes through the bed the charged particles are attracted to the surface of the strand. To manufacture the coated and filled strand the outer six wires are separated from the inner wire just prior to entering the fluidized bed. When the wires are re-stranded with the epoxy still in a plastic state the interstitial space between the wires is completely filled with epoxy.

The use of epoxy-coated strand in post-tensioning applications and stay cables requires the use of special wedges that bite through the epoxy coating and into the underlying strand. Concerns have been raised since the protective barrier of the epoxy is broken by the wedges at a very critical location. Experimental work has confirmed the occurrence of corrosion at locations where the wedge teeth were embedded in the steel.[33] Corrosion was also found under the epoxy coating between the wedge teeth marks. The significance of corrosion at the wedge locations may vary. In bonded post-tensioned construction, corrosion at the wedge locations should not have a significant effect on the integrity of the structure, particularly if coated and filled strands are used. However, in unbonded post-tensioned applications or stay cables, anchorage failure due to corrosion at the wedges could lead to failure of the tendon or cable. In these situations, additional protection must be provided for the strand at the wedge locations.[33]

Epoxy-Coated Prestressing Bars

High strength threaded bars commonly used for post-tensioning may be specified with epoxy coating. Epoxy-coated threadbars are coated according to ASTM A775-97, "Standard Specification for Epoxy-Coated Reinforcing Steel Bars."[81] This standard is the same one used for epoxy-coated mild steel reinforcement. Anchorage hardware, including bearing plates, nuts and couplers are also epoxy-coated.

Nuts and couplers are proportioned to allow free movement over threads without damaging the epoxy coating.

The fusion-bonded epoxy coating process is similar to that for prestressing strand. The bars are first cleaned and preheated, then the epoxy powder is applied on the bars electrostatically. The final thickness of the epoxy coating ranges from 0.18 to 0.30 mm (0.007 to 0.012 in.).[81]

Epoxy-coated prestressing bars face similar issues in quality control as epoxy-coated mild steel reinforcement. The effectiveness of the corrosion protection provided by the epoxy coating is dependent on the quality of the coating and the amount of damage to the coating. Transportation and handling are common sources of coating damage. Padded bundling bands, frequent supports, and nonmetallic slings are required to prevent damage during transportation. Care must also be taken during placement and stressing of bars to minimize coating damage. Damaged coating can be repaired on-site using a two-part liquid epoxy. However, it is more desirable to adopt practices that prevent damage to the coating.

5.4.1.2 Galvanized Prestressing Steel

Zinc galvanizing has proven to be the most effective metallic coating for corrosion protection. Zinc provides protection by sacrificially corroding in place of steel when exposed to a corrosive environment. Zinc is anodic to steel in the electromotive force (EMF) series and will corrode sacrificially to steel when there is electrical contact and a sufficiently conductive electrolyte is present. The advantage of a sacrificial protection system is that it theoretically does not have to completely cover the protected part, and nicks and abrasions in the zinc should not permit corrosion of the underlying steel.

Zinc is widely used to protect exposed steel from atmospheric corrosion. The effectiveness of zinc-coated mild steel reinforcing bars in concrete has been uncertain. Galvanized bars were found to increase time to concrete cracking in some cases, while reducing time to cracking in others. A detailed discussion is provided in ACI 222R-96.[27] There are additional concerns when using zinc-coated steel, especially high-strength steel, in contact with cement paste. In the high-alkaline environment of concrete or cement grout the corrosion rate of zinc can be very high. One product of zinc corrosion in this environment is hydrogen gas, raising concerns of hydrogen embrittlement of the high strength steel.

Galvanized Prestressing Strand

The use of galvanized prestressing strand is not common in North America and is currently prohibited by the Federal Highway Administration for use in bridges. However, the use of galvanizing in prestressing applications and stay cables is very popular in Europe as well as Japan.

In addition to concerns for increased risk of hydrogen embrittlement in galvanized strand, the galvanizing process may also affect the mechanical properties of the strand. Galvanizing of cold drawn wire for prestressing strand may reduce the tensile strength of the wire and degrade relaxation properties. The ultimate elongation of the wire may increase, and the elastic modulus of the seven-wire strand is normally decreased. Questions have also been raised about the effects of zinc galvanizing on the bond of prestressing strand. Mixed results have been reported.[48]

Galvanized seven-wire strand suitable for prestressing applications is commercially available in standard sizes from 10 to 16 mm (3/8 to 0.6 in.) diameter and in standard grades. The strand is stress relieved (normal relaxation) and conforms to all the requirements of ASTM A416[74] except that the wires are galvanized. During the production process, the wires are zinc coated individually and then stranded. The minimum weight of zinc coating for the strand ranges from 275 to 305 g/m^2 (0.90 to 1.0 oz/ft^2). The single wires before galvanizing meet the requirements of Grade 270 (1860 MPa, 270 ksi) strand (ASTM A416) when fabricated to the corresponding finished strand size.[82]

Galvanized Prestressing Bars

Threaded galvanized prestressing bars are commercially available in standard sizes and strengths of threadbar for prestressing.

Although prestressing bars are not cold drawn like prestressing wire (strand), the process of zinc galvanizing still raises concerns for hydrogen embrittlement. A specification for galvanizing prestressing bars to minimize the effects of galvanizing on the potential for hydrogen embrittlement and on mechanical properties has been developed.[83] The highest potential for damage due to hydrogen embrittlement occurs during acid pickling of the bars prior to hot-dip galvanizing. Flash pickling of the bars should be carefully controlled in terms of pickling time and acid temperature, and hydrogen inhibitors should be used in the acid bath. The bars should be galvanized immediately after pickling[83,84] The maximum weight of zinc coating is 0.82 oz/ft^2 (250 g/m^2).[83] Provisions for maintaining the threadability of the bars after coating should also be considered.[83]

Zinc galvanizing of prestressing bars has some effect on the mechanical properties of the bars. Galvanizing may lower the yield strength of the bar up to 5%, and may alter the stress-strain relationship. However, the ultimate strength and ductility of the bars is not adversely affected by the galvanizing process.[83]

5.4.1.3 Non-Metallic Prestressing Materials

Fiber reinforced plastic products have been used for pretensioning and post-tensioning in bridges, buildings, marine structures, pavements and rock anchors. The use of fiber reinforced plastic reinforcement in concrete structures can have many benefits. In most applications, the main benefit of using fiber reinforced plastics is that they do not corrode and therefore eliminate structural deterioration related to corrosion of steel reinforcement.

Fiber reinforced products normally consist of continuous fibers of glass, aramid or carbon embedded in a polymer matrix. The matrix transfers stresses between the fibers and allows them to work as a single element. The matrix also provides stress transfer between the fibers and concrete and protects the fibers. Common matrix materials are polyesters and epoxies.

The material properties of fiber reinforced plastic tendons for prestressing can be significantly different from prestressing steel, and thus their use requires special design considerations. An excellent source of information on this subject is ACI 440R-96, State-of-the-Art Report on Fiber Reinforced Plastic (FRP) Reinforcement for Concrete Structures.[85] The reader is referred to this reference for further information.

5.4.2 Ducts for Post-Tensioning

Ducts have several functions in post-tensioned concrete. The duct provides the void to allow placement and stressing of the tendons after concrete has been cast, and transfers stresses between the grouted tendon and the concrete. As corrosion protection, the duct works as a barrier to moisture and chlorides. In order for the duct to work effectively as a barrier, it must be impervious to moisture and must itself be corrosion resistant. Duct splices and connections to anchorage hardware must also be watertight. Requirements for ducts are provided in Clause 10.8 of the *AASHTO LRFD Construction Specifications*.[72]

5.4.2.1 Galvanized Steel Duct

The most widely used duct material is corrugated galvanized steel. The steel is sufficiently strong to prevent crushing and damage during concrete placement, and can withstand the frictional forces associated with post-tensioning. Galvanizing provides some resistance to duct corrosion. However, research studies[68] (also see Chapter 5 of this report) and field performance (see Chapter 7) have found that the corrosion protection provided by galvanizing is limited and severe corrosion damage, including corrosion through the duct, can occur in marine or deicing salt exposures.

Some galvanized steel ducts are manufactured with a longitudinal crimped seam. The crimped seam may not be watertight, allowing moisture ingress even if the steel duct is undamaged. Grout bleed water observed leaking from ducts with longitudinal seams has confirmed the potential moisture pathway in ducts manufactured in this manner.

Splices for galvanized steel ducts are performed in many different ways. Common practice is to wrap the joint between ducts with ordinary duct tape. Sometimes a short length of oversized duct is used to span the joint between duct segments to maintain alignment. Heat shrink tubing developed for sealing electrical connections has also been used for duct splicing. Laboratory tests[68] found that duct tape splices were not waterproof, while heat shrink tubing "...produced essentially water and chloride-tight joints..."[68]

In view of the limitations listed above, galvanized steel ducts should not be used in situations where exposure to deicing salts or seawater may occur.

5.4.2.2 Epoxy-Coated Duct

Epoxy-coated steel duct eliminates several of the problems associated with galvanized steel duct. The epoxy coating protects the duct from corrosion and seals the longitudinal duct seams. A laboratory study showed excellent performance of epoxy-coated ducts in comparison to galvanized steel ducts.[68] Performance was evaluated in terms of grout chloride levels and extent of duct and strand corrosion damage.

Epoxy-coated ducts are not widely used and may be faced with some shortcomings. Similar to epoxy-coated reinforcement, the quality of the epoxy coating and level of coating damage will influence the effectiveness of the coating as corrosion protection. Questions have also been raised regarding the ability of the epoxy coating to withstand the deformations associated with fitting the duct to the desired profile.

5.4.2.3 Plastic Duct

The use of plastic duct systems can provide the highest level of corrosion protection for post-tensioning tendons since they are non-corroding and provide an impermeable barrier to aggressive agents. Plastic ducts have been developed with sufficient strength, rigidity, abrasion resistance and bond properties to satisfy structural requirements. Testing has also shown lower friction losses[68,86] and reduced fretting fatigue[86,87] for plastic duct systems in comparison to steel ducts. Commercially available plastic ducts for post-tensioning are normally provided with fitted watertight couplers for duct splices and connection to anchorage hardware.

Plastic ducts made from polypropylene are available in a two-strand system for slabs, and in multistrand systems for tendon configurations of up to fifty-five 12.7 mm (0.5 in.) diameter strands or up to thirty-seven 16 mm (0.6 in.) diameter strands.[88]

5.4.3 Temporary Corrosion Protection

The time between stressing and grouting of internal tendons should be as short as possible to minimize the opportunity for corrosion while the tendons are unprotected. Many specifications limit the length of time between stressing and grouting. The *AASHTO LRFD Construction Specifications*[72] provides time limits for grouting ranging between seven days and twenty days, dependent on the ambient humidity (Clause 10.4.2.2.1). The PTI Guide Specification for Grouting[89] has a similar requirement, with time limits for grouting ranging between seven days and forty days. If the permissible time limits between stressing and grouting are exceeded, temporary corrosion protection measures are required by both specifications.

A range of temporary protection measures are available. The most common form of temporary corrosion protection is to coat the prestressing steel with water soluble oils or vapor phase inhibitors. Other materials, including sodium silicate and biodegradable soap (normally used as coolant for cutting metal), have been used for temporary corrosion protection. The ducts must be thoroughly flushed with water immediately prior to grouting to remove all traces of the temporary corrosion protection materials that may inhibit bond between the steel and grout. With the exception of the vapor phase inhibitor, these materials can have the added benefit of reducing friction losses during post-tensioning if they are applied on the strands before stressing.[90] Other options for temporary corrosion protection include sealing the ducts to prevent moisture entry, continuous pumping of dry air through the ducts and purging with compressed air or dry gas.[89]

A comprehensive study of materials for temporary corrosion protection and lubrication of post-tensioning tendons found that water soluble oils could not be flushed from the strands completely and adversely affected bond between the strands and grout.[90] Adhesion tests found that bond was reduced by 90% if the ducts were not flushed. When the ducts were thoroughly flushed with water, bond was still reduced by 75% in comparison to untreated strands. The effect of sodium silicate on bond was not as significant as the water soluble oils, reducing bond by 50% before flushing and 10% after flushing. Stearate soap did not affect bond. These findings illustrate the potential negative side effect of many agents used for temporary corrosion protection. The use of water soluble oils for temporary corrosion protection should be avoided if the tendons are to be bonded.

5.4.4 Cement Grout for Post-Tensioning

Cement grout bonds the post-tensioning tendon to the surrounding concrete and provides corrosion protection for the tendon. Corrosion protection is in the form of a barrier to moisture and chloride penetration and in the presence of an alkaline environment for the tendon.

An optimum grout for post-tensioning combines desirable fresh properties with good corrosion protection. The fresh properties of the grout influence how well the grout fills the duct. The corrosion protection provided by the grout is rendered ineffective if the duct is only partially or intermittently filled with grout. These situations can lead to severe conditions for corrosion (see Appendix B). The presence of voids or discontinuous grouting may also permit movement of moisture and chlorides along the length of the tendon. Important grout fresh properties are listed below.

Fluidity: Fluidity is a measure of how well the grout flows or pumps. Insufficient fluidity may lead to difficulties in placement, blockages and incomplete grouting. Excessive fluidity may lead to void formation near crests in draped tendon profiles and incomplete grouting. Grout fluidity also influences the ability of the grout to fill the space between strands in a multistrand tendon.

Bleed Resistance: Resistance to bleed is very important in grouts for post-tensioning. Unlike concrete where bleed water can evaporate, bleed water in grouted ducts tends to migrate to high points in the duct, forming bleed lenses. Eventually, the bleed water will be re-absorbed into the grout, leaving a void. Bleed lenses are particularly a problem where large vertical differences are encountered along the tendon profile.

Volume Change: Reduction in volume or shrinkage in the plastic state can lead to voids and must be avoided. In some cases, it may be desirable for the grout to possess expansive properties while in the plastic state to offset shrinkage and possibly fill voids resulting from entrapped air or bleed water collection.

Set Time: Rapid setting grouts lead to insufficient fluidity, hindering placement and leading to incomplete grouting.

The fresh properties of grout can be controlled through water-cement ratio, by the use of chemical and mineral admixtures, and by the type of cement. Without the use of admixtures, fluidity is primarily a function of the water-cement ratio. In most cases, it will be desirable to reduce the water content to lower permeability and minimize bleed water. In this situation, sufficient fluidity can be provided through the use of superplasticizer. Partial cement replacement with fly ash will tend to increase fluidity for the same ratio of water to total cementitious material. The addition of silica fume or partial cement replacement with silica fume tends to decrease fluidity due to its small particle size. Bleed can be minimized by reducing the water-cement ratio and by using fly ash or silica fume. Antibleed admixtures may also be used, particularly in situations where the tendon profile has large variations in vertical distance and bleed water accumulation may be severe. Antibleed admixtures are sometimes referred to as thixotropic admixtures. This class of admixture gives the grout gel-like properties to minimize bleeding, while permitting the grout to become fluid when agitated (mixed, pumped, etc.). Expansive properties may be provided through the use of chemical admixtures. Expanding or non-shrink admixtures are generally categorized as gas-liberating, metal oxidizing, gypsum forming or expansive cement based.[33] Set time is normally controlled through the use of set retarding admixtures. Some control may also be available through the selection of cement type.

The corrosion protection provided by the grout is primarily related to its permeability. Low permeability will reduce or slow the ingress of moisture and chlorides. Similar to concrete, the permeability of grouts may be lowered by reducing the water-cement ratio and by the use of mineral admixtures such as fly ash and silica fume. Reduced water-cement ratios may require the use of superplasticizers to provide sufficient fluidity. Admixtures such as corrosion inhibitors may also be used to improve the corrosion protection provided by the grout.

The selection of suitable grout proportions and admixtures requires careful consideration of the grout fresh properties and corrosion protection. The effects of various admixtures and grout proportions on fresh properties and corrosion protection have been studied by several researchers.[33,91] Schokker[10] performed an extensive research study to develop two optimized grouts for post-tensioning. The study was part of this overall research project. The first grout was developed for use in applications where the tendon profile was primarily horizontal. The grout contained 30% fly ash (by weight), superplasticizer and had a water-cement ratio of 0.35. This grout had excellent fluidity, good bleed resistance and provided excellent corrosion protection. The second grout was recommended for vertical tendons where resistance to bleed is critical. This grout had a water-cement ratio of 0.33 and contained a combined superplasticizer and antibleed admixture. Additional information on mix proportioning and guide specifications for grouts for post-tensioning is provided by the PTI "Guide Specification for Grouting of Post-Tensioned Structures"[89] and by the U.K. Concrete Society report "Durable Bonded Post-Tensioned Bridges."[92] The requirements for grout in the *AASHTO LRFD Construction Specifications*[72] are minimal.

The corrosion protection provided by the grout is also heavily dependent on the construction practices. Many corrosion problems have resulted from poor construction practices and inexperienced contractors (see Appendix B). An optimized grout design is of no use if it is not placed properly and the ducts are not completely filled with grout. Attention must be given to batching and grouting/injection equipment, locations of vents along the duct and grouting procedures. Requirements for grouting procedures are given in Clause 10.11 of the *AASHTO LRFD Construction Specifications*.[72] Excellent guidance for construction practices is provided in Ref. 89 and 92.

5.4.5 Anchorage Protection

The post-tensioning anchorages and end stubs of the strands must be carefully protected. Although anchorage corrosion may lead to failure of the anchorage, bond between the tendon and concrete will prevent a complete loss of prestress. However, corrosion of the anchorage hardware may lead to cracking and spalling of the concrete in the vicinity of the anchorage and increased corrosion action. Corrosion of the anchorage and strand stubs may also allow moisture entry into the duct and subsequent tendon corrosion. Multistrand anchorage systems may be fitted with a sealed end cap to protect strand ends. The cap is grouted or filled with corrosion inhibiting grease. Not all multistrand post-tensioning systems include an end cap. Anchorages are commonly recessed in a pocket at the end or edge of the concrete element. Corrosion protection for the anchorage normally consists of filling the anchorage recess or pocket with mortar or concrete. Common practice is to coat the anchorage and pocket surfaces with an epoxy bonding agent prior to filling the anchorage pocket with a non-shrink mortar.

The location of the anchorage within the structure can also play a role in corrosion protection and corrosion damage. In many structures, the anchorages are located at the ends of structural elements below expansion joints, or at exterior member ends or slab edges. These locations are prone to concentrated exposure with moisture and chlorides, and often lead to severe anchorage corrosion damage. The location of post-tensioning anchorages is often dictated by the method of construction. In instances where the anchorage can not be located away from a possible source of aggressive agents, the anchorage must be detailed to provide multiple layers of corrosion protection. The Concrete Society (U.K.) Technical Report No. 47 on bonded post-tensioned bridges[92] provides suggestions for anchorage protection details. The report discusses two approaches for anchorage protection. The first is to provide an anchorage that is not encased in mortar or concrete after stressing. Exposed anchorage hardware is protected by end caps and waterproof membrane, and has the advantage that the anchorage can be readily inspected for corrosion damage. The second approach provides a higher level of corrosion protection at the expense of inspectability by recessing the anchorage in a filled pocket. Details of multilevel corrosion protection for this form of buried anchorage are shown in Figure 5.9. The details of the member end can also be designed to minimize contact with moisture and chlorides draining through expansion joints, as shown in Figure 5.10. The member end is detailed to prevent water from dripping onto the anchorage region. An abutment gallery is provided to allow inspectors to gain access to the anchorage.

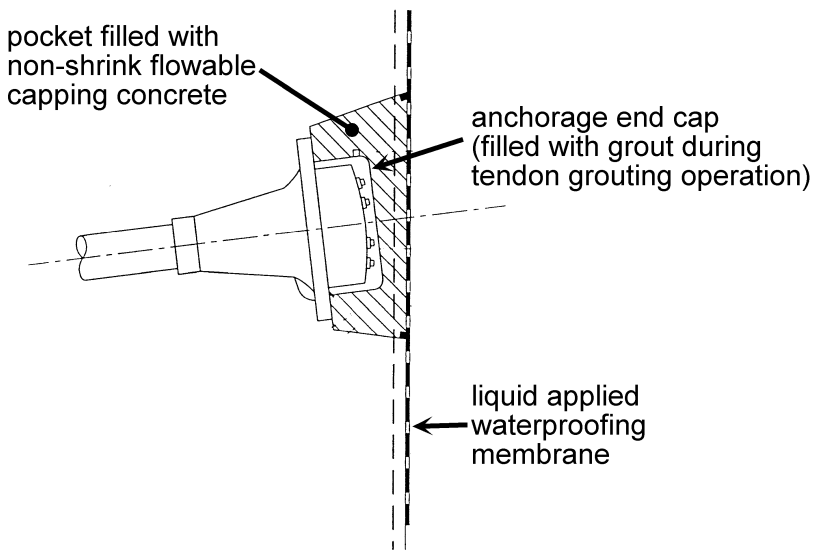


Figure 5.9 -Multi-Layer Corrosion Protection for Buried Post-Tensioning Anchorages[92]

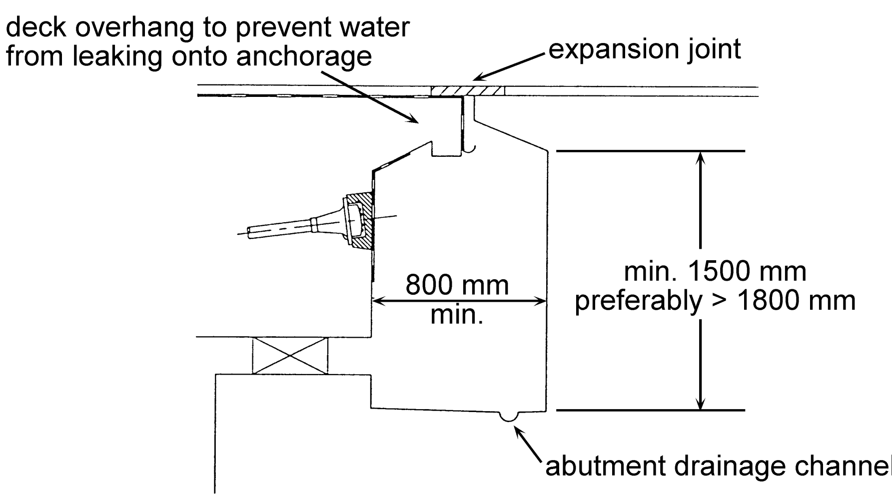


Figure 5.10 - Member End Details for Anchorage Corrosion Protection[92]

5.4.6 *Encapsulated and Electrically Isolated Systems*

Encapsulated and electrically isolated multistrand post-tensioning systems provide the highest level of corrosion protection. Encapsulation and electrical isolation are terms that are sometimes used interchangeably. However, it is important to note that encapsulated systems are not necessarily electrically isolated. Normally, encapsulation refers to encapsulation of the post-tensioning tendon only, and not necessarily the anchorage. Encapsulation of the tendon is provided by an impermeable duct, full grouting and sealed end cap over the strand or bar stubs. Complete encapsulation and electrical isolation includes coating of the anchorage or use of non-metallic anchorage components. Ideally, this approach provides an impermeable barrier around the entire post-tensioning system, protecting the system from aggressive agents and corrosion induced by stray currents or coupling with uncoated mild steel reinforcement.

The use of encapsulated, watertight post-tensioning systems also allows the system to be tested for leaks. Pressure testing is performed immediately prior to grouting with all vents closed and end caps sealed. The duct is pressurized with air and the leakage rate is monitored using a flowmeter. Sources of leaks can be identified using soapy water or other means. Significant sources of leaks should be repaired as applicable, including re-sealing of end caps, grouting or patching and crack sealing. The use of a watertight and airtight duct system also permits vacuum grouting. Vacuum grouting applies a negative pressure at one end of the duct while the grout is pumped under pressure from the opposite end. Vacuum grouting is particularly useful for long tendons or where large vertical distances are involved.

An encapsulated multistrand post-tensioning system is available in three configurations, each providing an increased level of corrosion protection.[88,93] All ducts, connections and trumpets are plastic. The bearing plate is a composite of metal and high performance mortar, and a sealed end cap is provided. The highest level of protection is provided by a configuration that provides electrical isolation for the tendon, and the ability to electrically monitor the tendon throughout the life of the structure.

5.5 UNBONDED POST-TENSIONING SYSTEM DETAILS

This section discusses corrosion protection for post-tensioning systems using unbonded multiple strand or bar systems that may be used in bridge substructures. This section does not address unbonded single strand or monostrand systems. Guide specifications for monostrand systems are provided by the Post-Tensioning Institute (PTI).[94,95] These comprehensive guide specifications include requirements for sheathing materials and corrosion preventive coating (grease). Additional information is provided in ACI 423.3R-96, “Recommendations for Concrete Members Prestressed with Unbonded Tendons.”[96] ACI Committee 423 (Prestressed Concrete) is currently in the process of producing a detailed specification for monostrand tendons that will supercede the PTI specifications for monostrand tendons.[95]

5.5.1 *Embedded Post-Tensioning*

Although not as widely used as bonded post-tensioned construction, unbonded multistrand post-tensioning systems and unbonded post-tensioned bar systems are available. Unbonded post-tensioning may be selected for various applications and structural design criteria. Common applications include flat slabs and foundations, joining precast concrete elements, precompression of bearings and structures that are to be later disassembled.

Multilevel corrosion protection in unbonded multistrand and post-tensioned bar systems is similar to bonded systems with the exception of the cement grout. Corrosion protection options include plastic or non-corroding ducts and epoxy-coated or galvanized strands or bars. Some multistrand systems may be fitted with greased and sheathed strands normally used in monostrand systems (see References 95 and 96). Anchorage protection for unbonded systems is the same as for bonded post-tensioning (see Section 5.4.5).

5.5.2 External Post-Tensioning

External post-tensioning has various applications, including precast segmental bridge construction and strengthening of structures. Stay cables may also be considered in this category. External post-tensioning tendons are not embedded in the concrete, but rather are bonded to the structure at discrete locations including anchorages and deviators.

Several options are available to provide multilevel corrosion protection for external tendons. Most external multistrand tendons are encased in a sheath, as shown in Figure 5.11. The steel or plastic sheath provides an exterior protective barrier around the tendon. Similar to considerations for post-tensioning ducts, the use of a plastic or other non-corroding material for sheathing provides the highest level of corrosion protection. Strands or bars used for external tendons may be epoxy-coated or galvanized. Greased and sheathed strands (as used in monostrand systems) are also commonly used for external multistrand tendons. The space between the strands or bars inside the outer sheathing can be filled with cement grout, grease or wax to provide additional corrosion protection. Grout properties should meet similar requirements as grouts used in bonded post-tensioned construction (see Section 5.4.4). Greases or waxes should be similar to those used for greased and sheathed monostrand tendons (see References 95 and 96).

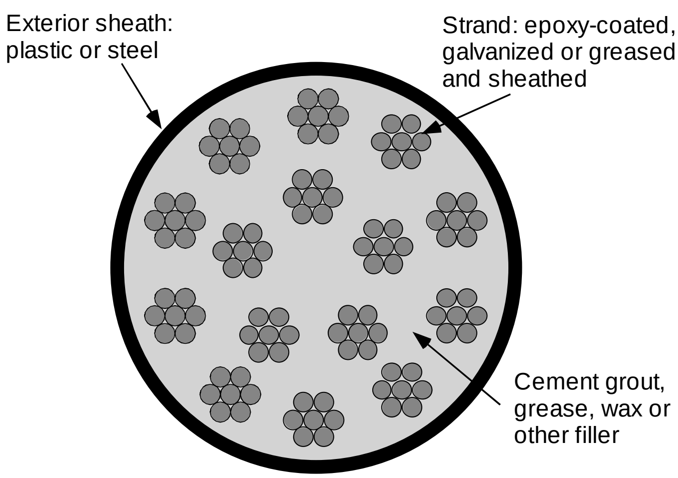


Figure 5.11 - External Post-Tensioning Tendon Corrosion Protection

Chapter 6:
Concrete Durability

6.1 SULFATE ATTACK

Sulfate attack is possibly the most common and widespread form of chemical attack on concrete.[55] Damage caused by sulfate attack normally occurs as cracking, crumbling and scaling of the concrete. In addition to physical deterioration, sulfate attack may also destroy the binding capability of the cement, thus affecting the mechanical properties of the concrete (strength, elastic modulus). Sulfate attack occurs as a chemical reaction of sulfate ions (aggressive substance) with the aluminate component of the hardened concrete (reactive substance). Sulfate attack may also occur as a physical attack on concrete due to the crystallization of sulfate salts within the cement matrix. Regions of concrete structures experiencing sulfate attack normally display a characteristic whitish appearance.[35] Damage is usually initiated in areas most susceptible to the ingress of contaminants, such as corners and edges of concrete elements. As the attack progresses, extensive cracking and spalling of the concrete may occur.

6.1.1 Exposure Conditions Causing Sulfate Attack

6.1.1.1 Soil and Groundwater

Many soils naturally contain sulfates of sodium, potassium, calcium and magnesium. Clays in particular may have considerable sulfate concentrations. These types of clays are often referred to as alkali or gypsiferous soils.[97] The groundwater in these soils acts as a sulfate solution. Significant sulfate solutions may also be found in areas where groundwater moves over rocks and sediments containing gypsum. Sulfates may also occur in soils and groundwater as a result of various forms of industrial waste such as mine tailings.

6.1.1.2 Seawater

Seawater typically has a high sulfate content. Sulfate attack on concrete structures located in or near seawater is generally limited to the tidal and submerged zones of the concrete structure.

6.1.1.3 Occurrence in Texas

Sulfate attack may occur where the structure is in contact with seawater or soils containing sulfates. The TxDOT Bridge Design Guide[18] contains a Texas map indicating districts where sulfate attack may occur, as shown in Figure 6.1. Sulfate attack due to sulfates in soils primarily occurs in the northwestern portion of the state. Sulfate attack due to seawater is a concern along the gulf coast. The source of this exposure map is not certain, but it was likely based on historical occurrences of sulfate attack in TxDOT structures.[98]

6.1.2 Mechanisms of Attack

Sulfate attack can occur as both a chemical and physical damage mechanism. Chemical sulfate attack may occur in soils or in seawater, while physical sulfate attack primarily occurs in structures located in seawater. Each is described below.

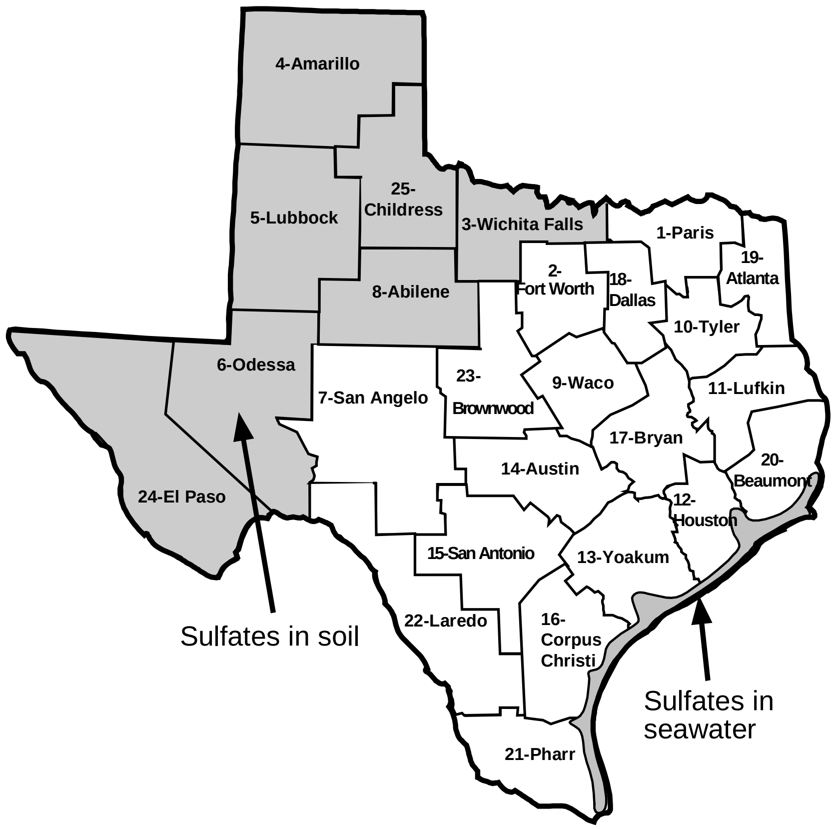


Figure 6.1 - Possible Sulfate Attack Exposure Conditions in Texas[18]

6.1.2.1 Chemical Attack

The mechanism of chemical sulfate attack is a complex process of chemical reactions between sulfate ions and the aluminate component of cement (C_3A) in the hardened concrete. The reaction products are expansive, causing disruption and cracking of the concrete. The actual form of chemical reaction is dependent on the type of sulfate.[37] Sodium sulfate reacts initially with calcium hydroxide ($Ca(OH)_2$) liberated during Portland cement hydration to form sodium hydroxide and calcium sulfate (gypsum). The calcium sulfate then reacts with C_3A to form tricalcium sulfo-aluminate (ettringite), which is expansive. Calcium sulfate reacts directly with C_3A to produce ettringite. Magnesium sulfate causes the most severe form of sulfate attack, initially reacting with C_3A and $Ca(OH)_2$ to produce ettringite and magnesium hydroxide. The presence of magnesium hydroxides creates an environment in which the calcium silicate hydrates (C-S-H, binder matrix in cement paste) are unstable, converting the C-S-H into cohesionless gel. This change may lead to a reduction in strength and stiffness of the hardened concrete.

The reaction between sulfates and $Ca(OH)_2$ to produce gypsum is referred to as gypsum corrosion. The combination of gypsum and C_3A to form ettringite is referred to as sulfo-aluminate corrosion. Both reactions are expansive, producing internal stresses in the concrete that can lead to disruption and cracking of the concrete. Although both reactions are expansive, the extent of concrete deterioration as a result of gypsum corrosion is not significant except at higher concentrations of sulfates (greater than 4000 ppm).[55] The importance of gypsum corrosion is the formation of gypsum, which places the sulfate ions in a form which facilitates the process of sulfo-aluminate corrosion.

6.1.2.2 Physical Attack

The mechanism of physical sulfate attack is due to crystallization of sulfate salts within the pore structure of the concrete. This form of sulfate attack does not involve any form of reaction with the cement. This mechanism generally occurs when concrete saturated with water containing sulfates is allowed to dry. Upon evaporation, crystallization of the sulfates occurs, producing an increase of solid volume which may lead to cracking of the concrete. Because evaporation is required for this form of deterioration to occur, physical sulfate attack is most severe at the level of the water line or in the tidal zone of structures located in seawater. This location allows alternating saturation and drying of the concrete.

6.1.3 Influencing Factors

The extent of deterioration caused by sulfate attack is related to the amount of aggressive substance (sulfates), the amount of available reactive substance (aluminates within the cement) and the rate of transportation between the two substances. Thus, the influencing factors for sulfate attack are primarily the exposure conditions, the type of cement and the permeability of the concrete. Each is described in detail below.

6.1.3.1 Severity of Exposure Conditions

The exposure condition primarily refers to the amount of available aggressive substance (sulfate) and the environmental conditions under which the sulfate is in contact with the concrete. Some guidelines for estimating the severity of the exposure conditions for sulfates are given by ACI Committee 201, "Guide to Durable Concrete,"[51] and by the "Durable Concrete Structures - CEB Design Guide."[17] These guidelines are shown in Table 6.1.

Table 6.1 - Assessment of the Degree of Sulfate Attack[17,51]

Sulfate Content		Mild	Moderate	Severe	Very Severe
ACI 201.2	Water soluble SO_4 in soil, %	0.00-0.10	0.10-0.20	0.20-2.00	Over 2.00
	SO_4 in water, ppm	0-150	150-1500	1500-10,000	Over 10,000
CEB	mg SO_4/kg of air-dry soil	2000-6000 (0.20-0.60 %)	6000-12,000 (0.60-1.2 %)	12,000 (1.2 %)	not found in practice
	mg SO_4/litre of water (ppm)	200-600	600-3000	3000-6000	>6000

The severity of sulfate attack is also affected by the immediate conditions around the structure. Table 6.2 gives some indication of the effect of the environment on the severity of sulfate attack.[99]

Table 6.2 -Effect of Environmental Conditions on Degree of Sulfate Attack

Environmental Condition	Severity of Attack
Always dry	Negligible
Almost always dry	Mild surface damage
Always wet	Continual degradation
Wet - Dry cycling	Accelerated degradation

For situations in which the concrete is always dry, sulfate attack will not occur because the transport mechanism of water is not present. In situations where the concrete is continually wet, the constant supply and transport of sulfates will cause steady deterioration of the concrete. The most severe environment for sulfate attack occurs when the concrete is saturated with sulfate water for considerable periods and occasionally allowed to dry. The drying process causes the sulfates to crystallize immediately beneath the surface of the concrete, causing physical deterioration. When the concrete is re-wetted with sulfate water, the concentration of sulfate ions near the surface is increased dramatically due to the dissolution of the sulfate crystals, further accelerating sulfate attack at the surface of the concrete. Moderate to severe sulfate attack may also occur just above the waterline in structures located in seawater due to capillary action which draws the sulfate rich water upwards into the concrete above the waterline. As the water evaporates, physical sulfate attack occurs as the sulfates crystallize at the surface of the concrete.

In certain situations, the exposure condition may be affected by the presence of other substances and thus the sulfate content alone may not be a complete gauge of the potential severity of sulfate attack. An example occurs in situations where a high concentration of chloride ions is present. When chloride ions are abundant, the formation of ettringite during sulfo-aluminate corrosion (second chemical reaction) is moderated by the preferential formation of chloro-aluminate, or Fridell's Salt.[17] This substance does not possess the detrimental expansive characteristics of ettringite formation. In addition, both gypsum and ettringite are more soluble in solutions with significant chloride concentrations, thus limiting the extent of solidification and the corresponding deleterious expansion.[55] It is for these reasons that sulfate attack on structures located in seawater has been found to be only moderately aggressive, in spite of high sulfate concentrations.

6.1.3.2 <u>Susceptibility of Concrete</u>

The susceptibility of the concrete to sulfate attack is dependent on the potential for gypsum corrosion and sulfo-aluminate corrosion. The process of gypsum corrosion is related to the amount of calcium hydroxide (by-product of the cement hydration process) available to combine with sulfates to form gypsum. The amount of calcium hydroxide ($Ca(OH)_2$) in the concrete is related to the ratio of the calcium silicates (C_3S / C_2S) in the cement.[35,37] The hydration of C_3S produces 2.2 times more $Ca(OH)_2$ than C_2S.[37] The ratio of C_3S / C_2S is a function of the relative amounts of various constituent materials in the cement manufacturing process.[35] At present, no limits are placed on maximum permissible C_3S content of cement.[100] The use of pozzolanic materials (silica fume, fly ash and others) has been found to improve resistance to sulfate attack by reducing the amount of free calcium hydroxide.[97,99,101] The pozzolans combine with $Ca(OH)_2$ in the production of calcium silicate hydrate gel, thereby limiting the amount of $Ca(OH)_2$ available for gypsum corrosion. As a result, less expansive material is formed and the amount of damage is limited, in spite of the presence of sulfates.

The chemical reaction of sulfo-aluminate corrosion requires the consumption of aluminates, and therefore is related to the amount of tricalcium aluminate (C_3A) in the cement. Experimental results have shown a distinct correlation between reduced C_3A content of the cement and improved resistance of the concrete to sulfate attack.[35]

6.1.3.3 <u>Permeability of Concrete</u>

The permeability of concrete controls the rate at which moisture and sulfates penetrate the concrete. An in-depth discussion of concrete permeability is provided in Section 5.3.

6.1.4 Protection Methods

In general, concrete may be protected from sulfate attack by preventing aggressive agents (sulfates) from penetrating concrete, and by limiting the amount of available reactive substance (aluminates within the cement). Sulfate attack protection mechanisms and methods are summarized in Table 6.3. A combination of protection measures is normally the best solution for preventing sulfate attack, particularly under severe exposure conditions.

Table 6.3 - Sulfate Attack Protection Mechanisms and Methods

Mechanism of Protection	Protection Method	
Prevent sulfates from entering concrete	**1.) reduce permeability:** - low w/c ratio - use fly ash/silica fume/slag - eliminate voids - moist or steam curing - reduce/control cracking	**2.) waterproof membranes** **3.) surface polymer impregnation**
Limit reactive substance (C_3A, $Ca(OH)_2$)	**1.) use low C_3A cement (Type II or V)**	**2.) use fly ash/silica fume/slag**

6.1.4.1 Limiting Penetration of Sulfates

The penetration of sulfates into concrete may be controlled in two general ways: by reducing the permeability of the concrete, and by providing a barrier at or near the surface of the concrete.

As mentioned previously, permeability is the single most important factor for improving the durability of concrete to most forms of deterioration. The factors that affect the permeability of concrete were presented in Section 5.3. The process of developing a specification type approach for ensuring concrete with low permeability involves consideration of all of these factors.

The penetration of sulfates into concrete may also be prevented or limited by providing a barrier at or near the surface of the concrete as discussed previously in Section 5.3. This approach has been successfully implemented for bridge decks but has not been applied to substructure elements.

6.1.4.2 Limitation of the Amount Of Available Reactive Substance

As discussed in Section 6.1.3, the extent of the chemical reactions of sulfate attack may be controlled by limiting the amounts of the available reactive substances for each reaction.

C_3A Content

Due to the observed correlation between the sulfate resistance of concrete and the tricalcium aluminate (C_3A) content of the cement, sulfate resistant cements have been developed by limiting the C_3A content. ASTM C150[100] specifies two types of sulfate resistant cement: Type II - Moderately Sulfate Resisting, and Type V - Sulfate Resisting. In Type II cement, C_3A is limited to 8%, and in Type V Cement, C_3A is limited to 5%. The CEB Durable Concrete Design Guide[17] specifies cements to have Moderate Sulfate Resistance (MSR) and High Sulfate Resistance (HSR) as follows:

MSR:	Portland Cement with $C_3A < 8\%$
	Portland Blast Furnace Cement
	Pozzolan Cement
HSR:	Portland Cement with $C_3A < 3\%$
	Portland Blast Furnace Cement (minimum of 65% slag)

No specifications are given for the content of pozzolan cement or Portland Blast Furnace Cement as a MSR cement.

Use of Mineral Admixtures (Pozzolans)

Due to the consumption of calcium hydroxide through reaction with pozzolanic mineral admixtures, silica fume and fly ash are recognized as a suitable additive for improving the sulfate resistance of concrete. In addition, the use of pozzolans also reduces the permeability of the concrete (see Section 5.3). The pozzolans may be either blended into the cement (inter-ground) or added directly. The necessary pozzolan content depends on many variables including the type of cement, the type of pozzolan and the severity of the exposure condition. ACI 201.2[51] suggests a pozzolan content of 15% to 25% of the Portland cement content (by weight). In this context, the pozzolans are used to replace the equivalent portion of the cement. The water-cement ratio therefore becomes a water-cement plus pozzolan ratio (w/(c+p) ratio). ACI 201.2 does not specify the mineralogical content of the pozzolan, but rather specifies: "Use a pozzolan which has been determined by tests to improve sulfate resistance when used in concrete containing Type V (ASTM C150) cement."[51] The only other recommendation by ACI 201.2 regarding the use of pozzolans is that ASTM C618[102] Class F fly ash appears to provide the "best results" for improving sulfate resistance. Other sources have made similar recommendations for the use of pozzolans.

Tikalsky[99] performed an extensive experimental study of the effect of fly ash on the sulfate resistance of concrete. Tikalsky found that fly ash conforming to ASTM C618 Class F specifications improved the sulfate resistance of concrete when used as a replacement for Type I and II cements or as a replacement for cement with 0% C_3A content. Improved resistance was found for replacement amounts ranging from 25% to 45%. However, ASTM[102] Class C fly ash, used in the same replacement amounts of 25% to 45%, was found to reduce the sulfate resistance of concrete made with Type I, II and V cements and with cement with 0% C_3A content. Tikalsky also examined the effect of pozzolans on the chloride ion permeability of concrete. Tikalsky reported that Type F fly ash (25% to 45% content), silica fume (9% added in excess of cement) and blast furnace slag (65% content) significantly reduced the permeability of the concrete to chloride ions in comparison to low C_3A cement (Type II or V). In situations where corrosion of reinforcement is a concern, cement pozzolan mixtures may provide a better alternative to Type II or Type V cement.

Freeman[97] also performed an experimental study on the use of fly ash to improve the sulfate resistance of concrete. The emphasis of the work by Freeman was to develop a criterion for the selection of fly ash to be used in concrete exposed to sulfates and to evaluate alternatives for improving the sulfate resistance of concrete with Class C fly ash used for strength purposes. Due to their wide availability and lower cost, Class C fly ash is commonly used as a cement replacement with the benefit of increased compressive strength.[97] However, as reported by Tikalsky,[99] Class C fly ash may have a detrimental effect on the sulfate resistance of concrete. Freeman reported that inter-grinding of Class C fly ash with cement improved the sulfate resistance of Type II cements. Freeman also reported improved sulfate resistance for concrete with Class C fly ash by including additional gypsum in the grinding process and by using sodium hydroxide and sodium sulfate as chemical additives. For the purposes of evaluating a particular fly ash for sulfate resistance, Freeman presented a model for predicting the effects of fly ash on the sulfate resistance of concrete. The model, termed Modified Calcium Aluminate Potential, computes a parameter based on the mineralogy of the fly ash. Limits are provided for the parameter to determine the effectiveness of the fly ash as sulfate resistant.

Experimental work has also shown silica fume to be highly effective for improving resistance to sulfate attack. Hooton[103] investigated cement replacement amounts of 10%, 15% and 20% (by mass) using silica fume. The concrete with silica fume was significantly more sulfate resistant than control concrete. Sulfate resistance increased with increasing silica fume content. Hooton recommended 10% cement replacement with silica fume for sulfate resistant concrete. Mangat and Khatib[104] also investigated the use of silica fume for sulfate resistance. Cement replacement amounts of 5%, 9% and 15% (by mass) were investigated. Test results showed a two-fold reduction in sulfate related expansions for concrete with 5% silica fume. Expansions were reduced by more than five-fold for the 9% and 15% silica fume concrete.

It should be noted that the fly ash and silica fume contents listed in the experimental research discussed in the preceding paragraphs are not necessarily those which will produce optimum resistance to sulfate attack. Rather, the recommendations of the cited researchers represent particular pozzolan contents, which appear to improve the resistance of concrete to sulfate attack and/or to lower the permeability of concrete. Obviously, other contents and procedures may also improve the sulfate resistance of concrete.

6.1.5 Recommendations for Preventing Sulfate Attack

Some general recommendations for the prevention of sulfate attack have been published by ACI Committee 201, "Guide to Durable Concrete,"[51] by the "Durable Concrete Structures – CEB Design Guide"[17] and by the U.S. Bureau of Reclamation (USBR)[105]. The protection guidelines are related to the severity of the exposure condition as defined previously in Table 6.1. The recommendations of ACI 201 and the CEB are summarized in Table 6.4 and Table 6.5.

Table 6.4 -ACI 201.2 R-92 - Recommendations for Concrete Subject to Sulfate Attack[51]

Exposure	Cement Type	w/c Ratio
Mild	no steps necessary	no steps necessary
Moderate	Type II, Type IP(MS), Type IS(MS)	0.50
Severe	Type V	0.45
Very Severe	Type V with pozzolan or slag	0.45

Table 6.5 -CEB Guidelines for Sulfate Resistance of Concrete[17]

Exposure	Cement Type	w/c Ratio
Mild	HSR Cement*	0.60
Moderate	HSR Cement*	0.50
Severe	HSR Cement*	0.40
Very Severe	not addressed	not addressed

* C_3A < 3%, or Portland blast-furnace cement with 65% slag

6.2 FREEZING AND THAWING DAMAGE

Freezing and thawing of moisture in concrete is a common form of physical deterioration of concrete. Damage due to freeze-thaw can occur in the form of scaling, popouts and D-cracking, as shown in Figure 6.2. Scaling refers to shallow fractures through the cement paste near the concrete surface. Scaling may occur in small patches or over large areas, and may or may not expose the coarse aggregate. Popouts refer to small pits on the concrete surface resulting from pieces of concrete "popping out." Popouts normally occur due to frost expansion of coarse aggregate near the concrete surface, and are characterized by a fracture surface through the aggregate, as shown in Figure 6.2. D-cracking consists of frost induced cracks running parallel to expansion joints and edges. The joints and edges allow greater moisture penetration, and provide less restraint for cracking. D-cracking is most common in pavements, and normally occurs due to expansion of frost susceptible aggregates. Freeze-thaw damage in structural members may also occur as general spalling along joints, corners and cracks in the member. Spalling may occur due to frost expansion of the cement paste and aggregate.

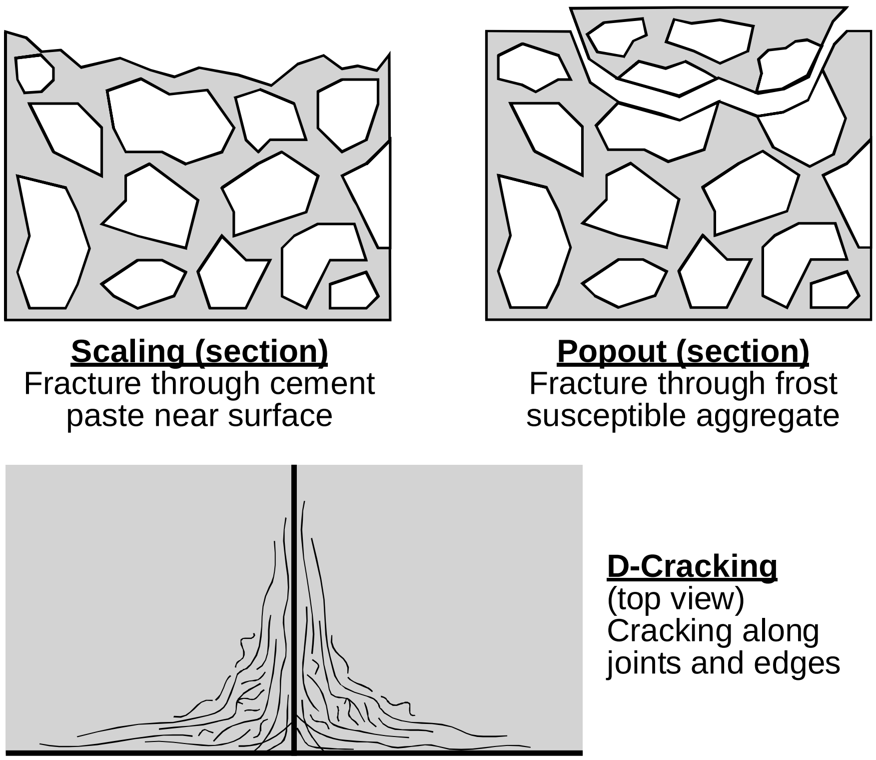


Figure 6.2 - Forms of Freezing and Thawing Damage

A very thorough treatment of the subject of freezing and thawing damage of bridges in Texas was provided by Watkins.[19] This reference addresses all aspects of the subject, and provides detailed design requirements for preventing freeze-thaw damage in Texas bridges. The subject of freeze-thaw damage in concrete is also well covered by ACI Committee 201[51] and Neville.[35] A general summary of the freezing and thawing deterioration in concrete structures is provided in the following sections, and the reader is referred to Ref. 35, 51 and 19 for more detail.

6.2.1 Exposure Conditions Causing Freezing and Thawing Damage

The obvious requirement for freeze-thaw damage is that the concrete must undergo freezing and thawing. The other condition required for freeze-thaw damage is the presence of moisture in the concrete. The severity of a given exposure for freeze-thaw damage is dictated by the annual number of cycles of freezing and thawing, and the concrete moisture content due to environmental conditions. Sources of moisture could be precipitation or direct contact with water as in the case of structures located adjacent to or over waterways. Both conditions of freezing and thawing and moist concrete must be present for freeze-thaw damage to occur. If the concrete is dry, the expansive stresses due to moisture freezing are absent and deterioration does not occur. If the concrete freezes and remains frozen for a considerable period rather than undergoing repeated cycles of freezing and thawing damage will be minimal. The effect of exposure conditions is discussed in more detail in Section 6.2.3

The areas where freeze-thaw deterioration of concrete structures may occur in Texas are indicated in Figure 6.3. The most severe conditions are in northern Texas where the highest number of freezing and thawing cycles occurs. Although the northeastern portion of the state has a milder climate, it has a moderate potential for freeze-thaw damage due to higher precipitation during the winter months.[19]

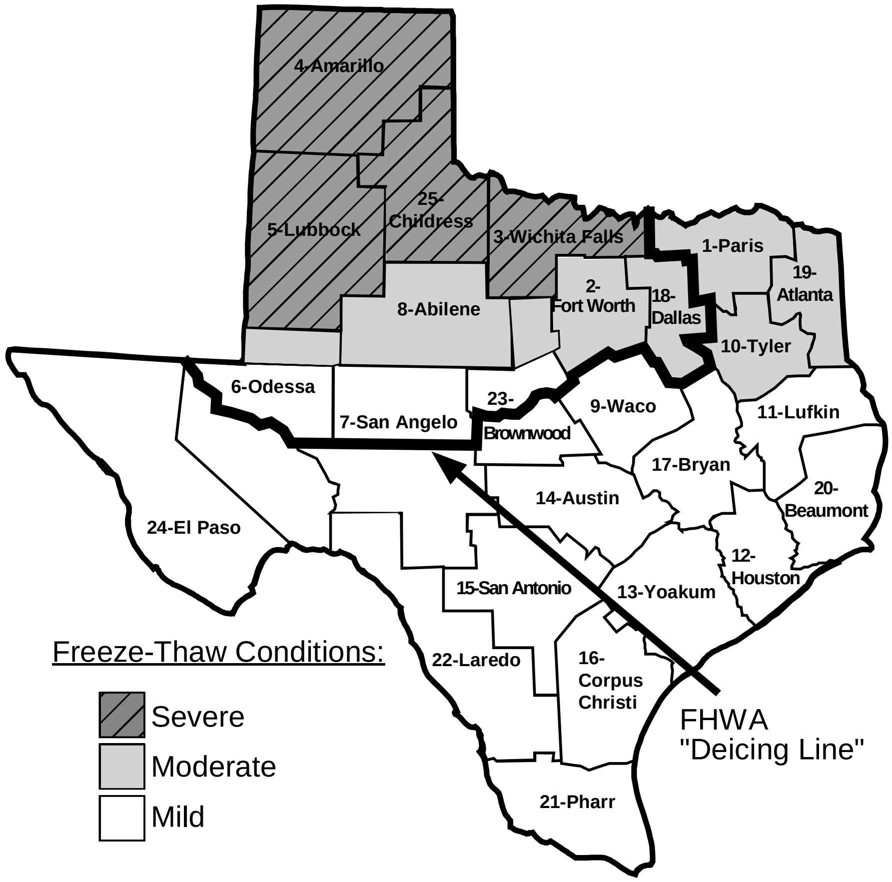


Figure 6.3 - Freeze-Thaw Exposure Conditions in Texas[19]

6.2.2 Mechanism of Attack

The mechanism of freezing and thawing damage has been debated. Three basic theories have been proposed for freezing damage in cement paste:[51,19]

1. **Hydraulic Pressure Theory:** Freeze-thaw damage results from the hydraulic pressure created by the expansion of freezing water. If the hydraulic pressure exceeds the tensile strength of the concrete, cracking and deterioration occurs. This theory is based on the principle that water expands nine percent when it freezes.[17,35,51] Pores or voids in the concrete more than 91% full of water will have insufficient space to accommodate the volume increase associated with freezing.

2. **Diffusion and Growth of Capillary Ice:** This theory states that as the pore water begins to freeze, the concentration of alkalis in the unfrozen water in the pore increases. This concentration creates an osmotic potential that draws water from nearby pores into the pores where freezing is occurring. The pore water alkali concentration is then diluted, and more freezing occurs. This process is referred to as ice-accretion. Once the pore is filled with ice and solution, further ice-accretion produces dilative pressure causing the paste to fail.

3. **Desorption Theory:** This theory states that water in very small pores or adsorbed in gel pores will not freeze due to capillary forces. This water is referred to as super-cooled. As water in larger pores begins to freeze, a difference in vapor pressure will occur, and water will attempt to migrate from small pores to larger pores. If redistribution of moisture is restrained due a high moisture content, rapid cooling or lack of entrained air bubbles in the concrete, the pressure of the water may exceed the tensile strength of the paste and lead to damage.

Freezing and thawing damage in aggregate is generally believed to occur according to the hydraulic pressure theory described above.[51]

6.2.3 *Influencing Factors*

The extent of freeze-thawing damage is related to the environmental conditions and the properties of the concrete, including the concrete pore structure, permeability and type of aggregates.

6.2.3.1 Exposure Conditions

As mentioned previously, freezing and thawing deterioration requires repeated freezing and thawing temperatures and the presence of moisture. Field experience and laboratory testing has shown that the amount of damage increases with the number of freezing and thawing cycles. For this reason, very cold climates where the concrete remains frozen for long periods may experience less freeze-thaw damage than climates with limited cold periods where the concrete experiences numerous freeze-thaw cycles. The temperature required for concrete to freeze and thaw is different from the ambient conditions required to freeze water. The freezing point for water in concrete is depressed because it is located in small pores and due to the typically high alkali concentration in the pore water. A range of freezing and thawing temperatures for concrete have been proposed. Watkins[19] defines the internal concrete temperature required to freeze pore water as –5° Celsius (23° F). The effective ambient freeze-thaw cycle is defined as a freeze temperature of –9° C (15° F) and a thaw temperature of 0° C (32° F). This cycle of ambient air temperatures is assumed to produce one complete freeze-thaw cycle for concrete at a depth up to 75 mm (3 in.).[19]

The moisture content of the concrete must exceed a “critical saturation” level for freeze-thaw damage to occur. The critical saturation concept is based on the notion that water will expand by nine percent as it freezes. Thus if the pores of the concrete are more than 91% filled with water, insufficient void space will remain to accommodate the expansion during freezing. Reported research has indicated the critical saturation level ranges between 87% and 91%.

The presence of deicing chemicals typically increases the severity of freeze-thaw damage.[17,51,19] The mechanism through which deicing chemicals affect freeze-thaw damage is generally assumed to be physical rather than chemical. The most widely accepted theory suggests that the presence of chlorides increases the saturation of the concrete and contributes to osmotic pressures similar to the diffusion and growth of capillary ice theory described above.[51] One other theory suggests that the application of deicing agents produces a large temperature differential between the exposed surface and the interior of the concrete, leading to internal stresses that may cause cracking.

Other environmental factors for freeze-thaw damage include:

- **low humidity:** possibility for water to evaporate
- **rate of cooling:** affects ability of water to redistribute to relieve expansion stresses
- **drying between freezing periods:** creates additional space (empties pores) for expansion

6.2.3.2 Concrete Pore Structure

The air void system and pore structure of the concrete have a very significant effect on the freeze-thaw durability of concrete. A suitable air void system provides a means to accommodate excess water during freezing. Air voids are provided by entraining air in the concrete, usually by using an appropriate air entraining admixture in the fresh concrete. Entrained air produces discrete, spherical bubbles in the cement paste. The resulting air void system is characterized by the volume, size and distribution of the air voids. The primary variable affecting the efficiency of the air void system is the spacing between air voids since this spacing affects the distance water has to travel. An average air void spacing of 200 μm (0.008 in.) is normally recommended.[106]

The volume of capillary pores within the structure of the cement paste should be minimized. The capillary pores provide the primary location for water accumulation. If the capillary pore volume is excessive, the entrained air void system will be inadequate to accommodate the excess water during freezing.

6.2.3.3 Concrete Permeability

The permeability of the concrete affects freeze-thaw damage in two ways. First, permeability influences the penetration of moisture into the concrete. It is assumed that concrete with low permeability will reduce the penetration of moisture into the concrete, reducing the severity of freeze-thaw damage. The second effect of permeability is its influence on movement of water within the concrete. Low permeability may restrict water movement between pores and air voids, leading to internal pressures and possible damage. For this reason, low permeability can not be relied upon as a sole method to prevent freeze-thaw damage and a balance must be drawn between limiting moisture penetration while allowing movement within the paste.

6.2.3.4 Mineral Admixtures

The effect of mineral admixtures such as fly ash and silica fume has been reported to be negligible to detrimental in some cases.[99,107,108] The presumed reason for this poor performance was the very low permeability that prevented moisture movement within the concrete. Others have conceded that the use of fly ash may decrease scaling resistance of concrete.[17,109] However, other researchers[35,109,110,111] have shown improved freeze-thaw resistance for concrete containing fly ash or silica fume.

The use of mineral admixtures will affect the required dosage of air-entraining admixtures. This sensitivity is due in part to the fineness of the mineral admixtures. The presence of carbon in fly ash will also increase the required dosage of air-entraining admixture. Guidance is provided in Ref. 35 and 106.

6.2.3.5 Aggregates

The freeze-thaw durability of the aggregate will have a large effect on the durability of the concrete. As described earlier and shown in Figure 6.2, freeze-thaw damage often results directly from inferior aggregates. The freeze-thaw resistances of aggregate are related to the absorption capacity, porosity, pore structure and size of the aggregate. The first three factors influence how much water may be present in the aggregate and whether the pore volume and structure can accommodate the actions of freezing. In general, larger aggregate sizes are less frost resistant than smaller sizes.[51] Some types of aggregates are more prone to freeze-thaw damage, and in particular D-cracking.[19] Aggregate of sedimentary origin, including limestone and dolomite, are particularly susceptible to D-cracking. Discussion on assessing the freeze-thaw resistance of aggregates is provided by ACI Committee 201.[51]

6.2.4 Protection Methods

Conceptually, the most simple method for avoiding frost damage is to prevent water from entering the pore structure of the concrete. This prevention can be done to some extent by lowering the permeability of the concrete or by sealing the surface of the concrete. Environmental effects such as the rate of cooling and frequency of freeze-thaw cycles are beyond the control of the designer. However, the capacity of the concrete to accommodate expansion stresses can be improved through the use of air entraining admixtures. The overall design concept and details of individual members may affect exposure to moisture, and thus could influence freeze-thaw deterioration. Methods for improving frost resistance are summarized in Table 6.6. A combination of design details, reduced permeability and entrained air should provide the best protection.

Table 6.6 - Frost Damage Protection Mechanisms and Methods

Mechanism of Protection	Protection Method	
Prevent moisture from entering concrete	**1.) reduce permeability:** - low w/c ratio - use fly ash/silica fume/slag - eliminate voids - moist or steam curing - reduce/control cracking	**2.) waterproof membranes** **3.) surface polymer impregnation** **4.) limit exposure to moisture**
Provide appropriate air void system	**1.) use air entraining admixtures to provide a total air content of 9% in the mortar**	

6.2.4.1 Limit Moisture Penetration

The presence or absence of moisture in concrete has a very significant effect on the extent of deterioration due to freezing and thawing. Therefore, protection against freeze-thaw damage must consider preventing moisture from entering the concrete.

Structural Form

The impact of structural form on corrosion protection was discussed in Section 5.1. The layout and geometry of the structure and structural elements influences exposure to moisture. The concepts discussed in Section 5.1, including drainage, joints, splashing and geometry are equally applicable to freeze-thaw durability.

Permeability

The penetration of moisture can be reduced by lowering the permeability of the concrete. Concrete permeability was discussed in detail in Section 5.3, and the reader is referred to that section.

Concrete Surface Treatments

Concrete surface treatments and overlays may be used to prevent or limit moisture from penetrating concrete. This subject was also discussed previously in Section 5.3. The reader is referred to that section and to references 27 and 51 for more information on surface treatments.

6.2.4.2 Air Void System

An appropriate air void system in the concrete plays a large role in freeze-thaw durability. The air content of concrete must be sufficient to provide protection against freezing and thawing, but must not be so excessive as to significantly reduce the concrete strength. The average reduction of concrete compressive strength is approximately five percent per one percent of entrained air.[35]

The required air content is normally recommended to be nine percent of the volume of mortar or cement paste.[17,35,51] The volume of cement paste in the concrete will vary with the concrete mix proportions, and therefore the required air content for the concrete will also vary. The maximum coarse aggregate size is often used as a parameter to determine the required volume of entrained air. This method will be discussed further in the following section.

As mentioned previously, an appropriate air void system can be obtained using an air-entraining admixture. Air entraining admixtures are usually hydrocarbon based, and form small, well distributed pores in the concrete that are initially free of water. During the freezing of water in adjacent pores in the cement paste, the expansive stresses cause some of the water to move into the entrained air pore, dissipating the stresses. Repeated freezing and thawing without drying may eventually cause the entrained air pores to become filled water, allowing frost damage to occur. Many factors influence the admixture dosage required to provide a desired air void content. These factors include the cement fineness, cement alkali content, the use of fly ash or silica fume, chemical admixtures, aggregate fineness, water hardness, concrete temperature, workability of the concrete, mixing equipment and mixing time.[35,51] Some guidance is provided in Ref. 35 and 106.

6.2.4.3 Aggregates

The selection of frost resistant aggregates is an important aspect of protection against freeze-thaw damage since air entrainment of the concrete (cement paste) will not prevent freeze-thaw damage from the aggregate. It is very difficult to make any general conclusions on aggregate characteristics that are prone to frost damage. As mentioned earlier, certain aggregates are prone to D-cracking and should be avoided. However, it is normally required to rely on past experience with aggregates from known sources, or laboratory testing must be used to determine appropriate aggregates. Several test methods may be used to evaluate aggregates for freeze-thaw resistance: ASTM C666,[112] C671[113] and C682.[114] ASTM C666 and C671 are test methods for evaluating freeze-thaw resistance of concrete, including the effects of the aggregate. ASTM C682 provides guidance for interpreting the results of ASTM C671 specifically for aggregates.

6.2.5 Recommendations for Preventing Freeze-Thaw Damage

Recommendations for protection against freeze-thaw damage are given by ACI Committee 201, "Guide to Durable Concrete"[51] and by the "Durable Concrete Structures – CEB Design Guide."[17] Watkins[19] also reported detailed recommendations for freeze-thaw resistance in Texas bridges.

6.2.5.1 ACI Committee 201[51]

ACI Committee 201 lists basic requirements for freeze-thaw resistant concrete. The main aspects include frost resistant aggregates and recommendations for required air contents. The water-cement ratio is limited to 0.45 for thin sections and members exposed to deicing chemicals. This limit may be raised to 0.50 for other conditions. Aggregates should be evaluated using the ASTM methods discussed in the preceding section. Recommended total concrete air contents are listed in Table 6.7. The exposure severities are defined as follows:

Severe: "Outdoor exposure in a cold climate where the concrete may be in almost continuous contact with moisture prior to freezing, or where deicing salts are used. Examples are pavements, bridge decks, sidewalks and water tanks."[51]

Moderate: "Outdoor exposure in a cold climate where the concrete will only occasionally exposed to moisture prior to freezing, and where no deicing salts will be used. Examples are certain exterior walls, beams, girders and slabs not in direct contact with soil."[51]

The total concrete air contents listed in Table 6.7 are based on a nine percent total air content in the mortar portion of the concrete for severe exposure, and seven percent mortar total air content for the moderate exposure.[51] The total air content is the sum of entrained air content and entrapped air content. The volume of entrapped air is estimated based on the concrete mix proportions and aggregate characteristics. Therefore, the necessary entrained air content is determined based on the desired total air content and the volume of entrapped air.

Table 6.7 - ACI 201.2 Recommended Total Concrete Air Contents for Frost-Resistant Concrete[51]

Nominal Maximum Aggregate Size	Average Total Concrete Air Content	
	Severe Exposure	Moderate Exposure
9.5 mm (3/8 in.)	7.5%	6%
12.5 mm (1/2 in.)	7%	5.5%
19 mm (3/4 in.)	6%	5%
25 mm (1 in.)	6%	5%
37.5 mm (1.5 in.)	5.5%	4.5%

6.2.5.2 CEB – Durable Concrete Structures Design Guide[17]

The CEB guidelines for frost-resistant concrete are similar in approach to the ACI requirements. Two exposure conditions are used: Normal and Severe. Normal conditions are defined as those were the concrete may dry out during freezing. Severe conditions are those where the concrete may be saturated, including waterway structures and structures exposed to deicing chemicals. The recommendations assume that the aggregates are frost resistant, but no guidance is provided. The CEB recommendations for frost resistant concrete are summarized in Table 6.8. Air entrainment is not required for normal exposure conditions. Two total concrete air contents are specified under severe exposure, depending on the region of Europe. The basic concrete air contents are increased if the maximum coarse aggregate size is smaller than 32 mm (1.25 in.). The additional concrete air content is increased linearly to a maximum addition of 2.5% as the aggregate size decreases from 32 mm to 8 mm (1.25 in. to 0.31 in.)

Table 6.8 - CEB Guidelines for Frost-Resistant Concrete[17]

Variable	Normal Exposure	Severe Exposure
maximum w/c ratio	0.60	0.50
minimum cement content	270 kg/m^3 (454 lb/yd^3)	300 kg/m^3 (505 lb/yd^3)
total concrete air content	no air entrainment required	provide air entrainment to achieve total air contents of: 3.5% (Central Europe) 5.5% (Northern Europe) increase by up to 2.5% for coarse aggregate < 32 mm

6.2.5.3 Watkins[19]

Watkins presented comprehensive concrete air content requirements for bridges in Texas. Watkins considered susceptibility of various bridge components to freeze-thaw deterioration and the overall environmental conditions for the bridge to assign necessary air contents.

Member exposure (micro-exposure) condition ratings were assigned based on the member's proximity to moisture and deicing chemicals, and its susceptibility to freeze-thaw damage. Member micro-exposure condition ratings are listed in Table 6.9. The various regions of the state of Texas were assigned regional exposure (macro-exposure) condition ratings to reflect the severity of the environmental conditions. These ratings were based on the combined effects of number of annual freeze-thaw cycles, precipitation during winter months and use of deicing chemicals. The macro-exposure condition ratings were shown previously in Figure 6.3. Depending on the member exposure severity and environmental exposure severity, Watkins assigned a recommended total air content for the concrete, as shown in Table 6.10. The medium and maximum total concrete air contents are a function of maximum aggregate size, as indicated in Table 6.11. The total concrete air content is provided by air entrainment with consideration for the volume of entrapped air.

Table 6.9 - Member Exposure Condition Ratings[19]

Exposure Condition	Member Type	
Mild Exposure	• Anchors • Back-up Walls • Culverts • Drilled Shafts • Driveways • Headwalls • Manholes • Prestressed Concrete Beams	• Prestressed Boxes • Prestressed Piling • Retards • Railroad Structures • Slurry Displacement Shafts • Small Road Signs and Anchors • Wing Walls
Moderate Exposure	• Bridge Piers • Bridge Railing • Bridge Substructure • Cast-in-Place Traffic Barriers • Columns	• Drilled Shafts in Water • Inlets • Precast Traffic Barriers • Riprap
Severe Exposure	• Approach Slabs • Bents • Bridge Slabs • Concrete Overlay • Concrete Pavement • Curbs	• Dense Concrete Overlay • Direct Traffic Culvert (Top Slab) • Gutters • Seal Concrete • Sidewalks

Table 6.10 – Total Concrete Air Content Requirements Based on Exposure Conditions[19]

Member Exposure (Table 6.9)	Environmental Exposure (Figure 6.3)		
	Mild	**Moderate**	**Severe**
Mild	None	None	None
Moderate	None	Medium Air	Maximum Air
Severe	None	Maximum Air	Maximum Air

Table 6.11 - Recommended Total Concrete Air Contents[19]

Air Content	Maximum Aggregate Size					
	9.5 mm (3/8”)	12.7 mm (1/2”)	19 mm (3/4”)	25 mm (1”)	38 mm (1.5”)	50 mm (2”)
Medium Air	6%	5.5%	5%	5%	4.5%	4%
Maximum Air	8%	7%	6.5%	6%	5.5%	5%

6.3 ALKALI-AGGREGATE REACTION

Alkali-aggregate reactions are a form of chemical attack on concrete. The mechanism of attack is somewhat similar to sulfate attack in that chemical reactions occur resulting in expansive products and stresses. However, in the case of alkali-aggregate reaction the reactive substance is the aggregate. The expansive stresses associated with alkali-aggregate reactions may result in concrete cracking. Cracking is typically in a map or spider web pattern, and damage may appear similar to that caused by sulfate attack or freezing and thawing. Crack widths can range from very small to very large (10 mm (0.4 in)), but are rarely more than 25 mm (1 in.) deep.[35] Cracking due to alkali-aggregate reactions rarely affects structural integrity. Rather, the cracks affect the appearance of the structure, and provide a means of ingress for moisture and other potentially deleterious agents.

6.3.1 Exposure Conditions Causing Alkali-Aggregate Reaction

Alkali-aggregate reaction may occur due to internal or external sources of alkalis. In the former situation, alkali-aggregate reactions can occur in any location provided that moisture is present. Thus, any exposure to moisture could lead to deterioration, including concrete located indoors. For this reason, alkali-aggregate reaction is not necessarily related to environmental conditions. In the latter case, external sources of alkalis may penetrate the concrete. Sources of alkalis include soils with high alkali contents and seawater. For structures located in seawater exposures, portions of the structure that are within the tidal zone or are submerged will have the highest probability for alkali-aggregate reactions. Severe alkali-aggregate reaction deterioration has been reported below the waterline in marine structures.[115]

6.3.2 Mechanism of Attack

Alkali-aggregate reactions are chemical reactions between alkalis and certain types of aggregates. The reactions may be alkali-silica or alkali-carbonate, depending on the type of aggregate. The alkali-silica reactions produce an alkali-silica gel that possesses expansive properties and can lead to cracking and deterioration of the concrete. The reaction of alkalis with carbonate aggregates is referred to as the dedolomitization of the aggregate. The alkali-carbonate reaction products are prone to swelling in a manner similar to clays. The alkali-aggregate reactions are described in detail in Ref. 17, 35, 51 and 55.

6.3.3 Influencing Factors

The extent and severity of alkali-aggregate reactions are a function of the presence of alkalis and the reactivity of the aggregate. Alkalis (sodium and potassium) may diffuse into the concrete from external sources, or they may be present in the concrete from the cement or admixtures. Obviously, the presence of silica or carbonate in the aggregate increases its susceptibility to alkali-aggregate reactions Aggregate types that may be susceptible to reactions with alkalis are identified by ACI Committee 201.[51] Several laboratory tests are available for evaluating the reactivity of aggregates, as described in Ref. 51. Separate test methods are required for alkali-silica and alkali-carbonate reactivity. Another influencing factor for alkali-aggregate reactions is the presence of moisture, since water is required for the reactions and expansions to occur. Movement of moisture also provides a transportation mechanism for alkalis into the concrete and within the concrete. For this reason, reduced concrete permeability can improve resistance to alkali-aggregate reactions.

6.3.4 Protection Methods

The protection methods for alkali-aggregate reactions are straightforward, and are summarized in Table 6.12. Lowered concrete permeability will limit the penetration and movement of alkalis within the concrete, in addition to limiting the presence of moisture. Low alkali cements may be specified to limit the amount of alkalis in the concrete. The general guideline is to limit cement alkali content to less than 0.6% of equivalent Na_2O.[17,51] The use of mineral admixtures such as fly ash, silica fume and blast furnace slag has been shown to improve resistance to alkali-silica reaction.[103,116,117,118] The improved resistance is in the form of lowered permeability and the preferential reaction of alkalis with the mineral admixtures (pozzolanic reaction) rather than the aggregate. Required mineral admixture contents are a function of many factors. ACI 201 recommends the use of ASTM C441[119] to evaluate the effectiveness of mineral admixtures for controlling the deleterious effects of alkali-aggregate reactions. Finally, the use of non-reactive aggregates is of prime importance. Several ASTM standards have been developed to assess the reactivity of aggregates. Guidance on aggregate selection is provided by ACI Committee 201[51] and Ref. 35.

Table 6.12 – Alkali-Aggregate Reaction Protection Mechanisms and Methods

Mechanism of Protection	Protection Method	
Prevent alkalis from entering concrete	**1.) reduce permeability:** - low w/c ratio - use fly ash/silica fume/slag - eliminate voids - moist or steam curing - reduce/control cracking	**2.) waterproof membranes** **3.) surface polymer impregnation**
Limit reactive substances in concrete (alkalis and reactive aggregates)	**1.) use low alkali cement** **2.) use fly ash/silica fume/slag**	**3.) use non-reactive aggregates**

6.3.5 Recommendations for Preventing Alkali-Aggregate Reactions

No comprehensive, concise durability design guidelines have been published for the prevention of alkali-aggregate reactions. Some guidance is provided by ACI Committee 201[51] and the "Durable Concrete Structures – CEB Design Guide."[17] The development of detailed recommendations for preventing alkali-aggregate reactions is beyond the scope of this report. The general protection approach described in the preceding section should be taken, and additional guidance is provided by ACI Committee 201,[51] the CEB Design Guide,[17] and Ref. 35.

Chapter 7:
Field Performance of Prestressed Concrete Bridges

The field performance of prestressed concrete structures can provide a useful perspective on the corrosion of prestressing steels. Observed corrosion problems can be used to identify deficiencies in the processes of design, construction and maintenance of the structure. In learning from the past, similar problems can be avoided in the future and the service life of structures can be extended.

7.1 INCIDENCE OF CORROSION IN PRESTRESSED CONCRETE STRUCTURES

The overall performance of prestressed concrete structures worldwide has been very good and the number of serious cases of corrosion has been limited. In 1970, the FIP Commission on Durability[120] surveyed 200,000 prestressed structures and reported "an extremely low proportion of cases causing concern," and "occurrences of corrosion where the consequences have been serious are rare." More recent reviews of research and experience have also concluded that the durability performance of post-tensioned structures is very favorable to date.[121]

It is not possible to obtain precise numbers on the incidence of corrosion in prestressed concrete structures because many cases are not reported and some occurrences of corrosion have not yet been detected. Attempts to estimate the occurrence of corrosion have been made based on surveys of reported problems. A survey[122] of almost 57,000 prestressed structures reported 0.4% incidents of damage and 0.02% incidents of collapse due to all causes, including corrosion. Another survey[123] of 12,000 prestressed bridges reported visual evidence of corrosion in less than 0.007% of the surveyed bridges. Schupack[124] reported incidents of corrosion in about 200 post-tensioning tendons, representing only 0.0007% of the estimated 30 million stress-relieved tendons in use in the western world up to 1977. A condition survey[125] of all bridge types in the United States found that overall 23.5% of all bridge types were structurally deficient due to all causes including corrosion. The survey considered steel, timber, reinforced concrete and prestressed concrete bridges at comparable ages and spans. As a group, prestressed concrete bridges had the best performance with only 4% deficient. Since these deficiencies include all sources, the actual incidence of corrosion will be much less than 4% in prestressed bridges. A 1994 survey of post-tensioned segmental bridges found 98% of segmental bridges in the U.S. and Canada had condition ratings of satisfactory or higher, with no reported corrosion problems.[126] These figures give a general indication of the overall good performance of prestressed concrete structures, although the data is not current for several of the references.

Quantifying the incidence of corrosion in prestressed concrete structures is further complicated by limitations in techniques for detecting corrosion, particularly in post-tensioned structures. Condition surveys of prestressed concrete structures are often limited to visual inspections for signs of cracking, spalling and rust staining. This limited inspection may overlook corrosion activity, particularly for post-tensioned structures. Corrosion damage in post-tensioned elements has been found in situations where no outward indications of distress were apparent. The collapse of a precast segmental, post-tensioned bridge in Wales was attributed to corrosion of the internal prestressing tendons at mortar joints between segments.[50] The bridge had been inspected six months prior to the collapse, and no signs of deterioration were apparent. The collapse of this bridge occurred due to highly localized corrosion of the prestressing tendons. Because the corrosion was localized, distress indicators such as spalling, rust staining or increased deflections of the structure were not present. Examples such as this one lead some to fear that figures reporting the incidence of corrosion in prestressed structures based on limited or visual inspections may be unconservative and produce a false sense of security.

7.2 LITERATURE REVIEW: CORROSION IN PRESTRESSED CONCRETE STRUCTURES

The field performance of prestressed concrete structures is not as well documented as that for nonprestressed or conventionally reinforced concrete structures. This lack of documentation may be attributed to a number of factors, including the larger proportion of reinforced concrete structures in service, and the generally shorter length of experience with prestressed concrete structures, limiting data on long-term behavior. In spite of these factors, a number of good sources of information exist on the field performance of prestressed concrete structures.

A brief summary of relevant literature on the field performance of prestressed concrete bridges is provided in Appendix B. The review addresses the following areas:

- corrosion of prestressing strand before construction
- pretensioned bridge components
- unbonded single strand (monostrand) tendons
- unbonded multiple strand and bar tendons in bridges
- external multistrand tendons in bridges
- bonded internal post-tensioned tendons

The objective of the literature review was to illustrate typical corrosion problems in prestressed concrete structures. The scope of the review includes pretensioned elements and monostrand tendons to highlight corrosion problems that are not necessarily limited to those particular prestressing systems. Examples of corrosion problems are described according to type of prestressing, time of occurrence and various aspects of the prestressing system. Where possible, specific case studies are provided for illustration.

In comparison to reinforced concrete, prestressed concrete may have more aspects affecting corrosion and the subsequent durability of the structure. This increased complexity is particularly true for post-tensioned concrete structures due to the larger number of elements in the prestressing system and additional steps in the construction process. These factors are reflected in the reported incidents of corrosion in prestressed structures, and in particular, post-tensioned structures.

The review of reported cases of corrosion in prestressed concrete bridges indicated that the source of corrosion problems is normally traceable to specific circumstances of poor design, construction or maintenance. The findings of the literature review are summarized in Table 7.1 in terms of general factors that influence corrosion in post-tensioned structures.

Table 7.1 - Common Factors for Corrosion in Post-tensioned Concrete

Influencing Factor	Potential Problems
Environment:	
Use of deicing salts	source of moisture and chlorides
Marine environment	source of moisture and chlorides
Soils with high salt content	source of chlorides
Chemical exposure (acids, materials with high sulfur content)	may lead to hydrogen embrittlement or hydrogen induced stress corrosion

Table 7.1 - Common Factors for Corrosion in Post-tensioned Concrete - Continued

Influencing Factor	Potential Problems
Materials Selection:	
Heat treated prestressing steel	prone to stress corrosion and hydrogen embrittlement
Low quality concrete	insufficient protection for steel
Low quality post-tensioning grouts	excessive bleed lens or air void formation, insufficient or excessive fluidity, chlorides in grout
Non-permanent void formers (ducts)	no corrosion protection
Corrosion susceptible duct materials	limited corrosion protection
Dissimilar metals used for anchorage components	prone to galvanic corrosion
Design Deficiencies:	
Low concrete cover	insufficient protection for steel
Congested reinforcement	poor concrete consolidation or honeycombing
Poor drainage	saltwater collects on structural elements
Joint locations and details	saltwater drips onto supporting structural elements not designed for severe exposure
Anchorage protection	insufficient protection provided
Location of post-tensioning anchorages	saltwater comes in contact with anchorage
Post-tensioning ducts	discontinuous ducts or poor splice details lead to ingress of moisture and chlorides
Vents for post-tensioning ducts	improper vents or lack of vents leads to incomplete grouting
Construction Deficiencies:	
Design concrete cover not provided	insufficient protection for steel
Blocked or damaged post-tensioning duct	incomplete grouting – insufficient protection for prestressing steel
Poor grouting procedures or inexperienced contractors	incomplete or non-existing grouting - insufficient protection for prestressing steel
Sustained period between stressing and grouting/construction	opportunity for corrosion while tendon is unprotected
Maintenance Deficiencies:	
Expansion joints	saltwater drips onto supporting structural elements not designed for severe exposure
Blocked or damaged drains	saltwater collects on structural elements or drips onto supporting structural elements not designed for severe exposure

7.3 CONCLUSIONS – FIELD PERFORMANCE OF PRESTRESSED CONCRETE BRIDGES

The literature review of field performance of prestressed concrete bridges allows several general observations or conclusions:

- Most corrosion problems encountered in recent times result from poor design details and construction practices that took place well in the past. In many cases, new developments and improvements in prestressing systems and specifications will prevent a repetition of such past deficiencies that are now resulting in corrosion problems. There is a great benefit of learning from past problems.

- Most corrosion problems in prestressed concrete structures result from specific sources that could be avoided.
- Corrosion problems resulting from inherent durability related deficiencies in prestressed concrete structures are extremely rare.

Avoiding corrosion problems in prestressed concrete and specifically post-tensioned concrete requires:

- Awareness of possible corrosion problems.
- Durability design guidelines and recommendations for post-tensioned concrete. These guidelines must address durability and corrosion protection throughout the design process, since durability is affected by decisions at all design stages from the conceptual design and selection of the structural form to final refinement of design details.
- Attention beyond the design stage. Many corrosion problems resulted from construction problems and inferior materials. Construction specifications must include provisions for the necessary procedures and materials to ensure durability, and construction inspection and quality control must ensure these requirements are met.
- Attention beyond the construction phase. Proper maintenance is required throughout the service life of the structure. Consideration for maintenance should be made at the final design stage. Where possible, designers should specify maintenance requirements and techniques.

Chapter 8:
Experimental Studies of Corrosion in Prestressed Concrete

The amount of research for corrosion in prestressed concrete is very limited in comparison to corrosion research for reinforced concrete. This chapter provides a brief review of relevant corrosion research for prestressed concrete. The selected research addresses issues unique to prestressed concrete, and in particular to post-tensioned concrete, including post-tensioning materials selection, anchorage protection and prestressing for crack control.

8.1 MOORE, KLODT AND HENSEN

Moore et al[48] prepared an extensive report on corrosion protection for prestressed concrete bridges in 1970. The report contained a literature review of corrosion prevention in prestressed bridges, field inspections of prestressed structures, laboratory testing and preparation of corrosion protection recommendations. The laboratory portion of the work included tests to evaluate corrosion mechanisms in prestressed concrete, coatings for prestressing steel, corrosion in pretensioned members and grouting for post-tensioned members.

8.1.1 Coatings for Prestressing Steel

Moore and his co-authors performed a large experimental study to evaluate protective coatings for prestressing steel. Metallic and organic coatings were investigated on a 6 mm (0.25 in.) diameter cold drawn, stress-relieved prestressing wire. Coatings were evaluated for their effect on strength and elastic properties, ductility, relaxation, coating damage during stressing and bond with concrete. The possibility of hydrogen embrittlement was evaluated for galvanized wire. Coating abrasion resistance was compared, and the corrosion protection provided by the coatings was evaluated using both stressed and unstressed specimens. The test results showed that zinc galvanizing and epoxy coatings had the best overall performance, but none of the investigated coatings were ideal. The epoxy coating provided excellent corrosion protection, but lacked abrasion resistance. The cost of the epoxy coating was deemed prohibitive. It should be noted that the perceived shortcomings of the epoxy-coated wire may be a function of the age of the research. In the nearly 30 years since this work was performed, improvements in epoxy coating for prestressing strands should alleviate concerns for abrasion resistance, and increasing use of epoxy-coated strand has made it more cost competitive. The zinc galvanizing lowered strength and increased relaxation of the wire. An important finding was that the zinc galvanizing did not cause hydrogen embrittlement of the wire even after long-term testing in a chloride environment.

8.1.2 Pretensioned Beam Corrosion Tests

The pretensioned beam corrosion tests were discussed previously in Section 4.4.2, and presented in Appendix A, Section A.2.3.2. The purpose of the tests was to evaluate:

- effect of voids between steel and concrete
- effect of concrete cover
- effect of live loads
- effect of sizable tensile cracks in concrete
- effect of accidental overloading (cracking followed by load reduction and cracks closing)

The experimental work used small-scale pretensioned beam specimens subjected to saltwater exposure. The main findings of the research are summarized below. More detail is provided in Section A.2.3.2.

1. The most serious corrosion was observed in the beams with open cracks. Pitting corrosion was observed at cracks as small as 0.004 in. (0.1 mm)
2. Cracks in beams caused by brief overloading tended to "heal" after ten months of exposure. No increase in corrosion was observed at these crack locations
3. No correlation between load level and corrosion was observed, with the exception of the specimens loaded to cracking
4. Concrete cover of 1.5 in. (38.1 mm) and larger prevented corrosion in the uncracked specimens over the ten months of exposure. Corrosion was found in all specimens with 0.75 in. (19 mm) cover or less.

8.1.3 Grouts for Post-Tensioning

Moore et al[48] performed a number of tests to evaluate grouts and grouting procedures. Experimental work involved fresh property tests for compressive strength, shrinkage and fluidity, and full-scale girder grouting tests. Corrosion tests were not performed. Based on the fresh property tests, a water-cement ratio of 0.40 to 0.44 was recommended. The full-scale pumping tests found extensive bleedwater voids in the grout along the duct. Proprietary admixtures were evaluated to reduce bleeding, but proved ineffective. Laboratory tests found bleedwater and voids could be reduced by curing the grout under pressure.

8.2 TANAKA, KURAUCHI AND MASUDA

Tanaka et al[127] reported the findings of a ten-year exposure study of unbonded post-tensioned elements. The test specimens had dimensions of 200 x 150 x 2000 mm long (8 x 6 x 79 in. long) and were centrally post-tensioned with a single 12.7 mm (0.5 in.) diameter greased and sheathed unbonded prestressing strand. The post-tensioning anchorages were recessed in a pocket. Pockets were filled with mortar (sand-cement-water) after stressing. An expanding admixture was used in the mortar for selected specimens. Anchorage hardware in selected specimens was coated with a tar-epoxy prior to filling the pockets. The specimens were subjected to a marine and/or industrial exposure for up to ten years. The specimens were not subjected to structural loading.

After ten years of exposure, all specimens experienced cracking due to corrosion of the mild steel reinforcement in the specimens. Cracking and corrosion damage was more severe in the anchorage regions of the specimens. Anchorage hardware was corroded in all cases, including those with tar-epoxy coating. The overall corrosion damage was attributed to insufficient concrete cover for the nonprestressed reinforcement and anchorage hardware.

In spite of the corroded post-tensioning anchorages, the greased and sheathed prestressing strands were found to be "perfectly protected from corrosion." Tanaka et al concluded that the grease was very effective in preventing penetration of moisture and chlorides through the corrosion damaged anchorages.

8.3 ETIENNE, BINNEKAMP, COPIER, HENDRICKX AND SMIT

Etienne et al[128] reported an extensive literature review and experimental study of corrosion protection for unbonded post-tensioning tendons. The report is very thorough, and is an excellent source of information for unbonded, greased and sheathed single strand tendons. One aspect of the report of interest to the research in this report is anchorage protection.

Anchorage protection methods were examined using exposure tests of two slabs with 12.5 mm (0.5 in.) diameter seven-wire strand unbonded tendons. The slabs had dimensions of 6.52 x 1.8 x 0.153 m (257 x

71 x 5.6 in.). One slab contained six tendons, and the second slab had eight tendons. Single strand tendon anchorages were recessed in pockets at the slab edges. The anchorage protection variables included various combinations of pre-treatment and mortars, as listed in Table 8.1. The exposure duration ranged from 27 to 54 months.

Examination of the test specimens during and after exposure testing revealed frequent shrinkage cracks between the mortar plug and slab concrete. These cracks were largest when the entire recess and anchorage hardware were pretreated with anti-corrosive primer paint. Shrinkage cracks increased in width as the cement content of the mortar increased. The non-shrink mortar performed poorly, showing wide shrinkage cracks. Virtually no shrinkage cracks were observed around the mortar plugs where the epoxy bonding agent was used. At the conclusion of testing, pull-out tests were performed on the mortar plugs. The plugs where paint was used as pre-treatment had lower pull-out forces than the anchorages where no pre-treatment was used. This decrease in pull-out force suggests that the anti-corrosive paint was detrimental to bond. Pull-out tests for mortar plugs where the epoxy bonding agent was used failed in the mortar, suggesting excellent bond. After the plugs were removed, anchorages were examined for corrosion damage. Corrosion was light to heavy on anchorages with the anti-corrosive paint pre-treatment. Corrosion was light to none where epoxy bonding agent was used. Heavy corrosion was found on anchorages with no pre-treatment, and pitting corrosion was found where the anchorages were left exposed.

Table 8.1 - Combinations of Anchorage Protection Mortars and Pre-treatment[128]

Mortar	Pre-treatment
rich normal lean	anti-corrosive primer paint on exposed steel anchorage components
rich normal lean	anti-corrosive primer paint on entire anchorage pocket and anchorage components
rich normal lean	epoxy bonding agent on entire anchorage pocket and anchorage components
rich normal lean	no pretreatment
non-shrink mortar	no pretreatment
no mortar	no pretreatment

	masonry sand	:	cement	:	water
rich mortar	1.8	:	1	:	0.35
normal mortar	3.4	:	1	:	0.55
lean mortar	4.8	:	1	:	0.75
non-shrink mortar	proprietary grouting mortar				

The final conclusions for anchorage protection were:

1. Use an epoxy bonding agent.
2. Do not use anti-corrosive paint as pre-treatment.
3. The non-shrink mortar investigated did not offer improved corrosion protection.

4. Special measures to protect the anchorage, including anchorage enclosures or caps should be investigated and used.

8.4 PERENCHIO, FRACZEK AND PFIEFER

Perenchio et al[68] reported an extensive laboratory study and literature review of corrosion protection for prestressing systems in bridges. The research addressed both pretensioned and bonded post-tensioned prestressing systems. The report also provides an excellent review of corrosion protection measures and field performance of prestressed bridges. Aspects of the experimental work of interest to this project research are accelerated corrosion studies of pretensioned beams, post-tensioning anchorage hardware and post-tensioning ducts and grouts.

8.4.1 Pretensioned Beam Specimens

The pretensioned beam tests were discussed previously in Section 4.4.2 and are summarized in Appendix A, Section A.2.3.3. The purpose of the pretensioned beam corrosion tests was to evaluate the effectiveness of epoxy coated strand and the effect of cracks on the durability of pretensioned members. The specimens were 3.66 m (12 ft) long with a 152 x 254 mm (6 x 10 in.) cross section. The beams were pretensioned with two 12.7 mm (0.5 in.) dia. Grade 270 (1860 MPa) seven-wire strands. The beams were subjected to varied levels of loading and exposed to saltwater. The main conclusions from these tests are listed below. Additional detail is provided in Section A.2.3.3.

1. Cracking (crack width = 0.01 in. (0.254 mm)) reduced the time to initiation of corrosion and increased corrosion severity. However, significant corrosion also occurred in companion uncracked specimens.
2. No conclusions regarding critical crack width can be made from the results of this study.
3. 1 in. (25 mm) of clear cover was not sufficient to prevent corrosion in either cracked or uncracked specimens.
4. Epoxy-coated strand showed no signs of corrosion during exposure testing. Several specimens contained both epoxy-coated and uncoated strands, with epoxy-coated strands in the layer closest to the exposed face of the specimen. Although no corrosion was found on the epoxy-coated strands, the uncoated strands experienced heavy to severe corrosion in the uncracked and cracked specimens, respectively. These results illustrate the importance of using epoxy coated strand throughout the member rather than just at the level of steel closest to the tension face.

8.4.2 Post-Tensioning Anchorage Specimens

Three different corrosion tests were used to evaluate a number of variables for corrosion protection of the anchorage hardware. The test variables included epoxy-coated and uncoated prestressing strand, epoxy-coated and uncoated anchorage hardware and galvanized and polyethylene ducts.

The first test used small specimens with fully assembled anchorages and unstressed strands. The specimens incorporated all typical anchorage components. The test specimens were similar to macrocell specimens, with the anchorage hardware closest to the exposed surface, and four #11 (35 mm dia.) reinforcing bars in the bottom layer to simulate mild steel in the structure and act as the cathode in macrocell corrosion.

The second test type consisted of large concrete members with stressed anchorages. The specimens had dimensions of 381 x 254 x 3048 mm long (15 x 10 x 120 in. long) and contained a typical four strand slab anchorage system. Two of the four strands were tensioned to $0.70f_{pu}$. Similar to the small assembled anchorage specimens, four #11 (35 mm dia.) bars were added to simulate the nonprestressed steel in the structure.

The third test was small disassembled anchorage specimens used to investigate the possibility of galvanic cells developing between the different anchorage components. Test specimens consisted of the anchorage, four chucks, four sets of wedges, four short strand pieces and short duct segment cast in a concrete block with dimensions of 381 x 254 x 635 mm long (15 x 10 x 25 in. long). All components were provided with a ground wire to facilitate various corrosion measurements. There was no electrical continuity between the different components. Concrete cover was the same as for components in the small assembled anchorage tests. Again, four #11 bars were included in the specimens.

Specimens in all three tests were grouted with a 0.44 water-cement ratio grout. Anchorages were recessed as in typical practice. Anchorage pockets were filled with a drypack mortar (1 part sand, 1 part cement, sufficient water for workability). Exposure testing was performed for one year for all three test methods. Exposure conditions consisted of weekly wet-dry cycles with 15% NaCl solution. During the dry portion of the cycle, specimens were air-dried at 37.8 deg. C (100 deg. F). The general findings of the anchorage corrosion tests are listed below.

1. Heavy rust stains and some cracking were found in all specimens with galvanized ducts and uncoated anchorages. Galvanized ducts and uncoated anchorages were heavily corroded upon removal from the specimens.
2. No corrosion was found on epoxy-coated anchorages.
3. Chucks and wedges were corroded to varied extents in all specimens. Epoxy coating of the chucks did not prevent corrosion.
4. Strand ends were corroded in all cases, some severely. Strand corrosion extended through the wedges but generally stopped at the grout.
5. Drypack mortar used to fill anchorage recesses was not sufficient to protect chucks, wedges and strand ends. Authors recommended improved techniques for protecting these elements should be developed.
6. No corrosion was found on the epoxy-coated strand in all cases.

8.4.3 Post-Tensioning Duct Specimens

A series of post-tensioning duct specimens were used to evaluate corrosion protection away from the anchorage regions. The duct specimens consisted of three uncoated, unstressed prestressing strands inside a 2.9 m (9.5 ft) long grouted duct (50 mm (2 in.) diameter). A duct joint was provided at mid-length. The specimens were placed in a tank such that the central 1.52 m (5 ft) could be completely exposed to saltwater. The specimens were subjected to the same seven day wet-dry cycle described for the anchorage specimens for a total duration of ten months. Test variables are listed in Table 8.2.

Table 8.2 - Post-Tensioning Duct Specimen Test Variables[68]

Duct Types	Duct Splice Types	Grout Types
bare steel galvanized steel epoxy-coated steel polyethylene	taped splice heat shrink tubing	normal (w / c = 0.44) silica fume (proprietary grout) calcium nitrite (w / c = 0.44, admixture dosage 72.9 l / m^3)

The findings of the duct corrosion tests are summarized below:

1. Bare and galvanized steel ducts were corroded through.
2. The epoxy-coated ducts were corroded along the spiral duct seams due to a break in the coating or incomplete coating at the seam.

3. Grout chloride levels for bare steel duct were two times higher than for galvanized steel duct.
4. Grout chloride levels for galvanized steel duct were thirty times higher than chloride levels from the epoxy-coated duct.
5. Grout chloride measurements indicated that the polyethylene duct was impermeable.
6. Taped duct splices allowed moisture and chloride entry to the duct. Heat shrink splices reduced chloride contents at the joint by an average of 1000%.
7. Silica fume grout had lower corrosion currents and chloride penetration than the other grouts on average. However, the bare steel duct specimens with silica fume grout had more than three times as much visible strand corrosion damage as the other bare steel duct specimens.
8. Calcium nitrite grout performed similar to normal grout in terms of corrosion currents and visible strand corrosion.

8.5 TREAT ISLAND STUDIES

Treat Island, Maine has been used by the U.S. Army Corps of Engineers and others as an exposure station for concrete materials since 1936. In 1961, a series of 20 post-tensioned beams were placed on the beach at Treat Island within the tidal zone. The beams were not subjected to structural loading during exposure testing. The original objective of the testing program was to evaluate different end anchorage protection methods for post-tensioned beams. The findings of the research have been reported at different times in various sources, including O'Neil,[129] Schupack[130] and O'Neil and Odom.[131] A total of eight beams were removed from exposure testing in 1973 and 1974 for structural testing and autopsy.[129] An additional three beams were removed for testing and autopsy in 1983.[131]

The post-tensioned beams are 2.44 m (8 ft.) long, and have a 406 mm (16 in.) deep I-section. Nineteen of the beams are bonded post-tensioned with a single tendon. The tendon types included three different multiple wire systems, and a single post-tensioned bar system. Tendons were located in a bright steel duct, and were grouted with plain grout (w/c = 0.40 to 0.49 with a small amount of aluminum powder for expansion). The twentieth beam was unbonded post-tensioned with a multiple wire, grease coated, paper-wrapped tendon. Concrete cover to mild steel reinforcement was 19 mm (0.75 in.). Minimum cover to the post-tensioning ducts was 43 to 48 mm (1.7 to 1.9 in.). Post-tensioning anchorages were either recessed in a pocket or external. Recessed anchorage pockets were filled with different materials for protection. External anchorages were protected by capping the beam ends with concrete after stressing. Twelve different anchorage protection schemes were investigated, as listed in Table 8.3.

Table 8.3 – Anchorage Protection Schemes[129]

#	Anchorage Location	Protection Material	Surface Preparation	End Reinforcement
1	External	Concrete	None	No
2	Pocket	Concrete	None	n/a
3	External	Concrete	None	Yes
4	External	Concrete	Bush-hammered	No
5	External	Concrete	Bush-hammered	Yes
6	External	Concrete	Retarder	No
7	External	Concrete	Sandblasted and Epoxy-coated	No
8	Pocket	Concrete	Sandblasted and Epoxy-coated	n/a
9	External	Epoxy Concrete	Sandblast and Primer	No
10	Pocket	Epoxy Concrete	Sandblast and Primer	n/a
11	External	Epoxy Concrete	Sandblast and Primer	Yes
12	Pocket	Mortar	Sandblast	n/a

Protection Materials:

Concrete:	w/c = 0.80, f'_c = 21 MPa (3000 psi)
Epoxy Concrete:	epoxy binder, f'_c = 64 – 78 MPa (9230 – 11230 psi)
Mortar:	sand and cement, w/c = 0.44, aluminum powder for expansion, f_m = 53 MPa (7750 psi)

A summary of the observations and conclusions from the Treat Island studies is provided below.

1. Heavy concrete spalling over mild steel reinforcement was reported within the first ten years of exposure. Spalling was attributed to reinforcement corrosion and low concrete cover (19 mm [0.75 in.]).

2. Structural testing of the first eight beams (1973 and 1974) indicated that the prestressing wires in the bonded post-tensioned beams were not structurally damaged by corrosion. The unbonded, greased and wrapped post-tensioned beam showed the most corrosion, and this fact was reflected in structural testing data.

3. Forensic examination revealed some amount of corrosion on all wires in all beams. For bonded tendons, corrosion was found along the entire length of the tendons with no particular concentrations of damage. Corrosion damage was found on the ducts, particularly at the joints, suggesting moisture and chlorides had penetrated through the duct to reach the tendons. Corrosion observed in 1983 was deemed no more severe than corrosion observed in 1973 and 1974. The beam with an unbonded tendon had heavier corrosion damage at the ends, suggesting moisture and chlorides had penetrated through the anchorage region.

4. Examination of the prestressing wires suggested no evidence of hydrogen embrittlement in spite of the use of aluminum powder in the post-tensioning grout.

5. The best anchorage protection for the external anchorages was provided by the epoxy concrete end caps. Most of the external anchorages were corroded. All end caps without reinforcement failed, while there were no failures for reinforced end caps. The retarder surface preparation produced the least amount of end cap failures.

6. All of the recessed anchorages were well protected by the concrete or mortar plugs. Some corrosion was found on several of the anchorages. It was concluded that the anchorage plugs only protected the anchorages as well as the surrounding concrete and that anchorage corrosion may not necessarily be due to a deficiency in the anchorage protection methods investigated.

7. Thermal expansion tests conducted in 1983 suggested differences in thermal expansion properties between the epoxy concrete and beam concrete could lead to premature deterioration of bond between the materials.

8. The most effective anchorage protection was deemed to be the recessed anchorage. No specific recommendations were made regarding the protection material or surface preparation.

8.6 R.W. POSTON

Poston[132] investigated the use of post-tensioning to improve corrosion protection in bridge decks. This research was discussed previously in Section 4.4.2 and is summarized in Appendix A, Section A.2.3.1. The research was a comparison between nonprestressed and post-tensioned bridge deck designs. Specimens were subjected to aggressive saltwater exposure, and examined for chloride penetration, half-cell potentials and incidence and extent of corrosion damage. More detail is provided in Section A.2.3.1. The conclusions of the study are summarized below.

1. Corrosion of nonprestressed reinforcement initiated and occurred only at the location of flexural cracks. In many cases, corrosion had spread over a distance of 6 to 10 bar diameters (uncoated bars). The incidence and extent of corrosion was much less for the epoxy coated bars.

2. For both nonprestressed and prestressed specimens loaded to produce crack widths of 0.38 mm (0.015 in.), the incidence and extent of corrosion was similar. Virtually no incidence of corrosion of nonprestressed reinforcement was observed in the prestressed specimens with a crack width of 0.051 mm (0.002 in.). This lack of corrosion with crack widths more typical of prestressed concrete represents the most significant effect of prestressing/crack control on reinforcement corrosion.

3. Prestressing had little effect on chloride ion penetration in regions of uncracked concrete. However, chloride ion concentrations at crack widths of 0.051 mm (0.002 in.) were approximately 60% less than at crack widths of 0.38 mm (0.015 in.).

4. For the conditions and time length of the exposure testing in this study, no difference was observed for the two levels of cover considered.

8.7 CONCLUSIONS – CORROSION OF PRESTRESSED CONCRETE RESEARCH

The findings of the reviewed corrosion research for prestressed concrete can be summarized into a number of general conclusions:

- Anchorage corrosion is a common and potentially significant problem.
- The incidence of corrosion was increased in cracked (flexural) specimens. Prestressing to reduce or eliminate cracking improved corrosion protection.
- Epoxy coating of anchorages, ducts and prestressing strand improved corrosion protection.
- Recessed anchorage pockets offer better corrosion protection than external anchorages.
- Plastic (polyethylene) ducts are virtually impermeable and can significantly improve corrosion protection.
- Low permeability grouts for post-tensioning improved corrosion protection.

The reviewed research did not address a number of factors of interest to the research topic of this project. Some questionable aspects include:

- The majority of previous research had a short exposure duration. Some aspects of corrosion in prestressed concrete may be significantly influenced by exposure duration, and longer exposures could affect research findings.
- Much of the reviewed research did not consider the combined action of structural loading and aggressive environment.
- The majority of previous research did not consider specimen sizes and details representative of typical, modern prestressed concrete construction.
- The Treat Island studies, although long term, are limited in their usefulness by the lack of structural loading and obsolete post-tensioning systems.
- There was a lack of comparison between reinforced concrete and prestressed concrete specimens in most cases. As a result, it is not possible to assess the effect of prestressing for corrosion protection.
- Some corrosion protection variables were examined at the component level only, and not in representative structural elements.
- The effect of damage to epoxy coatings was not investigated.

Chapter 9:
Crack Prediction In Structural Concrete Members

Many methods for predicting crack widths have been developed for reinforced concrete, and to a much lesser extent for prestressed concrete. Most crack prediction methods are fundamentally based on one of two approaches:[133]

- Methods relating crack width to the tensile stress in the nonprestressed (mild steel) reinforcement, and
- Methods relating the crack width to a fictitious tensile stress in the concrete.

The first method is more widely used, and is the focus of this section. The development of crack width prediction methods has traditionally used either a statistical analysis of test data or basic principles of cracking in concrete. In the latter case, the methods are normally refined using crack data.

Five different crack prediction methods are discussed in this chapter. The first is a widely used statistically based model for reinforced concrete. Two models based on cracking principles are presented. Finally, two statistically based models for crack prediction in partially prestressed structures are outlined. The crack prediction models in this section are used for the long-term beam exposure test specimens in Chapter 3.

9.1 GERGELY-LUTZ SURFACE CRACK WIDTH EXPRESSION

The Gergely-Lutz crack width expression[134] is a well-known method for estimating maximum surface crack widths for reinforced concrete members. A modified form of the Gergely-Lutz expression is used for the crack control provisions contained in Clause 5.7.3.4 of the *AASHTO LRFD Bridge Design Specifications*.[3] Clause 5.7.3.4 emphasizes reinforcement details (bar spacing and concrete cover) and the level of stress in the bars at service load levels, and does not explicitly compute crack widths. Clause 10.6.4 of the ACI Building Code Requirements for Structural Concrete (ACI 318-95)[47] also uses a modified form of the Gergely-Lutz expression. Although different from the AASHTO format, the ACI 318 approach also emphasizes reinforcement details and the level of stress in the bars rather than calculated crack widths. The ACI Publication ACI 224R-90, "Control of Cracking in Concrete Structures"[53] also recommends the Gergely-Lutz expression.

The Gergely-Lutz expression for maximum tension face surface crack widths was developed based on an extensive multiple regression analysis of data from six experimental investigations of cracking in reinforced concrete. The primary variables include the steel stress, concrete cover, area of concrete in tension and the number of reinforcing bars. Two expressions were proposed by Gergely and Lutz, with the simpler version adopted by AASHTO[3] and ACI.[47,53] This expression is shown in Eq. 9.1.

$$w = 7.6 \times 10^{-5} \frac{h_2}{h_1} f_s \left(d_c \frac{A_e}{m} \right)^{1/3} \qquad \textbf{Eq. 9.1}$$

where:

w = tensile face surface crack width, in.
A_e = $2b(h-d)$: effective area of concrete in tension surrounding tensile reinforcement, in^2
m = number of tensile reinforcing bars
b' = width of beam at centroid of tensile reinforcement, in.
h = overall depth of beam
d = effective depth of beam to centroid of tensile reinforcement, in.

d_c = thickness of concrete cover measured from the extreme tension fiber to center of bar located closest thereto, in.
f_s = steel stress calculated by elastic cracked section theory, ksi
h_2 = $h - c$
h_1 = $d - c$
c = distance from neutral axis to compression face, in.

The effective area of concrete in tension is illustrated in Figure 9.1. To use Eq. 9.1 with S.I. units, replace the multiplier 7.6 x 10^{-5} with 1.1 x 10^{-5}.

Armstrong et al[6] investigated the applicability of the Gergely-Lutz expression for structural concrete members with a mixture of prestressed and nonprestressed (mild steel) reinforcement. Armstrong et al found that Eq. 9.1 produced good results for this case when used with several simple modifications:

- When determining m, the number of reinforcing bars, it is recommended to use the actual number of nonprestressed bars in the tension zone, and then add to that number one fictitious nonprestressed bar for each bonded prestressing strand present.
- The steel stress, f_s, should be that for the nonprestressed reinforcement calculated by elastic cracked section theory accounting for the presence of prestressing forces and prestressed reinforcement.
- The effective depth of the beam should be calculated based on the primary flexural reinforcement, including mild steel reinforcement and prestressed reinforcement, but ignoring "skin steel" in large members.

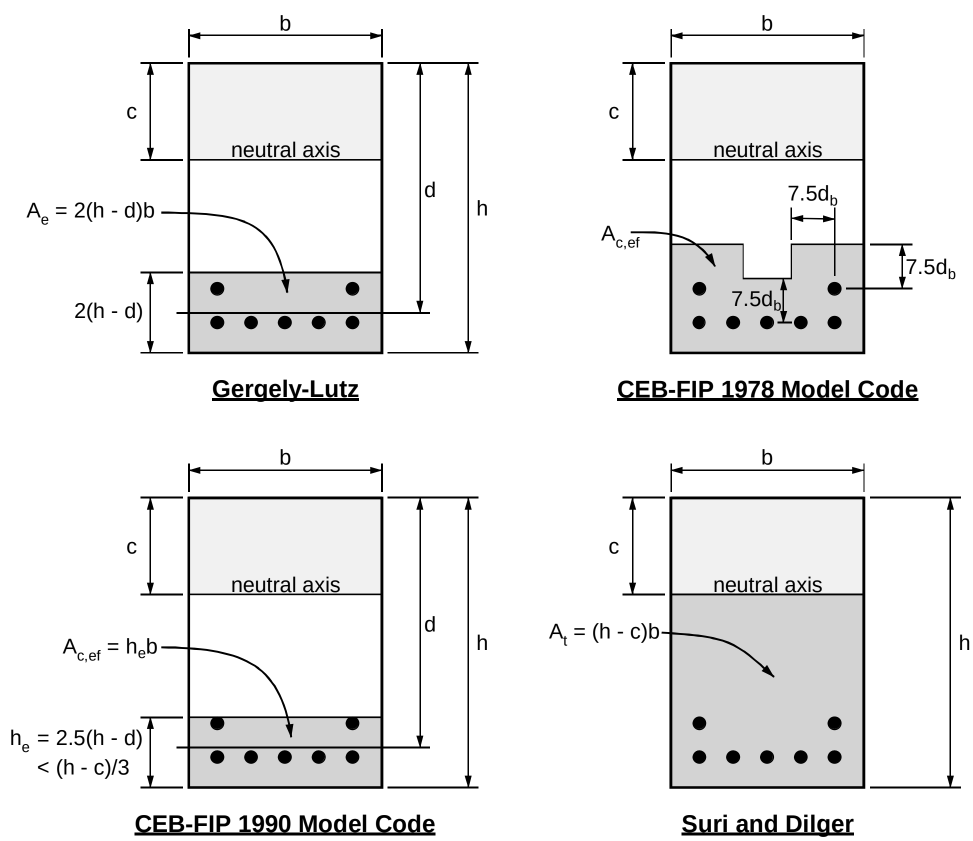


Figure 9.1 - Calculation of Effective Concrete Area in Tension for Various Models

9.2 CEB-FIP 1978 MODEL CODE CRACK WIDTH MODEL

The CEB-FIP 1978 Model Code (MC 78)[135] identifies cracking as a limit state in the design process. The MC 78 crack width model is based on general principles of cracking in concrete. Crack widths are determined as a function of average steel strain, accounting for tension stiffening from the concrete, and a predicted average crack spacing. The mean estimated crack width is the product of these two factors. The design maximum or characteristic crack width is taken as 1.7 times the mean crack width. The characteristic crack width is compared to allowable limits to satisfy the limit state. The MC 78 crack width model is given in Eq. 9.2. More detail on the MC 78 cracking model is provided in Reference 136.

$$w_k = 1.7\, w_m = 1.7\, s_{rm}\varepsilon_{sm} \qquad \textbf{Eq. 9.2}$$

where:

w_k = characteristic crack width, mm
w_m = mean crack width, mm
s_{rm} = average crack spacing, mm
ε_{sm} = mean steel strain for reinforcement situated in the effective embedment section, taking into account the contribution of the concrete in tension as shown in Figure 9.2.

with:

$$s_{rm} = 2\left(c + \frac{s}{10}\right) + \kappa_1\kappa_2\frac{\phi}{\rho_r} \qquad \textbf{Eq. 9.3}$$

$$\varepsilon_{sm} = \frac{\sigma_s}{E_s}\left[1 - \beta_1\beta_2\left(\frac{M_{cr}}{M}\right)^2\right] \geq 0.4\frac{\sigma_s}{E_s} \qquad \textbf{Eq. 9.4}$$

where:

c = concrete cover, mm
s = spacing of reinforcing bars, mm ($\leq 15\phi$)
ϕ = bar diameter, mm
κ_1 = coefficient for bond properties of steel
= 0.4 for deformed reinforcing bars
= 0.8 for smooth reinforcement, including prestressing strand
κ_2 = coefficient for strain profile within effective embedment zone
= $0.25(\varepsilon_1 + \varepsilon_2) / 2\varepsilon_1$
ε_1 = concrete strain at top of effective embedment zone
ε_2 = concrete strain at bottom of effective embedment zone
ρ_r = $A_s / A_{c,ef}$
A_s = total steel area within $A_{c,ef}$, including bonded prestressing steel, mm^2
$A_{c,ef}$ = effective embedment zone, mm^2: zone of concrete in tension where the reinforcement can effectively influence the crack widths. $A_{c,ef}$ is determined as shown in Figure 9.1. The procedure is to superimpose a square with dimensions of 15ϕ centered on each reinforcing bar / strand to determine the extent of $A_{c,ef}$. In slabs, the height of $A_{c,ef}$ is bounded by $(h - c) / 2$.
σ_s = stress in nonprestressed reinforcement calculated for a cracked section under the combination of actions being considered, MPa
E_s = elastic modulus of steel, MPa
β_1 = coefficient for bond properties of steel
= $1 / (2.5\kappa_1)$
β_2 = coefficient for influence of loading duration / application
= 1 for first loading

	=	0.5 for sustained or repeated loading
M_{cr}	=	cracking moment for the section under consideration, kN-m
M	=	applied moment for the combination of actions being considered, kN-m

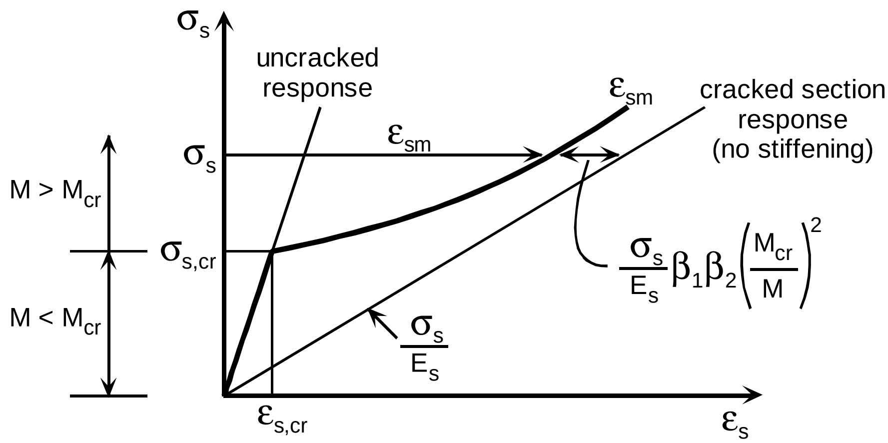


Figure 9.2 - Mean Reinforcement Strain, ε_{sm}, Accounting for the Contribution of Concrete in Tension (MC 78)

9.3 CEB-FIP 1990 MODEL CODE CRACK WIDTH MODEL

The CEB-FIP 1990 Model Code (MC 90)[137] also specifically identifies cracking as a limit state in the design process. Similar to MC 78, the MC 90 crack width model is based on general principles of cracking in concrete. However, the MC 90 model defines the characteristic crack widths as a function of the length over which slip between steel and concrete occurs near a crack, and the difference between the average steel and concrete strains within the length of slip. The characteristic crack width is compared to allowable limits to satisfy the limit state. The MC 90 crack width model also allows the effect of shrinkage strains to be introduced. Another difference between MC 90 and MC 78 is that MC 90 identifies different phases of cracking to better represent observed cracking behavior and crack formation in structural concrete, as shown in Figure 9.3.

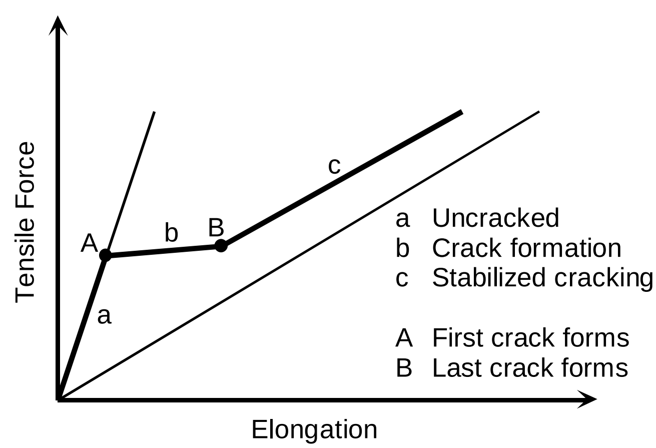


Figure 9.3 - Idealized Phases of Cracking Behavior for a Reinforced Concrete Tension Tie (adapted from Ref. 137)

The MC 90 crack width model is as follows:

$$w_k = L_{max}\left[\left(\varepsilon_{sm} - \varepsilon_{cm}\right) - \varepsilon_{cs}\right]$$ **Eq. 9.5**

where:

w_k = characteristic crack width, mm
L_{max} = length over which slip between the steel and concrete occurs, mm
ε_{sm} = average steel strain within L_{max}
ε_{cm} = average concrete strain within L_{max}
ε_{cs} = concrete strain due to shrinkage

with:

$$\begin{aligned}\left(\varepsilon_{sm} - \varepsilon_{cm}\right) &= \left(\varepsilon_{s2} - \beta\Delta\varepsilon_{sr}\right) - \beta\varepsilon_{sr1} \\ &= \left(\varepsilon_{s2} - \beta\varepsilon_{sr2}\right)\end{aligned}$$

where:

ε_{s2} = steel strain at the crack, calculated for a cracked section under the combination of actions being considered
β = empirical factor to assess average strain within L_{max} (see Table 9.1)
$\Delta\varepsilon_{sr}$ = ε_{sr2} - ε_{sr1}
ε_{sr1} = steel strain in the uncracked section under cracking forces reaching f_{ctm}
ε_{sr2} = steel strain at the crack, under forces causing f_{ctm} within $A_{c,ef}$. ε_{sr2} is analogous to the cracked section steel strain calculated at the cracking moment, and is approximated in MC 90 by Eq. 9.6. ε_{sr2} should not be taken greater than ε_{s2}.

$$\varepsilon_{sr2} = \frac{f_{ctm}}{\rho_{s,ef} E_s}\left(1 + \alpha_e \rho_{s,ef}\right)$$ **Eq. 9.6**

f_{ctm} = mean value of concrete tensile strength at the time of cracking, MPa
$\rho_{s,ef}$ = effective reinforcement ratio, $A_s / A_{c,ef}$
A_s = steel area within $A_{c,ef}$, mm^2
$A_{c,ef}$= effective area of concrete in tension, as illustrated in Figure 9.1, mm^2
α_e = E_s / E_c (E_c at the time of cracking)

The various steel strains are illustrated in Figure 9.4.

Table 9.1 - Values of β and τ_{bk} for MC 90

	Single Crack Formation		Stabilized Cracking	
	β	τ_{bk}	β	τ_{bk}
Short term / instantaneous loading	0.6	$1.8f_{ctm}$	0.6	$1.8f_{ctm}$
Long term / repeated loading	0.6	$1.35f_{ctm}$	0.38	$1.8f_{ctm}$

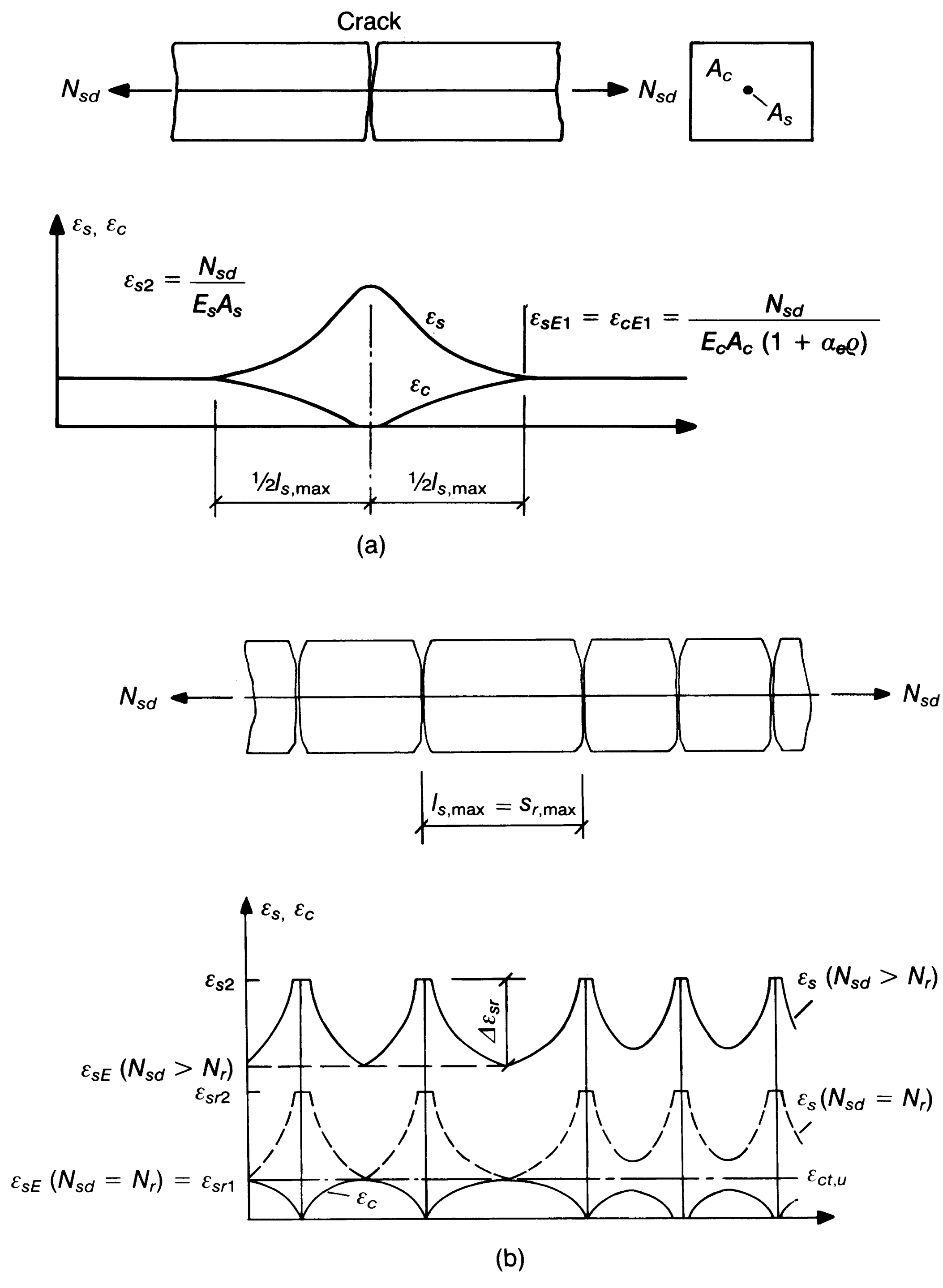


Figure 9.4 - Strains for Calculating Crack Widths Under MC 90: (a) For Single Crack Formation, (b) for Stabilized Cracking (from Ref. 137)

The length of slip, L_{max}, is dependent on the phase of cracking for the combination of actions being considered. Slightly different provisions are provided for reinforced concrete and prestressed concrete, but some simplifications are permissible to give a generalized form.

9.3.1 Single Crack Formation Phase

The single crack formation phase is defined as follows:

$\rho_{s,ef}\sigma_{s2} \leq f_{ctm}\left(1+\alpha_e\rho_{s,ef}\right)$ for reinforced concrete members

$\Delta F_{s+p} \leq f_{ctm} A_{c,ef}$ for prestressed concrete members

where:

σ_{s2} = steel stress at the crack, calculated for a cracked section under the combination of actions being considered, MPa

ΔF_{s+p} = force in tensile reinforcement after decompression, kN

= $A_s\sigma_s + A_p\Delta\sigma_p$ (expressions are provided in MC 90 to estimate σ_s and $\Delta\sigma_p$, or they may be calculated using first principles)

$$L_{max} = \frac{1}{\lambda}\left[\frac{\sigma_{s2}\phi_s}{2\tau_{bs,k}} + \frac{\Delta\sigma_p\phi_p}{2\tau_{bp,k}}\right]$$ for single crack formation **Eq. 9.7**

where:

λ = 1 for reinforced concrete

= 2 for combinations of mild steel reinforcement and prestressing steel

ϕ_s = reinforcing bar diameter, mm

ϕ_p = prestressing steel diameter, mm

$\tau_{bs,k}$ = characteristic bond stress for deformed reinforcing bars, MPa

= $1.8f_{ctm}$

$\tau_{bp,k}$ = characteristic bond stress for prestressing steel, MPa

= $0.36f_{ctm}$ for post-tensioning tendons with smooth bars or wires

= $0.72f_{ctm}$ for post-tensioning tendons with strands or indented wires

= $1.08f_{ctm}$ for post-tensioning tendons with ribbed bars

= $1.08f_{ctm}$ for pretensioned tendons with ribbed bars

= $0.72f_{ctm}$ for pretensioned tendons with strands

9.3.2 Stabilized Cracking Phase

The stabilized cracking phase is defined as follows:

$\rho_{s,ef}\sigma_{s2} > f_{ctm}\left(1+\alpha_e\rho_{s,ef}\right)$ for reinforced concrete members

$\Delta F_{s+p} > f_{ctm} A_{c,ef}$ for prestressed concrete members

$$L_{max} = \frac{\phi_s}{3.6(\rho_{s,ef} + \xi_1\rho_{p,ef})}$$ for stabilized cracking **Eq. 9.8**

where:

$\rho_{p,ef}$ = effective prestressed reinforcement ratio, $A_p / A_{c,ef}$

A_p = prestressed steel area within $A_{c,ef}$, mm^2

ξ_1 = $(\tau_{bp,k}\,\phi_s)/(\tau_{bs,k}\,\phi_p)$

9.4 BATCHELOR AND EL SHAHAWI CRACK WIDTH EXPRESSION

Batchelor and El Shahawi[133] developed a very simple crack width expression for use in partially prestressed members. The expression, shown in Eq. 9.9, was based on a regression analysis of data from five experimental investigations of cracking in partially prestressed members. Batchelor and El Shahawi investigated the influence of a large number of parameters, but concluded that the large scatter in the test data warranted a simple expression based only on the stress in the nonprestressed reinforcement.

$$w_{max} = \frac{0.96 f_s - 46}{1000} \qquad \textbf{Eq. 9.9}$$

where:

w_{max} = maximum surface crack width, mm

f_s = stress in nonprestressed reinforcement calculated for a cracked section under the combination of actions being considered, MPa

9.5 SURI AND DILGER CRACK WIDTH EXPRESSION

Suri and Dilger[138] developed a crack width expression to predict crack widths in partially prestressed concrete members. The Suri and Dilger expression, shown in Eq. 9.10, was based on a statistical analysis of test data from eighteen investigations. The regression analysis revealed that the controlling variables were similar to those used for the Gergely-Lutz expression (see Section 9.1). For practical design purposes, Suri and Dilger recommend increasing the crack widths obtained using Eq. 9.10 by 25%.

$$w_{max} = k\, f_s\, c \left(\frac{A_t}{A_s + A_p} \right)^{0.5} \qquad \textbf{Eq. 9.10}$$

where:

w_{max} = maximum surface crack width, mm

k = factor to account for different bond properties of various steels
= 2.55×10^{-6} for combinations of reinforcing bars and prestressing strands
= 3.51×10^{-6} for combinations of reinforcing bars and prestressing wires
= 2.65×10^{-6} for prestressing strands only
= 4.50×10^{-6} for prestressing wires only

f_s = stress in nonprestressed reinforcement calculated for a cracked section under the combination of actions being considered, MPa

c = concrete cover from the tensile face to the center of the nearest bar, mm

A_t = area of concrete in tension below the neutral axis (see Figure 9.1), mm^2

A_s = area of mild steel reinforcement in tension zone (A_t), mm^2

A_p = area of prestressing steel in tension zone (A_t), mm^2

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Appendix A
Crack Widths and Corrosion: Literature Review

This appendix provides a review of literature on the relationship between cracking and corrosion in structural concrete, a subject that has received much debate.[A.1] The literature review in this appendix covers the following sections:

A discussion of each of the two main sections is provided in Section 4.4 of this Report.

A.1 TECHNICAL ORGANIZATIONS

The following sections present the recommendations and guidelines, related to cracking and the prevention of corrosion, published by various technical organizations around the world. These publications include major building and bridge codes and specifications for structural concrete in North America, Europe and Japan, and the publications of several technical committees from the American Concrete Institute (ACI) and the Comite Euro-International Du Beton (CEB). Some of the publications do not address crack control as a method of corrosion protection, specifying other measures instead. These publications have been discussed briefly for the completeness of the literature summary.

A.1.1 Design Codes and Specifications

A.1.1.1 ACI 318-95

The American Concrete Institute (ACI) Publication ACI 318-95, "Building Code Requirements for Structural Concrete,"[A.2] is widely used for the design of structural concrete. Although ACI 318-95 has an entire chapter devoted to durability (Chapter 4), the effect of cracking on durability is not addressed. Rather, protection against corrosion of reinforcement is addressed through specification of a maximum water-cement ratio, minimum concrete strength and maximum chloride ion content for concrete mix constituents. The control of crack widths in ACI 318-95 is addressed implicitly in Clause 10.6 (Distribution of flexural reinforcement in beams and one-way slabs). For reinforcement with nominal yield strength less than 40 ksi (300 MPa), no specific requirements are necessary. For reinforcement with yield strengths exceeding 40 ksi, Clause 10.6.4 provides an empirical expression based on the Gergely-Lutz equation for crack widths,[A.3] written in a form that emphasizes reinforcement details (spacing and cover) and the level of stress in the bars at service load levels. The quantity "Z" is calculated using equation (10-4) of Clause 10.6.4 (shown below) and is compared to the limits of 175 kips/in (30.6 kN/mm) for interior exposure and 145 kips/in (25.4 kN/mm) for exterior exposure. A specific definition is not given for interior or exterior exposures. These limits for Z correspond to surface crack widths of 0.016 in. (0.4 mm) for interior exposures and 0.013 in. (0.34 mm) for exterior exposures. The basis of selection for the limiting values of Z (allowable crack widths) is not provided in either ACI 318-95 or its commentary.

$$Z = f_s \sqrt[3]{d_c A} \qquad \text{(kips/in. or kN/mm)} \qquad \text{(10-4)}^{A.2}$$

where:

f_s = calculated stress in reinforcement at service loads

d_c = thickness of concrete cover measured from extreme tension fiber to center of bar or wire closest thereto

A = effective tension area of concrete surrounding the flexural tension reinforcement and having the same centroid as that reinforcement, divided by the number of bars or wires

Equation (10-4) is a modification of the Gergely-Lutz[A.3] expression described in Chapter 9 of this report. By comparing Equation (10-4) with the Gergely-Lutz equation, the following definition is obtained for Z:

$$Z = \frac{w}{(7.6x10^{-5})\beta} \qquad \text{(units of kip/in. for Z and inches for w)}$$

For beams, β may be taken as 1.2, and the limiting values of Z corresponding to the desired maximum surface crack widths are obtained.

Although the approach of Section 10.6 of ACI 318 is to address reinforcement details and to de-emphasize crack widths, from the preceding discussion, it is clear that the underlying principle remains to be allowable crack widths. The move by ACI away from crack widths to the quantity Z was primarily for legal reasons. Due to the nature of cracking and the wide variability involved in predicting crack widths, comparison of predicted crack widths to the measured crack widths in situ often revealed inconsistencies which were used either justly or unjustly in cases of dispute. Thus, by utilizing the parameter Z which has no physical meaning, measured crack widths could no longer be compared to Code provisions for crack control, avoiding unnecessary legal action on the basis of cracking.

As mentioned previously, the limiting values of Z are proposed for interior and exterior exposures. For severe exposures, no explicit recommendations are made. Instead, Clause 10.6.5 states:

> "**10.6.5 -** Provisions of 10.6.4 are not sufficient for structures subjected to very aggressive exposure or designed to be watertight. For such structures, special investigations and precautions are required."

Unfortunately, no special investigations or precautions are suggested. The commentary for 10.6.5 provides the following comments:

> "**R10.6.5 -** Although a number of studies have been conducted, clear experimental evidence is not available regarding the crack width beyond which a corrosion danger exists. Exposure tests indicate that concrete quality, adequate compaction and ample concrete cover may be of greater importance for corrosion protection than crack width at the concrete surface. The limiting values for Z were, therefore, chosen

primarily to give reasonable reinforcing details in terms of practical experiences with existing structures."

The implicit provisions for crack control described above are intended for mild steel reinforcement. ACI 318-95 does not provide any form of similar requirements for the use of prestressing steel in prestressed elements or mixed reinforcement (partial prestressing) schemes. Clause 18.1.3 states that Clause 10.6 shall not apply to prestressed concrete. However, it should be noted that ACI 318-95 does not recognize the partial prestressing or prestressed components that are designed to be cracked under service load levels.

A.1.1.2 AASHTO LRFD Bridge Design Specifications

The American Association of State Highway and Transportation Officials (AASHTO) LRFD Bridge Design Specifications[A.4] list cracking as an action to be considered at the Service Limit State (Clause 5.5.2). The AASHTO LRFD Specifications address crack control implicitly through detailing requirements in Clause 5.7.3.4. This clause uses a similar approach to ACI 318-95,[A.2] with slight modifications. A modified form of the Gergely-Lutz expression is used, but re-arranged such that a limiting stress at service loads is computed as a function of reinforcement detailing and the quantity Z. The procedure is as follows:

$$f_{sa} \le \frac{Z}{\sqrt[3]{d_c A}} \le 0.6 f_y \qquad (5.7.3.4\text{-}1)^{A.4}$$

where:

f_{sa} = calculated stress in reinforcement at service loads

d_c = thickness of concrete cover measured from extreme tension fiber to center of bar or wire closest thereto

A = effective tension area of concrete surrounding the flexural tension reinforcement and having the same centroid as that reinforcement, divided by the number of bars or wires

Another difference between the AASHTO LRFD Specifications and ACI 318-95 is that the AASHTO document specifies slightly stricter limits on the value of Z. The limits for Z are 170 kips/in. (29.8 kN/m) for moderate exposure conditions, 130 kips/in. (23 kN/m) for severe exposure conditions and 100 kips/in. (17.5 kN/mm) for buried structures. These values correspond to allowable surface crack widths of 0.0155 in. (0.39 mm), 0.0118 in. (0.33 mm) and 0.009 in. (0.24 mm), respectively.

The commentary states the following regarding Equation 5.7.3.4-1:

> "There appears to be little or no connection between surface crack width and corrosion. Thicker or additional cover for reinforcement will result in greater surface crack widths. These wider crack widths are not detrimental to the corrosion protection of the reinforcement."[A.4]

The commentary goes on to state that in view of the above statement, Equation 5.7.3.4-1 should only be used for concrete cover less than or equal to 2 in. (50 mm). If the cover exceeds 2 in. (50 mm), a limiting value of 2 in. (50 mm) should be used in Equation 5.7.3.4-1.

For aggressive exposure or corrosive environments, the commentary states:

> "… additional protection beyond that provided by satisfying Equation 5.7.3.4-1 may be provided by decreasing the permeability of the concrete and/or waterproofing the exposed surface."[A.4]

The commentary also states

> "Structures subjected to very aggressive exposure are beyond the scope of these provisions. For such conditions, more restrictive limits on crack widths may be required. Narrower surface crack widths may be obtained by using the recommendations in ACI 350R."[A.4] (see Section A.1.2.4 for ACI 350R)

The AASHTO LRFD Specifications permit the use of partially prestressed members (i.e., prestressed members designed to crack under service load conditions (may or may not contain non-prestressed mild steel reinforcement)). Crack control for partial prestressed members is referred to Clause 5.7.3.4, described above. In this situation, bonded prestressing steel is included in the calculation of "A," and the increase in stress in the prestressing steel beyond decompression is used in place of f_{sa} in Equation 5.7.3.4-1. The values of Z are unchanged for partial prestressed members.

For fully prestressed members (uncracked at service load levels) Clause 5.9.4.2.2 limits the allowable concrete tensile stresses under service loads after losses to the following:

$6\sqrt{f'_c}$ psi ($0.5\sqrt{f'_c}$ MPa)	for components with bonded prestressing tendons or reinforcement subjected to moderate (or better) exposure conditions
$3\sqrt{f'_c}$ psi ($0.25\sqrt{f'_c}$ MPa)	for severe corrosive exposure conditions such as coastal areas
No tension	for members with unbonded prestressing tendons

In addition to crack control through detailing requirements, the AASHTO LRFD Specifications address corrosion protection in Section 5.12: Durability. Corrosion protection provisions consist of specification of minimum concrete cover for reinforcement. Requirements are given for various elements and exposure conditions. In general, these minimum cover requirements are the same as ACI 318-95 cover requirements. The minimum covers listed in Clause 5.12.3 may be reduced by 20% for water-cement ratios less than 0.40, and should be increased by 20% for water-cement ratios greater than 0.50. In addition to minimum cover requirements, Section 5.12 indicates that protection against chloride-induced corrosion can be provided by epoxy coating or galvanizing mild steel reinforcement, prestressing steel and post-tensioning ducts and hardware. Clause 5.12.4 allows the cover requirements for interior exposure to be used if epoxy-coated reinforcement is used. No other information is given regarding corrosion protection or implementation of the positive corrosion protection described above.

A.1.1.3 CAN3-A23.3-M84

The Canadian Standards Association Publication CAN3-A23.3-M84, "Design of Concrete Structures for Buildings,"[A.5] uses the same approach for crack control as ACI 318-95. No explicit guidelines for crack widths are given, and crack control is implicitly considered by the distribution of mild steel flexural reinforcement in Clause 10.6.4. The approach used in Clause 10.6.4 is identical to Clause 10.6.4 of ACI 318-95. Similarly, the implied allowable surface crack widths are also 0.016 in. (0.4 mm) for interior exposures and 0.013 in. (0.33 mm) for exterior exposures.

An important difference between the crack control provisions of CAN3-A23.3-M84 and ACI 318-95 is that the Canadian Standard provides information for crack control for partially prestressed members. Clause 18.9.3 of CAN3-A23.3-M84 applies to prestressed members which are cracked under service loads (i.e. the allowable concrete tensile stress under service loads is exceeded). Clause 18.9.3.3 is based on the same form of the Gergely-Lutz crack width expression as Clause 10.6.4 for non-prestressed sections (Equation (10-4)), but with stricter limits for Z. The specified maximum limits for Z in partially prestressed members are 20 kN/mm (115 kips/in.) for interior exposure and 15 kN/mm (85 kips/in.) for exterior exposure. These values of Z correspond to surface crack widths of 0.26 mm (0.010 in.) and 0.20 mm (0.008 in.). The commentary for Clause 18.9.3.3 states that the stricter limits applied to partially prestressed members are necessary due to “uncertainties in computing crack widths for partially prestressed members and because prestressing steel is more susceptible to corrosion.”[A.5]

A.1.1.4 Ontario Highway Bridge Design Code - 1991

Section 8-11 of the Ontario Highway Bridge Design Code (OHBDC)[A.6] contains the minimum requirements for durability of concrete structures. The emphasis of this section is the protection of reinforcement and other hardware from corrosion. The OHBDC addresses durability through concrete cover and protective coatings. Concrete covers are specified for various member types and exposure conditions. Protective coatings are required on reinforcement, anchorages, internal post-tensioning ducts and hardware specified for use within 100 mm (4 in.) of a surface subjected to moisture containing de-icing chemicals. Recommended protective coatings include, in general, epoxy, painting, galvanizing and metalizing. Corrosion resistant material (e.g. polyethylene) may be used for internal post-tensioning ducts in place of protective coating in this situation.

Crack control is addressed separately by the OHBDC in Section 8-12. Crack control is provided by emphasizing the distribution of reinforcement rather than by emphasizing crack widths. A parameter β_2 is computed based on the details of the section, the level of stress in the steel and the ratio of the moment at which the tensile concrete stress reaches $0.4f_{cr}$ to the moment under service loads.

$$\beta_2 = (0.9s_c + 2.0c_c)\, f_s \left[1 - \left(M_w / M_s\right)^2\right] \beta_2 = (0.9s_c + 2.0c_c)\, f_s \left[1 - (M_w / M_s)^2\right]$$

where

f_s = tensile stress in non-prestressed reinforcing bar nearest to the tensile face at the serviceability limit state, MPa (may be taken as 240 MPa (35 ksi) in non-prestressed members)

s_c = clear space between reinforcing bars nearest to the tension face, mm

c_c = clear cover to the reinforcing bar nearest to the tension face, mm

M_w = moment at a section when a tensile stress of $0.4f_{cr}$ is induced in the concrete, N-mm

f_{cr} = cracking strength of concrete

M_s = moment at a section at a serviceability limit state load, N-mm.

Limits for β_2 are given as 50 kN/mm (285 kips/in.) for non-prestressed members and 30 kN/mm (170 kips/in.) for prestressed members. These values correspond to average crack widths at the level of the reinforcement of 0.25 mm (0.010 in.) and 0.15 mm (0.006 in.) for non-prestressed and prestressed members, respectively.[A.7] It is important to emphasize that these crack width limits are specified for the crack width at the level of the steel rather than at the surface of the concrete, as in most cases. The crack widths specified above correspond to flexural crack widths at first loading of a member.

A.1.1.5 CEB-FIP Model Code 1990

The Comite Euro-International Du Beton (CEB) Bulletin No. 213/214 reports the CEB-FIP Model Code 1990.[A.8] The CEB-FIP Model Code addresses cracking as a specific limit state in Section 7.4 of the Model Code. The design criteria for cracking is that the functional requirements, durability and appearance of the structure should not be affected by cracking. This criteria may be satisfied by analytical procedures (calculated crack widths versus tolerable limits) or by practical detailing rules. This approach is applied to both non-prestressed and prestressed members, with slightly different requirements for each.

Analytical Procedures (Crack Widths)

A very complex crack prediction method is presented in the 1990 Model Code. The predicted characteristic crack widths are compared to allowable limits. The crack prediction method is described in detail in Chapter 9 of this report.

For non-prestressed reinforcement, the nominal limit of surface crack width is 0.30 mm (0.012 in.) for exposure conditions including humid environments, with or without de-icing agents and seawater environments. This limit is assumed to be satisfactory for both appearance and durability. For a dry environment, this limit may be relaxed.

For prestressed reinforcement, stricter requirements for crack widths are proposed. Surface crack width limitations are given for various exposure conditions for both post-tensioned and pre-tensioned members, as shown in Table A.1. As the environmental conditions become more severe, cracking is not permitted by not allowing tension in the section. This requirement may be relaxed to a surface crack width of 0.20 mm (0.008 in.) if coated tendons or an impermeable duct are used.

Table A.1 - Surface Crack Width Limits for Prestressed Members (CEB-FIP Model Code 1990)

<table>
<tr><th>Exposure Condition</th><th>Post-tensioned</th><th>Pre-tensioned</th></tr>
<tr><td>Dry Environment</td><td>0.008in. (0.20mm)</td><td>0.008in. (0.20mm)</td></tr>
<tr><td>Humid Environment</td><td>0.008in. (0.20mm)</td><td>Decompression</td></tr>
<tr><td>Humid Envir. with Frost & De-icing Agents</td><td colspan="2" rowspan="2">No Tension Allowed Within Section
OR
Use coated tendons or impermeable ducts (w = 0.2mm)</td></tr>
<tr><td>Seawater Environment</td></tr>
</table>

An important note to accompany this information is given in the commentary for subsection 7.4.1.1:

> "... due to the actual state of the art and the highly probabilistic nature of the related phenomena, such nominal crack width values may only serve as a means to apply the design criterion..., and can in no case be compared to actual crack widths measured in situ."[A.8]

Practical Detailing Rules

As an alternative to calculating crack widths, crack control may also be provided by satisfying the provisions of Section 7.4.4 of the CEB-FIP Model Code. Provisions for reinforcement details are given as maximum bar diameter and maximum bar spacing as a function of steel stress, for both reinforced sections and prestressed sections. This information is shown in Table A.2. The steel stresses are to be calculated under quasi-permanent loads for reinforced concrete and under frequent loads and the characteristic value of prestress for prestressed sections. When these detailing provisions are used, surface crack widths will not generally exceed the value of 0.30 mm (0.012 in.) for reinforced elements and 0.20 mm (0.008 in.) for prestressed elements.

Table A.2 - Maximum Bar Diameter and Spacing for Which No Crack Width Calculation is Needed (CEB-FIP Model Code 1990)

Steel Stress (MPa)	Maximum Bar Diameter (mm)		Maximum Spacing (mm)	
	Reinforced Sections	Prestressed Sections	Reinforced Sections	Prestressed Sections
160	32	25	300	200
200	25	16	250	150
240	20	12	200	100
280	14	8	150	50
320	10	6	100	--
360	8	5	60	--
400	6	4	--	--
450	5	--	--	--

A.1.1.6 British Standard CP110

The British Standards Institution Publication CP 110, "Code of Practice for The Structural Use of Concrete"[A.9] addresses cracking as a separate limit state. The general requirement of Clause 2.2.3.2 is that "cracking of concrete should not adversely affect the appearance or durability of the structure."[A.9] To meet this requirement, some "reasonable limits" are provided. A formula for computing surface crack width is provided in Appendix A of CP 110.

For reinforced concrete members, CP 110 limits surface crack widths to 0.3 mm (0.012 in.). For severe environments, such as alternate wetting and drying or exposure to seawater, the surface crack width is limited to 0.004 times the nominal cover to the main reinforcement. For 50 mm (2 in.) cover, surface crack widths would be limited to 0.2 mm (0.008 in.).

For prestressed concrete elements in which cracking is allowed (i.e. the design flexural tensile stress of the concrete is exceeded), the surface crack width is limited to 0.2 mm (0.008 in.) by CP 110. In severe environments, such as alternate wetting and drying or exposure to seawater, this limit is reduced to 0.1 mm (0.004 in.).

A.1.1.7 SIA Standard 162

The Swiss Society of Engineers and Architects Standard 162, "Concrete Structures,"[A.10] addresses cracking as a limit state. The general approach of SIA Standard 162 is to limit crack width through detailing and arrangement of reinforcement. Crack widths are not explicitly calculated and checked. Provisions are provided for normal requirements and for severe requirements. Normal requirements are defined as conditions where:

- Physical and chemical actions are insignificant
- Cracking produces no damage
- No special requirements regarding watertightness apply
- Cracking does not detract from the appearance of the structure

Severe requirements are defined as conditions where:

- Physical and chemical actions are significant
- Cracking can produce damage

- Watertightness is required
- Limiting crack widths is desirable for aesthetic reasons

For normal requirements, minimum reinforcement is specified to limit crack width in Clause 3.33.4. These requirements are based upon developing the tensile force corresponding to the initial formation of cracks, without yielding. The minimum area of reinforcement is given by

$$A_{s,min} = \frac{\alpha\beta f_{ct} A_{ct}}{f_y}$$

where:

α = factor to account for the influence of bar spacing

for spacing:	≤ 100 mm	$\alpha = 1.0$
	150 mm	$\alpha = 1.1$
	200 mm	$\alpha = 1.2$
	250 mm	$\alpha = 1.3$
	300 mm	$\alpha = 1.4$

β = factor used to calculate the tensile force corresponding to the initial formation of cracks, taking into account the stress distribution:

for rectangular sections in flexure:	$\beta = 0.5$
for T or I shapes in flexure:	$\beta = 0.3$
for box girders in flexure:	$\beta = 0.3$
for members in direct tension:	$\beta = 0.8$ for $h < 0.3$ m
	$\beta = 0.5$ for $h > 0.8$ m
	(interpolate for $0.3 < h < 0.8$ m)

f_{ct} = design value of concrete tensile strength

= 2 MPa for $f_{cube} < 25$ MPa ($f'_c < 21.25$ MPa)

= 2.5 MPa for $f_{cube} > 25$ MPa ($f'_c > 21.25$ MPa)

A_{ct} = critical tension zone of the uncracked concrete section

f_y = design value of yield stress of reinforcing steel (≤ 460 MPa (66.7 ksi))

For severe requirements, the minimum reinforcement required for normal conditions is to be increased by 30%. In addition, stresses in reinforcing steel and increases in stress in prestressing steel are limited according to Clause 3.33.5. Under the actions of all long term loads and one variable load, the stresses in the reinforcing steel and increases in stress in prestressing steel must not exceed the allowable stresses shown in Figure A.1.

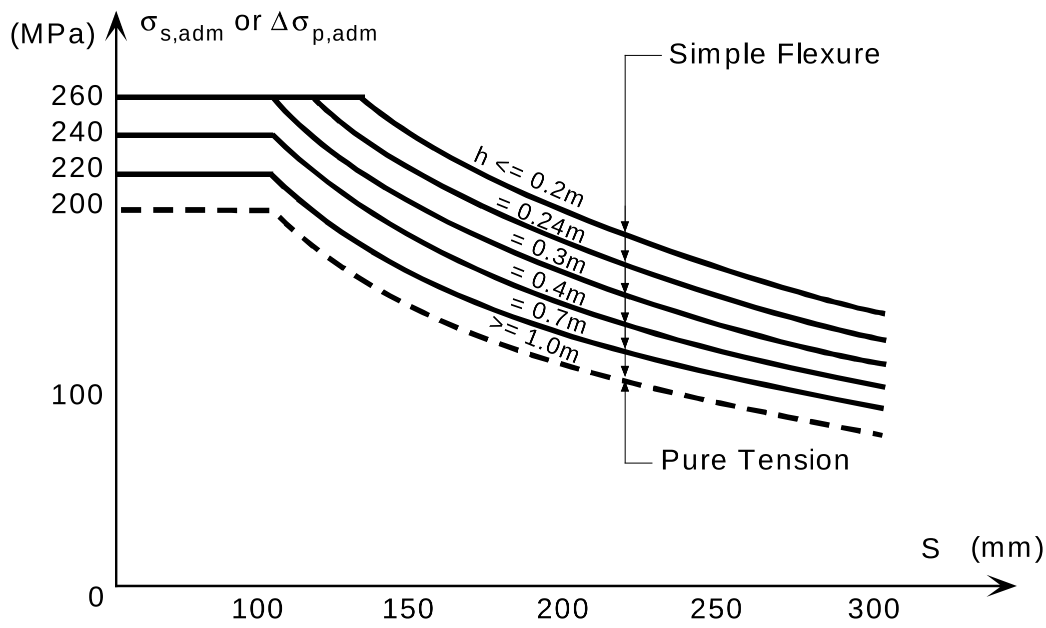

Figure A.1 -Limitation of Cracking Due to Loads (Severe Requirements): Stress in Reinforcing Steel and Increases in Stress in Prestressing Steel (SIA 162[A.10])

A.1.1.8 Standard Specification for Design and Construction of Concrete Structures-JSCE

The Japan Society of Civil Engineers (JSCE) publication SP-1, "Standard Specification for Design and Construction of Concrete Structures - 1986, Part 1 (Design),"[A.11] addresses cracking as a serviceability limit state. The provisions of Section 7.3 are intended to ensure that cracking does not impair the function, durability and appearance of concrete structures. The approach of the JSCE Standard is to ensure that computed surface crack widths satisfy permissible limits. For the purposes of defining permissible crack widths, three exposure conditions are considered: normal, corrosive, and severely corrosive. Severely corrosive environments apply to marine structures exposed to tides and splashing. A corrosive environment is defined as severe alternate wetting and drying or marine structures submerged in seawater. Normal conditions constitute all other exposures.

Permissible surface crack widths specified by the JSCE Standard are defined in terms of the amount of concrete cover in Clause 7.3.3. Different crack widths are specified depending on the exposure condition and the type of steel, as shown in Table A.3. A procedure for computing surface crack widths in reinforced concrete and prestressed concrete members is given in Clause 7.3.4.

The JSCE standard specifies the concrete covers as a function of exposure condition, as shown in Table A.4. Thus, for a reinforced concrete beam, the range of permissible surface crack widths would be 0.15 mm (0.006 in.), 0.20 mm (0.008 in.) and 0.21 mm (0.0083 in.) for normal, corrosive and severely corrosive environments, respectively. For a prestressed concrete beam in a normal environment, the permissible surface crack width would be 0.12 mm (0.0047 in.). A permissible crack width for prestressing steel in a corrosive or severely corrosive environment is not given in Table 3. The commentary to Clause 7.3.3 advises that prestressed concrete members used in these environments should be designed to prohibit the formation of flexural cracks.

Table A.3 -Permissible Surface Crack Width (JSCE SP-1[A.11])

Type of Reinforcement	Environmental Conditions Reinforcement		
	Normal	Corrosive	Severe Corrosive
Deformed and Plain Bars	0.005 x cover	0.004 x cover	0.0035 x cover
Prestressing Steel	0.004 x cover	No cracking	No cracking

Table A.4 - Basic Concrete Cover (JSCE SP-1[A.11])

Environmental Conditions	Slabs	Beams	Columns
Normal	25 mm (1.0 in.)	30 mm (1.2 in.)	35 mm (1.4 in.)
Corrosive	40 mm (1.6 in.)	50 mm (2.0 in.)	60 mm (2.4 in.)
Severe Corrosive	50 mm (2.0 in.)	60 mm (2.4 in.)	70 mm (2.75 in.)

A.1.2 Technical Committee Recommendations

A.1.2.1 ACI 201.2R-92

The ACI Publication ACI 201.2R-92, "Guide to Durable Concrete"[A.12] does not address the subject of crack control for the prevention of reinforcement corrosion in concrete. Rather, recommendations are given for corrosion prevention through the use of concrete with low permeability, mix proportioning (w/c ratio), workmanship, curing, concrete cover, drainage, chloride limitations for mix constituents and positive protection systems, including waterproof membranes and epoxy-coated reinforcement.

A.1.2.2 ACI 222R-96

The ACI Publication ACI 222R-96, "Corrosion of Metals in Concrete"[A.13] provides a discussion of the role of cracking on corrosion. However, it does not provide crack width or detailing requirements for prevention of corrosion. ACI 222R-96 provides the following discussion on crack widths:

> "Studies have shown that cracks less than about 0.3 mm (0.012 in.) have little influence on the corrosion of reinforcing steel.[A.14] Other investigators have shown that there is no relationship between crack width and corrosion.[A.15,A.16,A.34] Furthermore, there is no direct relationship between surface crack width and the internal crack width. Consequently, it has been suggested that the control of surface crack widths in design codes is not the most rational approach[A.43]."

A.1.2.3 ACI 224R-90

ACI Publication ACI 224R-90, "Control of Cracking in Concrete Structures",[A.17] gives additional guidance for crack control in concrete members. Chapter 4 of ACI 224R-90 provides several methods for predicting crack widths in flexural members, including the Gergely-Lutz equation.[A.3] In Section 4.4, a table of tolerable surface crack widths as a function of exposure condition is provided. The predicted maximum widths for a given member must meet the tolerable crack widths to ensure satisfactory durability. Tolerable surface crack widths are given for various exposure conditions including dry air, humidity or soil, de-icing chemicals, seawater with wetting and drying and water retaining structures. The recommended values are shown in Table A.5 of this report.

Table A.5 - Tolerable Surface Crack Widths at the Tensile Face of Reinforced Concrete Members (ACI 224R-90)

Exposure Condition	Tolerable Crack Width
Dry Air	0.016 in. (0.40mm)
Humidity or Soil	0.012 in. (0.30mm)
De-icing Chemicals	0.007 in. (0.18mm)
Seawater: wetting and drying	0.006 in. (0.15mm)
Water Retaining Structures	0.004 in. (0.10mm)

With regard to the recommended tolerable surface crack widths, Section 4.4 emphasizes the following:

> "It is important to note that these values of crack width are not always a reliable indication of the corrosion and deterioration to be expected. In particular, a larger cover, even if it leads to a larger surface crack width, may sometimes be preferable for corrosion control in certain environments. Thus the designer must exercise engineering judgment on the extent of crack control to be used."

The allowable surface crack widths proposed by ACI 224R-90 are based on a paper by E.G. Nawy.[A.18] Nawy summarizes typical allowable crack widths reported by several researchers, the CEB and the U.S. Bureau of Public Roads. The tolerable crack widths presented in ACI 224R-90 appear to be strongly influenced by those recommended by the U.S. Bureau of Public Roads,[A.19] shown in Table A.6. The USBPR values are for maximum crack width at the level of the reinforcement. No source is given for these recommendations. It should be noted that all of the work reported in the paper by Nawy, the basis of Table 4.1 in ACI 224R-90 (Table A.5 in this report), is in excess of 30 years old.

ACI 224R-90 addresses flexural crack control in prestressed members in Section 4.5. This section is intended for "partially prestressed members" which may be cracked under service loading. ACI 224R-90 suggests that crack control for this type of element is mainly for aesthetic reasons and that in situations where live loading is transitory, the residual crack width is normally small (0.001 in. to 0.003 in. (0.025 mm to 0.076 mm)) and crack control is not necessary. Some discussion is provided for calculation of crack widths using equations developed for reinforced concrete members (non-prestressed). With regard to allowable crack widths, ACI 224R-90 reports that some researchers indicate that corrosion may be a greater concern in prestressed concrete due to the smaller area of steel, thus suggesting stricter crack width limitations. However, ACI 224R-90 also indicates that other researchers have reported that there is no relationship between cracking and corrosion and thus does not offer any recommendations. In closing, ACI 224R-90 suggests that crack width limits for prestressed members should consider the magnitude and fluctuation of the live load. Unfortunately, no further guidelines are given.

Table A.6 - Maximum Permissible Crack Width at Level of Reinforcement at Working Load Level (U.S. Bureau of Public Roads[A.19])

Exposure Conditions	D.L. causes comp., L.L. causes tension	D.L. causes tension, L.L. causes tension
Air, or when a protective membrane is applied to surface	0.012 in. (0.30 mm)	0.010 in. (0.25 mm)
Salt air, water and soil	0.010 in. (0.25 mm)	0.008 in. (0.20 mm)
De-icing chemicals, humid tropical climate	0.008 in. (0.20 mm)	0.006 in. (0.15 mm)
Seawater and seawater spray; alternate wetting and drying	0.008 in. (0.20 mm)	0.006 in. (0.15 mm)

A.1.2.4 <u>ACI 350R-89</u>

ACI Publication ACI 350R-89, "Environmental Engineering Concrete Structures,"[A.20] provides recommendations for the structural design and construction of structures used in water and wastewater treatment facilities. For this type of structure, minimal cracking is a paramount requisite and emphasis is placed on structural design that minimizes the possibility of cracking and provides resistance to chemical attack. Thus, the guidelines provided by ACI 350R-89 are of some interest to the durability design of bridge substructures in severe environments.

The structural design process presented in ACI 350R-89 is based on the requirements of ACI 318-95[A.2] with special limitations for application to environmental structures to minimize leakage and improve durability in the extreme environment of environmental service. Both strength design procedures and working stress design procedures are addressed by ACI 350R-89 for this type of structure.

For strength design, the importance of the service limit state (i.e. minimized leakage and cracking) is considered indirectly by increasing the required strength of the members. The load factor for lateral liquid pressure (F) is increased from 1.4 to 1.7. The required strength to resist factored loads, U, is further increased by sanitary durability coefficients as follows:

a) In calculations for reinforcement in flexure, the required strength should be 1.3U

b) In calculations for reinforcement in direct tension, the required strength should be 1.65U

c) In calculations for the shear capacity provided by mild steel reinforcement, use $\phi V_s > 1.3(V_u - \phi V_c)$

d) In calculations for compressive regions of concrete in flexure or under axial load, the required strength should be 1.00U (unmodified)

The sanitary durability coefficients used to increase U were based on crack width requirements. No discussion as to the development of these coefficients is given in ACI 350R-89.

For working stress design, reduced allowable stresses are specified for both concrete and mild steel reinforcement. In addition, spacing requirements are specified for different bar sizes as a function of service load stress. Recommended allowable stresses for concrete are shown in Table A.7. These working stresses are specified in terms of f'c for all strengths of concrete. The minimum compressive strength for concrete recommended by ACI 350R-89 is 3500 psi. This value is increased to 4000 psi if the concrete is subjected to severe and frequent freezing and thawing. Maximum allowable working stresses for mild steel reinforcement are given in Table A.8 for a maximum bar spacing of 12 in (304.8 mm).

For both strength design and working stress design, crack control is addressed through the quantity Z, as defined in Section 2.1.1 of this report. For environmental structures, two levels of sanitary exposure are considered: normal and severe. Normal sanitary exposure is defined as "liquid retention (watertight), exposure to liquids with pH > 5 or exposure to sulfate solutions less than 1500 ppm. Severe sanitary exposures are conditions in which the limits defining normal sanitary exposures are exceeded."[A.20] For normal exposures, a maximum Z value of 115 kips/in. (20 kN/mm) is specified. This corresponds to a surface crack width of 0.010 in. (0.25 mm). For severe sanitary exposure conditions, a maximum Z value of 95 kips/in. (16.6 kN/mm) is specified, corresponding to a surface crack width of 0.0087 in. (0.22 mm).

Table A.7 - Recommended Allowable Concrete Stresses (psi) (ACI 350R-89)

Description	Stress
Flexure;	
Extreme fiber stress in compression	0.45 f'c
Extreme fiber stress in tension in plain concrete footings and walls	$1.6\sqrt{f'_c}$
Shear;	
Beams with no web reinforcement	$1.1\sqrt{f'_c}$
Joists with no web reinforcement	$1.2\sqrt{f'_c}$
Members with web reinforcement or properly combined bent bars and vertical stirrups	$5\sqrt{f'_c}$
Slabs and footings (peripheral shear)	$2\sqrt{f'_c}$
Bearing;	
On full area	0.25 f'c
On one-third area or less	0.375 f'c

Table A.8 - Recommended Stresses for Reinforcement at Service Loads for a Maximum Spacing of 12 in. (304.8 mm) (ACI 350R-89)

Bar Sizes	Exposure Condition	Maximum stress at service load	
		Grade 60	Grade 40
All sizes	Members in direct tension All Exposures	20 ksi (138 MPa)	14 ksi (96 MPa)
#3, #4, #5	Flexural Members Severe Exposure	22 ksi (152 MPa)	20 ksi (138 MPa)
	Flexural Members Normal Sanitary Exposure	27 ksi (186 MPa)	20 ksi (138 MPa)
#6, #7, #8	Flexural Members Severe Exposure	18 ksi (124 MPa)	18 ksi (124 MPa)
	Flexural Members Normal Sanitary Exposure	22 ksi (152 MPa)	20 ksi (138 MPa)
#9, #10, #11	Flexural Members Severe Exposure	17 ksi (117 MPa)	17 ksi (117 MPa)
	Flexural Members Normal Sanitary Exposure	21 ksi (145 MPa)	20 ksi (138 MPa)

A.1.2.5 CEB Information Report No. 183

The Comite Euro-International Du Beton (CEB) Publication No. 183, "Durable Concrete Structures,"[A.21] uses the same crack control provisions as the CEB-FIP Model Code 1990.[A.8] However, some additional comments are provided in CEB No. 183.

For ordinary reinforcement, CEB No. 183 reports that surface crack widths in the range of 0.30 mm to 0.40 mm (0.012 in. to 0.016 in.) are only of minor importance compared with the thickness and quality of the concrete cover. For severe exposure conditions, CEB No. 183 states:

> "... high corrosion rates may occur in the region of cracks. Again, limitation of crack width is not sufficient to avoid attack on the reinforcement. In such cases, special protective measures must be taken (e.g. sealing the concrete surface or the use of epoxy-coated reinforcement)."[A.21]

For prestressing steel, CEB No. 183 reports that due to the possibility of brittle failure, cracks crossing the prestressing steel in outdoor conditions can only be allowed in post-tensioned members, provided that the surface crack width is less than 0.20 mm (0.008 in.) and there is no source of chloride attack. Cracking must not be allowed in pre-tensioned members due to the lack of extra protection provided by the post-tensioning duct and grout. For exposures in which a source of chloride attack is present (de-icing salts or seawater), no tension is allowed for both post-tensioned and pre-tensioned elements, unless the tendons are coated. In this case, surface crack widths are to be limited to less than 0.20 mm (0.008 in.).

A.1.2.6 Durability of Concrete Structures - State of the Art

In the 1982 CEB State of the Art Report, "Durability of Concrete Structures,"[A.22] the importance of crack widths is addressed. It is simply stated that in the region of cracks in structural concrete, carbonation can penetrate much faster than in uncracked concrete. This leads to quicker corrosion initiation in the region. It is also said that cracks cannot be limited in such a way as to eliminate corrosion of the reinforcing steel during the life of the structure. The document further states that,

> "... the width of cracks is no longer to be regarded as a major factor in corrosion protection of the reinforcement. Carbonation in the region of smaller cracks will reach the reinforcement only at a later time than in that of wider cracks. From this point, however, the rate of corrosion is almost independent of the crack width, as the diffusion of oxygen is in general not influenced by the width of cracks."[A.22]

A.1.2.7 CEB Manual on Cracking and Deformations

In the CEB Manual on Cracking and Deformations,[A.23] there are four reasons cited for the control of crack widths. The reasons are listed in the document in the following order:

1. Appearance
2. Water tightness and Gas tightness
3. Corrosion Protection
4. Other Functional Requirements

Appearance was listed as the first reason to control cracking in structural concrete. It was suggested to limit cracks to those which cannot be seen in areas where concrete is exposed. There was question as to what this limit should be, and how it should be quantified.

A series of surveys to assess public reaction to cracks were carried out. The results showed that surface cracks wider than about 0.25 mm to 0.30 mm (0.010 in. to 0.012 in.) can lead to public concern. It was understood that this could not be a conclusive range of crack width levels for the following reasons. Very small cracks can be seen if material leaches from these cracks, or if dirt is present in the areas surrounding the cracks. The viewing distance to the crack also has an influence on whether or not the crack will cause public concern. Engineers are then advised to make their own assessment in each particular situation.

Corrosion protection is listed as the third reason to control cracking, and is referred to as the "most commonly quoted reason for controlling cracking". Again references such as Houston, Atimtay, and Ferguson[A.25] and Schiessl[A.38,A.39] are cited to show that crack control will only delay the onset of almost certain corrosion. It is stated that crack width has a negligible effect on the long term corrosive damage of a structure during its lifetime.

The crack control section concludes by saying that, "corrosion protection is probably the least convincing reason, since, at best, cracking is only of secondary importance in controlling corrosion."[A.23] Since aesthetic and functional requirements vary from situation to situation, the CEB model code leaves crack control as an issue for the engineer and client to come to agreement on.

A.2 RESEARCH - CRACK WIDTHS AND CORROSION

A tremendous amount of research has been performed over the last several decades on the subject of corrosion of steel reinforcement in concrete. The broad scope of the research programs including specimen types, exposure conditions, loading conditions, test duration, test methods, measurement methods and protection variables is staggering. In spite of the large amount of work in this area, little or no general conclusions have been made, largely due to the wide variation in testing methods and variables. In the following sections, an attempt has been made to review available literature on the subject of the influence of crack widths on corrosion. In Section A.2.1, research for reinforced (non-prestressed) concrete is reported. Section A.2.3 summarizes research in which the influence of prestressing on cracking and corrosion was investigated. For organizational purposes, the reviewed literature on reinforced concrete has been split into short term and long term tests. Experimental programs with duration's of 10 or more years have been designated as long term tests. All others have been classified as short term.

A.2.1 Reinforced Concrete Research - Short Term Corrosion Tests

A.2.1.1 Kahhaleh, K.Z.

Kahhaleh[A.24] performed a series of beam exposure tests to evaluate the performance of epoxy-coated reinforcement in structural members. The variables investigated in this program were reinforcement usage, loading condition (crack width), damage to epoxy coating and repair procedures. The reinforcement usage included longitudinal bars, stirrups and spliced bars with patched ends. The loading conditions considered were:

- no imposed load
- load applied to produce a specified crack width (0.013 in., 0.33 mm); load removed during exposure
- load applied to produce a specified crack width (0.013 in., 0.33 mm); load held during exposure

The concrete used in the test beams was designed to have a reduced strength and increased permeability to accelerate corrosion. The concrete had an average compressive strength f'c of 3700 psi (25.5 MPa) and used Type I cement with a water-cement ratio of 0.62. A concrete cover of 2 in. (50 mm) was used for all specimens. The midspan region of the beams was subjected to wet-dry cycles 14 days in length. The wet portion of the cycle consisted of 3 days of continuous wetting with 3.5% NaCl solution. The specimen was then allowed to dry for the remaining eleven days of the cycle.

After accelerated exposure testing for a period of 392 days, the following conclusions regarding cracking and corrosion were made:

1. **Cracking:** Corrosion of epoxy-coated bars was initiated much earlier in cracked members than in uncracked members. However, specific crack widths, measured during testing, did not show an influence on corrosion initiation and progression.
2. **Effectiveness of Epoxy Coating:** Cracked specimens with damaged coating showed the worst corrosion damage. Patched areas also experienced significant corrosion in cracked members. Where damaged or patched areas coincided with cracks, significant localized pitting was observed on longitudinal bars.
3. **Corrosion Mechanism**: In cracked specimens, corrosion was initiated at crack locations and spread to adjacent areas, undercutting the epoxy coating. Differential chloride distributions and moisture gradients influenced by the presence of cracks generated large potential differences when coupled with damage to the epoxy coating. This provided the driving force for macrocell corrosion to initiate and proceed.

A.2.1.2 Houston, J.T., Atimtay, E. and Ferguson, P.M.

Houston, Atimtay and Ferguson[A.25] performed an experimental study of corrosion in representative elements from highway structures. A large number of variables was considered: type of reinforcing steel, cement type, water-cement ratio, aggregate type, concrete permeability, bar size and spacing, cover, casting position, concrete cracking, steel working stress and prestressing. This broad approach was intended to isolate the most critical parameters. The experimental program involved exposure testing of 40 beam elements and 42 slab elements for periods of up to 34 months. Exposure conditions consisted of daily spraying with 3% salt solution.

On the basis of the experimental results, the following observations related to cracking and corrosion were made:

1. **Critical Crack Width:** In many cases, corrosion was initiated at flexural crack widths greater than 0.005 in. (0.13 mm). However, for specimens with a cover of 1 in. (25 mm), limitation of crack widths to less than 0.004 in. (0.10 mm) did not ensure prevention of corrosion.

2. **Influence of Concrete Cover:** For shallow covers (1 in. (25 mm)), uniform corrosion of reinforcement was observed. For covers larger than 2 in. (50 mm), corrosion initiation was associated primarily with crack locations. However, larger covers were effective in minimizing continued corrosion by inhibiting the development of longitudinal splitting. The range of covers considered was 1, 2 and 3 in. (25, 50 and 75 mm).

3. **Stress Corrosion:** For the investigated levels of working stress in the mild steel reinforcement (20, 30 and 35 ksi (138, 207 and 241 MPa)), stress corrosion was not an apparent factor in the corrosion process. (normally stress corrosion cracking in reinforcement is only a concern for prestressing steels - individual wire diameters less than 4 mm (0.16 in.), cold worked and subjected to a permanent tension exceeding 400 MPa (60 ksi))

4. **Level of Stress/Crack Widths:** For the reinforcement stress levels considered (20, 30 and 35 ksi (138, 207 and 241 MPa)), the increased crack width associated with the increasing stress was not a significant factor in the corrosion process. This suggests that corrosion was unaffected by the range of crack widths considered.

5. **Prestressing:** Corrosion of the 3/8 in. (9.5 mm) diameter prestressing strand was similar to that of the unstressed #6 (19 mm dia.) bars in the same specimens. The effect of improved durability (crack control) through prestressing was not investigated in this research.

A.2.1.3 <u>Vennesland, O. and Gjorv, O.E.</u>

Gjorv and Vennesland[A.26] reported the findings of an experimental study of the effects of cracks on corrosion in submerged concrete structures. An experimental technique was developed to simulate the conditions in a submerged concrete structure where a galvanic cell is developed between exposed steel at crack locations (acting as the anode) and the embedded system of rebar (acting as the cathode). In such a system, the corrosion rate is dependent on the rate of oxygen diffusion to the cathode and on the relative areas of the cathode and anode. In a large submerged concrete structure, the amount of embedded steel acting as the cathode may be considerably larger than the small exposed steel area at the crack locations, developing a considerable corrosion rate in spite of the low oxygen concentration and diffusion rate.

The experimental procedure utilized 10 x 10 x 50 cm (4 x 4 x 20”) cracked concrete prisms as the anodic region and a stainless steel plate as the cathode. The diffusion rate of oxygen onto a stainless steel plate was determined experimentally to be approximately 100 times larger than the oxygen diffusion rate through the concrete cover of the concrete prism. This allowed the use of a comparatively small stainless steel plate to model a large embedded rebar system with a cathode to anode area ratio of approximately 10.[A.5] The entire system was submerged in synthetic seawater and was monitored to determine corrosion rates and instantaneous potentials. The concrete prisms were reinforced with a single 10 mm (0.4 in.) diameter deformed bar. The water cement ratio of the concrete was 0.5. Prior to initiating the corrosion test, each specimen was cracked under three point loading. The range of surface crack widths considered was 0.1 to 2.0 mm (0.004 to 0.079 in.). The specimens where then sealed with a neoprene based glue, leaving only the crack location exposed. The corrosion tests were run for a period of four months.

In general, the findings of this research indicated that for crack widths less than 0.4 mm to 0.5 mm (0.016 to 0.020 in.) corrosion was not significant (critical crack width). This was attributed to “clogging” of the cracks. The cracks did lead to an initiation of corrosion, but the products from the corrosion reaction were found to deposit in the cracks, thereby inhibiting further corrosion. Several specimens indicated that the critical crack width for corrosion could be as high as 0.6 mm (0.024 in.).

A.2.1.4 Lin, C.Y.

Lin[A.27] reported an experimental study of the effects of crack widths on corrosion of reinforcing steel in concrete beams exposed to seawater. The beams had dimensions of 76 x 152 x 914 mm long (3 x 6 x 36" long). The concrete used Type I cement and had a water cement ratio of 0.50. The beams were precracked to desired crack widths of 0.10 mm, 0.15 mm, & 0.18 mm (0.004, 0.006 and 0.007 inches). Some of the beams were subjected to sustained loading to keep the cracks open, while others were cracked and then unloaded. The corrosion testing was performed by completely immersing the beams in a tank of seawater and impressing a constant current between the reinforcing steel and an aluminum counter electrode. The direction of the impressed current was such that the reinforcement served as the anode. The corrosion testing was continued until initiation of a longitudinal crack along the line of the reinforcing steel, resulting from deposition of the corrosion products, was observed. Due to the impressed current, formation of the longitudinal cracks occurred quickly. Testing periods ranged from two to ten days.

Related to crack widths and the effects on corrosion, the research produced two significant findings:

1. For the specimens subjected to sustained loading, the crack width did not affect the amount of corrosion (for the range of crack widths considered).
2. For the specimens which were cracked and subsequently unloaded, the amount of corrosion was significantly less (up to five times) than the beams subjected to sustained loading. On the basis of this observation, Lin recommended that for corrosion control, cracks should not be permitted under sustained or frequent loads.

A.2.1.5 Makita, M., Mori, Y. and Katawaki, K.

Makita, Mori and Katawaki[A.28] reported the results of an experimental program studying the marine corrosion behavior of reinforced concrete. Concrete specimens were partially submerged in seawater and were examined for a relationship between crack width and extent of corrosion. The specimens were 75 cm long (30 in.) and were made with concretes having water cement ratios ranging from 0.40 to 0.70. Several of the specimens were precracked. The initial crack widths were not reported by the authors. The specimens were left at the exposure site for a period of 1000 days.

At the completion of exposure testing, crack widths were measured in all specimens. The measured surface crack widths ranged from 0.05 mm to 0.3 mm (0.002 to 0.012 in.). The specimens were subsequently autopsied to determine the locations and extent of corrosion. Makita et al reported that there did not appear to be a correlation between surface crack width and corrosion, for both the precracked specimens and the initially uncracked specimens.

A.2.1.6 Misra, S. and Uomoto, T.

Misra and Uomoto[A.29] conducted three series of tests to clarify the characteristics of corrosion occurring under a combination of different conditions. Two of the series were unloaded (uncracked) and tested under various exposure conditions in a laboratory. The third series consisted of four beams tested at a marine exposure site and two beams subjected to aggressive exposure conditions in a laboratory setting. Two of the specimens at the marine site were precracked to examine the effect of cracking on corrosion. The beams had dimensions of 100 x 200 x 2100 mm long (4 x 8 x 84 in. long) and were reinforced with two D6 (< #3) bars top and two D16 (~ #5) bars bottom. The clear cover over the stirrups was 10 mm (0.4 in.). The concrete had a water/cement ratio of 0.55. The two precracked beams were subjected to four-point loading to produce a tensile strain of 0.0011 in the bottom reinforcement. The beams were unloaded prior to transportation to the exposure site and were not loaded during exposure allowing the cracks to "close". One uncracked and one cracked beam were removed from the exposure site after one year. The remaining two beams were removed after a total of two years exposure.

After one year of the study, it was found that corroded areas in uncracked specimens were the same as corroded areas in uncracked portions of the cracked specimens. However, in the cracked central portion of the precracked beams, increased corrosion was observed. This led to the conclusion that the increased corrosion could be attributed to the flexural cracking in the central portions of these members.

Misra and Uomoto did not make any conclusions regarding critical crack width values. However, their data suggested that crack widths above 0.5 mm (0.020 in.) are sufficient to cause a harsh cycle of crack initiation and propagation. The test results show that corrosion in specimens with crack widths greater than or equal to about 0.5 mm (0.020 in.) was significant enough to cause additional longitudinal cracks along the reinforcing steel. This process initiated a rapid growth, or vicious cycle, of corrosion along the reinforcement. It was shown that the presence of shear reinforcement provided sufficient confinement to restrain the growth of corrosion induced longitudinal cracks along the main reinforcing bars.

A.2.1.7 Okada, K. and Miyagawa, T.

Two series of tests to examine the influence of cracks on the mechanism and rate of corrosion of reinforcing steel were carried out by Okada and Miyagawa.[A.30] In the first series of tests, the influence of cracks on the corrosion rate of reinforcing steel was investigated. In the second series, the influence of chosen variables on the mechanisms of crack corrosion was investigated. The two main variables chosen in this study were water-cement ratio, and crack width. This experiment was based on the theory that cracks in reinforced concrete structures make reinforced concrete so heterogeneous as to cause macrocells to develop in the cracked regions. It was suggested that the region near a crack may act as an anode.

Two sizes of specimens were used in the first series. Continuous immersion specimens had dimensions of 150 x 100 x 1000 mm long (6 x 4 x 40 in. long), and were reinforced with D10 (~ #3) bars. The clear cover to the reinforcement was 15 mm (0.6 in.) and specimens had pre-formed cracks of 0, 5, 10 or 25 mm (0, 0.2, 0.4 or 1.0 in.). The water/cement ratio was 0.65. The wetting and drying specimens were macrocell specimens and had 100 x 100 mm (4 x 4 in.) sections, with lengths up to 3 m. Clear cover was 20 mm (0.8 in.) for these specimens. The water/cement ratio was 0.70. The relative lengths of anode steel and cathode steel were varied to examine effects of cathode to anode area ratio.

Measures of macrocell current in the first test series showed that during immersion of specimens in sodium chloride solution, macrocell potential and current dropped with time. However, when specimens were allowed to dry, the current density increased. Close agreement between observed loss of steel area, and that calculated from the quantity of macrocell activity was found. This confirmed that macrocell current density is directly related to the rate of corrosion.

The specimens in the second series had cross-sections of 50 x 50 mm (2 x 2 in.). Specimen lengths were varied up to 1 m (40 in.). Water/cement ratios of 0.4, 0.5, 0.6 and 0.7 were used. Specimens were loaded to in tension to produce cracks (up to 0.3 mm (0.012 in.)) or were provided with pre-formed cracks (10 mm (0.4 in.)). Some specimens had 3.13% NaCl added to the mixing water.

In the second series, in which the mechanisms of crack corrosion were observed, conclusions regarding the effects of water-cement ratio and crack width on macrocell corrosion were drawn. It was concluded that as the water-cement ratio of concrete increases, corrosion of reinforcing steel accelerates. In addition, water-cement ratio influences both the macrocell corrosion rate at cracks, and the mechanism of corrosion. When the ratio was 0.40 or 0.50, potential difference increased with increasing crack width. According to the experimental results, the critical crack width is between 0.1

and 0.2 mm (0.004 and 0.008 in.). However, as water-cement ratio increased, there was less correlation between crack width and potential difference.

A.2.1.8 Swamy, R.N.

There exists evidence that there is an interaction between cover-to-steel and crack-width on the durability of reinforcing steel in concrete. Swamy[A.31] carried out a series of tests on concrete specimens reinforced with plain, epoxy coated, and galvanized steel. The specimens were a series of square concrete prisms containing a central reinforcing bar with embedment length of 760 mm (30 in.). The reinforcing bars had varying epoxy coating thickness of 100 µm, 200 µm, and 300 µm (4, 8 and 12 mils). Tests were also conducted using galvanized reinforcement without epoxy coatings. The specimens were subject to two exposure conditions. The first was a natural marine exposure in a tidal zone, and the other was an accelerated cyclic sea water immersion test in the laboratory. The accelerated test was a wet and dry cycle of six hours each to simulate low tide. Prior to the tests, all of the test prisms were loaded in tension resulting in surface crack widths of 0.11 to 0.25 mm (0.0043 to 0.010 in.). During the tests, the specimens were subjected to their environments in the loaded condition, with steel stress adjusted to approximately 200 MPa (29 ksi).

According to the study, no corrosion protection measure could overcome the negative impact of insufficient cover, poor concrete quality, or excessive cracking. Although the study was primarily concerned with different protection measures (such as epoxy coatings and galvanized strand), conclusions were made regarding the role of critical crack widths and adequate concrete cover on corrosion of reinforcement. The results of the study suggest that a balance between both concrete cover and crack width is necessary. With large cover, there exists larger crack widths at the surface of the concrete, but also a lower number of 'critical' crack widths. This type of interaction lead to what was suggested as an optimum range of both covers and crack widths in concrete specimens. With optimum concrete cover ranging from 50 to 70 mm (2 to 2-3/4 in.), crack widths from 0.10 to 0.15 mm (0.004 to 0.006 in.) were considered critical. Although it was concluded that cover to steel is the most critical factor in preserving the electrochemical stability of steel, results obtained from this study suggest that concrete cover, quality, and crack width play all play an interactive role in the durability of reinforced concrete structures.

A.2.1.9 Berke, N.S., Dalliare, M.P., Hicks, M.C., and Hoopes, R.J.

Berke et al[A.32] performed a series of tests to address the corrosion of cracked reinforced concrete members and to evaluate the effectiveness of calcium nitrite corrosion inhibitor. The specimens used in this program were beams, based on the ASTM G109 macrocell specimens. Dimensions of the beams were 6 x 6 x 30 in. long (152 x152 x 762 mm long). Reinforcement was identical to standard macrocell specimens, with one #4 (0.5 in. dia.) bar as the anode and two #4 bars as the cathode. Clear cover was 1-1/2 in. (38 mm). As in typical macrocell specimens, electrical contact between the layers of steel was provided using wire and a 10 Ohm resistor. Electrical contact is necessary for macrocell corrosion to develop, and the use of the resistor allows direct measurement of the corrosion current. The beams were initially cracked with the single #4 bar in tension, using a loading machine. The average crack width was 0.008 in. (0.2 mm). Shims were placed in the cracks and specimens were unloaded during exposure testing. Exposure consisted of ponding the cracked surface with 3% NaCl solution on a 4 week cycle (2 weeks wet, 2 weeks dry). Exposure testing was continued for 16 months.

A total of 8 beams were tested using concrete meeting minimum ACI Specifications[A.2] (w/c = 0.40). Four of the beams had calcium nitrite added.

Destructive examination of the specimens at the conclusion of testing found the majority of corrosion below the crack. The beams without corrosion inhibitor showed more severe corrosion and spreading of the corrosion several diameters from the crack.

Comparison of average macrocell corrosion current and total macrocell corrosion showed a dramatic improvement using calcium nitrite. Observed corrosion in these specimens was negligible in comparison to the specimens without calcium nitrite.

Overall, the results show that a crack width of 0.008 in. (0.2 mm) was insufficient to prevent corrosion with a 1-1/2 in. (38 mm) cover. The addition of calcium nitrite proved effective in limiting corrosion at a crack width of 0.008 in. (0.2 mm).

A.2.1.10 Schiessl, P., and Raupach, M.

Schiessl and Raupach[A.33] performed a laboratory and theoretical study investigating the dominant variables influencing corrosion in cracked reinforced concrete.

The laboratory study consisted of small reinforced concrete beams subjected to saltwater exposure. The beam dimensions were 97 x 150 x 700 mm long (3.8 x 5.9 x 27.6 in.). The beams were subjected to sustained loading to produce one crack at midspan. Investigated crack widths were 0.1 mm, 0.2 mm, 0.3 mm and 0.5 mm (0.004 in., 0.008 in., 0.012 in. and 0.020 in.). The effect of water-cement ratio (0.5 and 0.6) and concrete cover (15 mm (0.6 in.) and 35 mm (1.4 in.) was investigated. The maximum exposure duration was two years. The number of specimens in the study was not clear, but data for twelve specimens was plotted in several figures. The results of the laboratory study summarized as follows:

1. **Corrosion Mechanism:** The anodic reaction occurs at the crack location. The regions outside the crack behave cathodically up to a distance of 200 mm (7.9 in.) on either side of the crack. The cathodic current density decreases with distance from the crack due to increasing electrolyte resistance.
2. **Effect of Concrete Cover:** Increased concrete cover significantly reduced the calculated steel mass loss due to corrosion.
3. **Effect of Water-Cement Ratio:** Reducing the w/c ratio from 0.6 to 0.5 reduced calculated mass loss in the crack zone, particularly within the first six months of exposure. The influence of w/c ratio was less pronounced after 2 years of exposure.
4. **Effect of Crack Width:** Corrosion rates and mass loss increased with increasing crack widths. However, the influence of crack width was deemed much less than the influence of concrete cover and w/c ratio.

The laboratory study was followed by a series of calculations to further investigate the influence of cracking on corrosion. Based on the observed corrosion mechanism in the laboratory study, the corrosion process at a crack was modeled as "macrocorrosion" cells with an anode at the crack and large cathodes between the cracks. The reader is referred to the paper for more information on the model and assumptions. The results of the corrosion theory calculations are summarized as follows:

1. **Effect of Crack Spacing:** The corrosion rate (mass loss) increased as the crack spacing increased. These results can be explained by the area effect, or the ratio of cathode area to anode area. For a given corrosion current flowing in the corrosion cell, the corrosion current density increases as the electrode area decreases. The corrosion rate or metal loss is related to the current density at the anode. As the ratio of cathode area to anode area increases the anodic current density increases. If the crack spacing decreases, the available cathodic area is reduced and the anodic current density decreases.
2. **Effect of Reducing Bar Diameter to Limit Crack Widths:** Limiting crack widths by reducing the bar size (keeping total steel area constant) resulted in an increase in loss of reinforcement cross-section. This occurrence was attributed to an increase in total surface area of steel.

   Schiessl and Raupach conclude their paper with the following statement:

"The calculations and the results of the laboratory tests clearly indicate that the problem of reinforcement corrosion in crack zones cannot solely be solved by crack width limitation in the range from roughly 0.3 to 0.5 mm; corrosion protection must be assured primarily through adequate concrete quality and cover."[A.33]

A.2.2 Reinforced Concrete Research - Long Term Corrosion Tests

A.2.2.1 Tremper, B.

Tremper[A.34] reported the findings of a long term exposure test on small scale reinforced concrete members. The members were concrete blocks with dimensions of 8 x 8 x 2.5 inches (200 x 200 x 63 mm), reinforced with steel wires or deformed bars. Different concrete mixes with either well graded aggregate or poorly graded aggregate were used to represent different placement qualities. The water cement ratios of the concrete ranged from 0.40 to 0.75. The minimum cover to the reinforcement was 1-1/8 in. (28.6 mm) The specimens were loaded as beams to produce surface crack widths of 0.005 in., 0.010 in., 0.020 in., and 0.050 in. (0.127 mm, 0.254 mm, 0.508 mm and 1.27 mm). The specimens were then unloaded and taken to a coastal exposure site. The specimens were placed on racks, cracked side upwards, but were not in direct contact with the seawater. Tremper indicated that the exposure conditions would not be considered as particularly severe.

After ten years of continuous exposure testing, the specimens were autopsied. All reinforcement was found to be free of corrosion, except in the regions of cracks. However, this corrosion was deemed minor. Tremper concluded that for the conditions considered in these tests, occasional "large cracks" (within the range of cracks considered) do not promote serious corrosion of reinforcing steel.

A.2.2.2 Ohta, T.

Long term exposure tests on reinforced concrete beams exposed to sea air were conducted by Ohta.[A.35] One hundred and forty nine pairs of reinforced concrete beams (1000 x 150 x 150 mm (39.4 x 6 x 6 in.)) with open cracks were exposed for two to twenty years. The main reinforcement was one or two deformed 13 mm (1/2 in.) bars with 6 mm (1/4 in.) stirrups. The cements used were ordinary Portland cement, blast furnace slag cement, and Portland fly-ash cement. The specimens were exposed near the beach line in Ramoi, facing the Sea of Japan. The variables measured were carbonation of the concrete, chloride ion content in concrete, electrical potential and corrosion of the reinforcing steel. The cross sectional area of the reinforcing steel was measured to determine the extent of corrosion.

The loss of sectional area of reinforcing steel after ten years of exposure is plotted on Weibull probability paper in Figure A.2 and Figure A.3. The results are summarized as follows:

1. Crack widths did not correlate significant loss of the cross sectional area of steel for 20 mm (0.8 in.) cover after ten years (Figure A.2).
2. For 40 mm (1.6 in.) cover, there appeared to be a relationship between the amount of corrosion and crack widths (Figure A.3).
3. Corrosion was only slight for covers of 50 mm and 68 mm (2 in. and 2-2/3 in.).

It was then concluded that the rate of progress of depassivation depends on crack width when cover is thick, and when term of exposure is short.

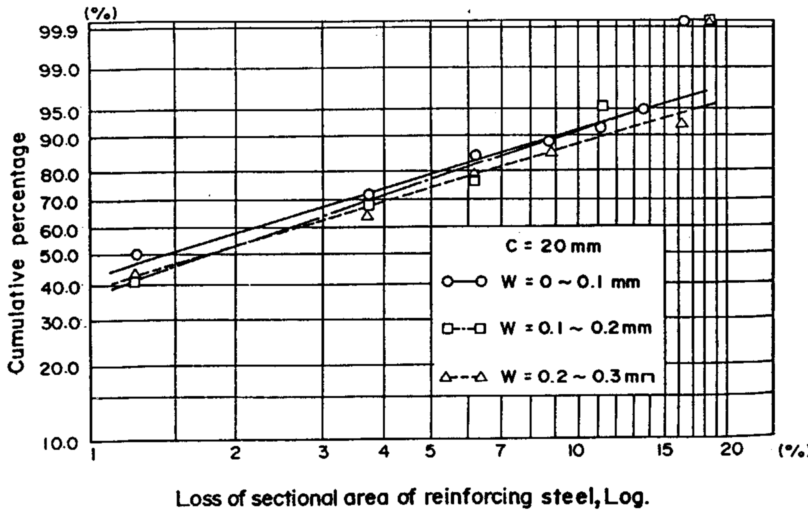

Figure A.2 - Loss of Cross-Sectional Area after 10 Years, 20 mm (0.79 in.) cover (Ohta[A.35])

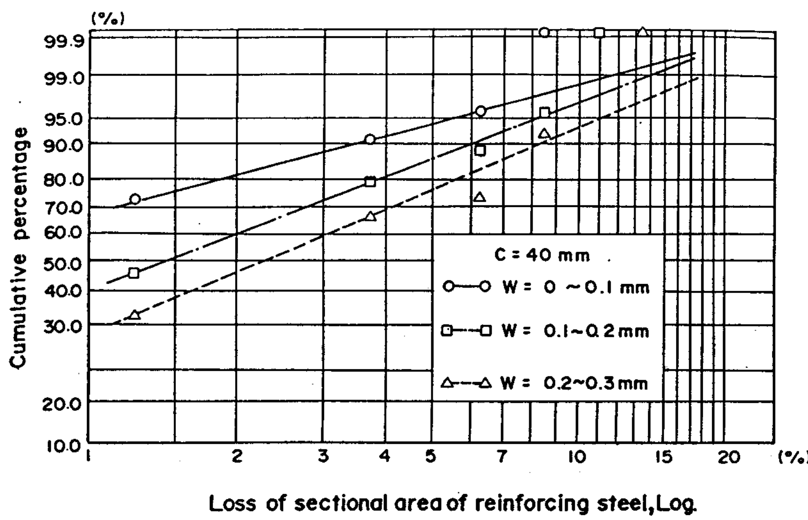

Figure A.3 - Loss of Cross-Sectional Area after 10 Years, 40 mm (1.57 in.) cover (Ohta[A.35])

Results after 20 year of the study:

1. The effect of crack widths on corrosion disappeared for even the 40 mm (1.6 in.) cover specimens (Figure A.4).
2. Specimens with 20 mm (0.8 in.) cover were very heavily corroded, and longitudinal cracks along the reinforcement were observed

It was concluded from these tests that cover, and not crack width played the most important role in the control of corrosion of reinforcing steel in concrete.

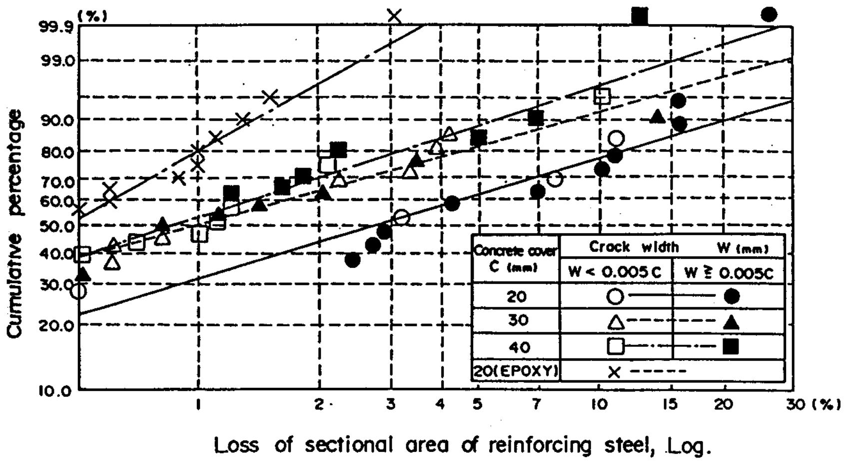

Figure A.4 - Loss of Cross-Sectional Area After 20 Years (Ohta[A.35])

A.2.2.3 Francois, R and Arliguie, G.

Francois and Arliguie[A.36] drew similar conclusions through a series of tests on cracked reinforced concrete specimens. The test program involved sixty-eight reinforced concrete beams, and covered a test period of ten years. The beams were 150 x 280 mm (6 x 11 in.) in cross section, and 3 m (118 in.) in length. Two exposure conditions were used. One was a salt fog produced by means of four vaporizers using compressed air at 0.1 MPa, and salt water at 3.5% NaCl by weight. The other environment was a mixture of 50% CO_2 and air, with relative humidity kept between 40% and 70%.

Results of their study suggest that the existence of cracks, and not their width, is the significant parameter in the corrosion of reinforcing steel. The study did not report how different values of crack width influenced corrosion, but only said that crack widths were less than 0.5 mm (0.02 in.). It was also observed that crack width influenced the speed at which corrosion began. Since this onset time was a relatively short, the influence of crack width was limited in general. Instead, the findings support the idea that concrete cover is directly related to the rate of corrosion. It was suggested that this is because the capacity of concrete to absorb aggressive agents increases with the thickness of cover.

A.2.2.4 O'Neil, E.F.

In a study by E.F. O'Neil,[A.37] several reinforced concrete beams were placed at the Weathering Exposure Station at Treat Island, Cobstock Bay, Eastport, and Lubec in Maine. The length of the study was 25 years (1950 to 1975). The purpose of the test was to obtain information on the long term weathering of air-entrained, and non-air-entrained concrete beams containing steels of different composition, types of deformation, and different levels of stress. Though the study of crack widths on corrosion was not a primary objective, data obtained during these tests show some relationship between stress levels in the steel and corrosion rates. These stress levels correspond directly to the crack widths observed in the specimens.

The test consisted of a series of 82 reinforced concrete beams. 22 beams were made with air-entrained concrete, and the rest were done without air-entrainment to evaluate the durability of air entrained concrete specimens in severe environments. Thirty nine percent of the beams had cover depths of 50 mm (2 in.), while the rest had 19 mm (3/4 in.) cover. The reinforcing steel used conformed to ASTM A 16-50T for rail-steel rebar, or to A 15-50T for billet-steel rebar. All specimens were loaded to put tensile steel under stress, with the exception of the control specimens. The loaded stress levels in the steel were 0, 138, 207, 276, and 345 MPa (0, 20, 30, 40 and 50 ksi). Beams were exposed to twice daily tidal cycles. Since tides reached as high as 9.15, the beams were also subjected to considerable head during wetting. In addition, the beams were subjected to freeze-thaw during the winter months.

After 5 years of exposure, all of the beams with non-air entrained concrete had extensively deteriorated due to freezing and thawing. After 25 years, 18 of the beams with air entrained concrete remained, however only 13 were in testable condition. At this time, 11 of the beams were tested and autopsied.

Corrosion of the reinforcing steel in beams stressed to 138 MPa (20 ksi) could not be matched with the flexural cracks in the beams. Beams stressed to 345 MPa (50 ksi) did show corrosion of steel matched with the flexural cracking in those regions. Since the 345 MPa (50 ksi) beams had crack widths of 0.4 mm (0.016 in.) or greater, it was concluded in the study that crack widths of 0.4 mm (0.016 in.) or higher were necessary to produce corrosion at flexural cracks.

The results could not show a definite relationship between the steel stress levels and corrosion that would indicate more or less corrosion for lower stress levels. The study generally did find that steel at higher stress levels produced larger flexural cracks, and therefore would allow for greater penetration of water and oxygen.

A.2.2.5 Schiessl, P.

Schiessl[A.38,A.39] found, through a series of long term tests, that the decisive parameter for the control of corrosion was the carbonation of the concrete, and not the width of cracks in the concrete.

The test specimens used in the comprehensive study consisted of 1.95 m rectangular beams (150 x 250 mm (6 x 9.8 in.)). The reinforcing steel consisted of 2 deformed 8 mm (0.31 in.) bars, and one plain 4 mm (0.16 in.) bar. Concrete covers were 2.5 cm and 3.5 cm (1 in. and 1.38 in.), and stirrups (8 mm deformed bars) were provided on only one-half of each specimen. After 28 days the concrete strength was 250 kg/cm^2 (3600 psi (24.8 MPa)). The beams were subjected to normal city air, polluted industrial air, and salty sea air. The pairs of beams were braced against each other at the quarter points by use of an intermediate piece, and stressed together with screw bolts. The beam pairs were situated such that the top of one beam faced upward, while the other faced downward. Permanent crack widths produced by this type of loading ranged from 0.15 mm to 0.40 mm (0.006 to 0.016 in.).

Schiessl concluded from these tests that the limiting of crack widths only serves to limit the amount of cracks with high corrosion intensity. These areas of high local corrosion intensity often give rise to longitudinal cracking along the reinforcing steel. This in turn can cause spalling of the concrete cover. Schiessl acknowledged that there is a wide range of opinion regarding acceptable crack widths, typically ranging from 0.1 mm to 1.0 mm (0.004 to 0.04 in.). He reasons, however, that the corrosion rate of reinforcing steel in carbonated concrete is almost independent of crack width and cover. That is, the influence of the crack width on corrosion intensity decreases with increased exposure time.

Schiessl explains that there are two phases in the corrosion of reinforcing steel. The first phase involves the depassivation of the reinforcing steel. This depassivation is dependent on both the cover depth and crack width. The alkaline nature of the surrounding concrete provides a passive layer around the reinforcing steel, thus preventing corrosion. CO_2 from the atmosphere reacts with hydroxides in the concrete (carbonation). Experiments showed that large covers slowed the process sufficiently enough to protect the steel inside. However, tests also revealed that cracks speed up the process, and that the rate was dependent on width.

The second phase begins once corrosion has initiated. Results from his studies showed that the rate of corrosion, once initiated, was independent of crack width and cover. Since the first phase is relatively short in duration compared to the length of the longer term exposure tests, and since it is only a matter of time before corrosion initiates regardless of crack widths, it is concluded that there is no long term relationship between crack widths and rates of corrosion.

Schiessl does not propose any specific limiting value of crack width because his research shows that there is no value of crack width below which protection against corrosion could be guaranteed. Test results show that, for even a specimen with 25 mm (1 in.) cover, and a crack width of 0.15 mm (0.006 in.), there still exists a 40% probability that corrosion will appear.

A.2.2.6 Tuutti, K.

Tuutti[A.40] studied the effects of cracks in the concrete cover on the corrosion of reinforcing steel through a review of available literature. Tuutti also agrees that permissible crack width reported in the literature range from 0.1 mm to 1.0 mm (0.004 to 0.04 in.). In addition, it was also noted that a few authors had shown that cracks widths as small as 0.01 mm (0.0004 in.) give rise to corrosion attacks. Shown in the report are the findings of Rehm and Moll (1964). In these findings, specimens with relatively low concrete quality (w/c ratio 0.8) showed signs of corrosion for crack widths above 0.1 mm (0.004 in.). When only the crack width varied, it was shown that the corrosion varied with the crack width. However, these results were then supplemented by the results of Schiessl's 10 year study.[A.38,A.39] These results showed that,

> "corrosion in the uncracked zones always occurred as soon as the carbonation front had penetrated to the steel, in which the thickness of the concrete cover determines the initiation time, given the same concrete."[A.38]

It was stated that for short term duration, corrosion was dependent on a limiting crack width value. The value of this limiting crack width was not explicitly defined, as it depends on the concrete quality and other factors.

Cited in this paper are the results of work by Tremper,[A.34] in which concrete quality, as well as crack widths were variables studied. Crack widths varied from 0.13 mm to 1.3 mm (0.005 to 0.05 in.), and w/c ratios varied from 0.40 to 0.75. Specimens were exposed for a period of 10 years. Results showed that corrosion attacks occurred in the cracks and in their immediate surroundings in all cases.

Tuutti concludes that cracks in the concrete cover do not change the basic mechanisms of corrosion, but instead only have local effects. It is theorized that corrosion initiates when a threshold concentration of an initiating substance is achieved at the surface of the steel. The rate of corrosion is then determined by the flows of these substances to the area. So basically it is only the local flows of these substances that are changed by the crack widths.

A.2.2.7 Beeby, A.W.

Beeby[A.41,A.42,A.43] examined the relationship of cracking to corrosion of reinforcing steel. Based on his own studies, the works of Schiessl,[A.38,A.39] and others, he concludes that crack widths have little influence on corrosion. He also says that current guidelines for controlling crack widths are unnecessary.

This argument is based on the idea that cracking influences the onset of corrosion locally, but has a negligible long term effect. He acknowledged that some short term tests (2 years) revealed an influence of crack widths on corrosion. Also cited were the results of Houston, Atimtay, and Ferguson[A.25] (see earlier discussion of this work). These results showed that crack widths had only a minimal effect on the corrosion of 84 beam and slab units exposed for up to two years with daily spaying of salt solution. Several other tests were cited to support the claim that crack widths have an overall minimal effect on the corrosion of steel in concrete.

Also used to support his claim that crack control guidelines are unnecessary is the work by Husain and Ferguson[A.44] regarding crack widths at the level of steel in concrete. In this study, it was found that there does not exist a definite relationship between the widths of surface cracks, and the widths of cracks at varying steel depths. Therefore, it is argued, the use of a surface crack width value can be completely arbitrary with regard to the actual situation in the field.

A.2.3 Prestressed Concrete Research

In comparison to corrosion research for reinforced concrete, very little research has been performed for prestressed concrete. In particular, the effectiveness of prestressing as corrosion protection through crack control has rarely been addressed. Some researchers have studied the corrosion of prestressed concrete, but in many cases, the specimens were not cracked or subjected to structural loading. Others, such as Houston et al[A.25] (see Section A.2.1.2) did not consider the effect of crack widths nor special measures for protection of the prestressing system. The research summarized in this section is all considered as short term exposure.

A.2.3.1 Poston, R.W.

Poston[A.45] performed an experimental program to examine transverse post-tensioning as a method for improving the durability of bridge decks. The effectiveness of the prestressing as a corrosion protection scheme was evaluated using representative full thickness (8") bridge deck specimens subjected to an aggressive chloride environment. The laboratory specimens modeled the negative moment region of the bridge deck over an interior support (girder). The durability of prestressed bridge decks (100% prestressing) and conventionally reinforced bridge decks (no prestressing) were investigated.

Since the postulated mechanism by which the durability of a prestressed bridge deck is improved involves the control or elimination of cracking, crack width was included as a variable. The non-prestressed specimens and some of the prestressed specimens were loaded to produce surface crack widths of 0.015 in. (0.38 mm). The remaining prestressed specimens were loaded to the same load used to open the cracks of 0.015 in. in the non-prestressed specimens. This produced surface cracks of 0.002 in. (0.051 mm) in the prestressed specimens. Other variables included the type of prestressing system (unbonded or bonded), black or epoxy-coated non-prestressed reinforcement (in all specimens) and concrete cover of 2 or 3 in. (50 or 75 mm). The concrete used in all of the specimens had a compressive strength f'c of 5100 psi (35.2 MPa) and used Type I cement with a water cement ratio of 0.44. The exposure conditions consisted of one wet-dry cycle every 14 days, continued for 17 cycles or 8 months. The wet portion of the cycle consisted of 3.5% saltwater solution ponded on the specimens for two days. The specimens were then allowed to dry for the remaining nine days of the cycle. On the second day (during the wet portion), the specimens were subjected to five repeated loading cycles to produce the desired crack width. The same loading pattern was also applied on the ninth day, during the drying portion of the cycle.

From the accelerated exposure tests, the following observations related to cracking and corrosion were made:

1. **Corrosion Occurrence:** Corrosion of non-prestressed reinforcement initiated and occurred only at the location of flexural cracks. In many cases, corrosion had spread over a distance of 6 to 10 bar diameters (uncoated bars). The incidence and extent of corrosion was much less for the epoxy-coated bars.
2. **Crack Width:** For both non-prestressed and prestressed specimens loaded to produce crack widths of 0.015 in. (0.38 mm), the incidence and extent of corrosion was similar. Virtually no incidence of corrosion of non-prestressed reinforcement was observed in the prestressed specimens with a crack width of 0.002 in. (0.051 mm). This represents the most significant effect of prestressing / crack control on reinforcement corrosion.
3. **Penetration of Chloride Ion:** Prestressing had little effect on chloride ion penetration in regions of uncracked concrete. However, chloride ion concentrations at crack widths of 0.002 in. (0.051 mm) were approximately 60% less than at crack widths of 0.015 in. (0.38 mm).
4. **Concrete Cover:** For the conditions and time length of the exposure testing in this study, no difference was observed for the two levels of cover considered.

A.2.3.2 Moore, D.G., Klodt, D.T., and Hansen, J.

As part of a large experimental program, Moore et al[A.46] performed a series of corrosion tests on pre-tensioned beams. The purpose of the tests was to evaluate:

- effect of voids between steel and concrete
- effect of concrete cover
- effect of live loads
- effect of sizable tensile cracks in concrete
- effect of accidental overloading (cracking followed by load reduction and cracks closing)

A total of 16 beams were tested. The dimensions of the beams were 4 x 6 in (102 x 152 mm) and 6.5 ft (1.98 m) long. The beams were pre-tensioned using two 3/8 in. (9.5 mm) dia., Gr 270 (1860 MPa) prestressing strands. One strand was placed near the compression face and the other near the tension face such that concentric prestressing was achieved. The concrete used Type I cement with w/c = 0.40. The compressive strength was 6300 psi (43.4 MPa). Concrete cover ranged from 1/2 in. to 2 in. (12.7 mm to 50 mm). Exposure conditions consisted of ponding with 3.5% NaCl solution. Exposure testing was continued for a period of 10 months.

The beams were loaded (3-point) in pairs, back to back. Various levels of load were used to evaluate increasing levels of extreme fiber stress. These included 1300 psi (8.96 MPa) compression, zero stress, 250 psi (1.72 MPa) tension, 500 psi (3.44 MPa)tension and 1000 psi (6.88 MPa) tension. The necessary applied load levels were calculated based on the desired stresses in an elastic uncracked section. The beams loaded to 1000 psi tension had a maximum average surface crack width of 0.008 in. (0.2 mm). All other specimens were "uncracked" under load. Two additional specimens were loaded to produce a surface crack width of 0.004 to 0.006 in. (0.1 to 0.15 mm), and then unloaded to zero extreme fiber stress (load corresponding to decompression at extreme fiber, assuming and elastic, uncracked section). No cracks were detectable upon load reduction.

The results of the exposure tests related to cracking and corrosion are summarized as follows:

1. **Effect of Cracks:** The most serious corrosion was observed in the beams with open cracks. Pitting corrosion was observed at cracks as small as 0.004 in. (0.1 mm).
2. **Effect of Temporary Overload:** Cracks in beams caused by brief overloading tended to "heal" after ten months of exposure. No increase in corrosion was observed at these crack locations.
3. **Effect of Load Level:** No correlation between load level and corrosion was observed, with the exception of the specimens loaded to cracking.
4. **Effect of Cover:** Concrete cover of 1.5 in. (38.1 mm) and larger prevented corrosion in the uncracked specimens over the ten months of exposure. Corrosion was found in all specimens with 0.75 in. (19 mm) cover or less.

A.2.3.3 Perenchio, W.F., Fraczek, J., and Pfiefer, D.W.

Perenchio et al[A.47] performed exposure tests to evaluate the effectiveness of epoxy-coated strand and the effect of cracks on the durability of pre-tensioned members. The specimens were 12 ft (3.66 m) long with a 6 x 10 in. (152 x 254 mm) cross section. The beams were pre-tensioned with two 0.5 in. (12.7 mm) dia. Gr 270 seven-wire strands. The strands were located symmetrically, one at the top of the member and one at the bottom, such that the eccentricity was zero.

A total of 8 beams were tested. In 4 of the specimens, the strand closest to the tension face was epoxy coated. The second strand in these specimens was bare. Half of the specimens, 2 with epoxy coated and 2 with all bare strands, were intentionally cracked under flexural loading to an average surface crack width of 0.01 in. (0.254 mm). The cracks were maintained during exposure testing by loading the specimens in pairs, back to back. Exposure consisted of a one week long wet-dry cycle (3.5 days wet, 3.5 days dry) using 15% NaCl solution. During the dry portion of the cycle, the specimens were subjected to a constant temperature of 100 deg. F (37.8 deg. C) to further accelerate corrosion. Exposure testing was continued for 10 months. The specimens were wired externally to allow direct measurement of macrocell corrosion current.

The experimental results for the beams with bare strands only indicated that corrosion was more severe in cracked beams. Macrocell corrosion currents were significantly higher in the cracked specimens. No quantitative corrosion current data was reported by the authors. In addition, half-cell potential readings indicated a high probability of corrosion in the cracked beams after only 30 days of exposure. Similar readings were obtained in the uncracked beams only after an exposure period of 60 days. Visual examination of the strands after autopsy at the conclusion of testing showed heavy corrosion on all strands near the tension face of the beam. The amount of corrosion in the cracked beams was not significantly more than in the uncracked beams. Some amount of corrosion was also observed on all strands nearest the compression face. Chloride content measurements indicated that chlorides had reached the strands at the compression face through cracks in the cracked specimens. In the uncracked specimens, high chloride levels at these strands were attributed to spillage of the salt water solution.

The macrocell corrosion current measurements were essentially zero for the specimens with epoxy-coated strands. Occasional non-zero measurements indicated a reversed corrosion macrocell had developed (i.e. the bare strand near the compression face was corroding). This occurrence was confirmed during destructive examination of the beams after conclusion of exposure testing. As in the specimens with bare strands only, this corrosion was attributed to penetration of chlorides at crack locations or due to spillage. Half-cell potential readings for the beams with epoxy-coated strands were erratic and could not be used to draw any conclusions.

From the reported experimental results, the following observations can be made:

1. **Effect of Cracking:** Cracking (crack width = 0.01 in. (0.254 mm)) reduced the time to initiation of corrosion and increased corrosion severity. However, significant corrosion also occurred in companion uncracked specimens.

2. **Critical Crack Width:** No conclusions regarding critical crack width can be made from the results of this study.

3. **Concrete Cover:** 1 in. (25 mm) of clear cover was not sufficient to prevent corrosion in either cracked or uncracked specimens.

4. **Epoxy-Coated Strand:** Epoxy-coated strand showed no signs of corrosion during exposure testing. However, experimental results illustrate the importance of using epoxy-coated strand throughout the member rather than just at the level of steel closest to the tension face.

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Appendix B
Field Performance of Prestressed Concrete Bridges: Literature Review

This appendix provides a brief review of available literature on the field performance of prestressed concrete structures, with an emphasis on bridges. The review addresses the following areas:

B.1 Corrosion of Prestressing Strand before Construction

B.2 Pretensioned Bridges

B.3 Unbonded Single Strand (Monostrand) Tendons

B.4 Unbonded Internal Tendons (Multistrand and Bar) in Bridges

B.5 External Multistrand Tendons in Bridges

B.6 Bonded Internal Post-Tensioned Tendons in Bridges

General occurrences of corrosion problems are described according to type of prestressing, time of occurrence and various aspects of the prestressing system. Where possible, specific case studies are provided for illustration.

B.1 CORROSION OF PRESTRESSING STRAND BEFORE CONSTRUCTION

Corrosion of prestressing strand occurring prior to construction can lead to failures before, during and after stressing. Corrosion prior to construction may result from improper storage and handling during shipping. Failures before stressing normally occur in cases where the prestressing strand is stored in tightly wound coils. This type of failure is generally attributed to stress corrosion cracking and is most common in quenched and tempered steel, which is more susceptible to stress corrosion. Quenched and tempered steel is not permitted by AASHTO or ACI (see Section 5.4.1 of this report), and is generally not available for use in North America. Reports of this type of failure have primarily been from Germany. Chemical contamination of the strand during storage, transport and handling can lead to embrittlement or pitting corrosion of the strand. Common sources of contamination are splashing with fertilizers, water containing lime and gypsum, animal wastes and raw oils.[B.1] Pitting corrosion may also occur as a result of exposure to moisture, saltwater or sea-mist during storage or transportation. Embrittlement and pitting corrosion may lead to failure prior to stressing. Corrosion occurring before stressing may also cause failure during stressing and after stressing in both pretensioned and post-tensioned structures.[B.1] Guidelines for assessing the degree of corrosion on prestressing strand before it is placed in the structure are provided by Sason[B.2] and PCI.[B.3]

B.2 PRETENSIONED BRIDGES

The types of corrosion problems in pretensioned structures are not significantly different from those in reinforced concrete structures. The absence of the post-tensioning duct and anchorages in pretensioned concrete makes it more similar to reinforced concrete in terms of the protection provided for the prestressing steel. The main influencing factors for corrosion in pretensioned structures are the prestressing steel, concrete and severity of environment. The effect of the concrete and environment is the same for pretensioned and reinforced concrete structures. Although prestressing steel is more susceptible to corrosion and the consequences of corrosion may be more severe than for mild steel reinforcement, corrosion of prestressing tendons in pretensioned structures is rare for two main factors. Pretensioned elements are always precast, generally resulting in improved overall quality control and good quality concrete. Also, pretensioned elements normally fit the classic definition of full-prestressing, that is, concrete tensile stresses are limited to prevent flexural cracking of the concrete. Where corrosion has been discovered in pretensioned structures, the cause is normally related to the structural form and details. Because pretensioned elements are precast, the structure may contain a large number of joints or discontinuities. Poor design and/or maintenance of these joints may direct moisture and chlorides onto the pretensioned elements of the structure in very localized areas.

Novokschenov[B.4] performed an extensive condition survey of several pretensioned and post-tensioned bridges, in both marine and de-icing salt environments. One bridge located in the Gulf of Mexico consisted of pretensioned girder approach spans and post-tensioned segmental box girder main spans. The bridge was 16 years old at the time of inspection. Corrosion damage consisting of concrete cracking caused by corrosion of the prestressing steel was found on the ends of the pretensioned girders adjacent to the expansion/contraction joint at the transition between the approach and main spans. Corrosion was attributed to chloride laden moisture from the deck leaking through the expansion/contraction joint onto the ends of the girders, producing highly localized, severe exposure conditions. Novokschenov[B.4] also examined a precast pretensioned box girder viaduct that had been exposed to de-icing salts throughout its service life. The bridge was 29 years old at the time of inspection. Examination revealed that almost all longitudinal joints between box girders were leaking, ranging from very minor to extensive. The leakage appeared to have resulted from moisture and chlorides penetrating through cracks in the cast-in-place concrete deck overlay and progressing through the longitudinal joints between the girders, as shown in Figure B.1. In the areas of heaviest leakage, extensive staining and white deposits were visible, accompanied by corrosion of the prestressing strands and deterioration of the concrete cover. In some areas, spalling exposed the prestressing strands, leading to severe deterioration and failure of up to six of the seven wires in several strands. The specified concrete cover for this bridge was 45 mm (1.75 in.), and measured cover was up to 6 mm (0.25 in.) less than this value. Novokschenov mentioned that this type of damage was common in other, similar bridges, and concluded that is an inherent problem to this particular bridge design. A third bridge examined in this report[B.4] consisted of precast pretensioned I-girders. Corrosion related damage consisting of concrete cracking and spalling was found in girders adjacent to longitudinal expansion joints and at the ends of most girders at transverse joints, both expansion and fixed. The path of chloride laden water at a longitudinal expansion joint and the resulting deterioration are shown in Figure B.2. In areas where the strand was exposed due to spalling, wire fractures were common. Corrosion damage at the ends of the girders was less severe at fixed joints in comparison to expansion joints, attributed to less leakage of chloride laden moisture from the bridge deck. The specified cover was 50 mm (2 in.), and measured covers were up to 6 mm (0.25 in.) less than this value.

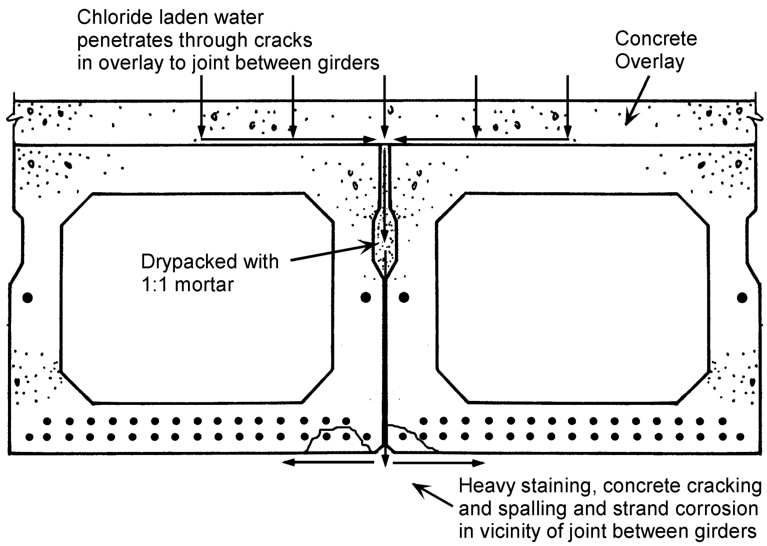

Figure B.1 - Mechanism for Moisture and Chloride Penetration Through Concrete Overlay in Precast Pretensioned Box Girders (adapted from Ref. B.4)

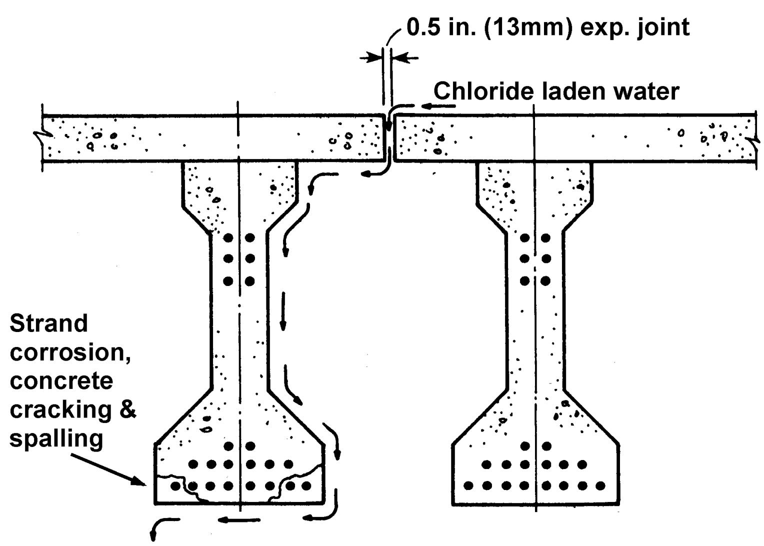

Figure B.2 - Mechanism for Moisture and Chloride Penetration at Longitudinal Expansion Joints in Precast I-Girder Bridges (adapted from Ref. B.4)

Others have performed similar condition studies of bridges with pretensioned elements.[B.5] In general, these surveys found corrosion related deterioration in pretensioned members to be localized in specific areas of the structure, primarily at transverse joints along the bridge. The findings of condition surveys of pretensioned bridges indicate that corrosion problems in this type of structure are primarily a function of the design of the structure, rather than the pretensioned elements. Improved joint design and maintenance or minimization of joints would appear to eliminate most corrosion problems.

B.3 Unbonded Single Strand Tendons

Unbonded single strand or monostrand tendons refer to greased and sheathed type single strand tendons commonly used in slabs. Monostrand unbonded tendons represented approximately 80% of post-tensioning used in the U.S. from 1965 to 1991.[B.6] The majority of this steel was used in buildings (including parking structures) and in slabs-on-grade. Although monostrand applications would be very limited in bridge substructures, they have been used for transverse post-tensioning in bridge decks and segmental box girders. Also, examination of corrosion problems in structures with monostrand tendons can provide insight into the overall picture of corrosion in post-tensioned structures, including bridges.

The evolution of the monostrand system for post-tensioning is shown in Figure B.3. Common locations of corrosion are indicated by the letter "C" in the figure. A very comprehensive discussion of corrosion of monostrand tendons is provided by ACI/ASCE Committee 423.[B.7] Corrosion problems in monostrand tendons can be grouped into four areas:

1. Damage to the sheathing,
2. Poor anchorage protection,
3. System deficiencies,
4. Structural aspects.

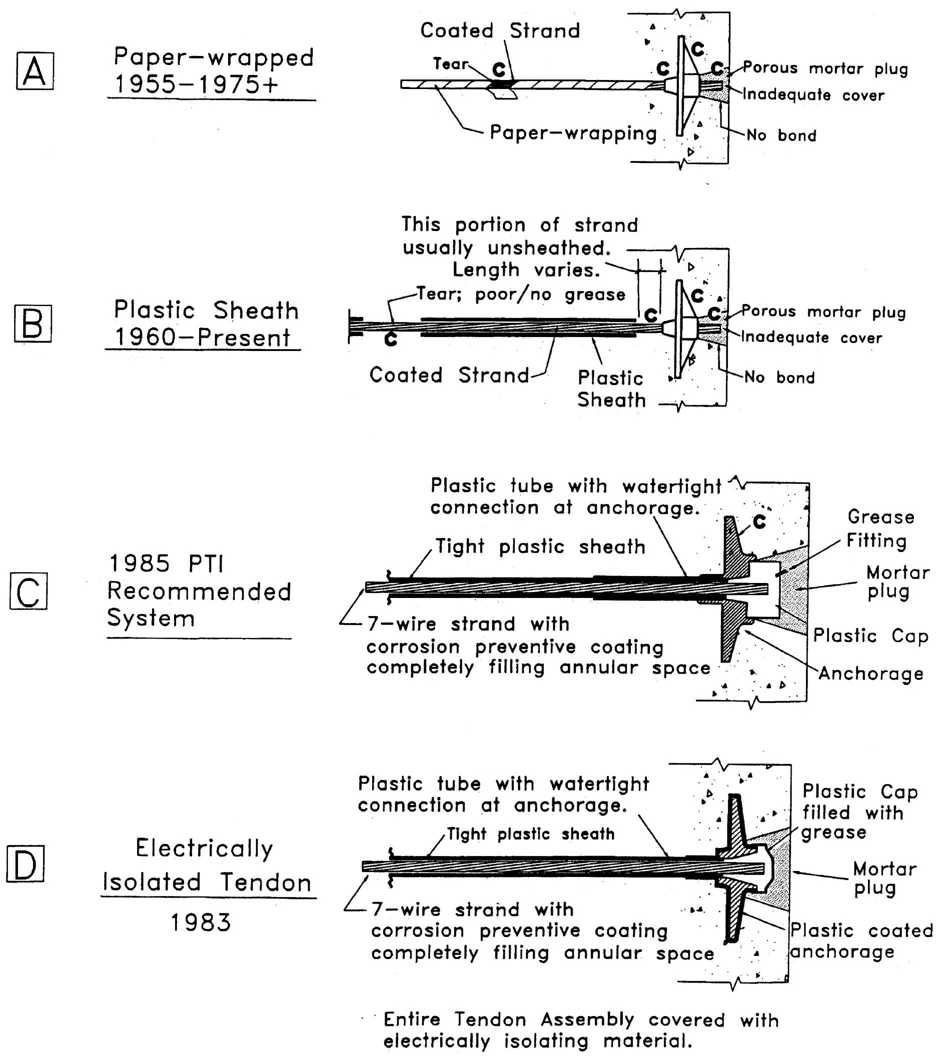

Figure B.3 - Evolution of Monostrand Systems for Post-Tensioning (Ref. B.6) (common locations for corrosion indicated by "c")

B.3.1.1 Sheathing Damage

Damage to the sheathing during transportation, handling and placement can lead to corrosion by allowing moisture and chlorides to reach the tendon. This situation is worsened when cracks in the concrete or concrete with high permeability provides easy access for moisture and chlorides to reach the tendon. In some cases, water has been found inside the sheath of tendons located buildings where they were not exposed to moisture.[B.6] In this situation, it is likely that water entered the tendon sheath during fabrication, handling or prior to concrete placement.

B.3.1.2 Anchorage Protection

Inadequate anchorage protection can lead to multiple forms of corrosion problems in monostrand systems. Corrosion of the anchorage itself is a common problem. Failure of the anchorage in an unbonded system obviously leads to loss of the tendon. Corrosion of the anchorage typically occurs due to lack of a protective barrier or insufficient concrete cover. Concrete or mortar used to cover anchorage recesses after stressing is often low quality, allowing moisture penetration to the anchorage. Placement of the anchorage in locations where exposure to moisture and chlorides may occur, such as at or below construction or expansion joints, has also lead to corrosion related anchorage failures of monostrand tendons. Typical moisture and chloride access to the monostrand system is shown in Figure B.4.[B.8] Schupack[B.6] reported corrosion of live-end anchorages at expansion joints and at dead-end anchorages where the concrete was cracked. Kesner and Poston[B.9] reported corrosion of live-end anchorages at the edge of balconies in a residential building. In this situation, the anchorages were not sufficiently protected for their exterior exposure.

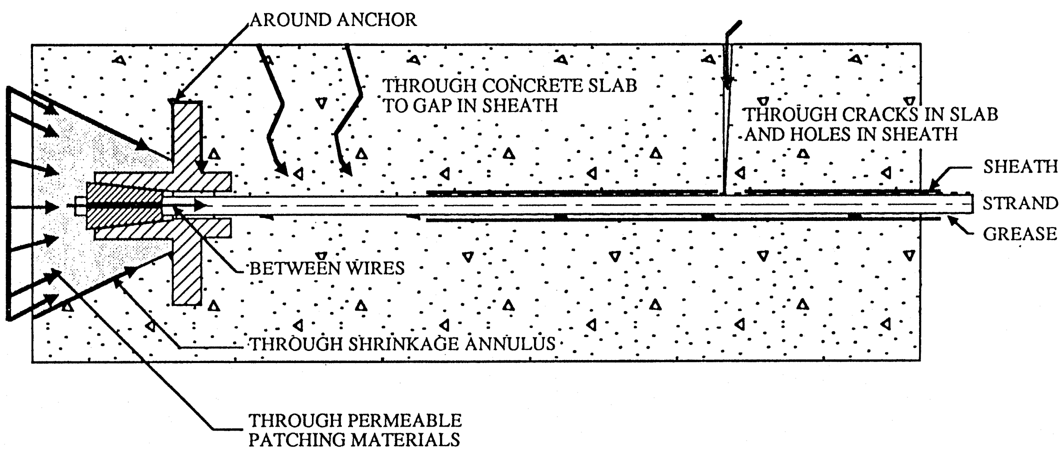

Figure B.4 - Possible Moisture and Chloride Access to Monostrand Systems (Ref. B.8)

Poor quality anchorage protection can also lead to corrosion of the strand stub that projects from the anchor. If the strand stub corrodes, it often provides a pathway for moisture to reach the anchorage, or it may allow moisture to move along the interstices between the wires, and into the greased and sheathed length of the strand. Schupack[B.10] reports a situation where water was leaking through a light fixture in a flat slab post-tensioned building. The source of moisture was rainwater penetrating a poorly protected end anchorage on the exterior of the building. Rainwater entered the tendon thought the anchorage and moved along the tendon inside the sheath, exiting the tendon where the sheath was damaged.

B.3.1.3 System Deficiencies

Many corrosion problems in monostrand systems have been related to the system itself. Most of these problems occurred in older monostrand systems, such as A and B shown in Figure B.3. Modern developments in monostrand systems (C and D, Figure B.3) have eliminated many of the problems found in older systems.

The evolution of sheath types used in monostrand systems is shown in Figure B.5. Earlier sheath systems have shown poor long-term corrosion protection. Paper wrapping is not waterproof and is easily damaged. Peterson[B.11] reports that paper wrapped monostrand tendons are a common corrosion problem in parking structures. Heat sealed sheaths have been found to split open over time, compromising the moisture barrier for the strand. A large number of monostrand corrosion problems have been encountered with the push-through sheath.[B.6] Even when the sheath is intact, the annular space around the strand allows movement of moisture and chlorides. Schupack[B.6] reported severe tendon corrosion and failures in a seven year old platform structure with push-through monostrand tendons. Water entered the sheathing at poorly protected end anchorages. Intermittent corrosion and wire failures were found throughout the structure. Tight fitting extruded sheaths should minimize this problem.

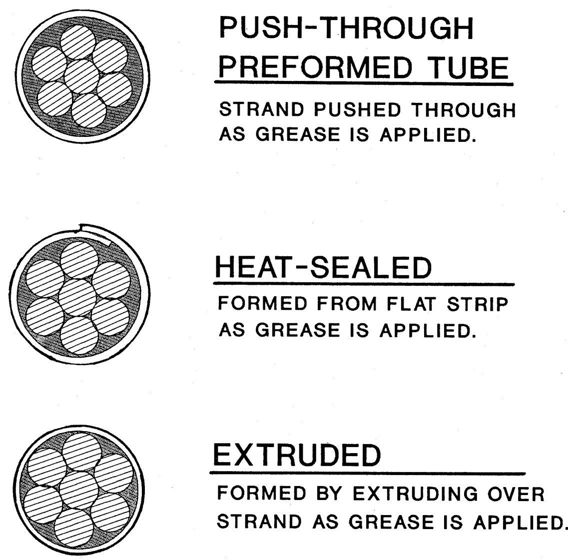

Figure B.5 - Evolution of Sheaths for Monostrand Systems (Ref. B.10)

Another common source of monostrand corrosion problems has been the discontinuity of sheathing and grease on the strand immediately behind the anchorage (see Figure B.3). Schupack[B.6] reported on a thirteen year old parking structure with extruded sheaths. The stressing anchorages in this structure were located at an expansion joint that permitted moisture and chlorides to come in contact with the anchorage. Removal of concrete behind the anchorages revealed severe corrosion of the prestressing strand where the sheath was not present. Examination of strand where the sheath was intact revealed bright strand with no evidence of corrosion. Schupack[B.6] also reported severe pitting corrosion on unsheathed strand at dead-end anchorages in a fourteen year old parking structure. The anchorages were located away form expansion and construction joints. Moisture appeared to reach the tendon through cracks in the vicinity of the anchorage, leading to corrosion.

The grease used in the monostrand systems also plays a critical role in corrosion protection in addition to providing lubrication. Grease related problems have included inadequate coverage, water soluble grease, contaminated grease and the lack of corrosion inhibitors in the grease.

B.3.1.4 Structural Design Aspects

Some aspects of the design process may lead to further corrosion problems. Electrical contact between the monostrand tendon and other reinforcement may provide the opportunity for macrocell corrosion with a large cathode (reinforcement) and small anode (monostrand tendon) in a non-isolated system. Large cathode to anode areas can lead to high corrosion rates and severe corrosion damage. The electrically isolated system shown in Figure B.3 should prevent this occurrence.

Reinforcement congestion or reinforcement ties may lead to sheathing damage during post-tensioning of the monostrand.

Inadequate concrete cover can play two roles in corrosion of monostrand systems. First, concrete is a barrier to penetration of moisture and chlorides. The second role is as protection for the monostrand system. Wiss, Janney, Elstener Associates, Inc.[B.12] reported a parking garage where corrosion of the mild steel reinforcement led to concrete spalling and delamination. A combination of low cover and severe spalling exposed the monostrand tendons at the high points of the tendon profile. Traffic wear and tear eventually damaged the tendons, allowing moisture penetration and corrosion of the tendons.

B.4 UNBONDED INTERNAL TENDONS IN BRIDGES

This section deals with unbonded internal tendons other than monostrand tendons. Included in this category are unbonded multistrand tendons and unbonded post-tensioning bars. Internal unbonded tendons are not commonly used for several reasons. The lack of grouting that provides bond between the tendon and concrete limits the ultimate load carrying capacity of the structure. Unbonded internal tendons also suffer a lack of corrosion protection options, primarily that provided by grout. Failure of an unbonded tendon, due either to tendon corrosion or anchorage corrosion, leads to a complete loss of prestress.

Novokschenov[B.4] reported a condition survey of a bridge with pretensioned and post-tensioned girders located in Salt Lake City. Post-tensioned girders were prestressed with unbonded post-tensioning bars. Two 25 mm (1 in.) diameter and two 38 mm (1.5 in.) diameter prestressing rods were used in each girder. Each post-tensioning bar was placed inside a galvanized steel duct, and no additional corrosion protection was provided. Bar anchorage was provided using end nuts and a steel bearing plate. Anchorages were located in pockets that were filled with mortar after stressing. The bridge was located in an environment where deicing salts were used. After thirteen years of service, failures of the post-tensioned bars began occurring. Failures were first indicated by loud noises heard by persons in the area, and by bars projecting from the ends of the girders. Additional failures were discovered by removing the mortar anchorage protection and checking for loose bar ends and nuts. No cracks or rust stains were found on the exterior of the girders. Twenty-one bar failures were found in total. Pitting corrosion was found on the fractured bars, and the absence of necking or cross-section reduction suggested the failure was brittle in nature. The source of corrosion was attributed to moisture and chlorides entering the ducts at the anchorage zones and moving along the tendon. Corrosion of the steel anchorage plates was rated from moderate to very severe. Corrosion of the plates caused cracking and spalling of the mortar cover. Chloride measurements in the mortar were very high. A malfunctioning drainage system and leaking expansion joints allowed chloride laden moisture to drip onto the ends of the girders and the anchorage areas. Examination of the duct exterior at locations away from girder ends found no sign of corrosion activity. It was concluded that penetration of moisture and chlorides through the concrete cover and galvanized steel duct was unlikely, and that the sole cause of corrosion was penetration at end anchorages.

B.5 EXTERNAL MULTISTRAND TENDONS IN BRIDGES

The most common forms of external multistrand tendons occur in bridges. Cable stays may also be considered in this category. Corrosion protection for multistrand external tendons typically consists of a plastic or metal sheath normally filled with grout or corrosion inhibiting grease. Observed corrosion related failures or problems have resulted from a breakdown in the sheathing system or insufficient protection of the anchorages. These situations are worsened by poor or incomplete filling of the void space around the tendon with grout or grease that allows movement of moisture along the tendon length after penetration.

Robson and Brooman reported[B.13] corrosion related distress in a precast segmental box girder bridge with external tendons. The external prestress was provided by 240 tendons, each consisting of nineteen wires (19 mm (3/4 in.) dia.) inside a plastic, grease filled sheath. Severe signs of distress were observed after approximately twenty years of service life. Two of the 240 tendons had failed completely, and evidence of individual wire fracture was observed in 121 of the remaining tendons. The fractures were attributed to corrosion of the wires in the anchorage zones. It was assumed that corrosion began during a ten month construction delay during which the tendon ends were left unprotected. Because the tendons were external, individual wire failures were detectable by visual inspection. Existing tendons were removed and the bridge was prestressed with new tendons after modifications to the anchorage areas.

B.6 BONDED INTERNAL POST-TENSIONED TENDONS IN BRIDGES

B.6.1 After Stressing, Before Grouting

The time period after stressing but before grouting provides an open opportunity for corrosion of post-tensioning tendons. During this period, the tendon is not fully protected, and tendon corrosion has occurred as a result of water penetrating the ducts through either the end anchorages or grouting ports and vents. Hydrogen embrittlement failures have been attributed to corrosion occurring during the period between stressing and grouting.[B.14] Many construction specifications limit the length of time between stressing and grouting of post-tensioned tendons to forty-eight hours to minimize the potential for corrosion during this period. Corrosion occurring after stressing but before grouting could also lead to failures after the structure has been in service for some period.

B.6.2 In Service

Incidents of corrosion in post-tensioned structures during service have been attributed to a variety of sources. The corrosion protection of a post-tensioning tendon in service is provided by a multi-layered system of variables, and a breakdown in any of the components may lead to tendon corrosion. In most cases, corrosion related deterioration is related to an inadequacy or breakdown in more than one component of the protection system.

B.6.2.1 Grouting

Many corrosion problems have resulted from various aspects of grouting. The effectiveness of the grout as corrosion protection is related both to its material properties and construction practices.

The most common grout related corrosion problems are attributed to incomplete grouting, that is, where the duct is not completely filled with grout. The extent of incomplete grouting may range from small voids to a complete lack of grouting. Common causes of incomplete grouting are construction difficulties, improper construction practices, blocked or damaged ducts and improper placement or usage of vents. The fresh properties of the grout may also affect the grouting process through insufficient or excessive fluidity and excessive bleed water, leading to entrapped air or the formation of bleed lenses. The severity of tendon corrosion is related to the extent of incomplete grouting and the availability of moisture, oxygen and chlorides. In general, the most severe attack occurs when the tendon is intermittently exposed and embedded in the grout. In this situation, a concentration cell may occur due to the variations in the chemical and physical environment along the length of the tendon. Concentration cells may result from differences in oxygen, moisture and chloride concentration, and often lead to severe macrocell corrosion.

Tendon corrosion may also occur in situations where the entire length of the tendon is well grouted. The most common cause of corrosion in these situations has been sources of chlorides in the grout itself. Examples include seawater used as the mixing water or chloride containing admixtures. A combination of severe exposure conditions and low cover may lead to corrosion of the duct and subsequent penetration of moisture and chlorides from an external source.

Isecke[B.15] described a detailed examination of a bonded post-tensioned bridge in Germany. The bridge was demolished after less than twenty years of service due to corrosion related deterioration. Isecke reported varying levels of grouting: full grouting, partial grouting, partial or total coating of the steel surface with a thin film of grout and complete absence of grout. No corrosion was found where grouting was complete and the steel fully embedded in grout. Varying amounts of corrosion damage were found under all other grouting conditions. The most severe corrosion was reported in partially grouted ducts at the boundaries between exposed and embedded steel. In ducts that were completely ungrouted, the prestressing steel was covered with a thin film of rust, but the reduction of area due to corrosion was deemed very small.

Schupack[B.16,B.17] performed an extensive forensic examination on a thirty-five year old post-tensioned bridge. The extent of corrosion damage in this bridge was not significant enough to affect structural behavior.[B.16] Corrosion deterioration was attributed to two sources: poor and incomplete grouting throughout the bridge, and the use of grout containing high levels of chloride in some girders. Schupack found a range of grouting, from fully grouted, to partial grouting to a complete lack of grout. The extent of corrosion was dependent on the completeness of grouting, the type of grout, and the availability of moisture. No corrosion was found in tendons where the ducts were completely filled with grout that did not contain chlorides. In partially grouted tendons (with no chlorides in the grout) and in ungrouted tendons, most exposed wires had surface corrosion. Severe corrosion was found at tendon low points where water had collected in the duct. Several tendons that were completely ungrouted, but free of moisture, showed no signs of corrosion. Several girders in the bridge were grouted using an expansive grout that contained high levels of chloride. This grout was not recommended for post-tensioning applications by its manufacturer, as expansive properties were achieved by adding iron filings and chlorides to provide expansion though corrosion of the iron. Chloride analysis performed on grout samples from the bridge found chloride levels as high as 8000 ppm by weight of grout. Very severe tendon and duct corrosion was found where this grout was used. Deep pitting corrosion and random wire breaks were found. Schupack also reported significant longitudinal cracks in the webs of the girders following the tendon profile. In most cases, the cracks were attributed to freezing of water in partially grouted or ungrouted tendons, rather than from tendon corrosion, illustrating additional deterioration that may result from poor grouting.

B.6.2.2 <u>Inadequate Concrete Cover</u>

Concrete cover provides an additional level of protection for the tendon. In situations where the protection provided by the duct is less than adequate, low concrete cover has contributed to tendon corrosion.

Novokschenov[B.4] reported a condition survey of the Gandy Bridge in Florida. This bridge consisted of precast post-tensioned girders with reinforced concrete deck slab. Post-tensioning was provided using 28.6 mm (1.125 in.) diameter prestressing bars. Each bar was located inside a 38.1 mm (1.5 in.) grouted metal duct. The bridge was less than thirty-five years old at the time of inspection, and had experienced significant cracking and spalling resulting from corrosion of the post-tensioning ducts and tendons. Measured values of concrete cover for the bottom tendons were less than the specified value of 70 mm (2.75 in.), ranging from 32 mm (1.25 in.) to 64 mm (2.5 in.), with an average of 53 mm (2.1 in.). Concrete in the girders was air-entrained with low water-cement ratio. Rapid chloride permeability measurements on concrete samples from the bridge indicated moderate to low permeability. Because the concrete was of good quality, Novokschenov concluded that insufficient concrete cover was the major cause of corrosion of the post-tensioning tendons.

B.6.2.3 <u>Duct Problems</u>

The post-tensioning duct is an important component of corrosion protection in post-tensioned structures. Many forms of ducts exist, ranging from non-permanent duct formers, to galvanized steel ducts, to plastic ducts, each providing an increasing level of protection. Incidents of corrosion have resulted from damaged ducts, improper splices between ducts, corroded ducts and situations where non-permanent duct formers have been used. Holes in the duct may allow concrete to enter the duct during casting. This may hamper placement and tensioning of the tendons, and may cause difficulties during grouting. Damage or misalignment during construction or concrete placing may also lead to post-tensioning and grouting difficulties.

As mentioned in the preceding section, Novokschenov[B.4] reported duct and tendon corrosion in a post-tensioned bridge in Florida. Novokschenov concluded that insufficient concrete cover led to severe corrosion of the metal ducts and post-tensioning tendons. If non-corroding plastic ducts had been used, it is possible that corrosion related deterioration of the post-tensioning system could have been eliminated in spite of low cover. Isecke[B.15] also reported total deterioration of metallic ducts due to corrosion in many areas of a post-tensioned bridge. Corrosion of the duct lead to moisture and chloride penetration into the grout. In most cases, deterioration of the duct corresponded to severe corrosion and occasionally fracture of the prestressing steel.

B.6.2.4 **Anchorage Protection**

Anchorage corrosion in bonded tendons is generally not deemed failure critical, unlike unbonded tendons. Bond between the tendons and concrete will prevent a complete loss of prestressing. However, anchorage corrosion and inadequate anchorage protection can lead to the ingress of moisture and chlorides into the tendon. This condition is particularly severe with poorly grouted ducts that may allow moisture to readily move along the length of the tendon. Corrosion of anchorage components can also cause cracking and spalling of concrete in the vicinity of the anchorage.

Most anchorage corrosion problems result from two factors: inadequate protection and location. Inadequate protection may include insufficient cover, permeable materials used to fill the anchorage recess and lack of bond between fill material and anchorage recess. The location of the anchorage plays a significant role. Normally anchorages are located at the end of the member. In many structure types, expansion joints are located over the member ends. Poor detailing and maintenance of the joints has permitted chloride laden moisture to come in direct contact with the anchorage zones of the member, creating particularly severe exposure conditions.

Dickson et al[B.18] reported a detailed evaluation of a thirty-four year old precast post-tensioned girder. The girder was removed from a bridge that had been subjected to deicing salts throughout its service life. The overall condition of the girder was excellent, and the observed corrosion deterioration was not deemed to affect structural behavior. The most severe corrosion was found on the anchorages of the girder. Anchorage protection was provided by a cast-in-place concrete end diaphragm. Surface corrosion was found on all anchorage and bearing plate surfaces. The post-tensioning wires within the anchorage were corroded more severely than the wires within the length of the duct. In general, the ducts were very well grouted with only one void found during dissection of the girder. Corrosion of the wires within the length of the tendon was very minor. Chloride analysis performed on grout samples indicated that chlorides had infiltrated the duct through one of the anchorages in spite of the cast-in-place concrete anchorage protection.

Isecke[B.15] also reported infiltration of moisture and chlorides through end anchorages during the examination of a post-tensioned bridge. The anchorages in this bridge were unprotected. Anchorages located in the vicinity of expansion joints were exposed to chloride laden moisture runoff from the bridge deck. Anchorages in these areas were heavily damaged by corrosion. Moisture and chlorides penetrated through the anchorages, leading to heavy corrosion on the post-tensioned bars used in the structure. In areas of the structure were the unprotected anchorages where not exposed to deck runoff, no corrosion was found on the anchorages.

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