
AASHTO-AGC-ARTBA Joint Committee

Subcommittee on New Highway Materials

Task Force 32 Report

Manual for Corrosion Protection of Concrete Components in Bridges



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INTRODUCTION

This document is disseminated upon the sponsorship of the Joint Committee of the American Association of State Highway and Transportation Officials, the Associated General Contractors of America and the American Road and Transportation Builders Association (the Joint AASHTO-AGC-ARTBA Committee) in the interest of information exchange, based on the combined technical expertise of the authors. As such, the opinions and conclusions implied or expressed herein do not necessarily reflect the official views or policies of the Joint Committee or its member organizations.

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DISCLAIMER

Readers and users of this Manual are advised that the information contained herein is based on current knowledge of the Task Group members, and is not intended to represent an exhaustive treatise on this subject, nor to indicate an endorsement of any of the corrosion protection systems or specific products included.

Where experience, research or test data on a particular system or product is limited, a list of "users" of the technology has been included to allow the reader to make contact with the appropriate agency official and to obtain firsthand information on the system or product for the intended use.

For the more commonly used corrosion protection systems, Appendix A summarizes the results of a nationwide survey of AASHTO Bridge Subcommittee member organizations that was conducted at the beginning of the Task Group's efforts.



FOREWORD

This manual was developed by a Joint Committee of the American Association of State Highway and Transportation Officials (AASHTO), The Associated General Contractors (AGC), and the American Road and Transportation Builders Association (ARTBA). The Joint Committee, AASHTO-AGC-ARTBA, formed Task Force Number 32, to address the need for a manual on corrosion protection of concrete components in bridges. This manual addresses the numerous systems available to provide corrosion protection for bridge components. Task Force members and their affiliation are as follows:

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James Cooper, Secretary	FHWA
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The Objective and Scope of this Task Force, as approved by the Subcommittee, are as follows:

OBJECTIVE:

Corrosion of reinforcing steel in bridge decks has caused both structural and rideability problems, and has been a recognized national problem for 15 years. In the past 5 years, an even more serious problem has surfaced with other load carrying members of highway structures. They are evidencing the same destructive effects of corrosion noted in bridge decks. The measures for bringing this problem under control involve considerations in new construction and rehabilitation of the present inventory. These considerations include rigid overlays, sealers, cathodic protection, epoxy coated bars, inhibitors, additives, etc., to delay or offset the effects of chloride infiltrations. The objective of this study is to document the types of systems being used for this purpose, noting the differences in techniques. Successes and failures of these systems will be noted by providing technical evaluation results of each system. With the above information, guidelines for use of each system will be developed.

SCOPE:

The Task Force will catalogue and evaluate concrete corrosion protection systems and develop criteria, specifications and construction procedures that will improve the ability of concrete superstructure and substructure elements, excluding highway pavements, to withstand corrosive environments, and the corrosive affects of chlorides from deicing chemicals or saltwater.

Special thanks to Edna Mae Boegli for the hard work and excellent typing in developing this document.

PREFACE

Almost 53 percent of the nation's highway bridges are identified as having concrete as their primary construction material. Of all bridges, 69 percent of the decks are concrete. As of 1991, the National Bridge Inventory (NBI) shows an existing total of over 577,000 bridges (includes 97,000 culverts), representing 2.9 billion square feet of bridge deck area.

Over 135,000 (24%) of these bridges are rated structurally deficient. The cost to replace or rehabilitate these structures would be staggering. Rehabilitation of concrete structural components is beyond the scope of this Task Force Manual. The Strategic Highway Research Program (SHRP) has contracted with a private firm to provide guidance in that area. This manual provides guidance for corrosion protection of new concrete bridges and components, such as bridge decks that are being totally replaced, and is intended to provide the bridge owner with the proper techniques to use from the initial step of deciding what to do, to the final step of evaluating the in-place product and evaluating its acceptability. In some cases, a rigid concrete overlay may be the appropriate protection strategy for a new deck. In that case, the reader is directed to the AASHTO/AGC/ARTBA Guide Specifications - Manual for Design and Construction of Concrete Overlays for Bridge Decks, May, 1989.

This Manual was written by leading experts in the field, identified in the Foreword, representing public and private sector engineers. Design engineers, material suppliers and contractors had input into the manual's development. Because of the adverse impact on the travelling public caused by premature bridge failures, it is imperative that a quality product be provided at the time of construction. Utilizing the criteria herein should ensure minimizing the number of concrete bridges or concrete components that will deteriorate to an unacceptable condition over their service life.

It is recognized that individual State practices, and specific environmental conditions may warrant modification of portions of this Manual. However, modifications should be exercised with care because of the interdependency of the many variables on the quality and performance of a good concrete corrosion protection system.

It is recommended that specifications be developed from these criteria.

AASHTO-AGC-ARTBA
Manual for Corrosion Protection of
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AASHTO-AGC-ARTBA

Task Force 32

CORROSION PROTECTION OF CONCRETE COMPONENTS IN BRIDGES

CHAPTER 1

Background

A. Introduction

In the U.S. there are almost 577,000 bridges and innumerable other highway structures - retaining walls, sign bridges, etc. Virtually all of these employ reinforced concrete in one manner or another as a structural component.

Each of these structures has a finite life. At some time a structure will be removed due to lack of need, replaced due to deterioration, destroyed by natural or man-made forces, or replaced because it no longer meets the functional needs at the particular site. Most bridges are replaced due to "functional obsolescence." A bridge too narrow to carry the required volume of traffic is an example. However, many structures, or portions of structures, are also replaced due to the effects of the environment indicating the need for adequate protective strategies. The 1989 FHWA Report to Congress "The Status of the Nation's Highways and Bridges," notes that 24% of highway bridges are "structurally deficient," and thus are candidates for repair, rehabilitation or replacement.

Webster defines deterioration as "becoming impaired in quality, state or condition." There is no question that the concrete in our older structures is deteriorating. This deterioration may be the result of poor mix design, construction practices, or environmental affects such as freezing and thawing of the concrete matrix or some other subtle and possibly less understood reason such as alkali-silica reactions, that is not apparent for many years. These types of deterioration of concrete can be slowed with simple material changes; i.e., sulphate resistant concretes in sea water applications; top quality aggregates to prevent the destructive alkali-silica reaction; or an air-entrainment admixture in a freeze thaw environment. It is not the purpose of this manual to address these types of deterioration. A properly qualified concrete materials engineer can provide solutions or mix designs to withstand these effects. The American Concrete Institute's (ACI) ACI 201 "Guide to Durable Concrete" tells how to achieve long lasting concretes.

There is another form of concrete deterioration that is caused by corrosion of the reinforcing steel in the structure. Concrete bridge decks are the most obvious example of this deterioration. Corrosion of reinforcing steel in bridge decks has caused both structural and rideability problems and has been recognized as a national problem since 1975. Of the 577,000 highway bridges, over 69% have concrete bridge decks. It is safe to say that most bridge decks over 25 years old in the northern latitudes of the U.S. either have been or are ready for rehabilitation. An equally serious problem has surfaced with other load carrying members of highway structures due to this corrosion. Thus, prevention of corrosion of all reinforcing steel must be considered by all bridge owners.

Good quality concrete protects reinforcing steel through its low permeability and highly alkaline environment. The high pH of the concrete provides a protective oxide film on the reinforcing steel. Corrosion of the reinforcing steel occurs when sufficient oxygen is available and the protective oxide film is damaged, or intruded upon, thereby allowing the electrochemical oxidation process to occur. Carbonation or chloride ions can destroy or penetrate this passive film. A sufficient amount of chlorides, oxygen from the air and moisture can result in detrimental corrosion (or rusting) of the reinforcing steel.

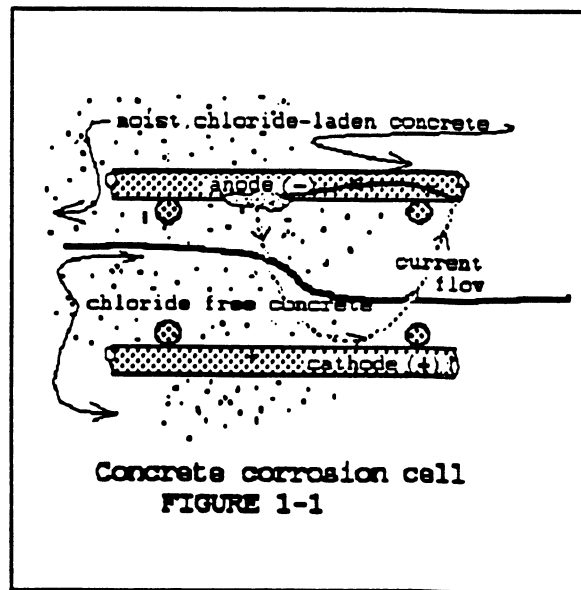
The primary source of corrosive ions is highway deicing chemicals containing chlorides commonly used in the winter. Chlorides can also come from sea water in marine exposures as well as from the admixtures, mixing water and concrete aggregates. The chloride ions reach the reinforcing steel through cracks and joints in the concrete as well as by permeating through the concrete. Over time, sufficient chlorides can accumulate near the reinforcing steel to induce corrosion. The rate at which chlorides permeate into the concrete depends upon the permeability of the concrete as well as the amount of moisture present to transmit the chloride ion. The extent of cracks and jointing in the concrete are a function of the structures size, concrete properties, structural design, and traffic and environmental loadings.

Once the chloride ions from the deicers, seawater or other sources have destroyed or penetrated the protective film on the steel, an electric cell, Figure 1-1, is formed along the steel, or between steel bars, and the electrochemical oxidation process of corrosion begins. Steel areas along the bar become the anode, discharging current in the electric cell. This is where the corrosion takes place and the iron goes into solution. The steel areas that receive current are the cathodes where hydroxide ions are formed. The iron and hydroxide ions then form iron hydroxide, which further oxidizes to form rust, or iron oxide.

In addition, the chloride ions combine with the iron ions to form soluble iron-chloride that carries the iron into the concrete, where it is oxidized and forms rust. Rusting is an expansive process (rust expands several times its original volume) which induces internal stress and eventual delamination and spalling in the concrete over the reinforcing steel. The cross-sectional area of the steel can also be significantly reduced resulting in a loss

of load carrying capability. [A more detailed discussion of the corrosion of metal in concrete can be found in ACI 222, "Corrosion of Metals in Concrete."] Once the corrosion starts, the rate of corrosion is influenced by the concrete's electrical resistivity, moisture content, temperature, and rate at which the oxygen migrates through the concrete to the steel.

Concrete in normal exposures always has oxygen present which is necessary in the corrosion process. Some moisture is also necessary, but concrete which is saturated with water and kept completely submerged will have only a minimal amount of corrosion. This is because of the slow transfer of dissolved air through water filled pores in the concrete. Concrete that has not been completely cured prior to application of chlorides offers more opportunity for corrosion to occur, because of the better transmission of air. As little as 0.15% water soluble chloride, or about 0.2% acid soluble chloride, by weight of cement is sufficient to initiate corrosion of the embedded reinforcing steel.



The purpose of incorporating a corrosion protection system into a structure is to insure that the concrete elements will survive at least until the end of the structure's functional life. Sometimes this functional life requirement is so short (20 years or less) that the only system required is a reasonably good concrete. However, this is not the norm and structures should be constructed of materials that will achieve a much longer "service" life (the time during which minimal, if any maintenance, needs to be done that is attributed to the effects of chloride penetration).

What is the length of this service life for designing our structures? Some say 50 years, others 75 years. An examination of the current inventory (577,000 bridges) noted that 28% are already over 50 years old and 10% over 75 years. At the present time (1991), the AASHTO Bridge Subcommittee is developing a new bridge design specification that will base structural analysis on a 75 year recurrence interval for certain critical loads (floods, earthquakes, etc.). Thus, it would seem logical to design the corrosion protection system to ensure the integrity of the structure for at least 75 years. In reality, a system that will do this most probably will provide a much longer service life. So, if there is a way to measure it, this manual is intended to provide corrosion protection strategies for at least a 75 year service life for the concrete elements. This does not mean that each and every system, installed at the time of construction, will not require any thought or maintenance only after 75 years. Effective corrosion protection strategies will require periodic "maintenance" to ensure their function is retained. Even though

some systems (e.g., cathodic protection) will require relatively frequent monitoring, maintenance and even replacement, they have certain benefits (moderate initial cost, rapid installation, etc.) and may in fact have a lower life-cycle impact (time and/or money) which may support their use. However, their use on new or existing concrete structural elements should be accompanied with proper long-term commitments for monitoring, and for periodic maintenance.

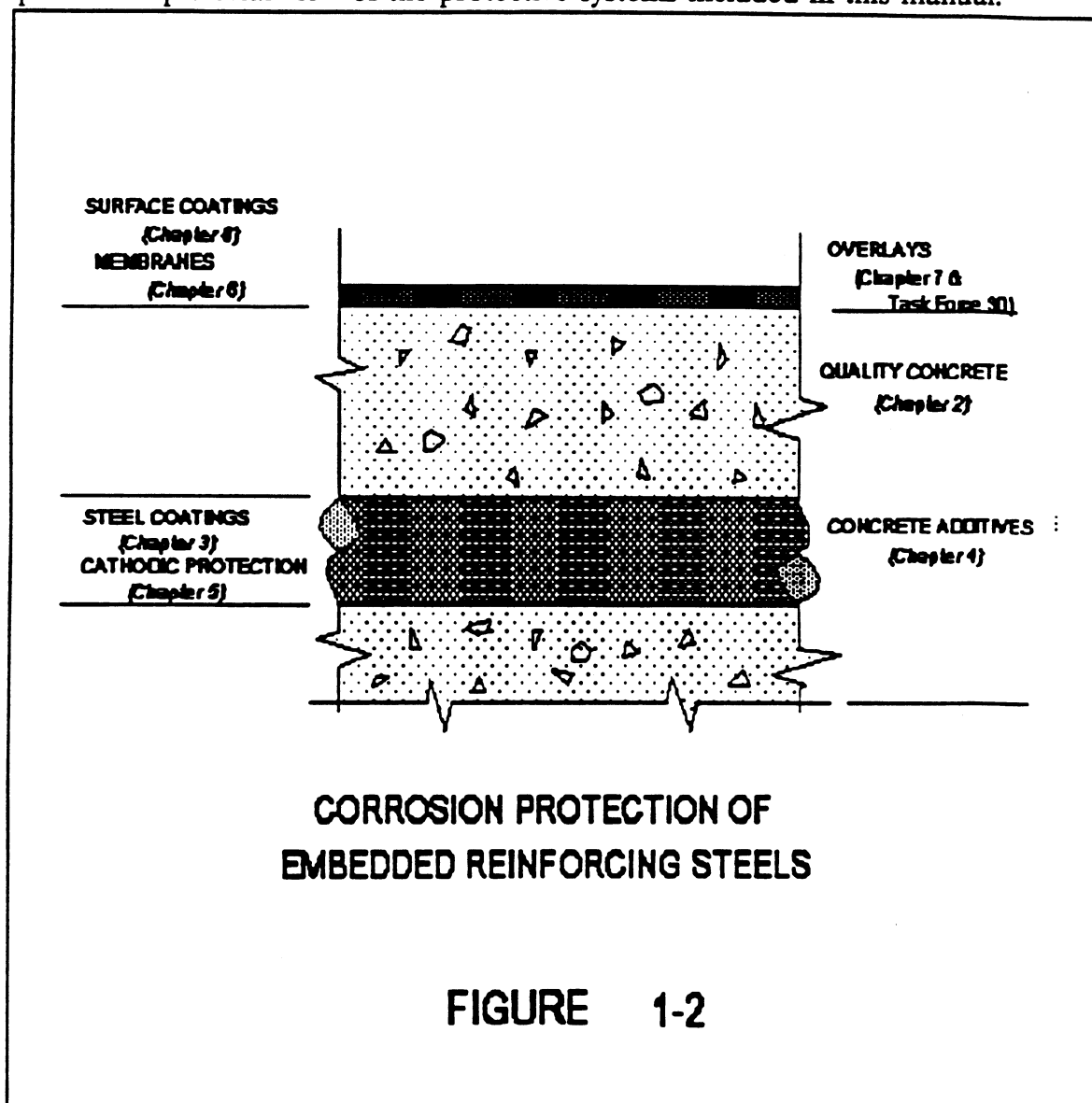
The first line of defense against chloride induced corrosion is the use of a low permeability concrete. Special admixtures such as microsilica (MS), can also be used to reduce the permeability of concrete. Corrosion protection strategies such as surface coatings, or low permeability overlays serve to slow down or prevent the ingress of chloride into the concrete. Corrosion inhibitors attempt to neutralize a certain level of chloride ion. Cathodic protection systems attempt to control the corrosion current flow thereby controlling the rate and location of corrosion. All of these corrosion protective systems have limits as to their ability to resist chloride induced corrosion. The choice of which system to use depends upon the initial and life-cycle costs, the level of maintenance desired, as well as the environment in which the structure will reside.

Each geographical area must be assessed to determine the potential for corrosion, and then the pros and cons of each available protection strategy carefully assessed prior to selecting the system(s) to be used. Many Bridge Owners have adopted a corrosion protection strategy that determines minimum levels of protection required, depending upon the environmental exposure conditions. Appendix B is an example, and shows how the States of Texas and California have established corrosion protection levels based on anticipated corrosion attack from above (deicing salts) or below (sulphate soils or saltwater).

B. Contents and Structure of Report

This manual catalogs concrete corrosion protection strategies and presents criteria, specifications and construction procedures intended to improve the ability of concrete superstructure and substructure elements to withstand the corrosive affects of chlorides from any source.

In addition to this Background chapter, the manual includes seven other chapters. Figure 1-2 presents a pictorial view of the protective systems included in this manual.



Corrosion protection systems can be classified as internal protection (Chapters 2 through 4) or external protection strategies (Chapters 5 through 8)

Chapter 2 is devoted to design details and construction procedures intended to enhance the corrosion resistance of concrete structures. The details and procedures discussed are basic requirements irrespective of the protection systems and strategies provided as discussed in subsequent chapters. Such details include concrete cover, quality concrete with low water-cement (w/c) ratio, adequate curing, elimination of joints, proper drainage, and prestressing.

Chapter 3 is concerned with coatings for steels embedded in concrete. The coatings include epoxy coating for reinforcing bars, wire mesh and prestressing steel, and zinc (galvanized) coating for reinforcing steel.

Precautions for handling these bars without damaging the coatings are discussed. Procedures for repair of damaged coatings are given.

Chapter 4 covers admixtures used in portland cement concrete added at the time of mixing the concrete for decks, superstructure or substructure components including precast and cast-in-place members. This chapter provides a description of each product, material specifications, construction requirements and references for additional information on the specific products.

Cathodic protection, although normally used for the rehabilitation of existing structures, can be used for the corrosion protection of new concrete structures. The different methods of cathodic protection are discussed in Chapter 5.

Chapter 6 deals with membranes applied to decks. These include membranes exposed to traffic and those protected from traffic with an overlay.

Bridge deck overlays have several functions; they improve rideability, skid resistance, deck drainage and also provide protection of the decks cross-section and structural integrity, in addition to providing a corrosion protection system.

AASHTO/AGC/ARTBA Task Force 30 has prepared Guide Specifications - Manual for Design and Construction of Concrete Overlays for Bridge Decks. Contents of this guide are summarized in Chapter 7. This chapter also includes a discussion on impermeable polymer modified asphalt overlays.

Surface coatings and sealers applied to a concrete surface are covered in Chapter 8. There are many new materials available to coat the surface of concrete substructures and superstructures to improve their impermeability to chloride ion migration.

CHAPTER 2

Design Details and Construction Procedures to Minimize Corrosion

A. Introduction

Presented in the following sections are recommendations concerning structural design details and construction procedures that will provide corrosion resistance, or will enhance the corrosion resistive characteristics of concrete structures. These structural details are considered to be separate and distinct from the other corrosion protection systems discussed in this Manual. Application of the recommendations in this section will improve the corrosion protective characteristics of concrete structures whether used independently, or used in conjunction with the other corrosion protection systems. The appropriate mix of corrosion protective features in a given project requires consideration of the related costs as well as the severity of the exposure conditions. However, in nearly all cases, the costs of including a reasonable mix of corrosion protective measures in new construction represents only a small fraction of the direct and indirect costs involved in the repair of a deteriorated structure. Indirect costs are those related to the inconvenience to users resulting from partial or complete closure of a facility. With respect to urban freeways, the indirect costs may be very substantial.

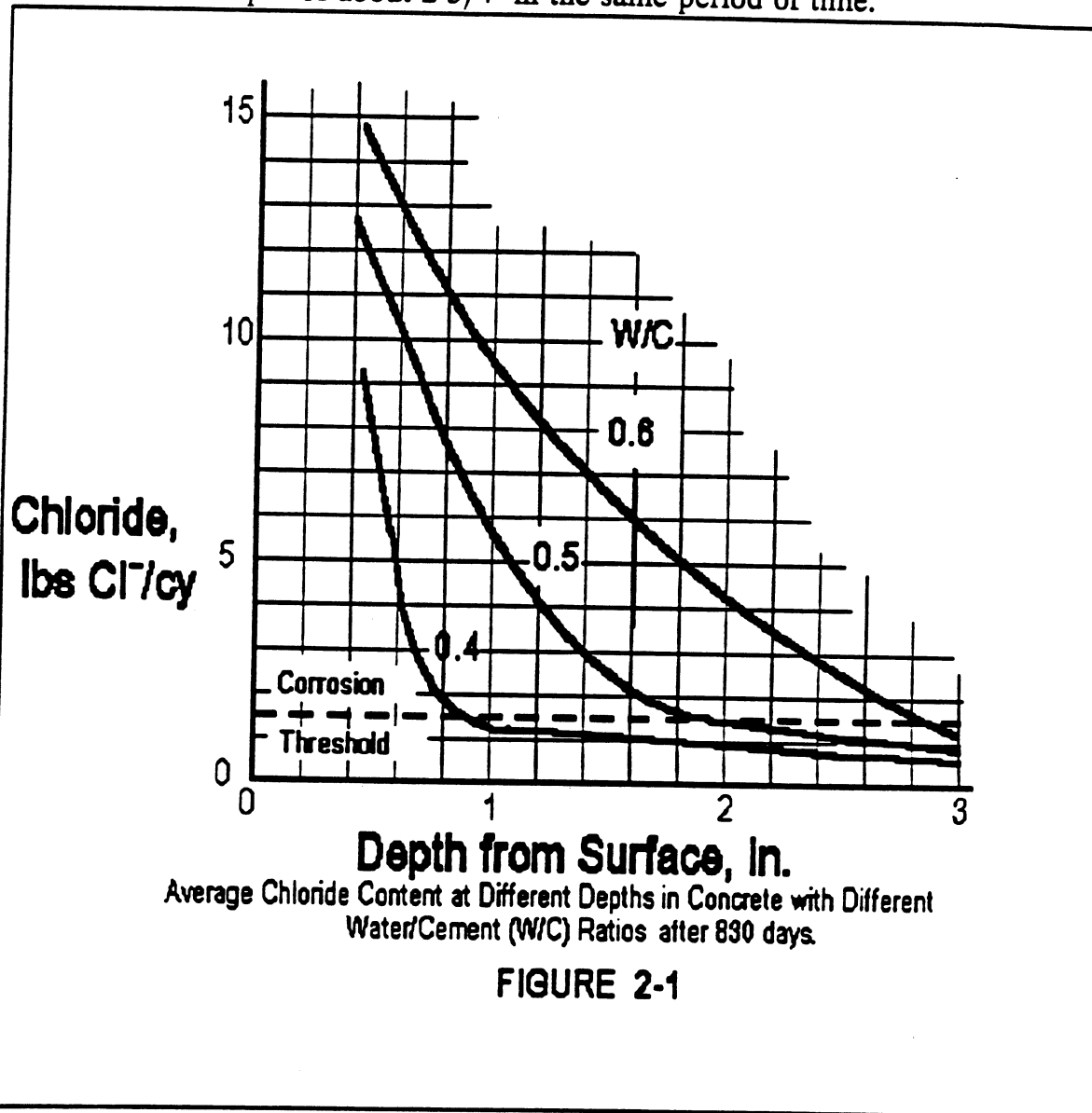
B. Concrete Cover

The AASHTO "Standard Specifications for Highway Bridges", 14th Edition, Section 8.22.1 of Division I, reinforcing steel cover requirements have, in recent years, been modified to provide additional protection for structural elements exposed to aggressive environments. Nonetheless, cover in addition to the values in the AASHTO specifications may be appropriate. Additional cover may be warranted to allow for placing tolerances, wear due to traffic, and depending on other protective systems used, the penetration of deicer chemicals into the concrete. Reduction of the cover specified by AASHTO as a trade-off for use of other corrosion protective systems is considered to be inappropriate. The AASHTO specified values are considered to be minimum values regardless of other corrosion protective systems employed on a project.

The importance of concrete cover on corrosion protection is illustrated in Figure 2-1, adapted from "Influence of Design and Materials on Corrosion Resistance of Steel in Concrete", by David Stark, Research and Development Bulletin RD 098, Portland Cement Association, Skokie, Illinois, 1989, 40 pages. The amount of chloride intrusion, in



pounds per cubic yard of concrete, versus depth from the surface, in inches, is given for three different water/cement ratios after 830 daily salt applications. The threshold of corrosion is depicted by the horizontal dashed line. For example, with a water/cement ratio of 0.4, threshold of corrosion is obtained at a depth from the surface of about 3/4 in. On the other hand, with a water/cement ratio of 0.6, the same chloride concentration will be found at a depth of about 2 3/4" in the same period of time.



Appendix G also provides a theoretical solution for evaluating the effectiveness of various corrosion protection systems, by estimating the time it will take for a threshold level of chlorides to reach the rebars.

C. Concrete Quality

A high quality concrete is needed to resist the normal effects of the environment such as freeze-thaw though durability is normally achieved by proper addition of an air-entraining admixture. Since freeze-thaw deterioration may be more severe than that caused by corrosion, care must be exercised that a proper entrained air-void size and distribution in the concrete is not compromised by other corrosion protection strategies, specifically admixtures. The Portland Cement Association's (PCA) Manual (p. 40 Reference 3 below) notes the generally accepted air-void characteristics necessary to produce durable concretes.

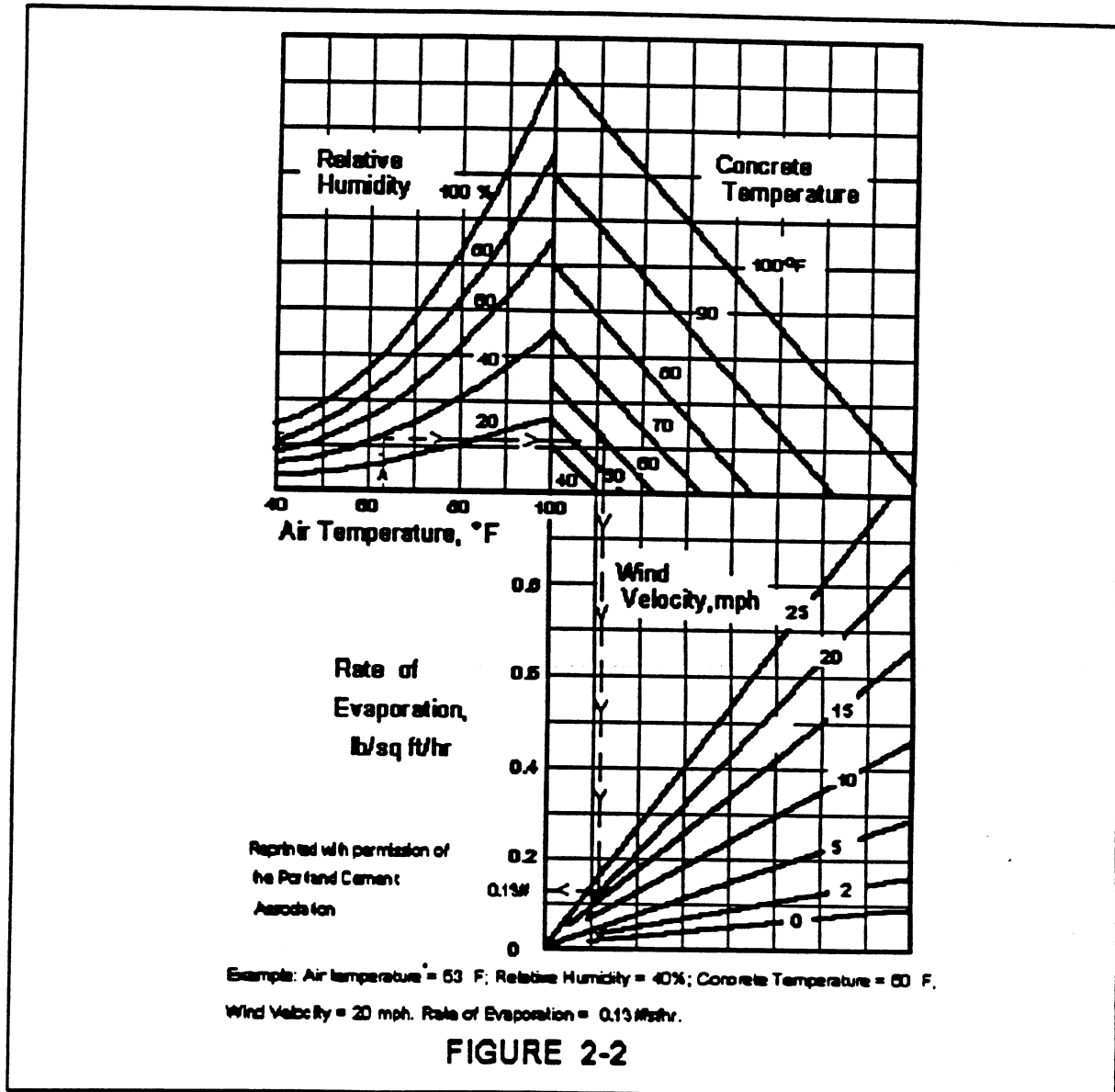
Much has been written about the need for high quality, dense concrete to provide protection against penetration of deicer chemicals and other corrosive agents. This concept is entirely valid if the concrete is not cracked. However, high quality dense concrete does not provide protection against corrosion when corrosive agents have access to the reinforcement through cracks. For a variety of reasons, cracks often form directly over and parallel to reinforcement, which allows ready access of corrosive agents, even when high quality, dense concrete is used. This is not to deny the corrosion protective advantages of high quality concrete, but is to suggest that high quality concrete and an adequate amount of cover, do not by themselves afford assurance against corrosion. Accordingly, the use of an adequate amount of cover, combined with high quality, dense concrete is recommended along with other measures to control cracking and the related possibility of the penetration of corrosive agents to the reinforcement.

Concrete shall be at least the quality of class as specified by the AASHTO Bridge Specification in Division II, Section 4.5 "Classes of Concrete." Proportioning for the various classes shall be in accordance with Section 4.6 "Proportioning of Concrete." During placement of the concrete, other applicable sections of Division II, Section 4 shall be closely followed to ensure top quality concrete, and with that, maximum corrosion protection.

Other references that provide controls for corrosion resistant concrete structures are:

- A. ACI 345-82 "Standard Practice for Concrete Highway Bridge Deck Construction".
- B. ACI 214-77 "Recommended Practice of Evaluation of Strength Test Results of Concrete".
- C. PCA "Design and Control of Concrete Mixtures", by Portland Cement Association, Thirteenth Edition.

Because of the importance of proper curing and protection of wet concrete to provide maximum quality control, figure 11-6 of the PCA reference is reproduced herein as Figure 2-2. This same figure appears in ACI 303, "Hot Weather Concreting Practices". Concrete placements should not be initiated when ambient conditions (wind/temperature) may produce an evaporation rate of mix water in excess of 0.2 lb/sq ft/hr from the exposed surface (some limit the rate to 0.15). In areas of the country where this restriction may restrict construction time too severely, alternate procedures must be considered such as night-time placements or continuous fogging during placements.



D. Trial Batch

To better ensure that the proposed mix design will provide the corrosion protection desired, a trial batch of at least 5 cubic yards should be required to be placed and cured in conditions as close to site conditions, particularly ambient temperature, as is possible, and far enough in advance that adjustments can be made in the mix design to improve workability, strength and/or permeability (See ACI 211.1-Recommended Practice for Selecting Proportions for Normal, Heavy Weight and Mass Concrete). Permeability of the concrete specimens taken from the trial batch should provide permeability rates of no more than 2000 coulombs for low and moderate chloride exposure conditions and 1000 coulombs for severe exposure conditions, including marine applications. Permeability tests shall be conducted in accordance with AASHTO Material Specification T-277.

It is important to note that some of the products included herein may not have approved ASTM or AASHTO material specifications to use in evaluating the product. In these cases the importance of the trial batch, and the testing of the hardened concrete samples, cannot be emphasized enough to insure the durability and strength of the structural components is not compromised.

E. Details of Conventional Reinforcement

Some durability studies have identified cracks and crack width as factors in the development of corrosion of reinforcement. Equation (8-61) in Section 8.16.8.4 of the AASHTO Specifications provides guidance on details to control flexural cracking of the concrete, and requires for structures exposed to corrosive environments or the use of deicer chemicals, that additional corrosion protective measures be utilized in addition to satisfying Equation (8-61). Such measures include "...increasing the denseness or imperviousness to water or furnishing other protection such as a waterproof protecting system." Also, the use of more smaller reinforcing bars to provide the required area, versus fewer and larger bars, will help control cracking.

F. Prestressing as a Means of Corrosion Protection

Many research projects have reflected the high corrosion resistance provided by prestressed concrete, both precast, pretensioned concrete products, and site-cast post-tensioned structures or structural elements. The favorable corrosion protective characteristics of prestressed concrete structures and structural elements is considered to reflect the elimination of cracks because of the prestressing. Factory produced, precast, prestressed concrete products generally have the additional advantage of the higher degree of quality control that is possible in a protected, factory environment. Where site-cast, post-tensioned structures have experienced corrosion problems, the cause has often been the use of inappropriate details, and/or inadequate construction practices,

particularly in the grouting of tendons. Accordingly, it is necessary to utilize appropriate details and provide qualified inspection to ensure corrosion resistance of site-cast post-tensioned structures or structural elements.

National Cooperative Highway Research Program (NCHRP) Report 314, "Corrosion Protection of Prestressing Systems in Concrete Bridges" has identified a variety of newer materials for use for improved corrosion resistance including polyethylene post-tensioning duct, epoxy-coated anchorages and epoxy-coated prestressing strand. The concluding paragraph of the summary of this report states in a concise way the present state-of-the-art concerning corrosion resistance of prestressed concrete construction:

"Most of the newer materials tested in this program provided significantly enhanced corrosion protection when compared to traditional materials; however, these tests and the nationwide survey showed that traditional materials can provide good protection in normal environments, even with chlorides present. On the other hand, codes have recently been modified to require lower permeability concrete and greater clear cover over the reinforcing and prestressing steel and its hardware. Thus, these new code provisions for better overall corrosion protection, supplemented by the newer materials evaluated in this severe tests program, provide designers with opportunities to produce highly corrosion-resistant bridges using pretensioning and post-tensioning methods".

In summary, the use of prestressed concrete structural alternatives, both the precast, pretensioned, and the post-tensioned variety, offer significant possibilities for achieving corrosion resistant construction when implemented properly. Of course, the inherent corrosion resistance of prestressed concrete will be further enhanced when used in conjunction with the other corrosion resistive systems discussed in this manual.

G. Jointless Bridges

Experience has shown that elimination of joints in bridge decks in corrosive environments provides great benefits in improved elements. While thermal stresses may require joints in long major structures, many structures would benefit from use of far fewer deck joints and appropriate structural design for any related increases in thermal stresses. The number of structural problems due to thermal stresses is minuscule compared to the widespread distress and great cost associated with corrosion problems caused by leaking joints. Accordingly, it seems apparent that designs with few or no deck joints should be developed, and that all deck joints should be eliminated when structurally feasible. Tennessee has had great success with jointless bridges of lengths of 400 feet for steel superstructures and 600 feet for concrete superstructures. Other States have reported similar success.

The Illinois Tollway may have been the first to use cast-in-place diaphragms to eliminate joints at piers of bridges using precast, prestressed I-Girders. The durability performance of these bridges has far surpassed the performance of bridges on the same tollway with joints at piers. The State of Tennessee has constructed many bridges with steel, precast and prestressed girders and cast-in-place decks without joints at either piers or abutments. Some of these bridges are quite long and there has been no reported thermal distress. Design technology has been developed for use of continuous concrete decks on simple-span, precast, prestressed I-Girders. Likewise, continuous bridges, which are generally without joints for several spans, could benefit from elimination of still more joints in aggressive environments.

H. Bridge Joints and Drainage Details

Although jointless steel and concrete bridges provide excellent corrosion protection for superstructure (other than the deck) and substructure elements in areas where deicing salts are used, there will still be many bridge joints used. To provide an adequate corrosion protection strategy, the Engineer must accept the fact that there is no such thing as a waterproof joint. Degradation over time, as well as lack of maintenance, has shown that the best designed joints will allow water to penetrate. Bridges should be detailed to minimize the effect of this leakage.

In addition to the concrete quality discussed earlier, and even if additional protection strategies are incorporated into the structure, the following details should be included whenever possible:

1. Slope abutment and pier caps a minimum of 1:12 to ensure drainage of the caps.
2. Place bearings on pedestals at least 1" high to protect bearing components.
3. Prevent leakage water from contacting ends of girders.
4. Provide troughs under joints with the maximum slope possible, but no less than 1:12, to divert deck drainage water, and insure "self cleaning" velocities.
5. When open joints (finger joints) are used, reduce deck scuppers to an absolute minimum, utilizing the deck joint as a drain. This increases water flow through the joint and promotes self cleaning.
6. Provide easily accessible openings in the joint at the deck level to allow periodic flushing of troughs with hoses.
7. Where troughs are used, ensure detailing will allow easy replacement from below the deck to minimize traffic disruptions during maintenance.

8. When scuppers are used for bridge deck drainage, design and detailing should be in accordance with Federal Highway Administration (FHWA) Report RD-87/014, "Bridge Deck Drainage Guidelines." Poorly designed scuppers, and using too many scuppers, causes them to rapidly become inoperative and contribute to bridge component deterioration due to the drainage coming into contact with unprotected members. Generally, too many scuppers are used. Figure 2 of the FHWA report is reproduced herein as Figure 2-3 as a primary guide in determining scupper spacing.
9. Because no bridge joint is waterproof, additional protection for abutments and piers under expansion joints can be provided by utilization of a surface sealer. (See Chapter 8)

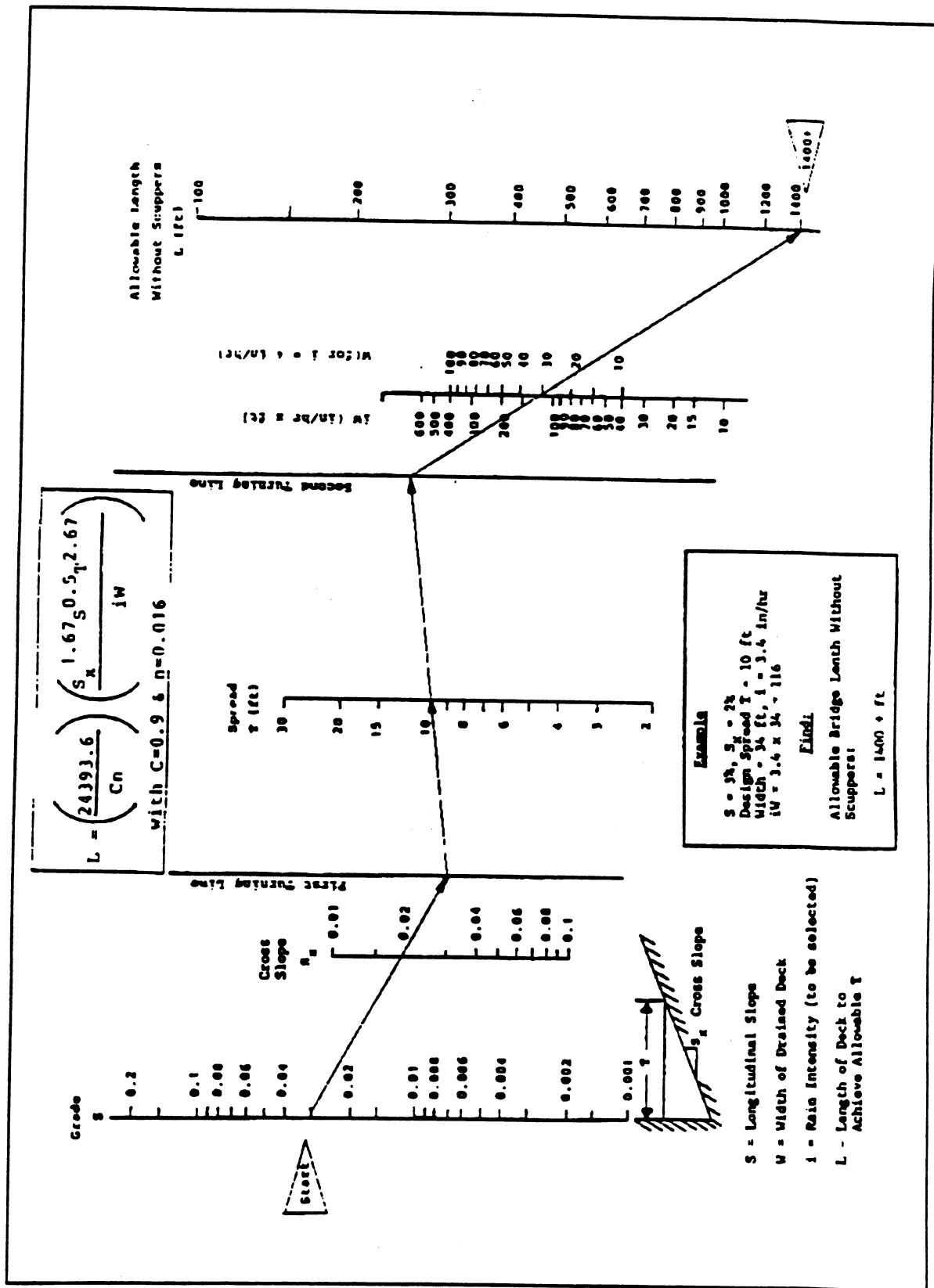


FIGURE 2-3 SCUPPER SPACING

CHAPTER 3

Steel Coatings

(Reinforcing Bars, Welded Wire Fabric, Prestress Strand)

A. Introduction

One commonly used means to protect steel in concrete against corrosion is to coat it with material that will isolate it from oxygen, moisture and corrosive chloride ions. Numerous non-metallic and metallic coatings for steel reinforcement have been evaluated.

The literature accurately documents the results of the extensive testing conducted during the 1970's. Of all the materials tested, the non-metallic fusion bonded epoxy and the metallic galvanized coatings have received acceptance as a corrosion protection mechanism.

Fusion bonded coatings have become the most widely used alternate, particularly on mild reinforcing steel. Recognizing the need for protection of welded wire fabric mesh and prestressing strand, industry developed procedures to apply this coating to these products as well. Standard specifications for coated mesh and prestressing strand have recently been developed and are now available for use by specifiers.

Fusion bonded epoxy coating has been used for prestressing strand on a limited experimental basis. Corrosion testing has shown that epoxy coated strand is highly corrosion resistant (See NCHRP Report 313 Corrosion Protection of Prestressing Systems in Concrete Bridges); however, questions still persist regarding the structural effects of coating the strand. Additional research is needed to fully determine whether epoxy coating adversely influences the creep, fatigue and/or fire resistance of the finished product. Epoxy coated strands, with coating thicknesses of 25 to 45 mils, are available in two types. For post-tensioned applications and stay cables for cable-stayed bridges, the epoxy coating is smooth. For pretensioned applications, the epoxy coating has a grit impregnated into it to improve bond characteristics. Recent tests at the University of Wisconsin-Milwaukee and University of Florida at Gainesville indicate that epoxies used in coating strands softens at temperatures of about 160 to 170 °F. Therefore, in pretensioned applications there is potential for loss of prestress at such temperatures.

B. Fusion Bonded Epoxy Coatings

1. Description

Fusion-bonded epoxy coating is a material which achieves its adhesion to the steel substrate through a heat catalyzed reaction. It is a thermo-setting material, as opposed to being thermo-plastic, meaning that its physical properties are stable over a wide range of temperature. Epoxy coating is an organic material comprised of epoxy resins, appropriate curing agents, various fillers, pigments and flow-control agents.

2. Material Specifications

a. General

Epoxy coating protects the steel in two distinct ways. First, it physically separates the steel from the harmful reactants which cause corrosion (i.e., water, oxygen and ions such as chloride). The coating is in fact a molecular filter. Water molecules, being extremely small, will penetrate through an organic coating. This penetration is dependent upon the moisture vapor transfer rate of the coating. The moisture will generally come to an equilibrium when there are as many water molecules passing into the material as are leaving. On the other hand, epoxy coating is highly resistant to penetration by chlorides, sulfides or similar ions.

The second way the coating protects is by blocking the electron flow in a corrosion cell through its high electrical resistance - functioning similarly to the insulation on an electrical wire. Dielectric strength is a key coating property. If electrons cannot travel to the cathode, corrosion is not possible. Typical resistance values for epoxy coating used on reinforcing steel can range from 200,000 to 800,000 ohms.

b. Surface Preparation

Adhesion is the key to performance of any protective coating (1). Most premature coating failures can be directly attributed to a lack of coating adhesion. Epoxy coating achieves its adhesion through a combination of physical and chemical forces which interact at the interface of the coating and steel substrate.

Proper surface preparation is essential in order to achieve maximum adhesion.

(1) Wire Mesh and Reinforcing Steel

Abrasive blast cleaning is used for mesh and reinforcing steel. Bars must be cleaned to a "near white" condition (ASTM D2200 and the pictorial standards of SSPC-VIS 1, degree of cleaning SSPC-SP107) meaning that 95% of the surface is free of all visible traces of rust, mill scale and other contaminants.

It is extremely important that the abrasive mix be kept free of chloride contamination. Any chlorides deposited on the steel surface during the cleaning operation could potentially react with the steel and initiate corrosion underneath the coating. In addition, any water soluble soil deposited between the steel substrate and the coating will have a tendency to draw water through the coating as a result of osmosis. This might cause the coating to blister and disbond.

Abrasive blast cleaning not only cleans the surface but produces a roughened texture to which the coating can bond. The roughness, or anchor profile, is a function of the peak to valley depth, the number of peaks and the shape of the peaks, all of which are determined by the selection of abrasive. A target profile of approximately 1.5 to 4.0 mils is considered suitable as an anchor pattern. Generally it has been found that the use of an all grit, or a grit-shot mixture of abrasive will yield the optimum anchor profile for epoxy coatings. Use of 100% shot abrasive mix is not recommended because it can tend to result in a more rounded profile with less exposed surface area of steel.

The blast cleaning operation should not leave residue on the steel that will interfere with the adhesion. Recent studies (2, 3, 4) have shown that some coating operations produce epoxy-coated bars which have a high level of contamination between the coating and steel interface (backside contamination). Extensive backside contamination will certainly reduce the adhesion of the coating and its effectiveness against corrosion.

(2) Prestressing Strand

Because of the concern for introduction of small "notches" in prestress strand if blast cleaning is used, and the inability of the grit to adequately clean the area between individual wires, chemical cleaning is generally used for prestressing strand. The requirements for cleanliness of strand are specified only to the extent that the finished product can meet the required tests.

c. Curing

(1) Wire Mesh and Reinforcing Steel

After the steel has been blast cleaned, it is normally heated to a temperature specified by the powder manufacturer (usually in the range of 450-475 degrees F), although it can be applied to a cold bar. The steel must be at the correct temperature, either before or after coating in order to achieve proper cure of the coating. If the temperature is too low, an incomplete reaction of the powder may result leading to lack of flow, long gel time and poor performance. Overheating is also undesirable. It can result in poor adhesion, bendability, excessive foaming and blistering.

(2) Prestressing Strand

When the dry epoxy powder comes in contact with the hot steel surface it melts and flows into the anchor profile. Within a few seconds, the liquid powder gels and changes to a solid and continues to cure.

d. Powder Application

The epoxy powder is mixed with moving air to transport it from storage bins to application equipment. Typically, powder is applied in a cabinet where the powder is aerated by an air stream pumped through thousands of tiny holes in the bottom of the cabinet. This keeps the powder flowing evenly and distributed equally.

The powder is discharged from a number of spray nozzles (guns). To keep it from blowing past the bars, the powder is given a small static electrical charge which attracts the particles to the oppositely charged steel surface.

After coating, the heated steel needs to be quenched. Quenching is simply a method of cooling the coated steel to a temperature at which it can be handled. Quenching is usually accomplished by passing the hot coated bars through a water spray or stream.

e. Grit Impregnation

When specified, inert particles (grit) shall be impregnated into the surface of the coating to insure proper transfer of stress. The grit particles shall be inert in concrete and non-reactive with concrete additives and soluble salts. The addition of grit greatly improves the bond between the concrete and the strand, thus reducing the required transfer length.

Grit has also been impregnated into the surface of coatings on reinforcing steel on a limited experimental basis. Although grit-impregnated epoxy coated reinforcing steel has not been used in actual structural applications, it does appear to provide additional bond capacity.

f. Holiday Testing

Low voltage holiday detectors are used to detect discontinuities and imperfections in the coating. A holiday detector functions by passing a low voltage DC current into a wet sponge which in turn is passed over the coating surface. As the sponge contacts a pinhole, electrical contact is made with the steel and an audible alarm is sounded or a visible marker is triggered so that it is possible to locate the imperfection.

A holiday is normally defined as a pinhole, discontinuity or other coating imperfection not visible to the naked eye. Under most specifications, abrasions, cuts and other damage incurred by handling during shipment and placement are not considered to be holidays.

Most holiday detection is accomplished with in-line detectors which operate continuously as part of the production line. Auxiliary hand-held units are often used to randomly check the in-line results. Holiday testing is normally considered to be a quality control and inspection technique intended for the application process in the plant; it is not generally intended for use in assessing damage to the coating in the field. Limitations on the number of holidays per foot of bar generally apply only to the plant situation in most common standard specifications.

For reinforcing steel, the limitation is 2 holidays per linear foot. For wire mesh, the limitation is 2 holidays per linear foot of wire. In addition, there shall not be more than an average of 4 holidays per linear foot of wire in the specimen of welded wire fabric being checked when the wire spacings are 4" or more. There shall be not more than 6 holidays per linear foot of welded wire fabric with spacings less than 4". For prestress strand the limitation is 2 holidays per hundred feet.

g. Film Thickness

(1) General

Coating thickness is another important performance parameter. The thicker the coating, the less likely it is that discontinuities and holidays will occur. In general, a thicker coating has a higher dielectric strength, is more resistant to passage of ions and water, and provides a higher degree of corrosion protection.

Film thickness is measured by using magnetic gages. These gages use a type of spring balance to pull off a small permanent magnetic from the surface of the

coated steel. They are based on the principle that the magnetic force varies inversely to the coating thickness.

(2) Reinforcing Steel

The maximum coating thickness is limited by structural considerations such as creep, fatigue and bond/development of the reinforcement. Research has shown that coating thicknesses within standard specification limits (i.e., 90% of the readings between 5 and 12 mils) have fatigue and creep performance equivalent to uncoated reinforcing bars.

Recently it has been proposed that the minimum for coating thickness be increased from 5 to 7 mils. This revision is currently under consideration by AASHTO, ASTM, NACE and others, and is expected to be incorporated into the future versions of each group's respective specifications for epoxy coated reinforcing steel.

Recent bond studies have shown that the basic development lengths for epoxy-coated reinforcing bars must be increased 15-50% to account for the reduced friction of the coating. AASHTO 1989 Standard Specification for Highway Bridges prescribes modification factors for the basic development length of epoxy-coated rebars.

(3) Wire Mesh

The required film thickness of the coating is dependent upon the class of coating specified. For a Class A coating (plain and deformed steel wire and welded wire fabric used for concrete reinforcement), the film thickness shall be 5-10 mils. For a Class B coating (plain and deformed steel wire and welded wire fabric used for reinforcement in earth reinforced construction) the coating shall be a minimum of 18 mils.

(4) Prestressing Strand

The film thickness for prestress strand is 25 to 45 mils. Epoxy-coated prestressing strand with imbedded grit has been shown to have a bond to concrete which is superior to uncoated prestressing strand.

h. Bend Test

(1) General

The bend test is one of the most important acceptance tests. Properly conducted, the bend test can reveal deficiencies in surface preparation, cleanliness, curing, and powder quality.

(2) Reinforcing Steel and Wire Mesh

Typically, a coated bar or welded wire fabric is bent 120 degrees to 180 degrees around a specified pin diameter. If cracking and disbondment of the coating occurs, it is indicative of a problem in the application process.

(3) Prestress Strand

Epoxy-coated prestressing strand is evaluated by bending it around a mandrel equal to 32 times the nominal diameter of the strand.

i. Fabrication

Reinforcing bars may be fabricated (bent) before or after coating is applied. If fabrication is done after coating, all cracks in the coating shall be properly repaired prior to installing the fabricated bar in the forms.

j. Specifications

(1) Reinforcing Bars

ASTM D3963 Epoxy-Coated Reinforcing Steel
ASTM A775 Epoxy-Coated Reinforcing Steel Bars
AASHTO M284 Epoxy-Coated Reinforcing Steel

(2) Welded Wire Mesh

ASTM A884 Epoxy-Coated Steel Wire and Welded Wire Fabric for Reinforcement

(3) Prestressing Strand

ASTM A882/A882M Epoxy Coated Seven-Wire Prestressing Steel Strand. Epoxy coated prestress strand is available from:
Florida Wire and Cable Company
825 North Lane Ave. P.O. Box 6835
Jacksonville, Florida 32205

(4) Epoxy Coatings

The coatings approved for use shall be prequalified in accordance with Annex A1 of AASHTO M284 or ASTM A882, as appropriate.

k. Prequalification Certification

It is highly recommended that all suppliers of epoxy-coated steels be prequalified to assure that the company has met at least minimum levels of quality control and is able to produce the coated product in accordance with the specifications. The Concrete Reinforcing Steel Institute (CRSI) has a voluntary program for epoxy coating companies which provides an independent assessment of a supplier's quality control procedures. The CRSI certification provisions, or equal, should be minimum requirements for the project specifications. The CRSI program is not intended to replace acceptance testing by the owner, but rather to complement the owner's efforts.

3. Criteria

Epoxy-coated reinforcement and wire mesh is recommended for any structural application where corrosion is a possibility. Since research on coated prestressing steel is still underway, and the effects on creep and temperature resistance still need to be determined, this cannot be fully endorsed at this time. It should be noted that Florida has experienced problems with corrosion of epoxy coated reinforcing steel when used in the marine environment of South Florida. Research to determine the cause of these problems is underway. However, as an interim measure, Florida has limited use of epoxy coated reinforcing steel to bridge decks.

The use of epoxy-coated reinforcement is applicable to new reinforced and prestressed concrete construction, and rehabilitation situations where concrete members are reconstructed.

4. Construction Specifications

Although epoxy coating is a fairly durable, abrasion resistant material, it is not indestructible. Improper handling and storage can damage a coating and expose the underlying steel, negating the coating's protective benefits. Precautions must be taken to avoid excess damage to the coating. For the most part, these precautions amount to simple common sense measures.

Epoxy-coated steel is typically trucked or shipped by rail to the project. Care must be exercised to minimize scraping the bundles. Bundles of steel must be lifted in a way to avoid excess sag which might cause bar to bar abrasion. Nylon slings or padded wire ropes should be used. In preparing the bars or mesh for shipment, bundles shall be tied with straps padded or made of a material that will not cut into the coating.

Epoxy-coated steels should be stored on timbers or other suitable protective cribbing. The steels should be stored as close as possible to the area where it will be placed in a structure to keep handling operations to a minimum. Coated bars, mesh or strand should not be dropped or dragged. Long-term storage (>6 months) of epoxy-coated reinforcing bars at the job site should be avoided if at all possible. Deliveries should be scheduled and coordinated with placing in the structure to avoid any need for long-term storage. If circumstances or other conditions make it necessary to store coated steel outdoors for an extended period of time, the bars must be protected from the direct rays of sunlight and sheltered from the weather. It is recommended that the steel be covered with an opaque polyethylene sheeting or other suitable protective material. For stacked bundles, drape the protective covering over the sides of the bundles and around the perimeter of the stack. Provisions should be made to provide for adequate air circulation around the bars to prevent condensation under the covering.

In general, placing of epoxy-coated steel is done similarly as for uncoated steel. The key exception is that coated steel requires more careful handling and placement. Once bundles of coated bars have been broken, dragging one bar over another or any other abrasive surface should be avoided. Bars or wire fabric should be placed on epoxy coated bar supports and tied with wire coated with a non-conductive material.

After the coated reinforcement is in place, walking on the bars shall be held to a minimum. Workman should be careful not to drop large hand-held tools or other heavy construction materials on the bars in place. Care should be exercised so that concrete conveying and placement equipment does not damage the coating.

When a damaged coating must be repaired, the patching or touch-up material should be applied in strict accordance with the instructions furnished by the manufacturer. Generally, surface preparation consists of a thorough manual cleaning of damaged spots and complete removal of rust. This is usually done with a wire brush and emery paper. Care should be exercised so that the bare areas are not made larger than necessary to accomplish the repair of the damaged spot.

Following these simple precautions should greatly minimize field damage. Although the standard specifications permit as much as 2% of the surface area of reinforcing steel to be damaged and exposed, with proper care much less damage should actually occur. Surveys of actual field installations of epoxy-coated bars show that field damage should normally be less than 0.05% of the surface area when these recommended practices are followed.

It has been recommended that the standard specifications for epoxy coated reinforcing steels of AASHTO, ASTM and NACE be revised to require that all visible damage to the coating be properly repaired prior to concrete placement.

These proposals have widespread support and are expected to be adopted in the near future.

"Bite-through" wedges or other devices which are specifically designed and manufactured for use with epoxy-coated strand must be used to provide proper tension in the strand without slippage. The preferred method of cutting tensioned epoxy-coated strand is with an abrasive saw. Gas cutting may soften the coating and cause slippage.

5. Maintenance Requirements

Normally no maintenance is required on epoxy-coated reinforcement during the service life of a structure.

6. References/ Other Information

Munger, C.G., Abstract - Surfaces, Adhesion, Coatings, The International Corrosion Forum, Anaheim, California, April 18-22, 1983.

Bell Evaluation Laboratory - Effectiveness of Epoxy-Coated Reinforcing Steel, Project No. L621/1449, July 6, 1990.

Sohanghpurwala, A.A. and Clear, K.C.; Effectiveness of Epoxy-Coated Reinforcing Steel, Ontario Report - MAT-89-02.

Scannell, W.T. and Clear, K.C.; Long-Term Outdoor Exposure Evaluation of Concrete Slabs Containing Coated and Uncoated Reinforcing Steel, Transportation Research Board, January, 1990.

Romano, David C.; Preliminary Investigation of Epoxy-Coated Reinforcing Steel Disbondment: Cause and Effects, Florida Department of Transportation, November, 1988.

Clear, K.C. and Virmani, Y.P.; Corrosion of Non-Specification Epoxy-Coated Rebars in Salty Concrete, Public Roads Magazine, June, 1983.

Hededahl, P. and Manning, D.G.; Field Investigation of Epoxy-Coated Reinforcing Steel, Ontario Report - MAT-89-02.

Guidelines for Inspection and Acceptance of Epoxy-Coated Reinforcing Bars at the Job Site, Concrete Reinforcing Steel Institute, 1987

Clear, K.C. and Bell Evaluation Laboratory; Fundamentals Related to Epoxy-Coated Reinforcing Steel, First Report, December, 1989.

Munger, C.G.; Corrosion Prevention by Protective Coatings, National Association of Corrosion Engineers, 1984.

Read, J.A., WS Atkins Consultants Ltd.; FBECR The Need for Correct Specification and Quality Control, Concrete Magazine, September 1989 Volume 23 Number 8.

Saint, E.R., Guidelines for Inspection, Patching and Acceptance of Epoxy-Coated Reinforcing Bars, EM-69, Ontario Ministry of Transportation and Communications, 1983.

Source of Epoxy-Coated Prestress Strand; Florida Wire and Cable Company, 825 North Lane Avenue, P.O. Box 6835, Jacksonville, Florida 32205.

C. Galvanized Rebar Barrier Coating System

1. Description

Hot dip galvanizing is a factory-controlled process of applying a zinc coating to steel by immersing the material in a molten zinc bath, resulting in a metallurgical combination of zinc and steel.

Galvanizing protects the base steel in two distinct ways: barrier and cathodic protection. It acts by isolating the metal from the electrolytes in the environment by providing adhesion to the base metal. Galvanizing provides cathodic protection by changing an element of the corrosion circuit, introducing a new element, and insuring that the base metal becomes the cathodic element of the circuit.

The galvanizing process is a three-step procedure consisting of caustic cleaning, acid pickling and fluxing, and galvanizing. First, the steel is immersed in a hot alkali solution to remove organic contaminants like dirt, paint markings, grease and oil from the metal surface. Second, the steel is chemically cleaned by acid pickling which removes scale and rust from the steel surface. Pickling is either in a dilute solution of hot sulfuric acid or ambient temperature hydrochloric acid. After the metal is removed from the pickling tank, it is rinsed thoroughly to remove any residual acid. Fluxing is the final surface preparation step in the galvanizing process. Fluxing promotes bonding of the zinc to the steel. Next, the steel is dipped in a galvanizing kettle which is filled with molten zinc, usually at a temperature of 830°F to 860°F (443°C to 460°C). The steel rebar remains in the molten zinc until it assumes the temperature of the bath. During this time, a complex structure of zinc and iron alloys are coated on the base metal. After the steel is removed from the galvanizing tank, it is either quenched with water or allowed to cool in air.

Essentially, once the metal is withdrawn from the kettle, the galvanizing process is complete. However, some specifications, including those of the Pennsylvania DOT, require the galvanized rebar to be chromate treated. Chromate treatment passivates the galvanized surface so no gas is released while the wet concrete is around the steel. The chromate treatment is usually done during the quenching operation by putting chromatic acid salts in the quench tank.

Construction practices, techniques, and equipment are essentially the same for galvanized rebar as with black steel. The only change in addition to the galvanizing, is that the wire ties and chairs are to be galvanized. This is done to prevent black steel from coming in contact with galvanized steel. If this occurs, a corrosion cell would be created at these contact points and the zinc coating would sacrifice itself for the protection of the black steel and, thus, exhaust the supply of zinc at a faster rate.

Concrete is placed using conventional placement techniques and materials.

Surface preparation is the most important step in the application of any coating. With galvanizing, the surface preparation process contains its own built-in means of quality assurance and quality control in that zinc will simply not react with a steel surface that is not perfectly clean.

The most extensive known user of galvanized reinforcing steel as a corrosion protection strategy in bridge structures, specifically decks, is the State of Pennsylvania. Results of their experiences are documented in Malasheskie's report listed in the reference section.

2. Material Specifications

ASTM A767- Standard Specification for Zinc Coated (Galvanized) Steel Bars for Concrete Reinforcement

ASTM A123- Standard Specification for Zinc (Hot-Dip Galvanized) Coatings on Iron and Steel Products.

AASHTO M111 - Zinc (Hot-Dip Galvanized) Coatings on Iron and Steel Products

The coating thickness shall provide a minimum weight of zinc coating of 2 oz. per ft².

3. Criteria

This method can be used for new construction or replacement of portions of an existing construction. To prevent galvanic action between coated and uncoated steels, all reinforcing in the components should be galvanized.

4. Construction Specifications

Galvanized rebar should not be in contact or proximity with bare rebar or steel. If bare rebar is present in the same structural unit, isolation techniques must be used.

Normally, the same construction practices used for uncoated reinforcement can be used for galvanized rebars as well.

Reinforcing bars may be fabricated before or after galvanizing is applied.

5. Maintenance Requirements



No maintenance should typically be required during the normal service life of a structure.

6. References/ Other Information

Malasheskie, G.J.; et al Bridge Deck Protective Systems Research Project FHWA-PA-88001+85-17, Commonwealth of Pennsylvania, Department of Transportation, July, 1988.

International Lead Zinc Research Organization, Galvanized Reinforcement for Concrete II, 1981.

American Galvanizers Association, 315 South Patrick Street, Suite 302, Alexandria, Virginia 22314

Portland Cement Association, Evaluation of the Performance of Galvanized Reinforcement in Concrete Bridge Decks, May 1982.

AASHTO M232 (ASTM A153) Standard Specification for Zinc Coating (Hot Dip) on Iron and Steel Hardware

CHAPTER 4

Admixtures

A. Introduction

Each of the admixtures in the material class under consideration here has certain advantages and limitations, and it is important that these be understood and taken into account if full benefit is to be realized and problems which could otherwise reduce service life avoided. Different admixture types (high range water reducers, Microsilica, Fly Ash, corrosion inhibitors, etc.) may function by different processes; and so each may be unique with regard to mix design, fabrication and curing. Thus, care must be exercised to assure, first, that different admixtures proposed to be used in the concrete mix are mutually compatible, and, second, that the manufacturer's recommendations are followed. As is indicated in Chapter 2, a trial batch well in advance of actual project placements is necessary to ensure problems will not exist. For example, specific requirements exist with regard to water evaporation from plastic Microsilica concrete; and if controls are inadequate, surface cracking may occur in a finished slab. Also, one type of calcium nitrite admixture exhibits accelerating set properties which, if not accounted for, may result in an unduly stiff mix and poor compaction or honeycombing. Further, it is important to recognize the distinction between, for example, a low permeability material and a low permeability structural component. A material with low permeability may be ineffective for protecting embedded steel in a structural component if it contains cracks or voids large enough to facilitate rapid chloride, water and oxygen ingress. At the same time, the maximum period for which aggressive exposure data for these products are available is approximately 15 years, which is well below the life existing structures have survived. Coupled with the known uncertainties of life projection based upon short term or accelerated tests data, it becomes clear that a definitive, quantitative evaluation procedure which has been confirmed by real-time service data is not available. This must be balanced, however, with the realization that new promising products have evolved and will continue to evolve; and bridge owners and engineers must make every effort to take advantage of these. The types of admixtures addressed herein have been evaluated by laboratories or by in-service performance.

In some cases, the admixtures included have specific requirements that must be met, such as AASHTO M 194, Chemical Admixtures for Concrete. In other cases the admixtures do not fit into the types listed in M 194. To insure the finished product meets the project requirements, the use of the trial batch as discussed in Chapter 2 becomes extremely important. In the absence of a specific material specification, it is suggested the M 194, Table 1, limitations be used as a guide when evaluating the affects of the admixture.

B. High-Range Water Reducers (HRWR)

1. Description

These admixtures are powerful dispersing agents, and their action in portland cement pastes, mortars and concretes can be attributed to this action. Commercially marketed HRWR's (also known as Superplasticizers) may be formulated by combining the dispersing agents with water-reducing admixtures containing lignin, or ones containing modifications of hydroxylated carboxylic acids (HC type), or carbohydrates and polysaccharides (PS type). These formulations may import retardation or slightly longer setting time to the cementitious mixture.

At dosages less than the manufacturer's recommended rates, the HRWR reduces water demand to the same extent as more conventional admixtures. However, since these admixtures are not inherently retarders, they may be used at much higher dosages. Water reductions of 20 to 30 percent are not uncommon. When used without reducing water content, concrete with very high workability, often termed flowing concrete, or fluid concrete, can be produced with a relatively low water/cement(w/c) ratio. (See ASTM C1017 Specifications for Chemical Admixtures for Use in Producing Flowing Concrete.) It is important to note that formwork may require special design considerations when using flowing concretes because of increased fluid pressures.

When used as a water reducer, the unblended naphthalene formaldehyde and melamine formaldehyde high-range water reducers will have little affect on time of setting as measured by ASTM C 403. When a blended HRWR (Type G) is used with conventional admixtures, concrete may exhibit retardation of one to three and a half hours at ambient temperatures in the 60° to 80°F range.

Since HRWR may be used to manufacture workable concrete at very low w/c, very high strength concrete can be manufactured in the field. Compressive strengths greater than 14,000 psi at 28 days have been attained.

2. Material Specifications

Chemical Admixtures for Concrete AASHTO M194 (ASTM C494) Type F, water reducing, high range, admixtures and Type G, water reducing, high range and retarding admixtures.

3. Criteria

The use of HRWR as the sole means of corrosion protection is appropriate only in areas of a low corrosive environment. As indicated above, 20 to 30 percent

reduction in water content is possible through use of HRWR. Research has clearly indicated that reduced permeability, and therefore less chloride penetration occurs with lower water-cement ratios (see Figure 2-1). Since HRWR produces a more workable mix at low w/c, it is often times used primarily as a construction expedient when low permeability concrete is desired. Primary corrosion protection is normally provided by other strategies such as epoxy coated reinforcing (chapter 3).

4. Construction Specifications

Much research has been conducted on the properties of superplasticized concrete. The University of Texas, under FHWA/Texas Highway Department HPR contract No. 3-5-87-1117, summarizes critical requirements in their Research Report 1117-2, "Guidelines for the Proper Use of Superplasticizers in Concrete".

Section 7.3 of these guidelines are reproduced below for the user's benefit. The entire report should be reviewed if extensive use of HRWR's are proposed.

Section 7.3 Guidelines for the Proper Use of Superplasticizers

This section consists of guidelines for the proper use of superplasticizers when producing flowing concrete in hot weather. These guidelines will be of particular interest to field personnel and resident engineers who are involved in projects where the use of the admixture is being contemplated or has been approved. Once testing is complete for both hot weather and cold weather conditions, a complete and detailed set of guidelines would be published as a separate report. At this time, this report contains test results to date as these affect or influence the proper use of superplasticizers in the field.

- (1) Choose material suppliers who can provide cement, aggregates, and admixtures with fairly consistent physical and chemical properties.
- (2) Evaluate in-place strength and durability requirements for the structural concrete. In particular, what amount, if any, of air entrainment is required?
- (3) Evaluate job site location with respect to ready mix facilities for estimation of a reasonable transit time to establish a time of addition for the superplasticizer to the fresh concrete mix.
- (4) This experimental program has shown that, unless the superplasticizer can be added 30 minutes after initial water:cement contact, some retarding admixture is needed.
- (5) If it is necessary for the concrete to be air-entrained, it has been shown that a naphthalene based superplasticizer can significantly reduce the effectiveness of vinsol resin and specially developed air-entraining agents. Therefore it is most likely necessary that the mix be proportioned for an air content of 1 to 4 percent higher than that recommended by the specifications.
- (6) Establish mix proportions for production of ready-mix concrete with an initial slump of 1 to 4 inches when the time of addition is approximately 60 minutes; if the time of addition is delayed to approximately 90 minutes, an initial slump of 4 to 6 inches is more appropriate. In either case, the slump of the concrete should not fall below approximately one inch before the superplasticizer is added.
- (7) Execute at least one full trial batch using actual materials to determine material compatibility and the adequacy of air-entrainment after the initial dosage to flowing concrete at the estimated time of addition. Conduct intermittent tests for air content before and after the addition of superplasticizer up until the time when placement is estimated to have been completed.

- (8) In addition to tests for air content, it is desirable to conduct intermittent slump tests, to monitor the fresh concrete temperature, and to cast cylinders and/or flexural beams. This will allow determination of a reasonable working time, detection of a temperature rise in the concrete, and a method to ensure that minimum strength specifications are met.
- (9) Evaluate the slump test results and limit the ready-mix truck load to a volume that may be placed and consolidated within the time frame for continuing acceptable workability.
- (10) If it is not necessary for the concrete to be air-entrained, a second superplasticizer dosage would not be detrimental to the properties of the concrete mix except for setting times. This, however, can extend finishing to significantly later times. This would not be a consideration for members with formed finishes, such as columns.
- (11) The study of different retarder dosages in this research program has shown that while a larger retarder dosage cannot delay slump loss, it can decrease the amount of superplasticizer required to produce flowing concrete. Larger retarder dosages will delay setting time further.
- (12) The concrete should be mixed thoroughly, at least five minutes, after the addition of superplasticizer. Placement should begin immediately.
- (13) No changes are necessary in standard recommendations for adequate consolidation and curing.

5. Field Testing

Prior to beginning field placements, a trial batch (see Chapter 2, Section D, Concrete Quality) using the proposed construction materials and approved mix design shall be prepared by the Contractor prior to placement. The manufacturer's recommendations should be used when adding a High-Range Water Reducer. The HRWR trial batch concrete is required to meet a maximum permeability requirement based on the expected chloride exposure conditions as noted in Chapter 2. If the test indicates a permeability greater than desired, a new trial batch, using a revised mix design shall be made and tested.

6. Maintenance

No specific maintenance requirements are necessary to retain corrosion protection attained by use of HRWR's.

7. References/Other Information

Evaluation of Super-Water Reducers for Highway Applications, FHWA/RD-80/132, March, 1981.

Phase one Interim Report Superplasticized Concretes for Rehabilitation of Bridge Decks and Highways Pavements, DOT/RSPA/DPB-50/81/3 and Supplement thereto OT/RSPA DMA-50/84/3.

Guidelines for the Proper Use of Superplasticizers in Concrete, FHWA/Texas HPR report FHWA/TX-89/1117-2.

Effects of High-Range Water Reducers on the Properties of Fresh and Hardened Concrete, FHWA/Texas HPR report 1117-3F.

Schutz, R. J., Durability of Superplasticized Concrete, Proceedings, Symposium on Superplasticizers in Concrete, Ottawa, May 1978.

Whiting, David, Effect of High-Range Water Reducers on Some Properties of Fresh and Hardened Concrete, R & D Bulletin No. RD-61-01T, Portland Cement Association, Skokie, Illinois, 1979.

C. Fly Ash/Pozzolan

1. Description

When used in concrete, Fly Ash generally contributes to strength gain and improves durability of the concrete. The Fly Ash may be added either as a replacement for a portion of the portland cement or as an addition to the usual amount of cement.

Strength of Fly Ash concrete and the rate of strength gain are affected by the characteristics and the amounts of the particular Fly Ash, and the cement with which it is used. Also, curing periods longer than those required for concretes without Fly Ash may be needed to obtain the full benefits of the pozzolanic properties of the Fly Ash.

Normally, all Fly Ash concretes proportioned to have 28-day compressive strengths equivalent to similar concrete without Fly Ash will have greater strengths when properly cured.

The use of Fly Ash tends to reduce the permeability of the concrete and thus retards the infiltration of aggressive liquids. Accordingly, better resistance to corrosion of embedded metals is provided as compared to non-Fly Ash concrete.

The permeability of Fly Ash concrete generally decreases with time because of the pozzolanic reactions that occur after the concrete has hardened. The amount of decrease is dependent on use of good curing practices and the temperature at which curing occurs. Laboratory tests have indicated that significant reduction in permeability of Fly Ash concrete to the chloride ion (as determined by AASHTO Test T 277) is attained by curing specimens at 100 degrees F in lieu of 73 degrees F, thus indicating the advantage of placing Fly Ash concrete in hot summer weather. Conversely, placement in cold weather may have adverse effects on the permeability.

2. Material Specifications

Fly Ash and Raw or Calcined Natural Pozzolan AASHTO M 295 (ASTM C 618) for use as a mineral admixture in concrete where cementitious or pozzolanic action, or both, is desired or, where properties normally attributed to finely divided mineral admixtures may be desired. The Fly Ash content is generally specified by the purchaser and is generally between 15 to 20% of the weight of cement. Blended cements are governed by ASTM Specification C 595-89 Standard Specification for Blended Hydraulic Cements.

3. Criteria

When used in sufficient quantity, Fly Ash is capable of preventing excessive expansion resulting from a alkali-silica reaction. As a general rule, Class F Fly Ash will improve the sulfate resistance of mixtures in which it is included. However, Class C Fly Ashes containing a high ratio of calcium to iron oxides or those with large amounts of chemically active alumina may lower sulfate resistance. Generally larger amounts of Class C over Class F fly ash is required for equivalent protection,

4. Construction Specifications

To obtain corrosion protection with Fly Ash concrete, the water/cement ratio must be limited to not more than 0.44. Depending upon the criteria, it can be used as a substitute for cement or as additional cementitious material.

5. Field Testing

Prior to beginning field placements, a trial batch (see Chapter 2, Section D, Concrete Quality) using the proposed construction materials and approved mix design shall be prepared by the Contractor prior to placement. The trial batch concrete is required to meet a maximum permeability requirement based on the expected chloride exposure conditions as noted in Chapter 2. If the test indicates a permeability greater than desired, a new trial batch using a revised mix design shall be made and tested.

Field tests are considered important elements in obtaining quality concrete. The tests required are those that are normally employed for all portland cement concretes. Qualified personnel are required for all testing operations, whether performed by the Engineer's representative or the Contractor. These tests are described in the PCA document "Design and Control of Concrete Mixtures".

6. Maintenance

No specific maintenance requirements are required to retain corrosion protection attained by use of Fly Ash.

7. References/Other Information

ACI 226.3R, Use of Fly Ash in Concrete, American Concrete Institute, Detroit, Michigan, 1987.

Halstead, W. J., Use of Fly Ash in Concrete, NCHRP Synthesis 127, Transportation Research Board, Washington, D.C., 1986.

Ozyildirim, C. and Halstead, W. J., Resistance to Chloride Ion Penetration of Concretes Containing Fly Ash, Silica Fume, or Slag, ACI SP-108, Permeability of Concrete, American Concrete Institute, 1988.

Berry, E. E. and Malhotra, V. M., Fly Ash in Concrete, SP85-3, Canada, Centre for Mineral and Energy Technology, Ottawa, Canada, December 1985.

D. Microsilica (MS)

1. Description

Microsilica is the fine pozzolan obtained as a by-product of the manufacture of silicon or silicon alloys, such as ferrosilicon. Microsilica has also been referred to as "condensed silica fume" or "silica fume".

MS admixtures for concrete are furnished either in the dry compacted form or as water-based slurries that are made from dry material.

MS is gray and is darker with increasing carbon content. The product is very finely divided, with most of the particles being in the 0.1 to 0.2 micrometer range, which is about 1/100 of the diameter of the average sized cement particle.

Loose bulk densities of undensified MS range from 5 to 27 pounds per cubic foot (pcf). MS intended for use in concrete is generally in the middle of this range as produced. Densified MS generally has loose bulk densities ranging from 25 to 40 pcf.

The principal ingredient of MS is silica (SiO_2), which is in the non-crystalline (amorphous) state. Generally, those that are used in concrete admixtures have silica contents exceeding 85 percent.

The following States have used MS in both dry densified and slurry forms for bridge deck overlays:

California
Illinois
Kansas
Kentucky
Maine
New Hampshire
New York
Ohio
Oregon
Tennessee
Virginia
Washington
Wyoming

In addition, the slurry form of MS has been used in structural components by the following:

Texas DHPT - Braker Lane Bridge (1987)
Alabama D of H - Perdido Pass Bridge (1988)
Washington State DOT - Lake Washington Floating Bridge (1991)
Ohio DOT - Route 161 in Plain City (1987)
New York DOT - (1989)
Florida, Pinellas County - 49th Street bridge (1991)

2. Material Specification

"Microsilica for Use in Concrete and Mortar", AASHTO M307, covers Microsilica for use as a mineral admixture in Portland Cement Concrete and Mortar to fill small voids and/or where pozzolanic action is desired. (To minimize the possibility of excessive shrinkage cracking, a retarder, at varying dosages, should be considered.)

3. Criteria

Microsilica dosages for most highway applications generally range between 5 percent and 10 percent of the weight of the cement.

In many cases the microsilica is considered to be cementitious material, and it is included in the water-to-cementitious materials ratio $[w/(c + MS)]$ determination

The mix design shall keep the water-cement(w/c) ratio to the lowest limit that will provide a workable mix of uniform composition and consistency. In no case shall the mix design exceed the specified maximum w/c. In calculating the w/c, all free water shall be included. Be aware that most Microsilica slurries are composed of approximately 50 percent water by weight.

It is possible to entrain a desired amount of air in microsilica concrete using commercially available air-entraining admixtures (AE). Microsilica added to concrete increases water demand, often requiring an additional pound of water for every pound of added microsilica to maintain constant slump. Therefore, microsilica is nearly always used in conjunction with relatively high dosages of water-reducing admixtures or HRWR. Other factors being equal, addition of microsilica to concrete increases strength. Also, incorporation of microsilica into a mixture with HRWR enables the use of lower water-to-cementitious materials ratio than may have been possible otherwise. Compressive strengths between 6,000 psi and 12,000 psi have been obtained and used in bridges. It has been found for air bubble size to provide freeze-thaw protection that the traditionally acceptable values for the spacing factor (0.008 in. or less), and the specific surface, (600 in²/in³ or more) apply to microsilica concrete. However, MS concretes do not get saturated easily, and may perform satisfactorily with AE outside the normally acceptable range. Addition of microsilica to concrete reduces the water permeability by at least one order of magnitude and in many cases the permeability is so low that existing equipment is not suitable for measuring the permeability of microsilica concrete.

Lower water permeability also results in greatly increased resistance to the penetration of chloride ions, and adequate resistance for concretes used in highway applications may be attained with relatively low microsilica dosages (5 to 7.5 percent). Achieving low permeability represents one of the primary reasons why microsilica concrete is used in bridges, parking garages, and marine structures. Microsilica concrete generally shows good resistance to abrasion, which can be attributed to high strength. For this reason it has been used for pavements traveled by vehicles with steel-studded tires and for a military tank test track in Florida. Bridge overlays made with MS concrete have proven to be resistant to abrasion, and runway repairs have used specialty abrasion-resistant MS concretes. The relationship between static modulus of elasticity and compressive strength for MS concrete is roughly the same as that observed for portland cement concrete. As with other concretes, at very high strength levels the ACI equation for elastic modulus ($E_c = w_c^{1.5} 33 \sqrt{f_c}$) appears to somewhat over-estimate the observed results.

4. Construction Specifications

a. Trial Batch

Prior to beginning field placements, a trial batch (see Chapter 2, Section D, Concrete Quality) using the proposed construction material and approved mix design shall be prepared by the Contractor prior to placement. The trial batch concrete is required to meet a maximum permeability requirement based on the chloride exposure conditions as noted in Chapter 2. If the test indicates a permeability greater than desired, a new trial batch using a revised mix design shall be made and tested.

The control tests shall demonstrate that there is no adverse effect of using Microsilica.

b. Preparation for Placement

Proper mixing is important to insure the best performance of the mix, particularly with MS in the dry, densified form. Standard placement practices are generally acceptable, although at times finishing may be a little more difficult than with 'normal' concrete. MS concrete is more cohesive than normal concrete. This coupled with the low water-cement ratios typically used with this type of concrete normally dictates the use of a high-range water reducer at rates higher than normal. This may result in an extension of setting time calling for the use of continuous fogging and/or an evaporation retardant until the concrete is ready for the final finishing operation. Due to the increased cohesiveness of MS concrete, it is typically placed at a slump 1-2 inches higher than the slump of a conventional mix used in a similar placement.

As weather/climatic conditions change, MS concrete may be adjusted for setting through the use of retarding or accelerating admixtures similar to normal concrete.

If MS concrete is hand screeded it should be done with a sawing rather than a pulling motion which will greatly help to close the surface and facilitate bull floating.

c. Placement

(1) Ambient Conditions

These concretes shall be placed only when the ambient temperature is above 45 degrees Fahrenheit and is forecast to remain above 45 degrees Fahrenheit for the curing period. The placing of concrete should not begin, if at any time during the placement, and prior to application of the appropriate curing, the combination of