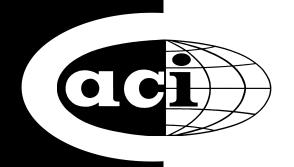# Guide to Design and Construction Practices to Mitigate Corrosion of Reinforcement in Concrete Structures

Reported by ACI Committee 222

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Guide to Design and Construction Practices to Mitigate Corrosion of Reinforcement in Concrete Structures

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Guide to Design and Construction Practices to Mitigate Corrosion of Reinforcement in Concrete Structures

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Corrosion of metals in concrete is a significant problem throughout the world. In many instances, corrosion can be avoided if proper attention is given to detailing, concrete materials and mixture proportions, and construction practices. This guide contains information on aspects of each of these. In addition, the guide contains recommendations for protecting in-service structures exposed to corrosive conditions. The guide is intended for designers, materials suppliers, contractors, and all others engaged in concrete construction.

Keywords: admixtures; aggregates; aluminum; cathodic protection; cement; chlorides; consolidation; corrosion; curing; epoxy coating; high-range water-reducing admixtures; mixing; mixture proportioning; permeability; reinforcing steel; water-cementitious material ratio.

CONTENTS

FOREWORD

This guide represents a compendium of technology to combat the problems of corrosion and is arranged into four major chapters. Chapter 2 discusses design considerations pertinent to corrosion, including environmental factors, performance of structural types, and the influence of structural details. Chapter 3 addresses the effects of concrete materials and mixture proportions on susceptibility to corrosion, including cements, aggregates, water, reinforcing steels, admixtures, and other embedded materials. Chapter 4 examines corrosion as it is influenced by the changes that concrete undergoes as it is mixed, transported, placed, consolidated, and cured. Chapter 5 describes several procedures available for protecting in-place structures.

This guide will aid in the design and construction of corrosion-resistant reinforced concrete structures and assist those involved in ensuring that reinforced concrete continues to function as a reliable and durable construction material.

CHAPTER 1—INTRODUCTION

Corrosion of metals in concrete is a serious type of deterioration that affects concrete in service. Corrosion is seen in parking structures, marine structures, industrial plants, buildings, bridges, and pavements. The Federal Highway Administration published a report in 2001 that the estimated cost of corrosion of highway bridges was between $6.43 and $10.15 billion (FWHA-RD-01-097 2001). This problem drains resources in both the public and private sectors. Implementation of solutions is needed, both in the design of structures resistant to corrosion and the rehabilitation of structures suffering the effects of corrosion.

Concrete provides a highly alkaline environment, which results in the formation of a passivating film that protects the steel from corrosion. Corrosion of embedded metals in concrete can occur, however, if concrete quality and details such as concrete cover and crack control are not adequate; if the functional requirement of the structure is not as anticipated or is not adequately addressed in the design; if the environment is not as anticipated or changes during the service life of the structure; or a combination of thereof. For more details on the mechanism of corrosion of metals in concrete, refer to ACI 222R.

Once corrosion begins, it is aggravated by factors such as moisture and elevated ambient temperatures. Cracking, stray currents, and galvanic effects can also exacerbate corrosion. Other causes of corrosion include steel directly exposed to the corrosive elements due to incomplete placement or consolidation of concrete, and industrial or wastewater chemicals that attack the concrete and the reinforcing steel. Reinforced concrete structures should be designed either to avoid these factors when they are present or be protected from these factors when they cannot be avoided.

CHAPTER 2—DESIGN CONSIDERATIONS
2.1—Structural types and corrosion

Corrosion of steel in concrete was first observed in marine structures and chemical manufacturing plants (Biczok 1964; Evans 1960; Tremper et al. 1958). The design considerations relevant to corrosion protection depend on the type of structure and its environment and intended use. Certain minimum measures—for example, adequate concrete cover and concrete quality—should always be specified, even for structures such as concrete office buildings completely enclosed in a curtain wall with no exposed structural elements. Depending on the type of structure and its expected exposure, additional design considerations can be required to ensure satisfactory performance over the intended service life of the structure.

2.1.1 *Bridges*—The primary issues in designing the deck, substructure, and superstructure of a concrete bridge for increased corrosion resistance are knowing the potential for chloride ion exposure while in service and the degree of protection required. In theory, the design considerations for a bridge located in a semi-arid region of the U.S., such as parts of Arizona, should be different from a bridge located in either Illinois or on the coast of Florida. ACI 318, ACI 345R, and AASHTO HB-17 recognize this and contain additional requirements for concrete structures exposed to different levels of chloride ions in service.

There are differences in interpretation when applying these provisions for corrosion protection of bridge structures. For exposure to deicing chemicals, the top mat of reinforcement is more susceptible to chloride-induced corrosion than the bottom mat and, therefore, acts as the anode with the bottom mat acting as the cathode in macrocell corrosion. AASHTO HB-17 recognizes this and requires greater concrete cover for the top mat of reinforcement. This basic premise of chloride-ion exposure is reversed for a bridge located in a warm climatic area over saltwater where the underside of the bridge deck can be more vulnerable to chloride-ion ingress. Consequently, the concrete cover should be increased for the bottom mat of deck reinforcement in this type of application.

So much has been written about the bridge deck problem since the early 1970s that corrosion protection of a bridge super-structure and substructure has sometimes been overlooked. Leakage of chloride-contaminated water through expansion and construction joints and cracks onto the superstructure,

substructure pier caps, abutments, and piers, can lead to corrosion of steel in these components. Additionally, snow-removal operations can pile chloride-containing snow around piers, and piers located in marine tidal splash zones are continuously subjected to wetting-and-drying cycles with chloride-laden seawater. To design a bridge deck, superstructure, and substructure with adequate corrosion protection over its intended service life of 75 years, as required by AASHTO HB-17, it is important to recognize the potential for chloride-ion ingress due to improper placement or functioning of joints, drains, and other openings in the structure.

2.1.2 *Parking structures*—In many respects, the potential for corrosion-related deterioration in a parking structure is greater than that for a bridge. Because of the intended function of a parking structure, chloride-laden slush on the underside of parked vehicles has ample time to drip onto parking decks, increasing the potential for chloride-ion penetration. Also, unlike bridge decks, parking structures, except for exposed roofs, are not rinsed by precipitation. Moreover, poor drainage permits chloride-laden water to pond on the concrete deck.

Design considerations pertinent to corrosion protection of a parking structure are similar to those of bridge design in that they depend on location and expected exposure. Corrosion-protection measures for a parking garage constructed in warm climates, where there is minor or no use of deicing salts, will be different from one constructed in cold climates, where deicing salts are heavily used.

All structural components for a parking structure located in a northern or mountainous climate where deicer chemicals are used should be provided with additional corrosion-protection measures. Additional corrosion-protection considerations are also needed for parking structures located in close proximity to marine areas where exposure to salt spray, salty sand, and high moisture conditions is probable. ACI 362.1R contains further recommendations.

2.1.3 *Industrial floors*—The type of expected exposure will impact design considerations necessary for corrosion protection of industrial floors. The primary concern in industrial and manufacturing facilities is exposure to acids or other aggressive chemicals that can lead to disintegration of the concrete. Membranes and coatings can protect these floors from their environment.

2.1.4 *Concrete façades*—Knowledge of the expected environmental exposure is the primary issue regarding satisfactory corrosion protection of concrete façades, such as architectural precast panels. The proximity of façades to heavily industrialized areas and geographical location is of particular importance. Some cities have higher levels of carbon monoxide, carbon dioxide, and pollutants from industrial smoke discharge, which can lead to a greater rate of concrete carbonation.

Concrete façades are occasionally exposed to chloride-induced corrosion. This can occur when chloride-laden snow piled against a parking structure façade subsequently melts and drips down the side of the structure. Steel reinforcement in the concrete façade and metal connections used to secure the façade to the structure are both vulnerable to this type of attack.

2.1.5 *Marine structures*—Concrete structures, such as docks, piers, and storage tanks, located in a marine environment are vulnerable to chloride-induced corrosion. Chloride ions and other ions in seawater can penetrate concrete. The portion of a marine concrete structure located in the tidal and splash zones is generally the most susceptible to corrosion because of the availability of water, oxygen, and chlorides. All segments of a marine structure are at risk for chloride-induced corrosion, but low oxygen concentrations significantly reduce corrosion rates in submerged portions.

2.1.6 *Concrete slab-on-ground*—Casting reinforced concrete in contact with chloride-contaminated soil can cause corrosion of the embedded reinforcement if chloride ions migrate into the concrete. Concrete with a high water-cementitious material ratio (*w/cm*) and high permeability is more vulnerable to attack.

2.1.7 *Other structures*—Other types of concrete structures can experience corrosion-related problems. Concrete in sewage and waste facilities can disintegrate after prolonged exposure to acids and sulfates in wastes and expose the steel. Prestressing steel in wire-wound concrete prestressed tanks has also corroded as a result of inadequate shotcrete protection and cracking (Schupack 1964; Schupack and Poston 1989). In these cases, the prestressing wires used to wrap the tanks had inadequate shotcrete cover to provide protection. Carbonation, water from rain, or leakage from inside the tank, along with oxygen, are sufficient to cause electrochemical corrosion of the prestressing wires. Improved design and construction have mitigated these problems.

2.2—Environment and corrosion

An important consideration in the design for corrosion protection is the type of environmental exposure to which a concrete structure will be subjected over its service life.

2.2.1 *Concrete not exposed to weather*—Concrete structures with the lowest corrosion risk are those not exposed to weather, such as a structural concrete frame of an office building. Reinforcement in concrete structures not directly exposed to moisture and benefiting from the drying effect of heating has a low risk of corrosion. Barring any unusual conditions, and using code-recommended concrete cover and concrete quality, concrete structures not exposed to weather and other outside environmental factors should have a low risk of corrosion during the building service life. Exceptions would be interior sections of buildings exposed to periodic wetting such as kitchens, bathrooms, or water fountain areas, and concrete members and floor slabs made with chloride additions. Care should be taken in areas such as boiler rooms where floor slabs can be subjected to continuous heating and exposure to higher-than-normal carbon dioxide concentrations. Severe carbonation of the concrete can occur in these cases.

2.2.2 *Concrete exposed to weather*—Concrete structures exposed to weather have a higher risk of corrosion than those not exposed to weather. An exception can occur in enclosed concrete parking structures, where carbonation-induced corrosion is possible. Moisture, along with oxygen, causes corrosion if the steel loses its passivity due to chloride ingress or carbonation.

Temperature also influences the corrosion risk. Given two identical concrete structures exposed to weather, corrosion would occur at a faster rate for the one exposed to the higher average ambient temperature. In addition, temperature variations can cause cracking in concrete that leads to the ingress of deleterious substances and potential corrosion. Concrete structures exposed to weather are also more vulnerable to carbonation, acid rain, and freezing and thawing.

2.2.3 *Concrete exposed to chemical deicers*—Sodium chloride (NaCl) is a commonly used chemical deicer. NaCl is applied in rock-salt form and is at least 95% pure. Calcium chloride ($CaCl_2$) is more effective as a deicer and is normally used when ambient temperatures are less than –25°F (–3.9°C). Although the relationship between the rate of steel corrosion, concrete alkalinity, and chloride-ion concentration is not completely understood, it is known that chloride ions from deicing salts promote corrosion of reinforcing steel. Chloride ions can initiate active corrosion because they break down the passive oxide film that initially forms on concrete-embedded reinforcement.

Bridges, parking garages, and other concrete structures exposed to chemical deicers are at a high risk for corrosion. Code-required minimum concrete quality and concrete cover for structures exposed to chlorides in service are needed to prolong service life. Depending on the expected maintenance, such as periodic freshwater washes of exposed surfaces and the aggressiveness of the exposure, additional measures, such as increased cover, low-permeability concrete, corrosion-inhibiting admixtures, corrosion-resistant steel reinforcement, or protective coatings on reinforcing steel or concrete, can be required to meet the proposed design service life of the structure.

2.2.4 *Concrete exposed to marine environment*—Because of the potential for ingress of chloride ions from seawater, concrete structures exposed to a marine environment have a corrosion risk similar to structures exposed to chemical deicers. The most vulnerable region of the structure is the tidal or splash zones, which goes through alternating cycles of wetting and drying.

Because of this greater risk of corrosion, AASHTO HB-17 recommends 4 in. (100 mm) of clear cover for reinforced concrete substructures that will be exposed to seawater. Other protective measures may be required to extend the service life.

2.2.5 *Concrete exposed to chemicals*—Industrial concrete structures exposed to chemicals that can lead to the disintegration of concrete are at high risk for corrosion. This type of exposure may require protective measures beyond those required for structures exposed to moisture only. For particularly aggressive chemicals, an impermeable coating on exposed concrete surfaces, polymer concrete, or polymer/sulfur-impregnated concrete may be required to ensure long-term corrosion protection (ACI 548.1R and ACI 548.2R).

2.2.6 *Concrete exposed to acid rain*—Prolonged release of industrial pollutants, such as sulfur dioxide and nitrogen oxides, has changed the chemical balance of the atmosphere. In North America, this problem is more pronounced in the industrialized regions of the northern U.S. and Canada. When precipitation occurs, rainwater combines with these oxides to form sulfuric acid, nitric acid, or both, known as acid rain. Acid attack of the concrete cover weakens the physical and electrochemical protection it offers to embedded steel, and the mechanisms of attack have been the subject of numerous studies (De Belie et al. 1997; Bakharev et al. 2003; Zivica and Bajza 2001) and is described in detail in ACI 201.2R.

2.3—Cracking and corrosion

In previous documents presented by ACI Committee 222, it was stated that the role of cracks in the corrosion of reinforcing steel was controversial. Following significant field and laboratory studies, it has been concluded by the committee that cracks, especially cracks parallel to reinforcing steel, reduce the service life of structures by permitting rapid and deeper localized penetration of carbonation and by providing a direct path for chloride ions, moisture, and oxygen to the reinforcing steel (Lindquist et al. 2006; Darwin et al. 2007). Thus, cracks can accelerate the onset and rate of active corrosion. It is not recommended that judicious placement of reinforcing steel to control crack width be used as a primary means of protecting against corrosion. It is essential to have concrete with a low permeability, low potential for plastic and drying shrinkage, low thermal expansion susceptibility, and sufficient cover to protect embedded steel reinforcement.

2.4—Structural details and corrosion

The two most important parameters for corrosion protection are concrete cover and concrete quality (Darwin et al. 1985). Concrete quality is discussed in Chapter 3.

2.4.1 *Cover requirements*—One of the easiest methods of improving steel reinforcement corrosion protection is to increase the amount of concrete cover. The required minimum cover for reinforcement in most concrete structures not exposed to weather is 3/4 in. (20 mm) (ACI 318). As the risk of corrosion increases, so does the required thickness of concrete cover.

2.4.1.1 *ACI 318 requirements*—The current ACI 318 minimum concrete cover requirements are summarized in Table 2.1. Where concrete will be exposed to external sources of chlorides in service or to other aggressive environments, a minimum concrete cover of 2 in. (50 mm) for walls and slabs and 2-1/2 in. (65 mm) for other members is required for corrosion protection. For precast concrete manufactured under plant control conditions, a minimum cover of 1-1/2 and 2 in. (40 and 50 mm), respectively, is recommended for walls and slabs.

2.4.1.2 *AASHTO bridge specification requirements*—Table 2.2 summarizes the minimum concrete cover requirements as specified in AASHTO HB-17. AASHTO HB-17 requires that the values reported in the table be modified depending on the *w/cm* being used. For *w/cm* values less than or equal to 0.4, a factor of 0.8 should be applied to the values in the table. For a *w/cm* greater than or equal to 0.5, a factor of 1.2 should be applied to the values in the table. These factors account for the changes in the rate of transport of deleterious agents in concretes with different *w/cm*. The

Table 2.1—ACI 318-required minimum concrete cover for protection of reinforcement

	Cast-in-place		Precast concrete,[†] in. (mm) (manufactured under plant-controlled conditions)
	Nonprestressed,[*] in. (mm)	Prestressed,[†] in. (mm)	
Concrete cast against and permanently exposed to earth	3 (75)	3 (75)	—
Concrete exposed to earth or weather	No. 6 to No. 18 bars: 2 (No. 19 to No. 57 bars: 50) No. 5 bar or smaller: 1-1/2 (No. 16 bar, MW200 or MD200 wire, and smaller: 40)	Walls, panels, slabs, joists: 1 (25) Other members: 1-1/2 (40)	Wall panels: No. 14 and No. 18 bars: 1-1/2 (No. 43 and No. 57 bars: 40) No. 11 bar and smaller: 3/4 (No. 36 bar and smaller: 20) Other members: No. 14 and No. 18 bars: 2 (No. 43 and No. 57 bars: 50) No. 6 to No. 11 bars: 1-1/2 (No. 19 to No. 36 bars: 40) No. 5 bar and smaller: 1-1/4 (No. 16 bar, MW200 and MD200 wire, and smaller: 30)
Concrete not exposed to weather or in contact with ground	Slabs, walls, joists: No. 14 and No. 18 bars: 1-1/2 (No. 43 and No. 57 bars: 40) No. 11 bar or smaller: 3/4 (No. 36 bar or smaller: 20)	Slabs, walls, joists: 3/4 (20)	Slabs, walls, joists: No. 14 and No. 18 bars: 1-1/4 (No. 43 and No. 57 bars: 30) No. 11 bar and smaller: 5/8 (No. 36 bar and smaller: 15)
	Beams, columns: Primary reinforcement, ties, stirrups, spirals: 1-1/2 (40)	Beams, columns: Primary reinforcement: 1-1/2 (40) Ties, stirrups, spirals: 1 (25)	Beams, columns: Primary reinforcement: d_b[‡] but not less than 5/8 (15) and need not exceed 1-1/2 (40) Ties, stirrups, spirals: 3/8 (10)
	Shells, folded plate members: No. 6 bar and larger: 3/4 (No. 19 bar and larger: 20) No. 5 bar and smaller: 1/2 (No. 16 bar, MW200 or MD200 wire, and smaller: 13)	Shells, folded plate members: No. 5 bar and smaller: 3/8 (No. 16 bar, MW200 or MD200 wire, and smaller: 10) Other reinforcement: d_b[‡] but not less than 3/4 (20)	Shells, folded plate members: No. 6 bar and larger: 5/8 (No. 19 bar and larger: 15) No. 5 bar and smaller: 3/8 (No. 16 bar, MW200 or MD200 wire, and smaller: 10)

[*]Shall not be less than that required for corrosive environments or for fire protection.

[†]For prestressed and nonprestressed reinforcement, ducts, and end fittings, but not less than that required for corrosive environments or for fire protection.

[‡]d_b = nominal diameter of bar, wire, or prestressing strand, in. (mm).

minimum cover should be in accordance with the provided table, but in no case should it be less than 1.0 in. (25 mm). Also, AASHTO allows that cover over stirrups or ties to be 0.5 in. (12 mm) less than the values reported in Table 2.2, but never less than 1.0 in. (25 mm). Unlike ACI 318, AASHTO HB-17 allows for reduced cover for epoxy-coated reinforcement. The standard allows for interior exposure cover requirements when the reinforcement is coated with epoxy.

2.4.2 *Drainage*—The long-term performance of concrete structures, particularly parking structures and bridges, is enhanced by adequate drainage. Unfortunately, this is one of the most overlooked design details. Adequate drainage reduces the risk of corrosion by reducing ponding and the quantity of water and deicing salts that can otherwise penetrate the concrete.

For both bridges and parking structures, the slope required for drainage is a function of both short-term and long-term deflections, camber, surface roughness, and the number and location of drains. Depending on the layout of the structural framing system, drainage can be provided by transverse or longitudinal slopes or both. No simple formula incorporates all the factors that influence slope and drainage. As a rule of thumb, the minimum slope should be 1.5%, with 2% being preferred (ACI 362.1R). Consideration of time-dependent deflections is a critical component of a good drainage system

Table 2.2—AASHTO-required minimum concrete cover for protection of reinforcement

Situation	Cover (in./mm)
Direct exposure to salt water	4.0/100
Cast against earth	3.0/75
Coastal	3.0/75
Exposure to deicing salts	2.5/65
Deck surfaces subject to tire stud or chain wear	2.5/65
Exterior other than above	2.0/50
Interior other than above —Up to No. 11 (No. 36) bar —No. 14 (No. 43) and No. 18 (No. 57)	1.5/40 2.0/50
Precast soffit form panels	0.8/20
Precast reinforced piles —Noncorrosive environments —Corrosive environments	2.0/50 3.0/75
Precast prestressed piles	2.0/50
Cast-in-place piles —Noncorrosive environments —Corrosive environments —General —Protected —Shells —Auger-cast, tremie concrete, or slurry construction	2.0/50 3.0/75 3.0/75 3.0/75 2.0/50 3.0/75

design. Time-dependent deflections are particularly important for prestressed concrete structures.

Drains should be placed to prevent the discharge of drainage water against any portion of the structure or onto

moving traffic below and to prevent erosion at the outlet of downspouts. For safety reasons, drains in snow-belt areas should also be located to prevent melted snow from running onto a slab and refreezing. Drains, downspouts, and other drainage components should be made of a rigid, corrosion-resistant material and be easy to unclog. Additional information on drainage in parking structures can be found in ACI 362.1R.

2.4.3 *Reinforcement*—Differences between various types of steel reinforcement—for example, prestressing, nonprestressing, different grades, manufacturers, and sizes—may be significant factors in the electrochemical corrosion of steel. The level of stress in the steel is not a significant factor in electrochemical corrosion, but can be a factor in certain circumstances related to stress-corrosion cracking of prestressing steel.

For any concrete structure, independent of the risk of corrosion, steel reinforcement should be free of loose rust before casting the concrete. Measures should be made to protect steel from exposure to chlorides and other contaminants. Additionally, prestressing steel should be protected from the weather. It is not uncommon for the steel reinforcement in an entire project to be delivered to the site and exposed to weather for months before use. This should be avoided.

Lubricants used to draw prestressing steel appear to raise the chloride-corrosion threshold (Pfeifer et al. 1987). These oils, however, can also adversely affect bond.

Engineering specifications for a project should clearly state quality-control procedures to ensure that the reinforcement is adequately tied and secured to maintain the minimum specified concrete cover.

ASTM A767/A767M requires galvanized reinforcing steel to be treated with a hexavalent chromate to prevent the evolution of hydrogen in fresh concrete. This can occur when an unpassivated zinc surface reacts with hydroxides in fresh concrete (Boyd and Tripler 1968). Hydrogen evolution can also be prevented by adding a small amount of chromate salt to the fresh concrete (Rosenberg and Gaidis 1978). Calcium nitrite has been used to prevent hydrogen evolution of galvanized precast concrete forms (ACI 222R). Additionally, procedures should be provided to minimize electrical connection with nongalvanized metals. The use of chromate may be regulated due to health concerns. Research by Tan and Hansson (2008) has found that the chromate treatment may not be necessary.

When epoxy-coated reinforcement is used, ACI 222R recommends that there should be no reduction in cover. Because macrocells can develop where defects occur in the coating, project specifications should clearly state quality control of the coating and provide procedures to minimize or avoid electrical contact with uncoated metals.

Structures that use unbonded post-tensioned construction require protective measures, especially in aggressive chloride environments.

Because the prestressing elements may not be directly protected by an alkaline environment, but instead by a combination of either duct and grout or sheathing and grease, project specifications should clearly indicate that protection be maintained for the full length of the tendon and that the duct or sheathing material should be impermeable to water. For unbonded tendons in aggressive environments, the project specifications should require that sleeves used to connect the sheathing to the anchorage of encapsulated systems should have a positive mechanical connection to the anchorage at all stressing, intermediate, and fixed ends.

ACI 423.7 and ACI 301 provide guidance for additional measures such as corrosion-resistant grease and anchorage protection. Corrosion-related failure of unbonded prestressing tendons in building and parking structures has occurred in the absence of chlorides (Schupack 1982; Schupack and Suarez 1982). Unbonded monostrand installed before 1985 often had inadequate sheaths and grease protection. When the provisions of Post-Tensioning Institute (PTI) "Specifications for Unbonded Single Strand Tendons" (1985) are rigorously followed, corrosion rarely occurs.

Grouted tendons have had corrosion incidents caused by poor grouting materials, techniques, and anchorage protection (Schupack 2004). The PTI grouting specification (1985) and ACI 423.9M provide more stringent requirements relating to grouting materials, techniques, and anchorage protection.

2.4.4 *Joints*—Joints should be constructed and sealed to prevent leakage. ACI 224.3R provides guidance on design and detailing of joints in concrete structures. Additional information on the design of joints for parking structures can be found in ACI 362.1R. A discussion on various coatings for waterproofing concrete can be found in ACI 515.1R.

2.4.5 *Overlays*—For concrete structures with a high risk of corrosion, particularly due to external chloride, low-permeability overlays have been used as an alternative protection method. The overlay provides additional concrete cover to protect embedded reinforcement. In some cases, however, such as bridge decks, the use of overlays has been shown to increase cracking (Darwin et al. 2004), with a commensurate increase in chloride content at the level of the reinforcing steel (Darwin et al. 2006).

Overlays intended to reduce chloride ingress have been made with concrete with a low *w/cm*, latex-modified concrete, polymer concrete, and concrete with pozzolan (ACI 224R). Designs should include the compatibility of the overlay and the substrate concrete in terms of mechanical properties and should consider potential shrinkage cracking caused by restrained volume changes.

2.4.6 *Embedded items*—In general, any embedded metal in concrete should have the same minimum concrete cover as that recommended for steel reinforcement for the anticipated exposure conditions. If this cannot be achieved, then additional protective measures are needed. Embedded items require additional protection when exposed to an aggressive environment.

Precast parking structures often contain weld plates used to connect components. In aggressive environments, consideration should be given to the use of galvanized or stainless steel for these plates or painting the plates with epoxy after field welding. If the connection plates are galvanized, consideration should be given to the possibility of developing galvanic cells if connections are made to nongalvanized steel. Similar galvanic cells can be created between architectural features, such as handrails, and the

reinforcement. Such architectural features should be galvanized or otherwise protected such that galvanic cells are not formed.

In submerged concrete structures with unbedded, freely exposed steel components in contact with reinforcing steel, galvanic cells can develop with the freely exposed steel forming the anode and the embedded steel forming the cathode. This can cause corrosion of the embedded, freely exposed steel. If exposed connections are necessary, then corrosion protection, such as the use of an epoxy coating, is necessary.

CHAPTER 3—IMPACT OF MIXTURE PROPORTIONING, CONCRETING MATERIALS, AND TYPE OF EMBEDDED METAL
3.1—Influence of mixture proportioning on corrosion of reinforcing steel

3.1.1 *Introduction*—The proportioning of concrete mixtures that enhance the corrosion resistance of reinforcing steel is not substantially different from the proportioning of mixtures for any high-quality concrete. The goal is to use available materials to develop a concrete mixture that will permit mixing, transporting, placing, consolidating, and finishing in the fresh state and, when cured properly, will have a low permeability in the hardened state. Mixture proportions should permit pumping of the concrete, if required, and control bleeding and minimize shrinkage.

ACI 201.2R describes in detail the general durability of concrete, determined largely by the selection of cement, aggregates, water, and admixtures. When considering the effects of reduced permeability, freezing-and-thawing resistance, alkali-aggregate reaction, and sulfate attack on the corrosion of reinforcing steel, the most important concrete property is reduced permeability. Permeability describes the rate of movement of liquids or gases through concrete and is related to the connectivity of pores and voids in the hardened concrete. Assuming there is adequate curing, permeability can be reduced primarily through the use of chemical admixtures to achieve the lowest practical w/cm, and secondly through the use of pozzolanic admixtures, supplementary cementitious materials, and polymers (ACI 212.3R, ACI 212.4R, ACI 232.1R, ACI 232.2R, ACI 233R, ACI 234R, and ACI 548.1R).

3.1.2 *Benefits of low* w/cm—The benefits of reducing the w/cm to delay the corrosion of reinforcing steel are demonstrated in ACI 222R, which shows that the reduction in the flow of oxygen through concrete is a function of the reduction in w/cm. The report also shows the effects of w/cm on reducing salt penetration and increasing time-to-corrosion. In each of these cases, the benefit of reducing the w/cm can be interpreted as a result of the reduction in the permeability of the concrete.

For these reasons, w/cm is limited to certain maximum values for concrete that will be exposed to a corrosive environment. ACI 201.2R and ACI 211.1 contain recommended values for w/cm, and Chapter 4 of ACI 318 gives maximum values for the w/cm. All three documents recommend that w/cm should not exceed 0.40 for concrete exposed to chlorides from seawater, deicing salts, and other sources. AASHTO HB-17 limits the w/c to 0.49 for Class P > 4000 psi (> 28 MPa) concrete.

In cases where the shrinkage of concrete is restrained, such as bridge decks, the use of a lower w/cm can have negative effects in the form of increased cracking (Krauss and Rogalla 1996; Darwin et al. 2004). The increased cracking is caused by a decrease in tensile creep that results from the increase in concrete strength obtained with the lower w/cm. High-strength concrete exhibits less creep, even at the same stress-strength ratio, than normal-strength concrete (Smadi et al. 1987).

3.1.3 *Proportioning mixtures for low* w/cm—Reducing the quantity of mixing water relative to the mass of cementitious materials decreases the w/cm. Simply removing water from a given mixture, however, will generally result in a less workable mixture. To preserve workability, which is typically characterized by slump, and reduce w/cm, it is necessary to maintain the water content while increasing the cementitious materials content. By doing this, the contractor's placement needs can be met, while at the same time providing the dense, low-permeability concrete required. This means that a low-w/cm mixture will have increased cement content and an accompanying increase in cost. For concrete with a low slump or low water demand, this approach is satisfactory for moderate reductions in the w/cm.

For mixtures requiring a greater slump for placement or finishing purposes, or for the establishment of a w/cm of 0.40 or less, an increase in cement content alone will lead to excessive cement factors, which can lead to concrete mixtures with high mortar contents and an increased tendency toward plastic and drying-shrinkage cracking. In addition, the heat of hydration developed with higher cement contents results in higher early-age temperatures, which can lead to thermal cracking if proper actions are not taken to minimize high thermal gradients in the concrete element. To reduce the water content at a given cement content, water-reducing admixtures, which effectively reduce the water content required to obtain a desired slump, are used. The reduced water content may then lead to reduced cement content for the same w/cm. For greater reductions in water, high-range water-reducing admixtures (HRWRAs) (ASTM C494/C494M Types F and G) are used.

The effects of aggregate size and gradation on the water content required for a particular level of workability should not be overlooked. Smaller aggregate sizes demand more water as do intentionally or unintentionally gap-graded aggregates. By using the largest aggregate size commensurate with the structural details of the members being placed and controlling gradations, it is possible to reduce the water and cement contents required for a particular w/cm. It can be more economical to design a low-w/cm, low-permeability mixture based on 1-1/2 in. (37.5 mm) coarse aggregates than with 3/8 in. (9.5 mm) coarse aggregates. Further, appropriate selection and gradation of aggregates permit pumping, placement, and finishing of concrete at a lower slump than required when less-than-optimum aggregate sizes and gradations are used.

3.2—Influence of selection of cement, aggregates, water, and admixtures on corrosion of reinforcing steel

3.2.1 *Selection of cement*—The influence of portland cement's chemistry on the corrosion of reinforcing steel is discussed in detail in ACI 201.2R, 222R, 225R, and Whiting

(1978). The characteristic alkaline nature of hardened cement paste normally maintains the corrosion resistance of steel in concrete; this protection is lost when chloride ions contaminate concrete or when carbonation occurs.

One of the mineral constituents of portland cement, C_3A, (tricalcium aluminate) has the ability to react with chloride ions to form chloroaluminates, thereby reducing the impact of chloride contamination on corrosion. C_3A can represent 4 to 12% of the mass of cement. ASTM C150/C150M cement types (Types I through V) contain varying amounts of C_3A, but the effect of this constituent is not sufficiently clear to warrant selecting a chloride-reducing cement on the basis of C_3A content. Further, other durability problems, such as sulfate attack, become more likely as the C_3A content is increased. There is work, however, that shows corrosion initiates early in Type V cements in the presence of chloride and in chloride environments (Masledhuddin et al. 2007; Page et al. 1981) Also, in the presence of chloride, there is no increase in the susceptibility to sulfate attack with higher C_3A content (Rasheeduzzafar et al. 1990; Sakr 2005).

Higher-alkali cements are effective in providing a higher pH environment around the steel and reducing the corrosion potential of steel in the presence of chloride ions. Similarly, the use of a cement with a higher alkali content increases the risk of alkali-aggregate reaction. Unless the producer is certain that the aggregate selected for the concrete mixture is non-alkali reactive, the use of high-alkali cement to enhance corrosion resistance is not recommended.

Any portland cement meeting the requirements of ASTM C150/C150M can be used to produce a high-quality concrete that will reduce or prevent the corrosion of embedded reinforcing steel. Factors such as the selection and maintenance of a low *w/cm* and proper placement, consolidation, finishing, and curing practices are more important than the selection of cement in regard to corrosion.

Blended cements, in which the portland-cement clinker is interground with a supplementary cementitious material, will result in reduced permeability in suitably designed concrete. Uniform dispersion of the blended cement is needed but is harder to maintain as the difference in particle-size distribution between the cement and the blended supplementary cementitious material increases. When properly dispersed, silica fume or other supplementary cementitious materials can significantly reduce chloride ingress.

3.2.2 *Selection of aggregates*—ACI 201.2R and 222R discuss aggregate selection for durable concrete. Issues such as soundness, freezing-and-thawing resistance, wear resistance, and alkali reactivity should be addressed in addition to other aggregate characteristics that relate to the corrosion protection for the steel.

Two primary issues govern the selection of aggregates for use in concrete exposed to a corrosive environment:

1. The use of aggregates that introduce chloride ions into the mixture, which is discussed in detail in ACI 201.2R and 222R; and

2. The development of corrosion-resistant concrete is the proper selection of aggregate size and gradation to enhance the workability of the mixture and reduce the required water content (Section 3.1).

The chloride-ion concentration limits discussed in this guide can be exceeded through the use of aggregates that contain absorbed chloride ions. Judicious materials selection requires that the chloride-ion content of the proposed aggregates be evaluated before use. Free chlorides on the surface or readily available from pore spaces in the aggregate can be determined by relatively simple analytical means (Gaynor 1986). Tightly bound chlorides, however, will not likely contribute significantly to corrosion. It has been shown, however, that the bound chlorides can be released and contribute to the corrosion (Glass and Buenfeld 2000). Determination of the amount of bound chlorides that can enter into the pore solution requires specialized testing procedures (Hope et al. 1985; ACI 222R).

Once an aggregate source has been selected, attention should be given to monitoring the moisture content of both the coarse and fine aggregates at the time of inclusion in the mixture. Errors in assessing the moisture content can lead to substantial increases in the *w/cm* of the mixture, resulting in dramatic increases in permeability.

3.2.3 *Selection of mixture water*—Seawater should never be used to make concrete for reinforced structures. Water with chloride contents exceeding values given in ASTM C1602/C1602M should not be used to make concrete for reinforced structures because it may contribute enough chloride ions to cause serious corrosion problems.

3.2.4 *Selection of chemical and supplementary cementitious admixtures*—A wide variety of chemical and supplementary cementitious admixtures is available that either directly improves the corrosion protection provided by the hardened concrete or modifies the properties of the fresh concrete, permitting the use of lower *w/cm* mixtures with their accompanying benefits in enhancing corrosion protection. Certain admixtures, however, can increase the chloride-ion content, lowering corrosion protection. It may be necessary to determine the chloride-ion contribution of the admixture before use. Additionally, it is wise to check the compatibility between cement, admixtures, and other concrete ingredients by making field trial batches before starting construction. Incompatibility of materials can lead to rapid slump loss, rapid set, and increased water demand, which can adversely affect the corrosion resistance of the concrete.

3.2.4.1 *Chemical admixtures*—These materials are generally added in liquid form either during batching or upon arrival at the job site. The quantities used are quite small relative to the mass or volume of other materials in the concrete mixture, and careful control of the dosage is required. For example, it would not be unusual to add less than 18 fl oz. (530 mL) of admixture to 4000 lb (1820 kg) of fresh concrete. ACI 212.3R contains specific guidance relating to the use of admixtures. Admixtures can be grouped into the following classifications:

• **Air-entraining admixtures**—The use of air-entraining admixtures to develop a proper air-void system in concrete is necessary in a freezing-and-thawing environment. In many concrete mixtures, air entrainment also permits a reduction in water content because the air bubbles increase the workability of the mixture. If the cement content of the mixture is held constant while the

water content is reduced, the net result is a decrease in the *w/cm* and permeability (Kosmatka et al. 2003). Air-entraining admixtures, therefore, have an indirect benefit on enhancing corrosion protection. In many cases, environmental conditions require both freezing-and-thawing resistance and corrosion protection. Air-entraining admixtures should be specified using ASTM C260/C260M.

- **Water-reducing admixtures**—These chemicals are formulated to increase the workability or fluidity of fresh concrete by breaking up and dispersing agglomerations of fine cement particles. High-range water reducers can increase the workability of a concrete mixture proportioned for a given water content. Alternatively, these admixtures permit a reduction in the quantity of water required to achieve a particular slump. When this water reduction is matched with a reduction in cement, the *w/cm* remains the same. If the cement factor is kept constant while the water content is reduced, workability is maintained with a reduction in *w/cm* and a reduction in permeability. Water-reducing admixtures are classified as Types A, D, or E in ASTM C494/C494M, depending on their effects on time-of-setting.

- **High-range water-reducing admixtures**—HRWRAs provide dramatic increases in workability at the same *w/cm* or at a reduced water content at the same slump. Through the use of HRWRAs, concrete with low *w/cm* and marked reductions in permeability can be produced while maintaining workability. ACI 318 and ACI 357R recommend a *w/cm* less than or equal to 0.40 for concrete that will be exposed to deicing salts or a marine environment. HRWRAs can be used to achieve low *w/cm* and are classified as Types F and G in ASTM C494/C494M, where the latter indicates a retarding effect.

- **Accelerating admixtures**—Accelerators reduce concrete setting times and improve early strength. They are typically used to compensate for slower cement hydration when temperatures are below 60°F (16°C). One of the most common accelerators is calcium chloride. For steel-reinforced or prestressed concrete structures, however, admixed chlorides can lead to severe corrosion, especially if the concrete is subjected to wetting and chloride ingress. Admixtures that contain intentionally added chloride or other halides should not be used. A nonchloride accelerator should be noncorrosive within its recommended dosage range. Accelerating admixtures are classified as Type C or E in ASTM C494/C494M.

- **Retarding admixtures**—When temperatures are above 80°F (27°C), set retarders increase the setting time, thereby extending the time during which the concrete can be transported, placed, and consolidated without the need for additional water. Thus, the desired *w/cm* and, consequently, the intended permeability and durability characteristics of the concrete, are maintained. Set-retarding admixtures are classified as Types B or D in ASTM C494/C494M.

- **Corrosion-inhibiting admixtures**—Corrosion-inhibiting admixtures delay the onset of corrosion and reduce the rate of corrosion of reinforcement due to chloride attack.

3.2.4.2 *Chemical admixtures*—Two main types of chemical admixtures extend the time to corrosion-induced damage of steel-reinforced concrete structures in chloride-laden environments: corrosion inhibitors and physical-barrier admixtures. Some corrosion inhibitors also act as physical-barrier admixtures.

3.2.4.3 *Corrosion inhibitors*—ISO 8044 defines corrosion inhibitors as chemical substances that decrease the corrosion rate when present at a suitable concentration, without significantly changing the concentration of any other corrosion agent. Recent research, however, indicates that for some inhibitors, such as calcium nitrite, their role is to delay the initiation of corrosion (Hansson et al. 1998; Mammoliti et al. 1999). Nonetheless, these admixtures provide protection to the steel surface, either electrochemically (anodic, cathodic, mixed-inhibitor), chemically (chemical barrier), or as a chloride scavenger within the concrete cover to inhibit chloride-induced corrosion initiation. Inorganic chemical compounds that protect steel against chloride attack in a basic pH concrete environment include borates, chromates, molybdates, nitrites, and phosphates. Calcium nitrite is the most researched inorganic inhibitor and the most widely used. Organic compounds used in admixtures to protect steel from chloride-induced corrosion include alkanolamines and an aqueous mixture of amines and fatty-acid esters (Berke et al. 1993; Bobrowski and Youn 1993; Mäder 1995; Martin and Miksic 1989; Nmai et al. 1992; Nmai and Krauss 1994).

Organic amine-based compounds, such as some amine salts and alkanolamine, were reported to be effective corrosion inhibitors for steel in concrete when used in a post-treatment process for chloride-induced corrosion of steel in concrete in laboratory studies (Al-Qadi et al. 1992; Collins et al. 1993; Dillard et al. 1992). Results of long-term field evaluations of topically-applied corrosion inhibitors are conflicting (Islam et al. 2002; Sprinkel 2003; Sharp 2004; Schiegg et al. 2000).

Physical-barrier admixtures reduce the rate of ingress of corrosive agents (chlorides, oxygen, and water) into the concrete. Bitumen, silicates, and water-based organic admixtures consisting of fatty acids such as oleic acid, stearic acid, salts of calcium oleate, and esters such as butyloleate are typically used in these types of admixtures. A liquid admixture containing a silicate copolymer in the form of a complex, inorganic, alkaline earth may also be effective in reducing the permeability of concrete and providing protection against corrosion of reinforcing steel (Miller 1995).

3.2.4.4 S*upplementary cementitious admixtures*—These solid admixtures reduce the rate of penetration of water and chloride-containing solutions through the formation of additional hydration-type products or by plugging capillary pores in the portland-cement paste. The admixtures are effective in reducing the rate of water transmission through concrete under externally applied hydraulic pressures. They include:

- **Fly ash**—Fly ash is widely used as a partial replacement for cement in concrete. Workability is often improved, especially for low-*w/cm* mixtures, and permeability to chloride ions is reduced. The use of Type F fly ash will

also reduce the maximum temperature rise of concrete. Fly ash should be specified using ASTM C618.

* **Slag cement**—Slag cement is added as a cement substitute or blended into cement. It reduces temperature rise in large members and decreases permeability to chloride ions. Slag cement should be specified using ASTM C989.
* **Natural pozzolans**—Natural pozzolans provide some improvement in permeability reduction but are not as effective as fly ash or slag cement. Natural pozzolan is also specified using ASTM C618.
* **Silica fume (condensed silica fume)**—Silica fume is an effective pozzolan in reducing concrete permeability to chloride-ion ingress when used in combination with HRWRAs, and will provide higher strengths when used as a partial cement substitute or as an addition. Because of its high water demand, the use of an HRWRA is needed to improve dispersion of the silica fume and workability of the concrete mixture, especially at the low water contents typically used. Silica fume should be specified using ASTM C1240.

3.2.4.5 *Polymers*—Polymer concrete and polymer-modified concrete are commonly used in concrete construction and repair of concrete structures. In polymer concrete, the polymer is used as a binder for the aggregate, whereas in polymer-modified concrete, the polymer is used along with portland cement. Low permeability and improved bond strength to concrete substrates and other surfaces are some of the advantages of polymers. More details on polymers and polymer concrete are given in ACI 548.1R.

3.3—Uncoated reinforcing steel

For most reinforced concrete construction in the U.S., deformed billet-steel reinforcing bars conforming to ASTM A615/A615M are used (ACI 318). Reinforcing bars can also meet requirements of ASTM A706A706M, A996/A996M, A955/A955M and A1035/A1035M specifications. Factors such as level of stress have not been found to play a major role in corrosion susceptibility in the concrete environment (ACI 222R).

3.4—Epoxy-coated reinforcing steel

3.4.1 *Introduction*—After several evaluations and a research study involving numerous types of coatings (Clifton et al. 1974), fusion-bonded epoxy coating emerged during the 1970s as an acceptable method of corrosion protection for uncoated reinforcing steel in concrete. Today, fusion-bonded coatings are one of the most widely used corrosion protection alternatives in North America, particularly for mild-steel reinforcing bars. As of 1998, there were approximately 100,000 structures containing epoxy-coated reinforcement (Virmani and Clemena 1998).

Because fusion-bonded epoxy coatings form barriers, some protection is lost when the coating is damaged. Breaks in the coating reduce the electrical resistance (Clear 1992b; Wiss, Janey, Elsner Associates, Inc. 1992) and permit contaminants to reach the steel surface. Long-term adhesion of the epoxy coating to the steel substrate is vital to corrosion performance. Studies have shown that corrosion performance is not impaired by loss of adhesion when there are no breaks in the coating, but it is reduced substantially in the presence of defects (Surface Science Western 1995; Martin et al. 1995).

Although proper handling and quality-control measures will reduce damage and other coating defects, it is unrealistic to expect defect-free coated bars in the field. Defects can result from imperfections in the steel surface, inadequate film thickness, improper fabrication, rough handling, and consolidation of the concrete.

3.4.2 *Corrosion-protection performance*—The degree of corrosion protection provided by epoxy coatings is controversial. Numerous laboratory (Clear and Virmani 1983; Clifton et al. 1974; Erdogdu and Bemner 1993; Wiss, Janney, Elstner Associates, Inc. 1992; Pfeifer et al. 1993; Scannell and Clear 1990; Sohanghpurwala and Clear 1990; and Virmani et al. 1983) and field studies have shown that epoxy-coated reinforcing steel has a longer time-to-corrosion than uncoated reinforcing steel. Many field studies undertaken in the 1990s examining the performance of bridge decks in service for 15 years or more reported excellent results (Gillis and Hagen 1994; Hasan et al. 1995; Perregaux and Brewster 1992; West Virginia DOT 1994).

There have also been examples of corrosion-induced damage in structures containing epoxy-coated reinforcement, most notably in the splash zones of the substructure components of five large bridges in the Florida Keys. These bridges began to exhibit corrosion spalling within 5 to 7 years of construction (Smith et al. 1993). Isolated examples of corrosion have also been reported in bridge decks, barrier walls, and a parking garage (Clear 1994). An investigation in Ontario (Manning 1996) showed loss of adhesion in bridges that had been in service for less than 15 years. The degree of adhesion loss of the coating correlated with the age of the structure and was found in bars embedded in chloride-contaminated and chloride-free concrete. Other studies have reported poor adhesion on bars removed from older structures (Clear 1994).

Extensive laboratory and field studies were undertaken to determine the cause of corrosion problems with epoxy-coated bars (Sagues and Powers 1990; Sagues et al. 1994; Zayed et al. 1989). Other studies attempting to identify the factors affecting the performance of coated reinforcement were funded by the Concrete Reinforcing Steel Institute (CRSI) (Clear 1992b; Wiss, Janey, Elsner Associates, Inc. 1992), the Canadian Strategic Highway Research Program (SHRP) (Clear 1992a, 1994) and the National Cooperative Highway Research Program (Clear et al. 1995).

These studies have significantly contributed to understanding the long-term field performance of epoxy-coated reinforcement, but they have not related this performance to specific production variables or to results of short-term laboratory testing. From investigations on laboratory and outdoor-exposure specimens containing bars manufactured according to the 1987 specifications, a failure mechanism was identified involving progressive loss of coating adhesion accompanied by under-film corrosion on coated bars (Clear 1994; Clear et al. 1995). This led to the conclusion that epoxy-coated reinforcement can extend the time-to-corrosion by 3 to 6 years in bridges exposed to

salt (marine and deicing) compared with uncoated steel in the same environment. Clear (1994) estimated that the time-to-corrosion damage could be extended to 8 to 11 years if the quality of the coatings were improved.

There is no dispute that epoxy coating will extend the time-to-corrosion damage compared with uncoated steel, but the long-term performance remains somewhat uncertain. Not all factors affecting corrosion performance are understood; there are many examples of good performance and premature corrosion damage. The dominant factors affecting performance are the number and size of defects in the coating. Where epoxy-coated reinforcement is used, it is essential that quality-control and assurance measures focus on these two properties. A number of specifications, such as ASTM D3963/D3963M, ASTM A775/A775M, and AASHTO M284M/M284 are available. These specifications continue to be updated with the progress of research studies.

Where a coated bar is used in a structure, it is advisable to coat steel that would otherwise be electrically connected (Scannell and Clear 1990). The onset of corrosion occurs independently of whether the coated bars are coupled to uncoated bars. During the propagation phase, however, an uncoated bottom mat electrically coupled to a top mat can facilitate macrocell action that can increase corrosion rates, compared with electrically isolated bars (Schiessl 1992).

Most bars are coated as straight bars and then fabricated as required by bending schedules. Studies have shown that bent bars generally exhibit more damage and do not perform as well in corrosion studies (Clear 1992b; McDonald et al. 1995). As a result, some users are now requiring bending before coating.

3.4.3 *Inspection and testing*

3.4.3.1 *Holiday testing*—Holidays are pinholes, discontinuities, or other coating defects not visible to the naked eye. Abrasions, cuts, and other damage incurred during handling, shipping, or placement are not considered holidays.

Most production lines are equipped with in-line holiday detectors that operate continuously. Hand-held holiday detectors are often used to spot check in-line results. Holiday testing is used primarily as a quality-control and inspection tool in the plant; normally, it is not intended for use in assessing coating damage at the job site.

3.4.3.2 *Coating thickness*—The thickness of the applied coating is also an important performance parameter. Thicker coatings generally have fewer holidays and other discontinuities as well as higher dielectric properties. Thicker coatings also provide a better barrier to water and chloride ions, thereby conferring a higher degree of corrosion protection.

Structural considerations such as creep, fatigue, and bond development of the coated reinforcing steel limit the maximum allowable coating thickness. Most standard specifications require that 90% of the thickness measurements be between 7 to 12 mils (175 to 300 mm). Fatigue and creep performance of coated reinforcing steel are comparable to uncoated reinforcing bar when coating thickness is within these limits. ACI 318 requires a 20 to 50% increase in the basic development lengths for epoxy-coated reinforcing

steel, depending on the bar spacing and concrete cover, to account for the reduced bond associated with the coating.

3.4.3.3 *Bend test*—The bend test is another quality-control technique used to evaluate the application process. A production-coated bar is bent 120 to 180 degrees around a mandrel of a specified size. If the coating cracks, debonds, or both, there is a problem in the application process.

3.4.3.4 *Coating adhesion*—The bend test has been the principal quality-assurance technique used to evaluate coating adhesion in the plant. Additional means may be needed, however, to adequately evaluate adhesion on a production bar. Three tests were adopted in ASTM A775/A775M to supplement the existing bend test:

1. Hot water;
2. Cathodic debonding; and
3. Salt-spray.

These tests were used in the evaluation of epoxy coatings for other applications such as pipeline coatings and are more discriminating than the bend test in identifying relative differences in adhesion.

3.4.3.5 *Coating repair*—Coating defects and damage caused during production are repaired in the plant. Patching or touch-up material should conform, as applicable, to ASTM A775/A775M or ASTM A934/A934M, as specified in ACI 301, and should be applied in strict accordance with the manufacturer's recommendations. Generally, surface preparation is accomplished with a wire brush, emery cloth, sandpaper, or file. The repair material is typically applied by brush. The same procedures are followed to coat bar ends.

3.4.4 *Field practice*

3.4.4.1 *Fabrication*—Reinforcing steel is most often fabricated or cut and bent to shape after coating because most production lines are designed for coating long, straight bars. Any cracks or other damage in the bend areas should be properly repaired before shipping the bars to the job site.

3.4.4.2 *Handling and transportation*—Epoxy coatings can be damaged by improper handling and storage. Epoxy-coated steel should be bundled using plastic-coated wire ties or other suitable material. Excess sag that can cause bar-to-bar abrasion should be avoided when lifting bundles of coated steel. Nylon slings or padded wire ropes should be used to lift bundles in the plant and at the job site. Coated steel is usually shipped by rail or trucked to the project. Precautions should be taken to minimize scraping bundles during transport. Dragging coated bars over other bars or any abrasive surface should be avoided.

3.4.4.3 *Storage*—Epoxy-coated steel should be stored on timbers or other noncorrosive material. The storage area should be as close as possible to the area of the structure where the steel will be placed to minimize handling. Coated steel should not be dropped or dragged. Epoxy-coated steel should not be stored outdoors for longer than 3 months. If long-term outdoor storage cannot be prevented, the steel should be protected from direct sunlight and sheltered from the weather by covering it with opaque polyethylene sheets or other suitable waterproofing material. Provisions should be made to allow adequate air circulation around the bars to minimize condensation under the covering.

3.4.4.4 *Installation*—Epoxy-coated bars should be placed on coated bar supports and tied with coated wire. After coated steel is in place, walking on the bars should be kept to a minimum, and tools, equipment, and construction materials should not be dropped or placed on the bars. ACI 301 requires that all visible coating damage be repaired before placing concrete.

Studies at The University of Texas at Austin (Kahhaleh et al. 1993) have shown that vibrators used to consolidate concrete can cause large bare areas on epoxy-coated bars in the course of normal operations. To minimize this type of damage, vibrators with a resilient covering should be used. Vibration times, however, need to be increased to consolidate the concrete. As a result, the susceptibility to damage from vibration increases.

3.4.4.5 *Maintenance*—There are no maintenance requirements for epoxy-coated reinforcing steel throughout the service life of a structure.

3.5—Embedded metals other than reinforcing steel

3.5.1 *General*—Metals other than steel are occasionally used in concrete. These metals include aluminum, lead, copper and copper alloys, zinc, cadmium, Monel metal, stellite (cobalt-chromium-tungsten alloys), silver, and tin. Galvanized steel and special alloys of steel, such as stainless steels and chrome-nickel steels, have also been used. Zinc and cadmium are used as coatings on steel (Tomlinson and Wedgbury 1988; Bird and Strauss 1967).

Free moisture is always present in concrete to some degree. The moisture can exist in vapor form, as in air voids. Internal relative humidity is a measure of the moisture content of hardened concrete that would be in equilibrium with air at the ambient relative humidity. The moisture level below which corrosion will cease has not been definitively established. Below approximately 55% relative humidity, however, there is probably insufficient moisture to sustain corrosion, or the corrosion rate is so slow that it is inconsequential (Tuutti 1982). Enevoldsen et al. (1994) showed that when internal humidity fell below 85%, active corrosion could not be sustained.

Corrosion of nonferrous metals or alloys can result from various phenomena. The metal may be unstable in highly alkaline concrete or in the presence of chloride ions. The former occurs when the concrete is relatively fresh and may be self-limiting. The latter can initiate corrosion, particularly when the metal is in contact with a dissimilar metal. When dissimilar metals are in electrical contact (coupled), a galvanic cell can occur, resulting in corrosion of the more active metal.

More detailed information on corrosion of nonferrous metals is available (Fintel 1984; Woods and Erlin 1987).

3.5.2 *Aluminum*—Aluminum reacts with alkali hydroxides in portland cement paste, resulting in the liberation of hydrogen gas and alteration of the aluminum to various hydrous aluminum oxides (Monfore and Ost 1965).

When aluminum powder is added to portland cement paste, the formation of hydrogen gas can be used to make highly air-entrained (cellular) concrete or mortar, or expansive grouts when used in lesser amounts. In each instance, the desired property is attained when the concrete or mortar is plastic. Because fine aluminum powder reacts completely when the concrete or mortar is plastic, no subsequent volume change occurs after hardening, unlike the continued corrosion of residual aluminum.

Aluminum in solid forms, such as conduit, sheets, or rods, chemically reacts similar to finely powdered aluminum, but at a slower rate due to a lower surface area. The reactions initially generate hydrogen gas, and sometimes small bubbles remain on the aluminum surface. These voids are inconsequential; however, reactions occurring after the concrete hardens result in the formation of hydrous aluminum oxides, such as gibbsite, boehmite, and bayerite, with an attendant in-place volume increase.

Significant corrosion of solid aluminum products can produce two important phenomena:

- Reduction of the aluminum cross section—the corrosion can be sufficiently extensive to completely corrode conduit or pipe walls; and
- Increase of the volume of the corrosion products—sufficient stress can cause the encasing concrete to rupture. A similar phenomenon is responsible for concrete cracking due to corrosion of aluminum posts and balusters (Wright 1955) inserted in concrete and due to aluminum window frames in contact with concrete.

The reported number of cases involving corrosion of aluminum (Copenhagen and Costello 1970; McGeary 1966; ENR 1964; Wright 1955; Wright and Jenkins 1963) is large enough to caution against using aluminum in or in contact with concrete, unless the aluminum is properly coated using certain plastics, lacquers, or bituminous compounds (Monfore and Ost 1965). Anodizing gives no protection (Lea 1971). The addition of calcium nitrite was also found to improve the corrosion resistance of aluminum in contact with concrete (Berke et al. 1988).

3.5.3 *Lead*—Lead in dry concrete does not corrode. In damp or fresh concrete, it reacts with hydroxides in the concrete to form lead oxides (Dodero 1949). If the lead is electrically connected to reinforcing steel, or if part of the lead is exposed out of the concrete, galvanic corrosion can occur and cause accelerated deterioration of the lead. Lead in contact with concrete should be protected by suitable coatings or otherwise isolated from contact to the concrete (Biczok 1964).

3.5.4 *Zinc*—Zinc chemically reacts with hydroxides in concrete; however, the reactions are usually self-limiting and superficial, unless chlorides are present. An exception is when zinc-coated forms are used for architectural concrete so that the early reactions produce disfigured surfaces.

Exposing zinc to chromate solutions, such as by dipping, as required by ASTM A767/A767M, may prevent initial reactions of the zinc and inhibit the formation of hydrogen gas (Boyd and Tripler 1968); however, this dipping was found by others not to be beneficial (Tan and Hansson 2008). In the presence of chlorides, however, the zinc will corrode.

The corrosion products of galvanized steel encased in or in contact with chloride-containing concrete are a combination of zinc oxide and zinc hydroxychloride. The former has an in-place solid volume increase of 50%, and the latter has a

solid volume increase of 300% (Hime and Machin 1993). Under certain circumstances, the volume increase due to the latter can develop sufficient stress to crack the surrounding concrete. The corrosion of the zinc layer subsequently exposes the underlying steel to the chloride environment, and corrosion of the steel will ensue.

Zinc-alloy beams used as joists in certain constructions, galvanized reinforcing steel bars, galvanized corrugated steel used for permanent forms, and galvanized steel ties in masonry, have deteriorated extensively when chlorides are present. The zinc coating corrodes initially, followed by the steel.

Field studies on the performance of galvanized steel in reinforced concrete structures exposed to chlorides in service have yielded conflicting results (Arnold 1976; Cook 1980). In general, galvanizing is an inappropriate means for providing long-term protection for steel in concrete if chlorides are present or will be introduced into the concrete from the environment (Arnold 1976; Griffin 1969; Mange 1957; Stark and Perenchio 1975).

3.5.5 *Copper and copper alloys*—Copper is chemically stable in concrete, except when chlorides or ammonia are present. Ammonia can cause stress corrosion of the copper and early failure under loads. Ammonia is usually environmentally derived and not a normal component of concrete; however, copper pipes embedded in air-entrained concrete have corroded because the air-entraining admixture released small amounts of ammonia, which enhanced and accelerated stress corrosion of the copper pipes (Monfore and Ost 1965). A galvanic cell is created when copper is electrically connected to reinforcing steel in which the steel becomes anodic and corrodes in the presence of chlorides. In a test of copper-clad reinforcement, McDonald et al. (1996) observed that the concrete in the region of the copper had turned a gray-green color and had a higher proportion of unhydrated cement than concrete further away from the copper.

3.5.6 *Stainless steels*—Stainless steels are usually considered noncorrosive in concrete. In the presence of chlorides, however, under certain circumstances, corrosion can occur. Type 316 stainless steel, one of a number of stainless steels that is qualified under ASTM A955/A955M, is the most corrosion-resistant variety commonly specified.

3.5.7 *Other metals*—Chrome-nickel steels, chromium-silicon steels, cast iron, alloyed cast iron, nickel, chrome-nickel, iron-chrome-nickel alloys, Monel metal, stellite, silver, and tin have good resistance to corrosion when used in concrete (Fintel 1984).

Nickel and cadmium-coated steel have good resistance to corrosion when chlorides are not present, but under certain circumstances, they can corrode when chlorides are present. If the coatings are scratched or not continuous, a galvanic cell can develop and corrosion can occur at this location, particularly when chlorides are present (Fintel 1984).

CHAPTER 4—CONSTRUCTION PRACTICES
4.1—Mixing and transporting concrete
Fresh concrete used in the construction of structures containing embedded metals at the time of placement should be a homogeneous mixture of the concreting materials specified in the mixture design. The measuring, mixing, and transporting of the concrete should be in accordance with ACI 301 and the procedures outlined in ACI 304R. To ensure the accurate measurement of materials, batching equipment should meet the requirements of ASTM C94/C94M. Other commonly used references for plant equipment requirements are the *Concrete Plant Standards* (Concrete Plant Manufacturers Bureau 2000) and the "NRMCA Plant Certification Check List" (NRMCA 1999).

When concrete is made at a low *w/cm* approaching 0.30 using high cementitious-material contents, HRWRAs, and silica fume, there may not be enough water to produce concrete with adequate slump until the HRWRA is fully effective (Gaynor and Mullarky 1975). One effective solution is to charge the coarse aggregate, the water, half of the HRWRA, and the silica fume before charging the cement. The cement and coarse aggregate are mixed for approximately 1 minute before the sand is charged. The remaining HRWRA is then added. The delay in charging half of the HRWRA is to avoid the rapid slump loss that is sometimes encountered when the admixture is mixed with the water that initially wets the dry cement. A delay of even 30 seconds in adding the HRWRA will increase its effectiveness. At a *w/cm* of 0.40 and 0.45, these problems are rarely encountered, and the normal charging sequences previously described perform well.

ASTM C94/C94M permits adding tempering water to bring the slump within the desired range when a concrete truck arrives at a job site, but only if the additional water will not cause the specified water content to be exceeded. In any case, good communication and coordination between the construction crew and the concrete producer should minimize problems due to delays between delivery to the job site and discharge of the concrete. Placement crews should not add water or admixtures to the concrete being placed without approval from the engineer responsible for the design of the concrete mixture.

4.2—Placement of concrete and steel
4.2.1 *Formwork*—Concrete formwork should be designed with sufficient strength and rigidity to support loadings and lateral pressures caused by the concrete, equipment, and workers. It also should be sufficiently supported to maintain specified tolerances and cover for the reinforcing steel and to resist excessive deflections that can cause cracking in the concrete. Additional information on formwork can be found in ACI 347.

Irregular formwork surfaces may make concrete consolidation difficult, resulting in surface honeycombing or voids. These may create potential entry points for water and chlorides.

Before placing concrete, the formwork should be cleaned of all construction debris (such as sawdust and wire scraps) and water, snow, and ice, as these loose materials may create voids in the concrete surface.

Stay-in-place metallic formwork is susceptible to corrosion if concrete with corrosive components (such as calcium chloride) is placed, or if corrosive substances penetrate the concrete subsequent to hardening.

4.2.2 *Reinforcing steel*—The cover requirements specified in ACI 318 and ACI 201.2R are minimum cover. The

maximum cover that can be realistically designed in the structure should be used for any concrete member exposed to potentially corrosive environments. As pointed out in Chapter 2, however, the distinction between corrosive and noncorrosive environments is not always obvious, and the engineer, when in doubt, is advised to take the more conservative approach. In regions of congested steel, the spaces between bars should be designed to allow the concrete to be placed while reducing the possibility of voids or honeycombing.

Nonprestressing reinforcing steel bars should be in good condition before placement in the formwork. The surfaces should be free of mud or dirt and less visible contaminants, such as oil. Some mill scale is acceptable on uncoated bars, provided it is tightly adhered. Concrete will typically passivate this surface layer of corrosion, and it is usually not a condition that will cause corrosion in later years.

Epoxy-coated reinforcing bars should follow the guidelines given in Section 3.4. Damaged epoxy bars should be rejected or repaired in accordance with ACI 301 requirements. All flaws should be repaired before concrete placement. Attention should also be given to the reinforcing accessories such as chairs, tie wires, and openings.

Post-tensioned reinforcing steel (multistrand, single wire, or bar) requires thorough attention to detail during placement. For sheathed wire or strand, care should be taken to avoid damaging the sheathing during transportation and handling. Minor tears or punctures of the sheathing should be repaired only with materials furnished or recommended by the fabricator.

All details should be carefully placed to the tolerances shown on the design drawings. Not only should the specified number of tendons be placed in their correct configuration, but the relationship of the reinforcement to other building components should be well coordinated. For example, the configuration of any mechanical or electrical embedded items should be designed to avoid interfering with tendon placements that could cause tight bends. Likewise, congested areas should be avoided so that the concrete can be placed without creating voids or honeycombing. Near end anchorages, the cable sheathing should be cut as close to the anchorage as possible to avoid exposed strands. If it is cut too far back, the sheathing can be patched with materials furnished or recommended by the post-tensioning fabricator.

Reinforcement should be adequately secured and supported in the formwork to maintain the tolerances and cover. An adequate number of ties, chairs, or other accessories should be provided to avoid movement of the reinforcement. After the reinforcement is placed, foot traffic by workers should be limited to minimize walking on reinforcing steel, potentially shifting it out of position.

4.2.3 *Concrete placement*—Concrete should be properly placed to protect the steel components from future corrosion. Workmanship is important, and a worker's attention to the concrete placement will have a great effect on the quality and performance of the concrete member. The guidelines for cold-weather (ACI 306R) or hot-weather protection (ACI 305R) should be considered before any concrete is placed. Chapter 5 of ACI 304R should be followed. During placement, the concrete should be placed so that segregation of aggregate and

mortar is minimized. The formation of voids should be avoided, as they can lead to cracking and loosely placed concrete, resulting in a high porosity. Voids can be avoided with advance planning, proper mixture proportioning, and proper placement techniques. Properly proportioned concrete can be allowed to freefall into place without the use of hoppers, trunks, or chutes when forms are sufficiently open and clear so that the concrete is not disturbed during placement (ACI 304R). Chutes, dropchutes, and tremies can be used in applications such as in tall columns or shafts where this is not possible. Freefall may need to be limited if the reinforcement is relatively congested. Pumping provides several advantages over freefall in some members because the discharge point can be very close to the final point of placement.

Sequencing of the concrete placement is also important. Cold joints result from the partial setting of an earlier concrete layer before a second layer of concrete is placed and should be avoided. Cracking and delamination are likely at cold joints and can provide easier access for corrosion initiators into the concrete. Concrete placements where water is present, such as in underwater structures or in locations where complete dewatering is impossible, should also be sequenced to avoid entrapping water. The actual entrapment of clean water probably will not cause serious corrosion, but it will create a void or honeycombing that would be deleterious.

Monitoring during concrete placement is suggested to maintain concrete quality and to help coordinate the sequencing. For example, any defect in the concrete that can impair its serviceability can be rejected. Ready-mixed loads more than 90 minutes old, excessive delays during placement of successive lifts of concrete, or other potentially harmful practices, should cause the inspector to reject the concrete.

4.3—Consolidation

Much information is available regarding the practices to be followed to achieve proper consolidation, including ACI 309R. ACI 309R includes a general discussion of the importance of consolidation, effects of mixture design and workability, methods and equipment used for consolidation, and recommended vibration practices for various types of construction. The reader is strongly urged to thoroughly study and implement the recommendations of this document, especially for those structures exposed to potentially corrosive environments.

Other good sources of detailed information include ACI 309.1R on the principles of vibration, ACI SP-96, and a report issued by the Federal Highway Administration (FHWA) (Whiting and Tayabji 1988). The latter also includes considerable information on techniques and instrumentation used to monitor consolidation, especially with regard to concrete pavement. Finally, manufacturers of concrete vibrators and other types of consolidation equipment have issued handbooks that can prove beneficial, especially for selecting equipment for a particular job.

4.4—Influence of curing on corrosion of reinforcing steel

4.4.1 *Introduction*—ACI 308R defines curing as "The maintaining of a satisfactory moisture content and temperature in

concrete during its early stages so that desired properties may develop." In normal construction, the desired properties usually include strength, elastic modulus, and freezing-and-thawing resistance. With regard to preventing or delaying the onset of the corrosion of reinforcing steel, the concrete properties that develop as a consequence of curing include high electrical resistivity and impermeability to liquid water, water vapor, chloride ions in solution, oxygen, and carbon dioxide through the improvement of porosity and pore-size distribution.

With adequate and continued temperature and moisture control, not only does the strength of concrete increase, but the porosity decreases, the remaining pores become increasingly smaller, the electrical resistivity becomes higher, and the permeability to both liquids and gases becomes lower. Therefore, through proper curing, the internal resistance that concrete provides against corrosion of reinforcing steel is enhanced (Hansson and Sørensen 1990) Conversely, the corrosion-resisting properties of concrete will not develop to their expected values if adequate curing is not provided.

4.4.2 *Guidelines*—Temperature and moisture conditions should be maintained in the interior and at the surface of concrete as recommended in ACI 305R, 306R, and 308R. The need for such control is directly related to the ambient environmental conditions at the job site. In particular, concern should be raised for thermal control whenever it is anticipated that the air temperature may drop below 50°F (10°C) at any time during the first several days after casting. Moisture-control measures should be put into effect whenever it is anticipated that moisture will evaporate rapidly from the surface of the freshly cast concrete.

In general, such evaporative conditions are influenced by the temperature and the relative humidity of the air, the temperature of the concrete, and the speed of the wind blowing across the surface of the concrete. Such conditions often prevail in hot, dry environments, but the need for moisture control can be acute during winter concreting due to the combination of strong winds, dry air, and heating of the concrete.

ACI 306R discusses controlling the temperature of freshly cast concrete during cold weather. Depending on weather conditions and characteristics of the concrete mixture, it may be necessary to heat the mixture water, aggregates, or both, or to modify conditions at the point of placement using shelters, heated enclosures, or insulating blankets.

ACI 308R discusses controlling the moisture content of freshly cast concrete. Methods are described for retarding surface evaporation such as applying liquid membrane-forming compounds, plastic sheets, and reinforced paper. More expensive, and in some cases more effective, methods go beyond the prevention of evaporation and actually provide additional moisture to the surface of the concrete. These methods include ponding or immersion, fog spraying or sprinkling, and using continuously soaked burlap.

The techniques that may be required under hot and dry conditions of concrete placement are found in ACI 305R. In such cases, it may be necessary to erect sunshades, wind screens, or both, in addition to using fog-spraying or other techniques, for cooling the surface of the concrete.

CHAPTER 5—EVALUATION AND PROTECTION OF IN-SERVICE STRUCTURES
5.1—Types of structures susceptible to corrosion-related deterioration

In structures where the steel is in direct contact with concrete, steel corrosion mainly occurs in concrete exposed to chlorides or concrete that is severely carbonated. In areas where deicing salts are used for winter maintenance of highways and roads, bridges and their substructures are particularly vulnerable to embedded-steel corrosion because of the direct application of chloride salts to the bridge-deck surface. These salts can readily penetrate to the depth of reinforcing steel at crack locations in as little as 1 year (Darwin et al. 2006). In unprotected structures, corrosion can occur within the first decade. The deicing salts also penetrate expansion joints and other discontinuities in the bridge deck so that chloride-ion contamination of the substructure and subsequent corrosion of embedded steel is initiated. Substructures of bridges that pass over highways, barrier walls, and retaining walls adjacent to the highway are at risk from the spray of saltwater caused by vehicles on the highway.

Parking garage structures in these environments are also susceptible to chloride-ion contamination, even though deicing salts are not necessarily applied directly to the parking-deck surface. Chloride-laden water and slush clinging to the underside of vehicles entering the garage can drop off and pond on the parking-deck surface.

All reinforced concrete structures that are in contact with seawater or exposed to wind-driven ocean spray are at risk from corrosion-induced deterioration caused by the ingress of chloride ions.

If precautions are not taken, conventionally reinforced or prestressed concrete structures, such as cooling towers, can suffer carbonation corrosion-related damage. The combination of relatively high temperatures and water mist present in these structures accelerates carbonation of the concrete, resulting in an early breakdown in the corrosion-defense mechanism of the concrete, even in the absence of chloride ions or other aggressive ions. Also, evaporation within cooling towers will concentrate any dissolved material. Any aggressive salts that are concentrated can cause corrosion.

Serious corrosion damage can occur as a result of cast-in chlorides, even when the service environment is considered dry. In one case, the structure was built using a high-*w/cm* lightweight-aggregate concrete that contained calcium chloride as an accelerating admixture. The construction was somewhat unusual and incorporated a beam and slab system where the beams were very wide and approximately 15 in. (381 mm) deep. Many floors of this building were covered with vinyl tiles, which acted as vapor barriers preventing the evaporation of water from the top surfaces of the slabs. This seemingly dry environment for the floor system turned out not to be the case. Serious delamination occurred approximately 8 years after construction because of the combination of chloride ions from the accelerating admixture and the excess water in the concrete that was partially trapped by the floor tiles.

5.2—Evaluation of in-service structures

Engineers are constantly faced with the challenge of detecting concrete deterioration, corrosion in its early stages in undamaged areas, and providing appropriate maintenance in the form of repair and long-term protection for a structure to extend its service life. Repair is defined as a process "to replace or correct deteriorated; damaged; or faulty materials, components, or elements of a structure" (ACI 546R). Repair methods alone, however, generally do little to address the cause of deterioration. Therefore, in the case of reinforcing steel corrosion, simple repairs typically fail prematurely when nothing is done to mitigate or stop the primary deterioration mechanism.

It is necessary to rehabilitate structures to effectively combat corrosion of reinforcing steel in concrete. ACI Concrete Terminology (ACI 2010) defines rehabilitation as "The process of repairing or modifying a structure to a desired useful condition." Because rehabilitation includes addressing the deterioration process itself, the additional service life is typically much greater than for repairs made without proper diagnosis of the underlying causes. Consequently, ACI 546R provides a repair methodology that requires evaluation of the existing condition of the concrete structure to be repaired, determination of the mechanism or mechanisms that caused the problem, and selection of the appropriate repair materials and methods.

Assessment and selection of a cost-effective rehabilitation option for a deteriorated concrete structure requires:
- Obtaining information on the condition of the structure and its environment;
- Applying engineering analysis to the information;
- Identifying options that are viable for that particular structure;
- Performing life-cycle cost analyses; and
- Defining the most cost-effective alternative for rehabilitating the structure.

The first step involves reviewing structural drawings, previous survey reports, and available information on the environmental conditions at the site. Acquired information should include:
- The location, size, type, and age of the structure;
- Any unusual design features;
- Environmental exposure conditions, including temperature variations, marine environment, and precipitation;
- Reinforcing-steel details;
- Type of reinforcement, including uncoated mild steel, epoxy-coated steel, galvanized steel, and prestressing steel;
- Repair and maintenance history; and
- Presence of corrosion-protection systems.

This information is then used to develop a specific scope for a thorough condition survey of the structure. In addition to a visual examination of structural distress, a typical condition survey involves corrosion evaluation and concrete evaluation. The objective of the condition survey is to determine the cause, extent in terms of total area affected, and magnitude in terms of severity of the problem. Based on the specific scope developed for the target structure, some or all of the following procedures listed are used in the condition survey.

Reinforcement corrosion evaluation methods include (ACI 365.1R):
- Visual inspection, including crack and spall surveys;
- Delamination survey;
- Depth of concrete cover measurements;
- Chloride-ion content analyses at various locations and depths;
- Depth of carbonation testing;
- Electrical continuity testing;
- Concrete relative-humidity and resistivity measurements;
- Corrosion-potential mapping;
- Corrosion-rate measurements;
- Determination of cross section loss on reinforcing steel; and
- Measurement of concrete pH, corrosion product pH, or both.

Further evaluation test methods specific to the concrete include:
- Visual inspection;
- Petrographic analysis;
- Compressive strength testing;
- Chloride permeability testing; and
- Measurement of specific gravity, absorption, and voids.

ACI 222R contains detailed information regarding these test and survey techniques.

The condition survey is followed by analysis of the field and laboratory test results and selection of potential rehabilitation alternatives based on technical viability and desired service life. The next step in the process is to conduct a life-cycle cost analysis (LCCA). LCCA compares and evaluates the total costs of competing solutions based on the anticipated life of each solution and the desired service life of the structure (Purvis et al. 1999; Genge 1994; Ehlen 1999, ACI 365.1R). The value of a potential solution includes not only consideration of what it costs to acquire it, but also the cost to maintain it over a specified time period. To perform an LCCA, one should estimate the initial cost, maintenance cost, and service life for each rehabilitation alternative being considered. Finally, based on the LCCA results, the most cost-effective rehabilitation strategy is selected.

5.3—Barrier systems for concrete

5.3.1 *Introduction*—Because the corrosion of reinforcing steel usually requires the ingress of water, aqueous salt solutions, oxygen, and carbon dioxide into concrete, protecting concrete with barrier systems is a potentially effective anticorrosive practice. Four general types of barriers are used (ACI 515.1R):
1. Waterproofing;
2. Damp-proofing;
3. Protective; and
4. Paint.

5.3.2 *Waterproofing and damp-proofing barriers*—A waterproofing barrier consists of materials applied to the concrete surface to block the passage of liquid water and significantly reduce the passage of water vapor. Liquid and/or vapor are driven through concrete by hydrostatic head, vapor gradient, or capillary action. Porous concrete, cracks, or structural defects and joints that are improperly

designed or constructed will increase the passage of liquids and vapors. ACI 515.1R states that "membrane water-proofing is the most reliable type of barrier to prevent liquid water under a hydrostatic head from entering an underground structure." To protect steel reinforcement from chloride ions, the waterproofing membrane is placed on the same side as the hydrostatic pressure. This is called a positive-side system.

Waterproofing barriers traditionally consist of multiple layers of bituminous-saturated felt or fabric cemented together with hot applications of coal-tar pitch or asphalt.

Cold-applied systems use multiple applications of asphaltic mastics and glass fabrics, or use liquids, sheet elastomeric materials, and preformed rubberized-bituminous sheets. Information on specific materials, compatibility of membranes with concrete, surface preparation of concrete, application methods, and performance of various membrane systems is given in ACI 515.1R and the National Roofing Contractors Association's Roofing and Waterproofing Manuals. The bitumens (asphalt or coal-tar pitch) used in hot-applied barriers are low-strength and need to be reinforced with fabrics or felts to withstand the stresses caused by temperature changes. Similarly, the mastic and emulsions used in the cold-applied barriers do not have enough strength and should be reinforced with fabric.

A damp-proofing barrier resists the passage of water in the absence of hydrostatic pressure. It will not be effective if subjected to even an intermittent head of water, and it will not bridge cracks. Because of these limitations, this system is not considered further.

5.3.3 *Protective barriers*—Protective-barrier systems protect concrete from degradation by chemicals and subsequent loss of structural integrity, prevent staining of concrete, or protect liquids from being contaminated by concrete (ACI 515.1R). At service temperatures, materials for protective barriers should (ACI 515.1R):

- Not swell, dissolve, crack, or embrittle upon contact with pertinent liquids or vapors;
- Prevent the permeation or diffusion of chemicals that are able to cause a loss of adhesion between the barrier and the concrete; and
- Have sufficient abrasion resistance to prevent it from being damaged during placement.

Acid-resistant brick sheathing with chemical-resistant mortar joints applied over the barrier material—for example, built-up asphalt membrane—is necessary to help prevent damage to the relatively fragile barrier material caused by mechanical abuse or exposure to excessive temperature. Damage to the floor and walls could occur due to cutting forces from mechanical action or creep from sustained loading. ACI 515.1R provides additional information including factors that affect the adhesion of a protective barrier to concrete and the effects of a concrete structure, such as cracks in the concrete reflecting through the barrier and foundation movements on the performance of protective barriers.

5.3.4 *Decorative paint-barrier and penetrating sealer systems*—A decorative paint-barrier system stabilizes or changes the appearance or color of a concrete surface. Such a system can resist the diffusion of gases such as water vapor, carbon dioxide, and oxygen into the concrete. Decorative paints can be applied to exterior concrete surfaces above grade. The types of paints used are usually water-based portland-cement paints, water-based polymer latex paints, polymer paints (epoxy, polyester, or urethane), and silane/siloxane-based coatings. Silane/siloxane coatings are water-repellent systems and usually are clear coatings (thus not true decorative paint-barrier systems). Silane coatings have been the subject of extensive research (Hewelett 1990) and are commonly used in practice. They are effective in resisting chloride-ion penetration, but they have no resistance to the ingress of carbon dioxide (Swamy and Tanikawa 1990). Instead, acrylic-elastomeric products are available that minimize the ingress of carbon dioxide.

5.3.5 *Degradation factors and durability of barrier materials*—Various degradation factors can reduce the ability of the barrier (waterproofing and protective) materials and sealant materials to perform properly throughout their design life. These include exposure to ozone, ultraviolet radiation, microbials, organic solvents, and nuclear radiation (Davis and Sims 1983; Hewelett 1990; Mathey and Rossiter 1983; Schnabel 1981; Swamy and Tanikawa 1990; Traxler 1964). Also, elevated temperatures can result in significant creep in a bitumen-based system (Mathey and Rossiter 1983).

5.4—Admixtures that extend life of reinforced concrete structures exposed to chloride environments

5.4.1 *General*—This section deals with admixtures that are added to concrete during batching to control the time-to-corrosion initiation, the rate of chloride-induced corrosion of fully embedded steel reinforcement, or both. These admixtures are effective in providing protection against chlorides from external sources, such as seawater and deicing chemicals. Any admixture used for corrosion protection of steel reinforcement should be tested to ensure that it does not adversely affect concrete properties.

Admixtures that extend the life of reinforced concrete structures exposed to chloride environments were reviewed by Treadaway and Russell (1968); Craig and Wood (1970); Griffin (1975); Slater (1983); Berke and Roberts (1989); Berke (1991); and Nmai and Attiogbe (1992).

5.5—Cathodic protection

5.5.1 *Introduction*—Cathodic protection (CP) is a widely used and effective method of corrosion control. CP reduces corrosion by shifting the potential of the metal by supplying a current from an external source to counteract the naturally occurring corrosion current.

Historically, its greatest use has been on underground pipelines and seagoing ship hulls, but these are applications with criteria and requirements that differ from those for the concrete structures considered within the scope of this document. Although CP has application in new building construction, it has been most extensively used in conjunction with rehabilitation of existing structures. Only spalled and delaminated areas of concrete structures need to be repaired before applying CP to protect the reinforcing steel. Structurally

sound concrete, even if high in chloride content, need not be replaced.

CP can be implemented using impressed-current systems and galvanic (sacrificial-anode) systems. Impressed-current CP drives a direct current at low voltage from an anode material that is consumed at a controlled rate, through the concrete or the reinforcing steel. The direct current is supplied by an external power source, most often a rectifier that converts alternating current to direct current. Recently, solar power and specially-designed batteries were used as external power sources, but these alternatives are still considered experimental (Lasa et al. 1994).

Galvanic CP is based on the principle of dissimilar metal corrosion and the relative position of specific metals in the galvanic series. More active metals in the galvanic series protect more noble metals. The most common galvanic anodes are zinc, aluminum, magnesium, and various alloys of these metals. As with impressed current systems, current flows from the anode, through the concrete, to the reinforcing steel. In the case of galvanic systems, however, no external power source is needed because the current is driven by natural potential differences between the anode and the reinforcing steel. As a result, galvanic CP systems typically require less maintenance than impressed current systems; however, the anodes may need more frequent replacement. Galvanic systems were shown to work in marine zones that are submerged or subjected to salt spray (Kessler and Powers 1995; Sagues 1995).

CP systems for reinforced concrete consist of current-distribution hardware (anode), instrumentation (reference cells, reinforcing-bar probes, and null probes), electrical connections to the reinforcing steel and anodes, and a direct-current power source (for impressed-current CP systems only). Components inherent in the structure include the reinforcing steel (cathode) and the concrete-pore solution, which serves as a conductive electrolyte. Several cost-effective and durable CP systems are available for specific applications. Bennett et al. (1993) summarize the estimated cost and service life of various CP systems for reinforced concrete structures. Development of additional CP systems remains an active area of research.

Cathodic protection has been applied on a routine basis on normal reinforcing bars; however, two generalized problems can arise when applying CP to prestressing steel. The first is hydrogen embrittlement, and the second is loss of bond between the steel and concrete in situations with excessive polarization. Experiments to determine if the latter is significant have not yet been performed. Several investigations have addressed the susceptibility of prestressing steel to hydrogen embrittlement under CP of prestressed concrete (Galvez et al. 1985; Hartt et al. 1989; Hope 1987; Kliszowski and Hartt 1996; Kumaria and Hartt 1990; Parkins et al. 1982; Scannell and Hartt 1987; Young 1992).

For smooth prestressing steel with a low (0.02% weight) chromium content, a conservative lower potential limit of –0.974 volts versus copper-copper sulfate electrode (CSE) has been defined (Hartt et al. 1989; Kliszowski and Hartt 1996). Microalloyed prestressing steel, which often contains chromium at a concentration near 0.24% by mass, may require a more conservative lower potential limit (Kliszowski and Hartt 1996). The amount by which the prestressing steel cross section has been reduced at sites of localized corrosion has been proposed as a parameter for assessing the appropriateness of CP for a specific prestressed concrete structure or component. Although research continues in this area, the first full-scale application of CP to prestressed concrete bridge components in the U.S. has been completed (Scannell et al. 1994, 1995).

A limitation of CP-impressed current, which needs to be considered by designers, is the risk of stray current and unintended corrosion of discontinuous steel components.

5.5.2 *Performance history*—Cathodic protection applied to reinforced concrete has a performance history of more than three decades. Since its first major application in 1973 (Stratfull 1974), more than 275 bridge structures have been cathodically protected throughout the U.S. and Canada (Broomfield and Tinnea 1992; Eltech Research Corp. 1993). By the late 1980s, CP had been applied to a total of approximately 9,000,000 ft^2 (840,000 m^2) of above-ground concrete surfaces (Eltech Research Corp. 1993). Most of the applications are on bridge decks, but installations on bridge-substructure components are increasing (Kessler and Powers 1995; Rog and Swiat 1987; Sagues 1995; Scannell and Sohanghpurwala 1993).

A survey conducted in the early 1990s concluded that 90% of CP installations were operating satisfactorily (Pastore et al. 1991). According to another report (Zivich et al. 1992), CP systems installed on 14 bridges 16 to 18 years ago continue to operate successfully. The FHWA is continuing to evaluate the long-term performance of CP on bridge structures throughout the U.S. and Canada.

Since the mid-1980s, the application of CP technology for corrosion control has been extended to several other types of reinforced concrete structures, including parking garages, reinforced concrete buildings, wharves, and docks (Broomfield et al. 1987; Daily 1987; Schutt 1992; Tighe and Ortlieb 1991).

5.5.3 *Selection of CP candidate*—Almost any reinforced concrete structure of virtually any geometry, or portion thereof, can be cathodically protected. Even though CP has the ability to control corrosion, existing structures should be considered individually with regard to the need for and applicability of CP. General guidelines for determining if a given structure is a candidate for CP are:

• The projected remaining life of the structure should be greater than or equal to 10 years (cathodic protection is usually most cost effective when a long-term rehabilitation is desired);

• The majority of the reinforcing steel should be electrically continuous;

• A significant percentage of potentials should be active or more negative than –0.35 volts with respect to a copper-copper sulfate reference electrode. Interpretation of half-cell potentials, however, may vary with the type of structure involved and exposure conditions;

• Total acid-soluble chlorides should be more than 0.20% by mass of cement at a reinforcing bar depth over 20%

of the components' surface area, or expected to reach this level within 5 years;

• The concrete cover over reinforcing steel should be more than 1/2 in. (12.5 mm);

• The concrete distress should be caused solely by corrosion of reinforcing steel. For example, if cyclic freezing and thawing is a problem, CP alone may not be appropriate. In addition, if alkali-silica reaction is encountered, or possible, CP may not be appropriate; and

• The structure should be close to AC power if impressed-current CP is anticipated (solar power and specially designed batteries may provide acceptable alternatives to AC power).

In addition, several structural and civil engineering considerations can combine to indirectly influence the decision to apply CP. If CP is chosen, then another determination should be made to select the most appropriate system for the conditions encountered.

5.5.4 *Design*—The design of a CP system for a reinforced concrete structure depends on numerous parameters, including:

• The dimensions and as-built plans of the structure;

• The pattern and location of the embedded reinforcing steel;

• The results from a condition survey of the structure, that is, structural analyses, chloride-ion concentrations, concrete cover, electrical continuity of the reinforcing steel and other metallic fixtures, location and extent of delaminations and spalls, petrographic analysis of the concrete, entrained air-void parameters, and half-cell potentials;

• The availability, location, and type of AC power;

• The specified design life of the CP system;

• Repair and maintenance history. For example, type of patching material, area of repairs completed with epoxy-coated reinforcing bar, and type of bonding agent used with patch, if any; and

• An estimate of the direct current required to achieve CP levels.

Once the current required for protection has been calculated and the anode material selected, various other parameters should be considered in the design of the CP system, including zone size, voltage drop, rectifier sizing, proximity of anode to steel, interference corrosion, codes and standards, and specifications and drawings.

More details regarding the design of various impressed-current CP systems are provided in Bennett et al. (1993) and AASHTO-AGC-ARTBA Task Force #29 (1994). CP systems should be designed by personnel specializing in CP design for reinforced concrete structures.

5.5.5 *Installation and inspection*—For the successful operation of CP systems, all materials and equipment should be installed in accordance with the specifications, drawings, and manufacturer's recommendations. Detailed information regarding installation of impressed current CP systems is provided in AASHTO-AGC-ARTBA (1994) and Bennett et al. (1993). Testing and inspection should be conducted throughout construction to ensure that the design and manufacturer's specifications have been followed. A detailed construction inspection guide is available from the FHWA (1995).

5.5.6 *Testing and energizing*—Anodes, instrumentation, wiring, and all other system components should be tested to verify that they are in good working order. Data for future monitoring of the CP system should be collected, then the CP system energized. The effectiveness of a CP system is only as good as the criteria used to establish the protection level and the monitoring methods used to evaluate the criteria. Details on protection criteria and monitoring methods are provided in NACE RP0290 and Bennett (1994).

5.5.7 *Monitoring and maintenance*—All CP systems should be monitored periodically and maintained as required for acceptable long-term performance. Monitoring methods and frequency and maintenance information are provided in several publications (AASHTO-AGC-ARTBA Task Force #29 1994; Bennett et al. 1993; Eltech Research Corp. 1993; NACE RP0290).

5.6—Electrochemical chloride extraction

5.6.1 *General*—The concept of applying electrical current to concrete to move chloride ions away from reinforcement has been known for many years. Studies in the 1970s (Morrison et al. 1976; Slater et al. 1976) indicated that electrochemical extraction of chloride ions was a promising rehabilitation technique, but it might induce undesirable effects on the concrete and steel, such as increased permeability, reduced bond to reinforcement, alkali-aggregate reactions, or thermally induced cracking. In recent years, these concerns have been addressed and practical techniques for electrochemically treating corrosion-damaged structures have been developed.

Electrochemical chloride extraction (ECE) consists of applying an anode and an electrolyte to the surface of a reinforced concrete structure and passing current between the anode and the reinforcement, which acts as the cathode. The technique is similar to CP but differs in two important respects: ECE is a treatment process and not a permanent installation, and the current density is approximately 100 times that used in most CP installations.

Although the technique is commonly termed electrochemical chloride extraction, not all the chlorides are removed from the concrete, nor do they have to be for effective treatment. Chloride ions are moved away from the steel and some are removed from the concrete. At the same time, hydroxyl ions are generated at the steel surface so that the chloride-hydroxyl ion ratio (Cl^-/OH^-) is substantially reduced. Criteria for measuring effectiveness should be based on the reduction in corrosion activity or the extension of service life rather than the proportion of chlorides removed from the concrete.

5.6.2 *Extraction techniques and efficiency*—Two basic anode systems have been used, though combinations of the two are possible. The first is a commercial system developed in Norway. It consists of a cellulose-fiber spray applied to the concrete, a consumable steel-mesh anode, and a second layer of cellulose fiber (Manning and Pianca 1991). The second system was developed under contract at the Strategic Highway Research Program (Bennett et al. 1993) and consists of an anode blanket forming a sandwich of four

layers: a highly absorbent layer in contact with the concrete, a free-draining geotextile material to drain acid formed at the anode, a catalyzed-titanium mesh anode, and a high-strength geotextile material to hold the other materials in position. The electrolyte, usually lime water, is normally recirculated by a pump and appropriate hardware to ensure that concrete in the area of treatment remains wet. Where site conditions permit, regular wetting of the anode system is an alternative to continuous recirculation of the electrolyte. In one installation where the concrete was exhibiting alkali-silica reactivity, a 0.2 molar lithium-borate buffer was added to the electrolyte in an attempt to not aggravate damage to the concrete (Manning and Ip 1994).

Chloride removal is relatively rapid at first, but the rate of removal declines quickly as the transference number decreases and the circuit resistance increases. The transference number is the ratio of the quantity of electricity carried by the chloride ions to the total quantity of electricity passed. The transference number is proportional to the chloride content of the concrete, but relatively independent of the current density and only slightly affected by temperature (Bennett and Schue 1990). Typical parameters for ECE treatment, according to laboratory and field experience conducted since 1989, can be summarized as follows.

The maximum voltage that can be applied is limited by electrical codes to 40 to 50 volts, depending on jurisdiction. Chloride removal should not be undertaken without knowledge of the steel surface area, and treatment zones should be designed so that the maximum current density on the steel surface is in the range of 0.09 to 0.4 A/ft^2 (1 to 4 A/m^2). The total charge passed should be 56 to 111 A·h/ft^2 (600 to 1200 A·h/m^2), which means that treatment times will be approximately 10 to 80 days, assuming the average current density is 60% of the maximum. Electrochemical-chloride extraction treatment is most suitable for components that remain in service while the treatment is in progress. In the case of highway structures, substructure components are of primary interest. In the case of parking structures, it is possible to take parking bays out of service so that the time for treatment is not a serious constraint. Model studies and practical experience have shown that the ideal candidate structure is contaminated with chlorides, but the majority of chloride ions have not penetrated deeper than the reinforcement, and corrosion has not progressed to the stage where widespread delamination of the concrete has occurred (Manning and Ip 1994). Where the structure remains exposed to chlorides after treatment, sealing the concrete is advisable to prevent a further increase in the chloride content of the concrete.

The service life extension resulting from electrochemical chloride extraction is expected to be long term, but is not fully known at this time, as the treatment has only been in use for 20 years. Projects completed in the late 1980s remain passive to date.

5.6.3 *Realkalization*—Carbonation-induced corrosion results from the interaction of atmospheric gases with concrete and the protective highly alkaline pore solution pH of the concrete is gradually neutralized from approximately pH 12 to 13.5 to pH 8.5, as described in Section 2.2.4.2 of ACI 222.1. This deterioration mechanism is addressed with the realkalization of the concrete cover using an electrochemical treatment similar to ECE. Realkalization differs from ECE in that the system is kept wet using an alkaline electrolyte. The alkaline electrolyte is drawn into the carbonated concrete by a combination of ion migration and electro-osmosis. The treatment time for realkalization is typically 3 to 7 days and the typical total charge varies from 7 to 19 A·h/ft^2 (75 to 200 A·h/m^2). Realkalization is used to mitigate corrosion due to carbonation without requiring the removal and replacement of the carbonated concrete and is therefore often considered as a preservation technique for historic concrete structures (Bertolini et al. 2008).

CHAPTER 6—REFERENCES
6.1—Referenced standards and reports
The standards and reports listed below were the latest editions at the time this document was prepared. Because these documents are revised frequently, the reader is advised to contact the proper sponsoring group if it is desired to refer to the latest edition.

American Association of State Highway and Transportation Officials
HB-17   Standard Specifications for Highway Bridges, 17th Edition
M284M/M284   Standard Specification for Epoxy-Coated Reinforcing Bars: Materials and Coating Requirements

American Concrete Institute
201.2R   Guide to Durable Concrete
211.1   Standard Practice for Selecting Proportions for Normal, Heavyweight, and Mass Concrete
212.3R   Chemical Admixtures for Concrete
212.4R   Guide for the Use of High-Range Water-Reducing Admixtures (Superplasticizers) in Concrete
222.1   Provisional Standard Test Method for Water-Soluble Chloride Available for Corrosion of Embedded Steel in Mortar (withdrawn)
222R   Protection of Metals in Concrete Against Corrosion
224R   Control of Cracking in Concrete Structures
224.3R   Joints in Concrete Construction
225R   Guide to the Selection and Use of Hydraulic Cements
232.1R   Use of Raw or Processed Natural Pozzolans in Concrete
232.2R   Use of Fly Ash in Concrete
233R   Slag Cement in Concrete and Mortar
234R   Guide for the Use of Silica Fume in Concrete
301   Specifications for Structural Concrete
304R   Guide for Measuring, Mixing, Transporting, and Placing Concrete
305R   Guide to Hot Weather Concreting
306R   Guide to Cold Weather Concreting
308R   Guide to Curing Concrete

309R	Guide for Consolidation of Concrete
309.1R	Report on Behavior of Fresh Concrete during Vibration
318	Building Code Requirements for Structural Concrete and Commentary
345R	Guide for Concrete Highway Bridge Deck Construction
347	Guide to Formwork for Concrete
357R	Guide for the Design and Construction of Fixed Offshore Concrete Structures
362.1R	Guide for the Design of Durable Parking Structures
365.1R	Service Life Prediction
423.7	Specification for Unbonded Single-Strand Tendon Materials and Commentary
423.9M	Test Method for Bleed Stability of Cementitious Post-Tensioning Tendon Grout
515.1R	A Guide to the Use of Waterproofing, Damp-proofing, Protective, and Decorative Barrier Systems for Concrete (withdrawn)
546R	Concrete Repair Guide
548.1R	Guide for the Use of Polymers in Concrete
548.2R	Guide for Mixing and Placing Sulfur Concrete in Construction (withdrawn)
SP-96	Consolidation of Concrete

ASTM International

A615/A615M	Specification for Deformed and Plain Carbon-Steel Bars for Concrete Reinforcement
A706/A706M	Specification for Low-Alloy Steel Deformed and Plain Bars for Concrete Reinforcement
A767/A767M	Specification for Zinc-Coated (Galvanized) Steel Bars for Concrete Reinforcement
A775/A775M	Specification for Epoxy-Coated Steel Reinforcing Bars
A934/A934M	Specification for Epoxy-Coated Prefabricated Steel Reinforcing Bars
A955/A955M	Specification for Deformed and Plain Stainless-Steel Bars for Concrete Reinforcement
A996/996M	Specification for Rail-Steel and Axle-Steel Deformed Bars for Concrete Reinforcement
A1035/A1035M	Specification for Deformed and Plain, Low-Carbon, Chromium, Steel Bars for Concrete Reinforcement
C94/C94M	Specification for Ready-Mixed Concrete
C150/C150M	Specification for Portland Cement
C260/C260M	Specification for Air-Entraining Admixtures for Concrete
C494/C494M	Specification for Chemical Admixtures for Concrete
C618	Specification for Coal Fly Ash and Raw or Calcined Natural Pozzolan for Use as a Mineral Admixture in Concrete
C989	Specification for Slag Cement for Use in Concrete and Mortars
C1240	Specification for Silica Fume Used in Cementitious Mixtures
C1602/C1602M	Specification for Mixing Water Used in the Production of Hydraulic Cement Concrete
D3963/D3963M	Specification for Fabrication and Jobsite Handling of Epoxy-Coated Steel Reinforcing Bars

International Organization for Standardization

8044	Corrosion of Metals and Alloys—Basic Terms and Definitions

NACE International

RP0290	Standard Recommended Practice—Impressed Current Cathodic Protection of Reinforcing Steel in Atmospherically Exposed Concrete Structures

National Roofing Contractors Association

Waterproofing Manual
Roofing Manual: Membrane Roof Systems
Roofing Manual: Metal Panel and SPF Roof Systems
NRCA Roofing Manual: Steep-Slope Roof Systems
NRCA Roofing Manual: Architectural Metal Flashing, Moisture Control and Reroofing

These publications may be obtained from these organizations:

American Association of State Highway and Transportation Officials
444 N. Capitol St. NW
Suite 249
Washington, DC 20001
www.transportation.org

American Concrete Institute
38800 Country Club Drive
Farmington Hills, MI 48331
www.concrete.org

ASTM International
100 Barr Harbor Drive
West Conshohocken, PA 19428
www.astm.org

International Organization for Standardizations
1, Rue de Varembé
Case postale 56
CH-1211, Genéva 20
Switzerland
www.iso.org

NACE International
1440 South Creek Drive 77084
Houston, TX 77084
www.nace.org

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