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

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Corrosion of metals in concrete is a serious problem throughout the world. In many instances, corrosion can be avoided if proper attention is given to detailing, concrete materials and mixture designs, 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 design; permeability; reinforcing steel; water-cementitious material ratio.

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FOREWORD

This guide represents a compendium of technology to combat the problems of corrosion and is arranged into four major chapters. Chapter 2 discusses the most important design considerations pertinent to corrosion, including environmental factors, performance of particular structural types, and the influence of particular 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 many changes that concrete undergoes as it is mixed, transported, placed, consolidated, and cured. Chapter 5 describes a variety of 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 one of the most serious types of deterioration that can affect concrete in service. Corrosion can be seen in parking structures, marine structures, industrial plants, buildings, highway bridges, and pavements. In the United States, about 173,000 bridges on the interstate system are structurally deficient or functionally obsolete, in part due to deterioration caused by corrosion of reinforcing steel (Bhide 1999). 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 already 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, however, can occur 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 these factors.

The passive film on steel embedded in concrete forms as a result of the high alkalinity of concrete pore water. Several conditions can disrupt the stability of this passive film, resulting in the corrosion of steel in the presence of adequate moisture and oxygen. From a civil engineering point of view, the presence of a sufficient concentration of chloride ions and a reduction in pH as a result of carbonation of the concrete at the steel surface are the two conditions of most concern.

Sources of chloride ions in excess of the quantity required for corrosion include admixtures containing chlorides at the time of batching, chloride-bearing aggregates, or saline as mixing water. These sources of chloride ions usually can be controlled by judicious selection of the concrete mixture ingredients. Other major sources, which are not as easily controlled or quantified, include the ingress of chloride ions from either deicing salts or a marine environment. In the latter case, wind-borne spray also becomes a source of chloride ions for concrete structures that are located some distance from the ocean, generally within 5 miles (10 km).

Carbonation is the result of a chemical reaction between carbonic acid, formed by the dissolution of atmospheric carbon dioxide, and calcium hydroxide within the cement-paste phase of concrete. This reaction causes a significant reduction in the concentration of hydroxyl ions, resulting in a pH value that no longer supports the formation and stabilization of the passive layer on the steel surface. Carbonation is a time-dependent phenomenon that starts from the surface of the concrete and penetrates inward. Carbonation progresses slowly in concrete with low porosity paste; therefore, concrete at the level of the embedded steel generally is not carbonated during the design life of the structure. In concrete with more porous paste, carbonation can progress fairly rapidly. This cause of steel corrosion can be very important, particularly in warm, moist regions where carbonation is accelerated.

Once corrosion begins, it is aggravated by factors such as moisture in the environment and high temperatures. Cracking, stray currents, and galvanic effects can also aggravate corrosion. Other causes of corrosion include steel directly exposed to the 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; and Tremper, Beaton, and Stratfull

1958). The design considerations relevant to corrosion protection depend on the type of structure and, to a significant degree, its environment and intended use. Certain minimum measures, which are discussed later in this chapter (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, however, 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 and substructure of a concrete bridge for increased corrosion resistance are knowing the potential for chloride ions in service and the degree of protection required. In theory, the design considerations for a bridge located in a semi-arid region of the United States, such as parts of Arizona, should be different from those for a bridge located in either Illinois or on the coast of Florida. ACI 318, ACI 345R, and the American Association of State Highway and Transportation Officials (AASHTO) Specifications for Highway Bridges (AASHTO 1998) recognize this and contain special requirements for concrete structures exposed to chloride ions in service.

There can be differences in interpretation, however, when applying these provisions for corrosion protection of bridge structures. Generally, for exposure to deicing chemicals, the top mat of reinforcement is more susceptible than the bottom mat to chloride-induced corrosion and, therefore, acts as the anode with the bottom mat as the cathode in macrocell corrosion. The AASHTO bridge specifications recognize this and require greater concrete cover for the top mat of reinforcement. The basic premise of chloride-ion exposure, however, 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 substructure has sometimes been overlooked. Chloride-contaminated water can leak through expansion and construction joints and cracks onto substructure pier caps, abutments, and piers, which can lead to corrosion of steel in these components. Additionally, snow-removal operations can pile chloride-containing snow around piers, while piers located in marine tidal splash zones are continuously subjected to wetting-and-drying cycles with chloride-laden seawater. To design a bridge deck and substructure to ensure adequate corrosion protection over its intended service life of 75 years, as required by the AASHTO Bridge Design Specification, 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. And unlike bridge decks, parking structures, except for exposed roofs, are not rinsed by precipitation. Moreover, drainage provided in parking decks is quite often either inadequate or does not function properly.

Similar to a bridge, design considerations pertinent to corrosion protection of a parking structure 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 that for one constructed in cold climates, where deicing salts are heavily used.

A parking structure located in a northern or mountainous climate where deicer chemicals are used should be provided with additional corrosion-protection measures for all structural components. 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 highly probable. ACI 362.1R contains further recommendations.

2.1.3 *Industrial floors*—Design considerations necessary for corrosion protection of industrial floors depend largely on the type of expected exposure. 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*—The primary issue regarding satisfactory corrosion protection of concrete façades, such as architectural precast panels, is knowing the expected environmental exposure. The proximity of façades to heavily industrialized areas and geographical location is of particular importance. Some cities in the United States have higher levels of carbon monoxide, carbon dioxide, and pollutants from industrial smoke discharge, which can lead to a greater rate of concrete carbonation.

In some cases, concrete façades are exposed to chloride-induced corrosion. A typical example is of parking structure façades when chloride-laden snow piled at the edge of the structure melts and drips down the side of the structure. Not only is the steel reinforcement in the concrete façades vulnerable to attack but so are the metal connections used to secure the façade to the structure, which are often unprotected.

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 the concrete. Because both water and oxygen must be available for electrochemical corrosion to occur, that portion of a marine concrete structure located in the tidal and splash zones is generally the most susceptible to corrosion. All segments of a marine structure, however, 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*—When reinforced concrete is cast in contact with chloride-contaminated soil, chloride ions can migrate into the concrete, causing corrosion of the embedded reinforcement. This occurs more often in

concrete with a high water-cementitious material ratio (*w/cm*) and high permeability.

2.1.7 *Other structures*—Other types of concrete structures can experience corrosion-related problems. For example, in sewage and waste facilities, the concrete can disintegrate after prolonged exposure to acids in wastes and expose the steel. Prestressed-concrete, water-storage tanks have caused corrosion problems (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.

2.2—Environment and corrosion

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

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. Without direct exposure to moisture, coupled with the drying effect of heating and air-conditioning, reinforcement in concrete structures of this nature 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 for 30 or more years. 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. Additionally, 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 the moisture changes of weather have a higher risk of corrosion than those not exposed to weather. The exception is carbonation-induced corrosion in enclosed concrete parking structures. Moisture along with oxygen causes corrosion if the steel loses its passivity.

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. Temperature variations can cause cracking in concrete leading to the ingress of deleterious substances and potential corrosion. Exposure to weather also makes concrete structures 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 –3.9 °C (25 °F). 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 make the steel in concrete more susceptible to corrosion because they disrupt protective oxide film that initially forms on reinforcement.

Bridges, parking garages, and other concrete structures exposed to chemical deicers are at a high risk for corrosion. At a minimum, 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 on exposed surfaces and the aggressiveness of the exposure, additional measures, such as increased cover, low-permeability concrete, corrosion-inhibiting admixtures, 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 zone, which goes through alternating cycles of wetting and drying.

Because of this greater risk of corrosion, AASHTO (AASHTO 1998) recommends 100 mm (4 in.) of clear cover for reinforced concrete substructures that will be exposed to seawater for over 40 years. Other protective measures can be required to extend the service life.

2.2.5 *Concrete exposed to chemicals*—Industrial concrete structures exposed to chemicals, such as acids, that can lead to the disintegration of concrete are at high risk for corrosion. This type of exposure requires protective measures beyond those required for structures exposed to moisture only. For particularly aggressive chemicals, an impermeable coating on exposed concrete surfaces or 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 United States and Canada. When precipitation occurs, rainwater combines with these oxides to form sulfuric acid, nitric acid, or both, known as acid rain. Prolonged exposure to acid rain can lead to and accelerate deterioration of concrete and corrosion of steel in concrete.

2.3—Cracking and corrosion

The role of cracks in the corrosion of reinforcing steel is controversial (ACI 222R). One viewpoint is that cracks 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. Thus, cracks accelerate the onset of corrosion.

The other viewpoint is that while cracks accelerate the onset of corrosion, corrosion is localized. With time, chlorides and water penetrate uncracked concrete and initiate more widespread corrosion. Consequently, after a few years of

service for concrete with moderate to high permeability, there is little difference between the amount of corrosion in cracked and uncracked concrete.

To some extent, the effect of cracking on corrosion depends on whether cracking is oriented perpendicular or parallel to the reinforcement. In the case of flexural cracking, where cracking is perpendicular to the reinforcement, the onset of corrosion is likely accelerated, but deterioration in the long term is often not impacted significantly. If cracking occurs over and parallel to the reinforcement, however, as in the case of shrinkage or settlement cracks, corrosion will not only be accelerated but more significant, and widespread deterioration can be expected.

The use of provisions for controlling crack width by judicious placement of embedded steel as the primary means of protecting against corrosion is not recommended. It is essential to have concrete with a low *w/cm*, and with 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. Concrete cover is discussed as follows.

2.4.1 *Cover requirements*—One of the easiest methods of improving corrosion protection of steel reinforcement is to increase the amount of concrete cover. The minimum cover for reinforcement in most concrete structures not exposed to weather is 19 mm (3/4 in.). As the risk of corrosion increases, so does the required concrete cover. Because development length of reinforcing bars is known to be a function of cover (ACI 318), it may be desirable to use larger than minimum concrete cover, even if there is little risk of corrosion.

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, however, a minimum concrete cover of 50 mm (2 in.) for walls and slabs and 64 mm (2-1/2 in.) for other members is required for corrosion protection. For precast concrete manufactured under plant control conditions, a minimum cover of 38 and 50 mm (1-1/2 and 2 in.), respectively, is recommended for walls and slabs.

2.4.1.2 *AASHTO bridge specifications requirements*—Table 2.2 (AASHTO 1996) summarizes the current minimum AASHTO concrete-cover requirements. In corrosive marine environments or other severe exposure conditions, AASHTO recommends that the amount of concrete protection be suitably

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

	Cast-in-place		Precast concrete,[†] in. (mm) (manufactured under plant-control 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: 15)	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).

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

Reinforced concrete, in. (mm)	Prestressed concrete, in. (mm)
Concrete cast against and permanently exposed to earth: 3 (75)	Prestressing steel and main reinforcement: 1-1/2 (40)

Concrete exposed to earth or weather
  Primary reinforcement: 2 (50)
  Stirrups, ties, and spirals: 1-1/2 (40)
	Slab reinforcement
	  Top of slab: 1-1/2 (40)
	  When deicers are used: 2 (50)
	  Bottom of slab: 1 (25)

Concrete deck slabs in mild climates
  Top reinforcement: 2 (50)
  Bottom reinforcement: 1 (25)
	Stirrups and ties: 1 (25)

Concrete deck slabs that have no positive corrosion protection and are frequently exposed to deicing salts
  Top reinforcement: 2-1/2 (65)
  Bottom reinforcement: 1 (25)	

Concrete not exposed to weather or in contact with ground
  Primary reinforcement: 1-1/2 (40)
  Stirrups, ties, and spirals: 1 (25)	

Concrete piles cast against earth, permanently exposed to earth, or both: 2 (50)	

amplified by increasing the imperviousness to water of the protecting concrete or by other means. This can be accomplished by increasing concrete cover. Other methods for providing positive corrosion protection, which are specifically recommended, are epoxy-coated reinforcing bars, special concrete overlays, impervious membranes, or a combination of these measures.

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 amount 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 in the range of 1.67%; that is, 25 mm in 1.5 m (1 in. in 5 ft). To design a good drainage system, it is imperative that time-dependent deflections be considered. This is particularly true 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 should also be located to prevent melted snow from running onto a slab and refreezing in snow-belt areas. 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 is in ACI 362.1R.

2.4.3 *Reinforcement*—Differences between different types of steel reinforcement (for example, prestressed, nonprestressed, different manufacturers, and diameters) are not 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 steel reinforcement for an entire project to be delivered to the site and be exposed to the elements for months before use; this should be avoided.

Lubricants used in the drawing prestressing steel appear to raise the chloride-corrosion threshold (Pfeifer, Landgren, and Zoob 1987). These oils, however, can also adversely affect bond.

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

If galvanized reinforcing steel is used in concrete, a small amount of chromate salt can be added to the fresh concrete to prevent hydrogen evolution, which can occur when an unpassivated zinc surface reacts with hydroxides in fresh concrete (Boyd and Tripler 1968). Additionally, procedures should be provided to minimize electrical connection with nongalvanized metals.

If epoxy-coated reinforcement is used, the code-required minimum concrete cover still applies; there should be no reduction in cover. Because macrocells can develop where defects occur in the coating, project specifications should clearly spell out quality control of the coating and provide procedures for minimizing inadvertent electrical connection with noncoated metals.

Structures that use unbonded post-tensioned construction require protective measures, especially in aggressive chloride environments. Because the prestressing elements are not directly protected by the alkaline environment of concrete, but instead by some form of duct, project specifications should clearly indicate that the duct should be impervious to penetration of water and should be maintained for the full length between anchorages. The project specifications should show positive methods for attaching the duct to the anchorage to prevent water intrusion. The Post-Tensioning Institute (1985) and ACI Committee 423 (ACI 423.4R) provide guidance for additional measures, such as corrosion-resistant grease and anchorage protection.

There have been several cases of corrosion-related failure of unbonded prestressing tendons in building and parking structures in the absence of chlorides (Schupack 1982; Schupack and Suarez 1982). In one case, water and oxygen were available to the prestressing strands that were surrounded by plastic duct. Corrosion occurred because the strands were not protected by the alkaline environment of the concrete or by corrosion-resistant grease. Bonded systems generally exhibit excellent corrosion resistance, except when

located in severe environments or where construction deficiencies have occurred (Novokschenov 1988; Whiting, Stejskal, and Nagi 1993).

2.4.4 *Joints*—Because joints, especially construction joints, are often sources of leakage, they should be properly constructed and sealed. ACI Committee 224 (ACI 224.3R) has issued a comprehensive state-of-the-art report on proper design and detailing of joints in concrete structures. Additional information on design of joints for parking structures is in ACI 362.1R. ACI 515.1R discusses various coatings for making concrete more watertight, and ACI 504R discusses seals and sealants.

2.4.5 *Overlays*—For concrete structures with a high risk of corrosion, particularly due to external chloride, the use of low-permeability overlays can be the best protection method. The overlay provides additional concrete cover to protect embedded reinforcement.

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. As an example, it is difficult to achieve 50 mm (2 in.) or more of cover around the anchorage and strand extensions in an unbonded post-tensioned structure. In an aggressive environment, these components need additional protection.

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.

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 (cathode). This can cause corrosion of the unbedded, 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—The influence of mixture design on the corrosion of reinforcing steel

3.1.1 *Introduction*—The design of concrete mixtures that enhance the corrosion resistance of reinforcing steel is not substantially different from the design of mixtures for any high-quality concrete. The goal is to use the materials available to develop a concrete mixture that will permit mixing, transporting, placing, consolidating, and finishing in the fresh state and, if 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 *The benefits of low* w/cm—The benefits of reducing the *w/cm* to delay the corrosion of reinforcing steel have been 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 salt penetration and 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.

Water-cementitious material is fundamental to reducing the permeability of concrete because it defines both the relative masses of cementitious materials and water, and the relative volumes of these two components. The greater the *w/cm*, the easier it is for gases or solutions to pass through the concrete. For example, in a cement paste with a *w/cm* of 0.35, the cement particles occupy 47% of the volume of the paste. In paste with a *w/cm* of 0.60, they occupy only 34%. The initial water volume, 53% in the case of a *w/cm* of 0.35 and 66% in the latter case, gives rise to the capillary system in the hardened concrete. When the pores are large and interconnected, they form a system of continuous channels through the paste, which permits the passage of water, water vapor, dissolved salts, oxygen, and carbon dioxide. Therefore, it is beneficial to reduce both the size and total volume of these capillaries. This can be done effectively by reducing the *w/cm* and providing adequate curing to ensure sufficient hydration.

For these reasons, *w/cm* are 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* not exceed 0.40 for concrete exposed to chlorides from seawater, deicing salts, and other sources.

3.1.3 *Proportioning mixtures for a low* w/cm—The *w/cm* decreases by reducing the quantity of mixing water relative to the mass of cementitious materials. Simply removing water from a given mixture, however, will generally result in an unworkable 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 an 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 very high mortar contents and an increased tendency towards 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 a reduced cement content for the same *w/cm*. For greater reductions in water, high-range water-reducing admixtures (HRWRAs) (ASTM C 494 Types F and G) are used.

The effects of aggregate size and graduation 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 by 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 37.5 mm (1-1/2 in.) coarse aggregates than with 9.5 mm (3/8 in.) 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—The influence of the selection of cement, aggregates, water, and admixtures on the 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. While it is true that ASTM C 150 cement types (I-V) contain varying amounts of C_3A, 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.

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. At the same time, 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 nonalkali reactive, the use of high-alkali cement to enhance corrosion resistance is not recommended.

Any portland cement meeting the requirements of ASTM C 150 can likely 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*, 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. If 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. These other issues are not addressed further in this guide.

Two primary issues govern the selection of aggregates for use in concrete exposed to a corrosive environment. The first is the use of aggregates that introduce chloride ions into the mixture, which is discussed in detail in ACI 201.2R and 222R. 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 means using Quantab chloride titrator strips (Gaynor 1986). Tightly bound chlorides, however, will not likely contribute significantly to corrosion. Determination of the amount of bound chlorides that can enter into the pore solution requires specialized procedures (Hope, Page, and Poland 1985).

The second issue in 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.).

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*—Drinking water can be safely used in concrete. Seawater should never be used to make concrete for reinforced structures because it will contribute enough chloride ions to cause serious corrosion problems.

3.2.4 *Selection of chemical and mineral admixtures*—A wide variety of chemical and mineral 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 can 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 530 mL (18 fl oz.) of admixture to 1820 kg (4000 lb) 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. Therefore, air-entraining admixtures 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 C 260.

- **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. Concrete mixtures that have increased workability can be produced at 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 C 494, 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 still 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 C 494, 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 16 ºC (60 ºF). 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. Therefore, nonchloride accelerators should be used when accelerators are needed. A nonchloride accelerator should be noncorrosive within its recommended dosage range. Accelerating admixtures are classified as Type C or E in ASTM C 494.

- **Retarding admixtures—**When temperatures are above 27 ºC (80 ºF), set retarders increase the setting time, and thereby extend 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 C 494.

- **Corrosion-inhibiting admixtures—**Corrosion-inhibiting admixtures delay the onset of corrosion and reduce the rate of corrosion of reinforcement due to chloride attack. Refer to Section 5.4 for a more detailed discussion on corrosion-inhibiting admixtures.

3.2.4.2 *Mineral admixtures*—These finely divided materials enhance concrete properties in the fresh or hardened state, or both, and in some cases improve economy. 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 fly ash will also reduce the maximum temperature rise of concrete. Fly ash should be specified using ASTM C 618.

- **Ground-granulated blast-furnace slag—**Ground-granulated blast-furnace slag is added as a cement substitute or blended into cement. It reduces temperature rise in large members and decreases permeability to chloride ions. Ground-granulated blast-furnace slag should be specified using ASTM C 989.

- **Natural pozzolans—**Natural pozzolans provide some improvement in permeability reduction but are not as effective as fly ash or ground-granulated blast-furnace slag.

Natural pozzolan is also specified using ASTM C 618.

- **Silica fume (microsilica, 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 C 1240.

3.2.4.3 *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, while 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 United States, deformed billet-steel reinforcing bars conforming to ASTM A 615 are used (ACI 318). Factors such as steel composition, grade, or level of stress have not been found to play a major role with regard to corrosion susceptibility in the concrete environment (ACI 222R). Presently, no available information suggests any cost-effective modifications to the inherent properties of conventional reinforcing steel that would aid in resisting corrosion.

3.4—Epoxy-coated reinforcing steel

3.4.1 *Introduction*—After several evaluations and a research study involving numerous types of coatings (Clifton, Beeghly, and Mathey 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. There are approximately 100,000 structures containing epoxy-coated reinforcement (Virmani and Clemena 1998).

A fusion-bonded epoxy coating cures and adheres to the steel substrate as a result of chemical reactions initiated by heat; it is a thermo-setting material. Fusion-bonded epoxy coatings are composed of epoxy resins, curing agents, various fillers, pigments, and flow-control agents. The epoxy coating resists the passage of charged species, such as chloride ions, and minimizes moisture and oxygen transport to the steel. The coating increases the electrical resistance of any corrosion cell that tries to form between damaged areas on the steel surface.

Because it is a barrier, some protection is lost if the coating is damaged. Breaks in the coating reduce the electrical resistance (Clear 1992b; Wiss, Janney, Elstner Associates, Inc. 1992) and permit contaminants to reach the steel surface. Long-term adhesion of the epoxy coating to the steel substrate is very important to corrosion performance. Studies have shown that corrosion performance is not impaired by loss of adhesion if 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. Should corrosion occur at a defect, the coating should resist undercutting (further progression of corrosion beneath the coating). This resistance of the coating to undercutting is strongly dependent on its adhesion to the steel at the time corrosion initiates. A well-adhered coating will keep the corrosion confined to the vicinity of the defect so that the corrosion has a minimal effect on the life of the structure.

3.4.2 *Corrosion-protection performance*—The degree of corrosion protection provided by epoxy coatings is controversial. Numerous laboratory (Clear and Virmani 1983; Clifton, Beeghly, and Mathey 1974; Erdogdu and Bemner 1993; Pfeifer, Landgren, and Krauss 1993; Scannell and Clear 1990; Sohanghpurwala and Clear 1990; and Virmani, Clear, and Pasko 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 examined the performance of bridge decks in service for 15 years or more and reported excellent performance (Gillis and Hagen 1994; Hasan, Ramirez, and Cleary 1995; Perregaux and Brewster 1992; and 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, Kessler, and Powers 1993). Isolated examples of corrosion have also been reported in bridge decks, barrier walls, and a parking garage (Clear 1994). An investigation in Ontario 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 also have reported poor adhesion on bars removed from older structures (Clear 1994).

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

While these studies have significantly contributed to understanding the long-term field performance of epoxy-coated reinforcement, 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, a failure mechanism was identified involving progressive loss of coating adhesion accompanied by under-film corrosion on coated bars meeting 1987 specifications (Clear 1994; Clear et al. 1995). This led Clear 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 was improved.

While there is no dispute that epoxy coating will extend the time-to-corrosion damage, compared with uncoated steel, the long-term performance remains somewhat uncertain. Not all the factors affecting corrosion performance are understood, and there are many examples of good performance and examples of premature corrosion damage. The dominant factors affecting performance are the number and size of defects in the coating and the long-term adhesion of the coating to the steel. Where epoxy-coated reinforcement is used, it is essential that the quality-control and assurance measures focus on these two properties. A number of specifications such as ASTM D 3963, ASTM A 775, and AASHTO M 284 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 to 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, Sherman, and Pfeifer 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, and 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 175 to 300 μm (7 to 12 mils). Fatigue and creep performance of coated reinforcing steel is 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 have been proposed to supplement the existing bend test: the hot water test, the cathodic debonding test, and the salt-spray test. These tests have been used in other coating fields 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 A 775/A 775M or ASTM A 934/A 934M 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 typically is 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 (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, and bundles of coated steel should be lifted to avoid excess sag that can cause bar-to-bar abrasion. 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 keep handling to a minimum. Coated steel should not be dropped or dragged. Epoxy-coated steel should not be stored outdoors for longer than three 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, as described in Section 3.4.3.5.

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.

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.

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).

Corrosion of nonferrous or specialty steels can result from one of several 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 if 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.

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 form, 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 generate hydrogen gas initially, 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).

3.5.3 *Lead*—Lead in dry concrete does not corrode. In damp or wet 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, may prevent initial reactions of the zinc and inhibit the formation of hydrogen gas (Boyd and Tripler 1968). 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 joints 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 agent 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.

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 (ASTM A 615) 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 those locations, 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 C 94. Other commonly used references for plant equipment requirements are the "Concrete Plant Standards" (Concrete Plants 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 about a 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 s in adding the HRWRA will increase its effectiveness. At *w/cm* of 0.40 and 0.45, these problems are rarely encountered, and the normal charging sequences described previously perform well.

ASTM C 94 permits adding tempering water to bring the slump within the desired range when a ready-mixed 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. Not only should the formwork have the strength to support the concrete during construction and maintain its configuration, but it also should have sufficient strength to maintain tolerances for the reinforcing steel cover or resist excessive deflections that can cause cracking. For example, excessive deflections in slab formwork can create areas of low concrete cover that will be more susceptible to cracking above the low-cover reinforcing bars. The cracks would be potential water and chloride-ion entry points that, in a corrosive environment, could lead to extensive corrosion in a period of a few years. The formwork should be mortar-tight to avoid leakage of cement paste during consolidation.

The materials used to fabricate the formwork will not have any significant effect on the potential for future corrosion. The formwork, however, should be clean and textured to the minimum roughness needed. On irregular surfaces, it is difficult to consolidate the surficial concrete. The formation of surface honeycombing or voids creates potential entry points for water and chlorides.

For some members, the support formwork should be designed to avoid excessive deflections during concrete placement. Camber may be required to offset deflection when the formwork is loaded. Rigorous attention is needed to prevent low areas in formwork, which can result in loss of cover. One way to ensure this is by paying careful attention to horizontal and vertical tolerances after placement. The

tolerances should be spot-checked by measurement before concrete placement; the engineer should not rely on a visual appearance only.

Before placing concrete, the formwork should be cleaned of all construction debris, such as sawdust and wire scraps, and water, snow, and ice. All loose materials can create voids following placement. Stay-in-place metallic formwork, such as that used for slab decking in buildings, 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. The situation can be aggravated if the formwork is composed of materials other than mild steel such as galvanized steel or aluminum.

4.2.2 *Reinforcing steel*—Reinforcing steel bars should be placed to the configuration shown in the design drawings. The specified tolerances should be followed, with particular attention paid to concrete cover and closely spaced reinforcing. 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, if 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.

Non-prestressing 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 wire, and openings.

Post-tensioned reinforcing steel (multistrand, single wire, or bar) requires thorough attention to detail during placement. For sheathed wire or strand, special 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 must 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.

Special care should be taken to avoid using chloride-releasing tapes, such as some of the polyvinyl-chloride tapes used in earlier years. Some systems have sheathing encasing the entire anchorage assembly. These areas should be inspected for nicks and scratches before placement (Perenchio, Fraczek, and Pfeifer 1989).

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 avoid 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 free fall into place without the use of hoppers, trunks, or chutes if forms are sufficiently open and clear so that the concrete is not disturbed during placement (ACI 304R). Chutes, drop chutes, and tremies can be used in applications where this is not possible such as in tall columns or shafts. Free fall may need to be limited if the reinforcement is relatively congested. Pumping provides several advantages over free fall in some members because the discharge point can be very close to the final point of placement.

The sequencing of the concrete placement is also important. Cold joints result from the partial setting of an earlier
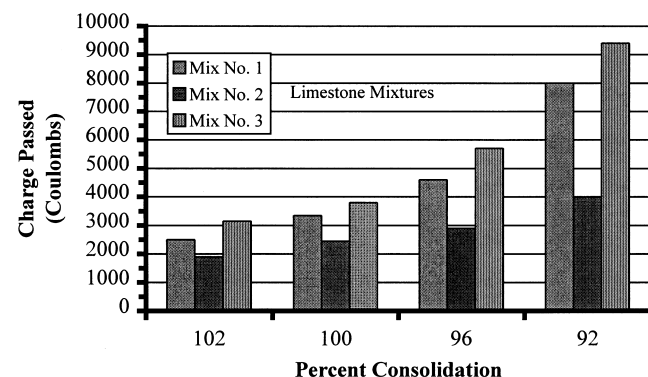

Fig. 4.1—Effect of degree of consolidation on rapid chloride permeability of limestone concrete mixtures (Whiting and Kuhlman 1987)

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 to the concrete serviceability.

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 min 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

4.3.1 *Influence of consolidation on concrete quality*—Concrete should be fully consolidated to increase the probability of realizing its full potential with regard to strength, durability, and impermeability to substances that can promote the corrosion of steel and other embedded metals. Fresh concrete, when initially placed, contains significant amounts of entrapped air. Consolidation, achieved through mechanical vibration, is needed to eliminate air voids, which otherwise would result in a weak, porous, and nondurable material.

Strength does not contribute to the corrosion-resistant properties of concrete, but a weak concrete may fail to sustain loads for which the structure was initially designed, leading to cracking, spalling, and disruption of protective cover, thereby exposing embedded metals to corrosive agents. A number of studies (Kaplan 1960; Whiting and Kuhlman 1987) have demonstrated that compressive strength is reduced by 30% for only a 5% decrease in the degree of consolidation. Bond to reinforcing steel is reduced even more dramatically; a loss of approximately 50% in bond strength results from a 5% reduction in consolidation.

As the *w/cm* of concrete decreases, its permeability decreases, resulting in a concrete that offers a greater degree of protection to reinforcing steel (Clear 1976; Whiting 1978). The benefits of consolidation are such that the greater the compactive effort employed, the stiffer the mixture that can be placed. Therefore, as the water content of concrete is reduced, the quality (that is, strength, durability, and impermeability) will improve, provided that the concrete is properly consolidated and adequately cured.

4.3.2 *Influence of consolidation on permeability*—Properly consolidated concrete is better able to resist the penetration of moisture, ions, gases, and other deleterious substances than concrete that has been poorly compacted. Figure 4.1 shows an example of the effect of consolidation on permeability of concrete to chloride ions. The figure represents three mixtures with cement contents ranging from 310 to 360 kg/m^3

(520 to 610 lb/yd^3) that were consolidated on a laboratory vibrating table. The group labeled "100" was consolidated in accordance with procedures described in ASTM C 192. Those labeled "102" were given an extended period of vibration. Those mixtures given less consolidation (96 and 92) showed an increase in coulomb charge passed as measured by AASHTO T 277.

Studies by the Federal Highway Administration (Clear and Hay 1973) have shown that the performance of mixtures specifically designed to have low permeability, such as low-slump dense concrete (LSDC), can be compromised if full consolidation is not achieved because much greater amounts of chloride ions would penetrate into the concrete, compared with a properly consolidated mixture.

The increase in permeability to chloride ions brought about by poor consolidation would make it easier for moisture and oxygen to enter the concrete, promoting rapid onset and progress of corrosion. In extreme cases, honeycombing that extends to the level of the reinforcement, or large voids in the vicinity of the reinforcement, would remove virtually all protection offered by the concrete to the steel, and corrosion would proceed as if the steel was not embedded in the concrete at all.

4.3.3 *Guidelines for achieving satisfactory 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 an ACI document on the principles of vibration (ACI 309.1R), proceedings of an ACI symposium held in 1986 (ACI SP-96), and a report issued by the Federal Highway Administration (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—The influence of curing on the corrosion of reinforcing steel

4.4.1 *Introduction*—ACI Committee 308 has proposed the following definition: "Curing is the maintaining of a satisfactory moisture content and temperature in concrete during its early stages so that desired properties may develop" (ACI 308R). 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. Conversely, the corrosion-resisting properties of concrete will not develop to their expected values if adequate curing is not provided.

4.4.2 *Background*—Adequate curing has a beneficial impact on various properties, ranging from compressive strength to chloride-ion permeability because a controlled moisture and temperature environment permits the continued development of the internal microstructure of the concrete. Hydration reactions between portland cement and water continue to produce hydration products that fill the spaces between cement grains, blocking capillary channels that would otherwise have provided passageways for water, water vapor, dissolved salts, oxygen, and carbon dioxide. The hardened concrete becomes denser, stronger, and less permeable.

Chemical reactions that produce the fundamental building blocks (or hydration products) of the hardened portland cement can take place only in water-filled spaces (Powers 1949). In other words, the hydration reactions can occur only when water is available at the surface of the individual cement grains. At very low *w/cm*, failure to provide water to the concrete from an external source can result in self-desiccation, which means that the cement uses up all of the available mixture water before hydration has achieved its full potential. A more common occurrence, however, is when inadequate moisture control permits the loss of water from the surface of freshly cast concrete, which then robs the mixture of water required for hydration. In either event, failure to provide sufficient water to fresh concrete will result in hardened concrete in which the cement has not completely hydrated, but may have hydrated enough to achieve the required performance. Failure to control the loss of moisture from the surface of freshly placed concrete can result in surface cracking and future avenues for corrosive agents if these cracks are large enough.

Powers, Copeland, and Mann (1959) studied the relationship between curing and permeability. In laboratory tests, Powers evaluated the duration of continuous wet curing required to develop cement pastes to the point of being essentially impervious to water under low pressure because the capillary system had become discontinuous.

w/cm of cement paste	Duration of wet-curing for Type 1 cement at 23 °C (73 °F) to achieve capillary discontinuity
0.40	3 days
0.45	7 days
0.50	14 days
0.60	6 months
0.70	1 year
Greater than 0.70	Impossible

Powers observed that the duration of wet curing required for development of capillary discontinuity was dependent on the initial *w/cm* of the cement paste. The *w/cm* defines not only the relative masses of the cement and water, but also defines the relative volumes of these two components (through their specific gravity) and the average spacing between the cement grains. The greater the *w/cm*, the greater the void space between grains of cement and the longer the time required to develop hydration products to fill that space.

Recent tests (Whiting and Kuhlmann 1987) demonstrated that chloride permeability, as measured by the AASHTO T 277, was strongly influenced by the curing duration. The tests also demonstrated that "measurements of permeability of concretes cured under standard laboratory conditions may be optimistic and that permeability of field-cured concrete is significantly greater than that of companion laboratory specimens."

4.4.3 *Guidelines*—Adequate 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 10 °C (50 °F) 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 velocity of the wind blowing across the surface of the concrete. While such conditions often prevail in hot, dry environments, 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 special 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 and concrete that is severely carbonated. In areas where deicing salts are used for winter maintenance of high-ways and roads, bridges and their substructures are particularly vulnerable to embedded-steel corrosion because of the direct application of sodium chloride or calcium chloride to the bridge-deck surface. These salts can readily penetrate typical concrete cover in just a few years. 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 salty water caused by vehicles on the highway.

Parking garage structures in these areas 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 cooling towers can suffer 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, cooling towers will concentrate any dissolved material. Mechanical towers are sometimes operated until dissolved materials have been concentrated by a factor of approximately 20. Any aggressive salts that are concentrated can cause corrosion.

Serious corrosion damage can occur in the interior of buildings. 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 381 mm (15 in.) deep. Many of the 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. Serious delamination occurred about eight years

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[*]Source: Bernard Erlin, The Erlin Co., Latrobe, Pa.

after construction because of the 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 (that is, 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, 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 116 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 (temperature variations, marine environment, precipitation), reinforcing-steel details, type of reinforcement (uncoated mild steel, epoxy-coated steel, galvanized steel, 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 procedures listed below are used in the condition survey.

Corrosion evaluation test methods include:

- Visual inspection including crack and spall surveys;
- Delamination survey;
- Depth of concrete-cover measurements;
- Chloride-ion content analyses;
- 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, corrosion product pH, or both.

Concrete evaluation test methods 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. 1994; Genge 1994; Ehlen 1999). 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 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 can be selected.

5.3—Barrier systems for concrete

5.3.1 *Introduction*—Because the corrosion of reinforcing steel usually requires the ingress into concrete of water, aqueous salt solutions, and air, treating concrete with barrier systems is a potentially effective anticorrosive practice. Four general types of barriers are used (ACI 515.1R): waterproofing, damp-proofing; protective; and 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 or vapor can be 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 waterproofing 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 consisted 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 Manual (NRCA 1989).

The bitumens (asphalt or coal-tar pitch) used in hot-applied barriers have very little strength and, therefore, 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 need to be reinforced with fabric.

A dampproofing 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 due to mechanical abuse or exposure to excessive temperature. For example, 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 (for example, cracks in the concrete reflecting through the barrier) and foundation movements on the performance of protective barriers.

5.3.4 *Decorative paint-barrier system*—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 recent extensive research (Hewelett 1990). While they are effective in resisting chloride-ion penetration, they have no resistance to diffusion by carbon dioxide (Swamy and Tanikawa 1990).

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 systems (Mathey and Rossiter 1983).

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

5.4.1 *General*—This section deals with admixtures that can be 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 have been 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).

5.4.2 *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.

5.4.2.1 *Corrosion inhibitors*—These are chemical substances that decrease the corrosion rate when present at a suitable concentration, without significantly changing the concentration of any other corrosion agent (ISO 8044). These admixtures act on the steel surface, either electro-chemically (anodic, cathodic, mixed-inhibitor) or chemically (chemical barrier) to inhibit chloride-induced corrosion above the accepted chloride-corrosion threshold level. 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 alkano-lamines and an aqueous mixture of amines and fatty-acid esters (Berke, Hicks, and Tourney 1993; Bobrowski and Youn 1993; Mäder 1995; Martin and Miksic 1989; Nmai, Farrington, and Bobrowski 1992; Nmai and Krauss 1994).

Organic amine-based compounds, such as some amine salts and alkanolamine, are effective corrosion inhibitors for steel in concrete when used in a post-treatment process for chloride-induced corrosion of steel in concrete (Al-Qadi et al. 1992; Collins, Weyers, and Al-Qadi 1993; Dillard et al. 1992).

5.4.2.2 *Physical-barrier admixtures*—These 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).

5.4.3 *Mineral 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. Mineral admixtures include fly ash, ground-granulated blast-furnace slag, and silica fume. Mineral admixtures are discussed in Section 3.2.4.2.

5.5—Cathodic protection

5.5.1 *Introduction*—Cathodic protection (CP) is a widely used and effective method of corrosion control. CP is a reduction or elimination of corrosion by shifting the potential of the metal to the open-circuit potential of the local anodes. The polarization is achieved 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 (electrolyte), to the reinforcing steel (cathode). 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 have been used as external power sources, but these alternatives are still considered experimental (Lasa, Powers, and Kessler 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 have been 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), return circuitry (positive and negative DC wiring), 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 currently available for specific applications. Bennett et al. (1993) summarizes 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.

CP has been applied on a routine basis on normal reinforcing bars; however, two generalized problems can arise 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 by (Galvez, Caballero, and Elices 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 United States has been completed (Scannell et al. 1994, Scannell, Sohanghpurwala, and Powers 1995).

5.5.2 *Performance history*—CP applied to reinforced concrete has a performance history of more than two decades. Since its first major application in 1973 (Stratfull 1974), more than 275 bridge structures have been cathodically protected throughout the United States and Canada (Broomfield

and Tinnea 1992; Eltech Research Corp. 1993). By the late 1980s, CP had been applied to a total of approximately 840,000 m^2 (9,000,000 ft^2) of above-ground concrete surfaces (Eltech Research Corp. 1993). Most of the applications are on bridge decks, but installations on bridge-substructure components is 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 the CP installations were operating satisfactorily (Pastore et al. 1991). According to another report (Zivich, Walker, and Ahal 1992), CP systems installed on 14 bridges 16 to 18 years ago continue to operate successfully. The Federal Highway Administration is continuing to evaluate the long-term performance of CP on bridge structures throughout the United States and Canada.

Since the mid-eighties, 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, Langford, and McAnoy 1987; Daily 1987; Schutt 1992; Tighe and Ortlieb 1991).

5.5.3 *Selection of a CP candidate*—Almost any reinforced concrete structure, or portion thereof, of virtually any geometry can be cathodically protected. Even though CP has the ability to stop corrosion, existing structures should be considered individually with regard to the need for and applicability of CP. Some very general guidelines to determine whether or not a given structure may be a good candidate for CP are:

- The projected remaining life of the structure should be greater than or equal to 10 years (CP is usually most cost effective when a long-term rehabilitation is desired);
- The majority of the reinforcing steel should be electrically continuous;
- The area of delamination should be around 5% or more;
- A large percentage of potentials should be 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 five years;
- The concrete cover over reinforcing steel should be more than 12.5 mm (1/2 in.);
- 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 determina-

tion 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 is dependent on numerous parameters including:

- The dimensions and as-built plans of the structure;
- The pattern, location, and schedule 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 CP systems are provided in Bennett et al. (1993) and AASHTO-AGC-ARTBA (1994). CP systems should be designed by personnel specializing in CP design for reinforced concrete structures. Corrosion engineering firms or their subsidiaries responsible for the design of a CP system should not be engaged in the manufacture or supply of corrosion-control materials or equipment (AASHTO-AGC-ARTBA 1994).

5.5.5 *Installation and inspection*—For the successful operation of CP systems, all materials and equipment must be installed in accordance with the specifications, drawings, and manufacturer's recommendations. Detailed information regarding installation of CP systems is provided in recent publications (AASHTO-AGC-ARTBA 1994; 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 (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 is 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 for acceptable long-term performance. Monitoring methods and frequency and maintenance information are provided in several publications (AASHTO-AGC-ARTBA 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, Lankard, and Moreland 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 treating corrosion-damaged structures have been developed.

Electrochemical chloride extraction consists of applying an anode and 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: the anode is temporary, and the current density is approximately 100 times that used in most cathodic-protection 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 would be 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 a contract in 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 can be 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 efficiency 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). It is possible to suggest a practical range for electrochemical treatment, according to laboratory and field experience conducted since 1989.

The maximum voltage that can be applied is limited by electrical codes to 30 to 50 V, 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 1 to 4 A/m^2 (0.09 to 0.4 A/ft^2). The total charge passed should be 600 to 1200 $A.h/m^2$(56 to 111 $A.h/ft^2$), which means that the treatment times will be approximately 10 to 80 days (assuming the average current density is 60% of the maximum). The efficiency of chloride removal typically is between 10 to 20% because of the presence of other negative ions, mainly hydroxyl ions, in the concrete.

Electrochemical-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 chloride ions have not penetrated deeper than the reinforcement, and corrosion has not progressed to the stage where 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.

Currently, most treatments have been relatively small demonstration projects. Consequently, costs have not been well established, though they are expected to be competitive with other rehabilitation methods. The extension of service life resulting from treatment is uncertain because the treatment has only been in use for a few years.

CHAPTER 6—REFERENCES
6.1—Referenced standards and reports

The documents of the various standards-producing organizations referred to in this document are listed with their serial designation. The documents listed were the latest effort at the time this document was written. Because some of these documents are revised frequently, generally in minor detail only, the user of this document should check directly with the sponsoring group if it is desired to refer to the latest revision.

AASHTO

T 277   Standard Method of Test for Rapid Determination of the Chloride Permeability of Concrete

M 284   Standard Specification for Epoxy Coated Reinforcing Bars

AASHTO-AGC-ARTBA Task Force #29, 1994, *Guide Specification for Cathodic Protection of Concrete Bridge Decks*, U.S. Department of Transportation, Federal Highway Administration, Washington, DC.

AASHTO, 1996, *Standard Specifications for Highway Bridges*, 16th Edition, American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO, 1998, *Interim Revisions to Standard Specifications for Highway Bridges*, 16th Edition, American Association of State Highway and Transportation Officials, Washington, DC.

American Concrete Institute

116R   Cement and Concrete Terminology

201.2R   Guide to Durable Concrete Chemical Admixtures for Concrete

211.1   Standard Practice for Selecting Proportions for Normal, Heavyweight, and Mass Concrete

212.3   Chemical Admixtures for Concrete

212.4   Guide for the Use of High-Range Water-Reducing Admixtures (Superplasticizers) in Concrete

222R   Protection of Metals in Concrete Against Corrosion

224R   Control of Cracking in Concrete Structures

224.3R   Joints in Concrete Structures

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   Ground Granulated Blast-Furnace Slag as a Cementitious Constituent in Concrete

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   Hot Weather Concreting

306R   Cold Weather Concreting

308R   Guide to Curing Concrete

309R   Guide for Consolidation of Concrete

309.1R   Behavior of Fresh Concrete During Vibration

318   Building Code Requirements for Structural Concrete

345R   Guide for Highway Bridge Deck Construction

357R   Guide for the Design and Construction of Fixed Offshore Concrete Structures

362.1R   Guide for the Design of Durable Parking Structures

423.4R   Corrosion of Unbonded Monostrand Tendons

504R   Guide to Sealing Joints in Concrete Structures

515.1R   A Guide to the Use of Waterproofing, Dampproofing, Protective, and Decorative Barrier Systems for Concrete

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

SP-96   *Consolidation of Concrete*

ASTM

A 615 Standard Specification for Deformed and Plain Billet–Steel Bars for Concrete Reinforcement

A 775 Standard Specification for Epoxy-Coated Reinforcing Steel Bars

A 934/ Standard Specification for Epoxy-Coated Prefabricated
A 934M Steel Reinforcing Bars

C 94 Standard Specification for Ready-Mixed Concrete

C 150 Standard Specification for Portland Cement

C 192 Practice for Making and Curing Concrete Test Specimens in the Laboratory

C 260 Specification for Air-Entraining Admixtures for Concrete

C 494 Specification for Chemical Admixtures for Concrete

C 615 Specification for Granite Dimension Stone

C 618 Specification for Coal Fly Ash and Raw or Calcined Natural Pozzolan for Use as a Mineral Admixture in Concrete

C 989 Specification for Ground Granulated Blast-Furnace Slag for Use in Concrete and Mortars

C 1240 Specification for Silica Fume for Use in Hydraulic-Cement Concrete and Mortar

D 3963 Specification for Epoxy-Coating Reinforcing Steel

ISO (International Organization for Standardization)

8044 Corrosion of Metals and Alloys—Vocabulary

NACE International

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

The publications listed above can be obtained from the following organizations:

AASHTO
444 N. Capitol St. NW, Suite 249
Washington, DC 20001

American Concrete Institute
P.O. Box 9094
Farmington Hills, MI 48333-9094

ASTM
100 Barr Harbor Drive
West Conshohocken, PA 19428-2959

ISO
1 Rue de Varembe
Case Postale 56
CH-1211, Geneva 20, Switzerland

NACE International
P.O. Box 218340
Houston, TX 77218-8340

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