ACI 222R-19

Guide to Protection of Reinforcing Steel in Concrete against Corrosion

Reported by ACI Committee 222

American Concrete Institute
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Guide to Protection of Reinforcing Steel in Concrete against Corrosion

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American Concrete Institute
38800 Country Club Drive
Farmington Hills, MI 48331
Phone: +1.248.848.3700
Fax: +1.248.848.3701

www.concrete.org

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Guide to Protection of Reinforcing Steel in Concrete against Corrosion

Reported by ACI Committee 222

David Trejo, Chair
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Michael C. Brown
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Arpad Savoly
David G. Tepke
Richard E. Weyers
David W. Whitmore
John B. Wojakowski

Consulting Members
Richard O. Lewis
Andrea J. Schokker

This guide reviews the most recent developments of metal corrosion, specifically reinforcing steel, in concrete. Individual chapters are devoted to corrosion of metals in concrete, protective measures for new concrete construction, procedures for identifying corrosive environments, active corrosion in concrete, and remedial measures.

Keywords: allowable chloride; carbonation; chloride; chloride threshold; corrosion; corrosion-resistant reinforcement; durability; prestressed concrete; reinforced concrete; reinforcing steels.

CONTENTS

CHAPTER 1—INTRODUCTION AND SCOPE, p. 2
1.1—Introduction, p. 2
1.2—Scope, p. 3

CHAPTER 2—NOTATION AND DEFINITIONS, p. 3
2.1—Notation, p. 3
2.2—Definitions, p. 3

CHAPTER 3—MECHANISM OF CORROSION OF STEEL IN CONCRETE, p. 3
3.1—Introduction, p. 3
3.2—Principles of corrosion, p. 3
3.3—Reinforcing bar, p. 9
3.4—Concrete environment, p. 11

CHAPTER 4—PROTECTION AGAINST CORROSION IN NEW CONSTRUCTION, p. 13
4.1—Introduction, p. 13
4.2—Design approaches and choices, p. 13
4.3—Methods of excluding external sources of chloride from concrete, p. 19
4.4—Corrosion control methods, p. 22
4.5—Summary, p. 24

CHAPTER 5—PROCEDURES FOR IDENTIFYING CORROSIVE ENVIRONMENTS AND ACTIVE CORROSION IN REINFROCED CONCRETE STRUCTURES, p. 24
5.1—Introduction, p. 24
5.2—Condition evaluation of reinforced concrete structures, p. 26
5.3—Corrosion evaluation methods, p. 26
5.4—Concrete evaluation test methods, p. 35

CHAPTER 6—REMEDIAL MEASURES, p. 36
6.1—Introduction, p. 36
6.2—Applicability, p. 36
6.3—Remedies and their limitations, p. 37
6.4—Summary, p. 40

CHAPTER 7—REFERENCES, p. 40
Authored references, p. 42

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CHAPTER 1—INTRODUCTION AND SCOPE

1.1—Introduction

The corrosion of metals in concrete can be a serious problem because of its widespread occurrence in certain types of structures and the high cost of repairing such structures. This holds true especially for reinforcing steel. Some of the first widely documented cases of steel reinforcement corrosion were associated with marine structures and chemical manufacturing plants (Tremper et al. 1958; Evans 1960; Biczók 1964). Later, numerous reports of its occurrence in bridge decks, parking structures, and other structures exposed to chlorides emphasized the problem (Crumpton and Pattengill 1969; Fruggiero 1972; Stratfull 1973; Litvan and Bickley 1987). Extensive research on factors contributing to steel corrosion has increased understanding of the mechanisms and causes of corrosion, especially concerning the role of chlorides. It is anticipated that application of the research findings will result in fewer instances of corrosion in new reinforced concrete structures and improved methods of repairing corrosion-induced damage in existing structures. For these improvements to occur, the research information should be disseminated to those responsible for the design, construction, and maintenance of concrete structures.

The high-alkaline environment of concrete results in the formation of a tightly adhering film over the reinforcing steel that generally protects it from extensive corrosion. Therefore, the corrosion of reinforcing steel is not a significant concern in most concrete elements or structures. Corrosion of steel, however, can become a problem if:

a) The concrete does not resist the ingress of corrosion-inducing substances
b) The structure is not properly designed for the service environment
c) The environment is not as anticipated
d) The structure exhibits changes during the service life of the structure

Corrosion of steel reinforcement is the primary subject of this guide. Although several types of metals can corrode under certain conditions when embedded in concrete, the corrosion of steel reinforcement is the most common and of greatest concern.

Exposure of reinforced concrete to chlorides is the major cause of premature corrosion of steel reinforcement, although corrosion can also occur in some circumstances in the absence of chlorides. For example, sufficient amounts of other substances such as other halides, chlorate, and hypochlorite can result in corrosion (Kerkhoff 2007; ACI 515.2R). Carbonation of concrete or other exposure conditions that reduce the concrete's alkalinity can lead to corrosion of the embedded steel reinforcement. Carbonation damage can be extensive in structures with low cover and is at times repaired without consideration of the mechanism of deterioration. As the concrete infrastructure ages, systematic carbonation is likely to become a more apparent and wider problem, particularly in locations away from salt ingress (Sagüés et al. 1997a). Widespread deterioration and consideration for holistic repair for carbonation-induced corrosion is not as common in North America as that for corrosion induced by chlorides. Chlorides are common in nature and very small amounts are generally present in the constituents of concrete. Chlorides can also be intentionally added into the concrete, most often as a constituent of accelerating admixtures. In addition, dissolved chlorides can penetrate hardened concrete in structures exposed to marine environments, salt-laden soils, or deicing or anti-icing salts.

The rate of corrosion of steel reinforcement embedded in concrete is influenced by environmental factors. Corrosion of reinforcement in concrete is an electrochemical process that generally depends on the presence of oxygen and moisture. Reinforced concrete with significant gradients of corrosive ions, such as chloride, is vulnerable to corrosion, especially when subjected to cycles of wetting and drying that is often prevalent in highway bridges and parking structures exposed to deicing or anti-icing salts, and in structures located in marine environments. Other factors that affect the rate and level of corrosion are:

(a) Heterogeneity in the concrete and reinforcing steel
(b) pH of concrete pore water
(c) Carbonation of concrete cover
(d) Cracks in the concrete
(e) Stray currents
(f) Time of wetness
(g) Galvanic effects due to contact between dissimilar metals
(h) The presence of other corrosive ions

Design features and construction practices also play an important role in the corrosion of embedded steel. Concrete mixture proportions, thickness of concrete cover over the reinforcing steel, crack-control measures, and implementation of measures designed specifically for corrosion protection are some of the factors that help control the onset and rate of corrosion.

Deterioration of concrete due to corrosion of the reinforcing steel results because the solid products of corrosion (rust) occupy a greater volume than the original steel and exert expansive stresses on the surrounding concrete. The outward manifestations of the rusting include staining, cracking, and spalling of the concrete (Torres-Acosta and Sagüés 2004). Concurrently, the cross-sectional area of the reinforcing steel is reduced. With time, structural distress may occur either because of loss of bond between the reinforcing steel and concrete due to cracking, delamination, and spalling, or because of the reduced steel cross-sectional area. This latter effect can be of special concern in structures containing high-strength prestressing steel because a small amount of metal loss could trigger a failure (Pillai et al. 2010a,b).

The National Research Council (2011) reported that one challenge facing the United States is the development of a cost-effective, environmentally friendly, corrosion-resistant material. Structures are now being constructed with alternative forms of reinforcement such as stainless steels (McDonald et al. 1995; Bower et al. 2000; Wenzlick 2007) and fiber-reinforced polymer reinforcing bars (Thippeswamy et al. 1998; Trejo et al. 2006; Benmokrane et al. 2007; ACI 440R). In addition, practice and research indicate the need for quality concrete, careful design, good construction practices, and reasonable limits on the amount of chlorides in the concrete mixture constituents. Measures that are being taken and further investigated include the use of corrosion inhibitors, protective coatings on reinforcing steel, concrete coatings, cathodic protection, chloride extraction, and realkalization. Although these measures have been successful in general, problems resulting from corrosion of embedded reinforcing steel and other metals have not been eliminated. Thus, research into new measures to mitigate corrosion of reinforcing steel in concrete are vital.

1.2—Scope

This guide discusses the factors that influence corrosion of reinforcing steel in concrete, measures for protecting embedded reinforcing steel in new construction, techniques for detecting corrosion in in-service structures, and remedial procedures. Consideration of these factors and application of the discussed measures, techniques, and procedures should assist in reducing the occurrence of corrosion and result, in most instances, in the satisfactory performance of reinforced and prestressed concrete structural members.

CHAPTER 2—NOTATION AND DEFINITIONS

2.1—Notation

A = area of reinforcing steel polarized, in.^{2} (cm^2)
A_w = atomic mass, lb/mol (kg/mol)
a = constant
B = proportionality constant, mV/mA
b = Tafel slope, V/decade
c = mass of cement, lb (kg)
cm = mass of cementitious materials, lb (kg)
d_c = depth of carbonation, in. (mm)
$D_{a,28}$ = apparent chloride diffusion coefficient at 28 days, ft^2/s (m^2/s)
D_{OPC} = apparent chloride diffusion coefficient of ordinary portland cement concrete, ft^2/s (m^2/s)
D_{SF} = apparent chloride diffusion coefficient of concrete containing silica fume, ft^2/s (m^2/s)
E = applied potential, V
E_{corr} = corrosion potential, V
F = Faraday's constant, 96,500 coulombs/mol
i = current density, mA/ft^2 (μA/cm^2)
i_{corr} = corrosion current density, mA/ft^2 (μA/cm^2)
k = carbonation rate constant, in./yr$^{0.5}$ (mm/yr$^{0.5}$)
M = mass of dissolved metal, lb (kg)
n = number of equivalents
R_P = polarization resistance, ohm-ft^2 (ohm-cm^2)
R_s = concrete resistance, ohm
t = time, seconds
t_y = time, years
w = mass of water, lb (kg)
β_a = anodic Tafel constant, V/decade
β_c = cathodic Tafel constant, V/decade
η = polarization or overpotential, mV
ΔE = voltage change resulting from the applied current, mV
ΔI = applied current required to obtain ΔE, mA

2.2—Definitions

Please refer to the latest version of "ACI Concrete Terminology" for a comprehensive list of definitions.

CHAPTER 3—MECHANISM OF CORROSION OF STEEL IN CONCRETE

3.1—Introduction

This chapter describes the thermodynamics and kinetics of the corrosion of steel reinforcement embedded in concrete. Subsequent sections explain the initiation of active corrosion by chlorides, carbonation of the concrete cover, and the rate-controlling factors for corrosion after it has been initiated. Finally, the influence of reinforcement types and of the concrete environments on corrosion of steel in concrete are discussed.

3.2—Principles of corrosion

3.2.1 *The corrosion process*—The corrosion of steel reinforcement in concrete is an electrochemical process that involves electron transfer between different species at the steel-concrete interface. In the absence of an external electrical source, the electron transfer takes place between two half-cell reactions—one capable of producing electrons and one capable of consuming electrons. The anodic half-cell reaction involves the oxidation or dissolution of iron, namely

$$\mathrm{Fe} \rightarrow \mathrm{Fe}^{++} + 2e^{-} \qquad (3.2.1a)$$

Under normal atmospheric exposure conditions, the Fe^{++} ions then react further with oxygen and water to form oxides or hydroxides, and the most likely cathodic half-cell reaction is oxygen reduction

$$2\mathrm{H_2O} + \mathrm{O_2} + 4e^{-} \rightarrow 4(\mathrm{OH})^{-} \qquad (3.2.1b)$$

When oxygen is not available, the cathodic half-cell reaction can take place in the form of hydrogen evolution via

$$2\mathrm{H}^{+} + 2e^{-} \rightarrow \mathrm{H_2} \qquad (3.2.1c)$$

When the two reactions occur at widely separated locations, they are termed a macrocell (Fig. 3.2.1a). When they occur close together or essentially at the same location, they are termed a microcell (Fig. 3.2.1b).

The cathodic reaction that occurs in any specific case depends on the availability of oxygen and on the pH of the cement paste pore solution near the steel reinforcement. This is shown by the Pourbaix (E-pH) diagram (Pourbaix 1974), illustrated in Fig. 3.2.1c, which delineates the thermodynamic areas of stability for each of the iron oxide types as a function of electrochemical potential and pH of the environment. The electrochemical potential is a measure of the ease of electron charge transfer between a metal and its environment; in this case, between the steel reinforcement and the cement paste pore solution. It is a property of the steel rein-

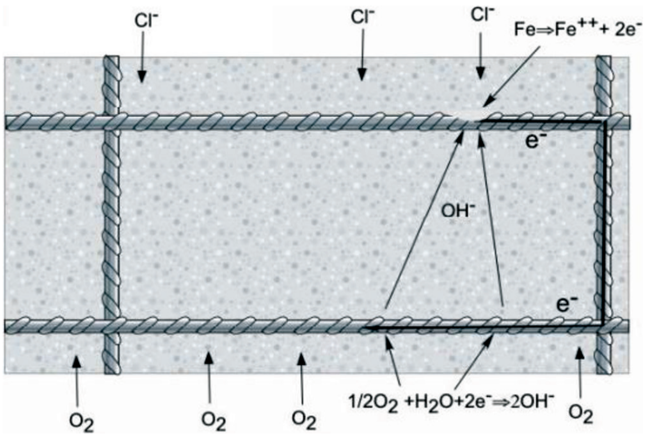

Fig. 3.2.1a—Macrocell corrosion (Hansson et al. 2006).

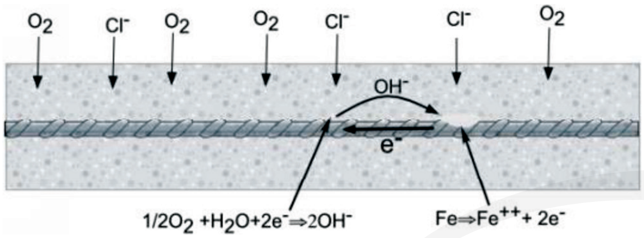

Fig. 3.2.1b—Microcell corrosion (Hansson et al. 2006).

forcement-concrete interface and not of the steel reinforcement itself. It is not possible to determine the absolute value of the potential and, therefore, it is necessary to measure the potential difference between the steel reinforcement surface and a reference electrode. This might be a standard hydrogen electrode (SHE), a saturated calomel electrode (SCE), a Cu/$CuSO_4$ electrode (CSE), or an Ag/AgCl electrode (SSCE). The value of the potential in a freely corroding system is commonly known as the corrosion potential, the open circuit potential, the free potential, or the half-cell potential. For the reaction shown in Eq. (3.2.1b) to occur, the potential should be more negative than that indicated by the upper dashed line, whereas the reaction shown in Eq. (3.2.1c) can only proceed at potentials more negative than the lower dashed line. In general, if all other factors are kept constant, when more oxygen is available, the electrochemical potential will be more positive (anodic).

For sound concrete, the pH of the pore solution is equal or greater than 13.0 and the half-cell potential more positive than –200 mV (CSE). Within this range, in the absence of any other factors, the iron oxides—Fe_3O_4 and Fe_2O_3 or hydroxides of these compounds—will form as solid phases and may develop as a protective (passive) layer on the steel reinforcement. If the pH of the pore solution is reduced, for example, by carbonation or by pozzolanic reactions, the system may be shifted to an area on the Pourbaix diagram in which these oxides do not form a protective layer and active dissolution is possible. Corrosion could theoretically be induced in very high pH environments at high temperature if the potential of the steel is held near –1.0 V (SHE), as per the following reaction

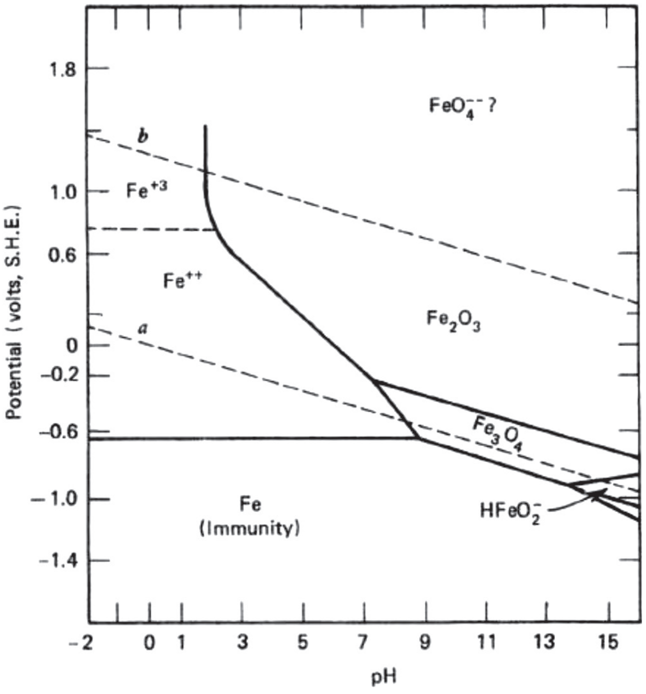

Fig. 3.2.1c—E-pH Pourbaix diagram 77°F (25°C) showing the potential pH ranges of stability of the different phases of iron in aqueous solutions (Revie and Uhlig 2008). The lines a and b represent the equilibrium potentials for the hydrogen evolution and oxygen reduction reactions, respectively.

$$Fe + OH^- + H_2O \rightarrow HFeO^{2-} + H_2 \qquad (3.2.1d)$$

where iron dissolves as $HFeO^{2-}$ (refer to Pourbaix diagram in Fig. 3.2.1d) (Townsend 1970). This condition is highly unlikely to exist in a reinforced concrete structure.

Corrosion of reinforcing steel in concrete can be caused by stray current corrosion or other environmental factors in addition to chemicals (Gummow and Meyers 1986; Bertolini at al. 2007). Stray current is received by the reinforcing steel in concrete or any metal electrically connected to the embedded steel and discharged elsewhere on its way to the source. Reinforcing steel corrodes at the point of current discharge. The most common sources of stray currents for reinforced concrete structures include DC-powered electric railways and electroplating plants. This type of corrosion most commonly occurs in structural elements in contact with the earth.

3.2.2 *Nature of the passive film*—A passive film can be relatively thick and inhibit active corrosion by providing a diffusion barrier to the reaction product of the reacting elements (Fe and O_2). Alternatively, and more commonly, it may be thin, often a few monolayers thick. In this case, the oxides simply occupy the reactive atom sites on the metal surface and prevent the metal atoms at these locations from dissolving. A passive film does not stop corrosion, but it does reduce the corrosion rate to an insignificant level. For steel reinforcement in concrete, the passive corrosion rate is typically ~4 × 10^{-5} in./year (~1 µm/year) or less; without the passive film, the steel reinforcement can corrode at rates at least three orders of magnitude higher than this (Hansson

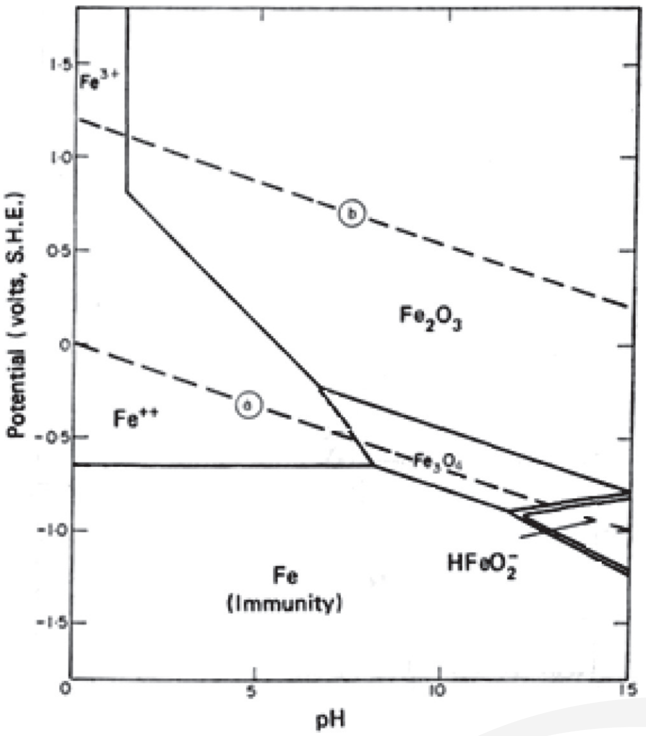

Fig. 3.2.1d—E-pH Pourbaix diagram 140°F (60°C) showing the potential pH ranges of stability of the different phases of iron in aqueous solutions (adapted from Townsend [1970]). The lines a and b represent the potentials for the hydrogen evolution and oxygen reduction reactions, respectively.

1984). The product FeOOH is considered to be the passive film formed at high pH. In recent years, numerous studies reported that the passive film formed on steel in alkaline solutions presents a bilayer structure (Sánchez-Moreno et al. 2009; Ghods et al. 2011a, 2012; Gunay et al. 2013) with an inner Fe^{2+} rich oxy-hydroxide (1 to 3 nm) and an outer Fe^{3+} rich film (5 to 10 nm), with the overall thickness typically in the range of 3 to 15 nm (Rossi et al. 2007; Ghods et al. 2011a, 2012; Gunay et al. 2013; Yu et al. 2015). Ghods et al. (2011a; 2012) and Gunay et al. (2013) reported that the inner film is dense and protective whereas the outer layer is relatively porous and unprotective.

3.2.3 *Kinetics of corrosion*—All metals, except very noble metals such as gold and platinum, are thermodynamically unstable in normal atmospheric conditions and will eventually revert to their oxides or other compounds, as indicated for iron in the E-pH (Pourbaix) diagram in Fig. 3.2.1c. Therefore, the information of importance to the engineer who would use a metal is not whether the metal will corrode, but how fast the corrosion will occur. The Pourbaix diagram only provides thermodynamic information on the stability of iron and its oxides, but it does not provide any information about corrosion kinetics. The corrosion rate can be determined as a corrosion current by measuring the rate at which electrons are removed from the iron in the anodic reactions described previously. The corrosion current can be converted to a rate of loss of metal from the surface of the steel using Faraday's law

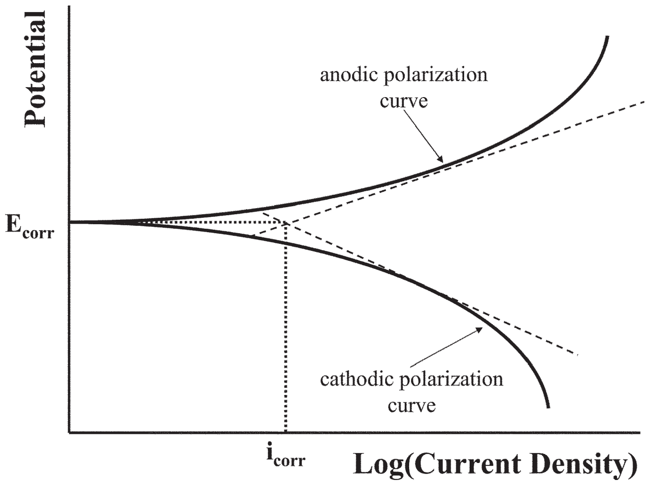

Fig. 3.2.3a—Schematic polarization curve for an actively corroding system (adapted from Jones [1996]).

$$M = \frac{itA_w}{nF} \tag{3.2.3a}$$

By dividing the mass of metal dissolved, M, by the metal's density and the corroding area of the steel reinforcement, the mass can be converted to an average thickness of the dissolved or oxidized layer.

For iron (or steel):

$$0.09\ \text{mA/ft}^2\ (1\ \text{mA/m}^2) = 0.046\ \text{mils/year}\ (1.16\ \mu\text{m/year}) \tag{3.2.3b}$$

The current density cannot be determined directly. This is because the requirement of a charge balance means that the rates of production and consumption of electrons by the anodic and cathodic half-cell reactions, respectively, are always equal and, therefore, no net current can be measured. Consequently, to determine the corrosion current density, the system should be displaced from equilibrium by applying an external potential and measuring the resultant net current (potentiostatic measurements). Alternatively, a known current can be applied and the resulting shift in electrochemical potential can be measured (galvanostatic measurements). The difference between the applied potential E and the original corrosion potential E_{corr} is termed the polarization and given the symbol η.

In the absence of passivity, but assuming the continued availability of oxygen and water, the net current would increase with anodic polarization, as shown by the upper curve in Fig. 3.2.3a and cathodic polarization would result in the lower curve. Note that the cathodic polarization curve current values are the absolute value of the actual current values. Tafel has shown that for values of η in the range ± 100 to 200 mV, η is directly proportional to the logarithm of the current density as follows (Burstein 2005)

$$\eta = a + b\log(i) \tag{3.2.3c}$$

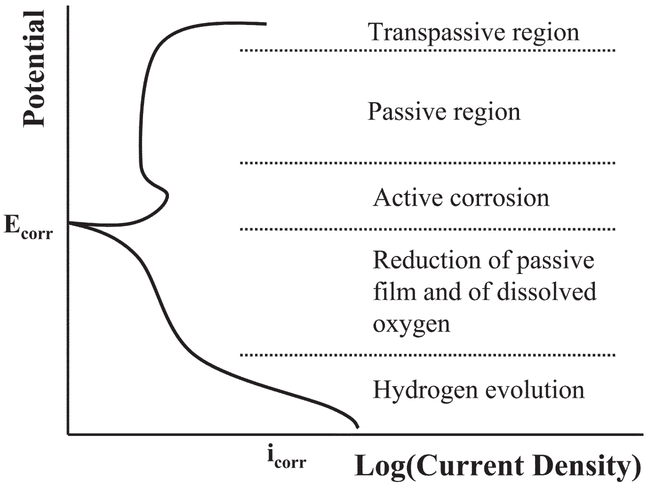

Fig. 3.2.3b—Schematic polarization curve for passive system with limited access of oxygen (adapted from Jones [1996]).

For metals in concrete, González et al. (1985) recommend values of η less than those recommended by Burstein (2005). A value of the corrosion current density i_{corr} (the current density when η is zero) can be obtained by extrapolating the linear portions of the anodic and cathodic polarization curves to E_{corr}, as shown by the dashed lines in Fig. 3.2.3a.

For steel reinforcement in concrete, however, the chemical protection provided by the formation of a passive film reduces the anodic current density by several orders of magnitude, as shown in Fig. 3.2.3b. The transition from the active corrosion part of the polarization curve to the passive region occurs because of the formation of a passive metal oxide film. Moreover, the physical barrier of the concrete and mill scale on the steel reinforcement can limit the oxygen access for the cathodic reaction and can result in a decrease in the cathodic current, also illustrated in Fig. 3.2.3b. These factors can significantly reduce the corrosion rate. They can also limit the accuracy by which the actual corrosion rate can be determined because the linear part of each curve no longer exists. This lack of accuracy is irrelevant, however, because a precise knowledge of the passive corrosion rate is of no practical interest. Polarization resistance (also known as linear polarization) (Andrade and Gonzalez 1978) and electrochemical impedance spectroscopy (EIS) (Mansfeld 1981; Ghods et al. 2010) measurements can be used to determine the passive corrosion current densities where they are needed for scientific reasons. Polarization curves (Tafel plots) for steel reinforcement in concretes of different qualities have been documented by Al-Tayyib and Khan (1988) and Martínez and Andrade (2011).

As illustrated in Fig. 3.2.3b, the value of the net anodic current density is approximately constant over a wide range of potential values but increases at potentials away from E_{corr}. This increase, called transpassive dissolution, can result from dielectric breakdown of the passive film. It can also be due to the potential being above that indicated by the upper dashed line in Fig. 3.2.1c. At these potentials, O_2 can be evolved at atmospheric pressures by the reverse of the reaction shown in Eq. (3.2.1b) or by the hydrolysis of water

$$2H_2O \rightarrow O_2 + 4H^+ + 4e^- \qquad (3.2.3d)$$

adding a second anodic reaction to that of the passive corrosion of iron. A third reaction would involve the oxidation of ferrous ions (Fe^{2+}) to ferric ions (Fe^{3+}), which is an anodic reaction.

3.2.4 *Initiation of active corrosion*—Active corrosion of steel reinforcement in concrete is typically preceded by the breakdown of the protective, passive film. This can occur over large surfaces of the steel due to general changes in the thermodynamic conditions or locally due to chemical attack or mechanical failure of the protective film. The former is usually a result of a decrease in pH to a level at which the passive film is no longer stable. The latter is usually caused by attack by aggressive ions such as chlorides but could also result from cracking in the concrete cover.

3.2.4.1 *Corrosion initiation by chlorides*—The most common cause of initiation of corrosion of steel reinforcement in concrete is the presence of chlorides. The source of chlorides may be marine environments, deicing and anti-icing salts, industrial brines, contamination, or chloride-containing admixtures.

The exact breakdown mechanism of the passive film by chlorides has not been well defined because of the difficulties in examining the process on an atomic scale in the extremely thin passive layers. Early research (Venu et al. 1965; Hausmann 1967; Gouda 1970) indicated that in the thicker films, the chlorides become incorporated in the film at localized weak spots, creating ionic defects and allowing easy ionic transport. In the case of submonolayer passivity, the chlorides may compete with the hydroxyl ions for locations of high activity on the metal surface, preventing these reactive sites from becoming passivated (Rosenberg et al. 1989). More recent studies indicate that changes in surface and local critical chemical conditions enable charge exchange pathways that eliminate the blocking ability of the passive film, thereby depassivating the surface (Díez-Pérez et al. 2006; Marcus et al. 2008; Ghods et al. 2011a, 2012; Gunay et al. 2013). Ghods et al. (2011a, 2012) and Gunay et al. (2013) have shown that chlorides cause the decrease of the relative amount of Fe^{+2} oxides with respect to Fe^{+3} oxides, eventually causing depassivation of the steel surface.

The net result is that active corrosion can occur at these locations and, once started, it proceeds autocatalytically, that is, in a self-feeding manner. The chlorides and ferrous ions react to form a soluble complex that diffuses away from the anodic site. When the complex reaches a region of high pH, it decomposes, precipitating an insoluble iron hydroxide and liberating the chloride to remove more iron from the steel reinforcement. Moreover, because the region of local breakdown of the passive film becomes anodic, more chlorides are attracted to that surface of the steel than to the surrounding cathodic areas and so the local concentration of chlorides increases. Because of this process, the solid corrosion products may be formed within the concrete cover, rather than at the steel reinforcement-concrete interface (Marcotte and Hansson 1998; Jaffer and Hansson 2009).

The initial precipitated hydroxide has a low state of oxidation and tends to react further with oxygen to form higher oxides. Evidence for this process can be observed when concrete with active corrosion is broken open. A light green semisolid intermediate reaction product (likely $3Fe(OH)_2 \cdot Fe(OH)_2Cl \cdot nH_2O$) is often found near the steel reinforcement that, upon exposure to air, turns brown/black (likely a combination of $Fe(OH)_3$ and Fe_3O_4), and subsequently rust colored (likely a combination of $Fe(OH)_3 \cdot 3H_2O$, α-FeOOH, β-FeOOH and Fe_2O_3) (Herholdt et al. 1985; Marcotte 2001). The iron hydroxides have a larger specific volume than the steel from which they were formed, as indicated in Fig. 3.2.4.1. In practice, the corrosion products have been found to have volumes of 2 to 3.5 times that of the steel from which they were formed (Marcotte and Hansson 1998). Consequently, the increases in volume as the corrosion reactions proceed leads to internal stresses within the concrete that may be sufficient to cause cracking and spalling of the concrete cover. A second factor in the corrosion process that is often overlooked because of the more dramatic effect of the spalling is the increased acidity in the region of the anodic sites that can lead to local dissolution of the cement paste (Bertolini et al. 1996).

3.2.4.1.1 *Incorporation of chlorides in concrete during mixing*—Using calcium chloride ($CaCl_2$) as a set accelerator for concrete is the most common source of intentionally added chlorides. Because chlorides promote reinforcement corrosion, the use of chloride-containing admixtures for reinforced concrete is strongly discouraged, and for many applications, it is not permitted (ACI 318-14). When chlorides are added to concrete during mixing, intentionally or otherwise, rapid corrosion can occur.

Chlorides added to the mixture have additional effects on subsequent corrosion rates. First, the chlorides increase the ionic concentration of the pore solution and its electrical conductivity (Enevoldsen et al. 1994). These factors lead to an increase in the corrosion rate. Second, chloride-bearing salts can alter the pH of the concrete pore solution; sodium chloride (NaCl) and potassium chloride (KCl) increase the pH whereas $CaCl_2$, in high concentrations, reduces the pH (Browne 1980). This affects both the chloride binding and the chloride threshold value for corrosion (Tritthart 1989b).

3.2.4.1.2 *Transport of chlorides from the environment into mature concrete*—Transport of chlorides into concrete can occur in sound concrete and proceed through the capillary pore structure of the cement-paste phase. Therefore, cracks in the concrete are not a prerequisite for transporting chlorides to the steel reinforcement. The rate of transport depends strongly on many factors, including the water-cementitious materials ratio (*w/cm*), type of cement, presence of supplementary cementitious materials (SCMs), specific cation associated with the chloride, temperature, and concrete maturity (Goto and Roy 1981; Page et al. 1981, 1986; Hansson 1984; Midgley and Illston 1984; Schonlin and Hilsdorf 1988; Mehta 1980, 1989; Balabanić et al. 1996; Li et al. 1999; Nokken and Hooton 2006; Jaffer and Hansson 2008).

Although cracks are not necessary for the ingress of chlorides, reinforced concrete generally contains cracks. The presence of cracks can enhance the ingress of chlorides, decrease the initiation time for corrosion, localize the corrosion attack to the intersection of the crack with the steel reinforcement, and reduce (but not eliminate) the advantages of concrete (Weiermair et al. 1996; Schiessl and Raupach 1997; Thuresson et al. 1997; Marcotte and Hansson 1998; Gérard and Marchand 2000; Gagne et al. 2001; Boulfisa et al. 2003; Marcotte and Hansson 2003; Okulaja and Hansson 2003).

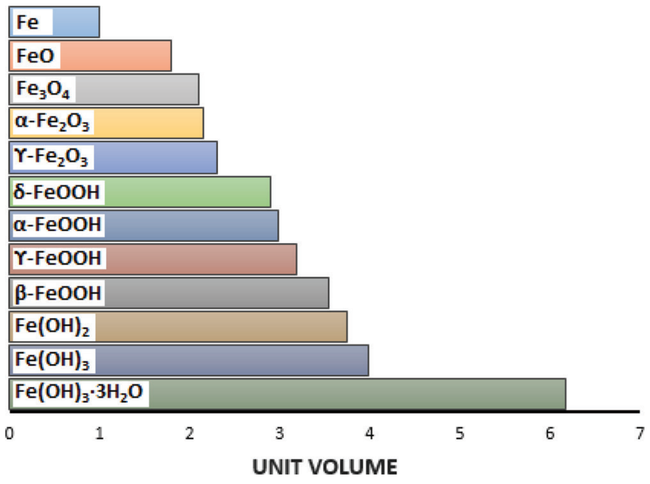

Fig. 3.2.4.1—The relative volumes of iron and its reaction product (adapted from Marcotte [2001]).

3.2.4.1.3 *Chloride binding and threshold values*—Not all chlorides in the concrete contribute to the corrosion of the steel reinforcement. Some of the chlorides react chemically with cement components, such as the calcium aluminates, to form calcium chloroaluminates, and are effectively removed from the pore solution. As the concrete carbonates, however, the chlorides can be released and become involved in the corrosion process. Research indicates that some chlorides also become physically trapped either by adsorption or in unconnected pores (Midgley and Illston 1984; Zhang and Gjørv 2005). The fraction of total chlorides available in the pore solution to cause breakdown of the passive film on steel is a function of numerous parameters, including the tricalcium aluminate (C_3A) and tetracalcium aluminoferrite (C_4AF) contents (Tritthart 1989a), pH (Tritthart 1989b; Byfors 1986), and *w/cm* (Tritthart 1989a,b; Byfors 1986). Higher pH values require more chlorides to initiate corrosion and pitting. Some of these effects are summarized in Fig. 3.2.4.1.3, which shows the effects of relative humidity and quality of the concrete cover on the critical chloride threshold (CEB 2000). The threshold value of 0.4 percent Cl^- by mass of cement proposed by CEB (2000) (approximately 2.4 lb/yd^3 [1.4 kg/m^3] of concrete), however, is higher than the acid-soluble chloride threshold value typically used in the United States, which is 1.0 to 1.5 lb/yd^3 (0.6 to 0.9 kg/m^3) of concrete. Moreover, research has indicated a wide range of values for chloride threshold (Hansson and Sørensen 1990; Glass and Buenfeld 1997; Alonso et al. 2000; Trejo and Pillai 2003; Poupard et al. 2004; Angst et al. 2009).

It has been well documented that initiation of steel reinforcement corrosion is not only dependent on the chlo-

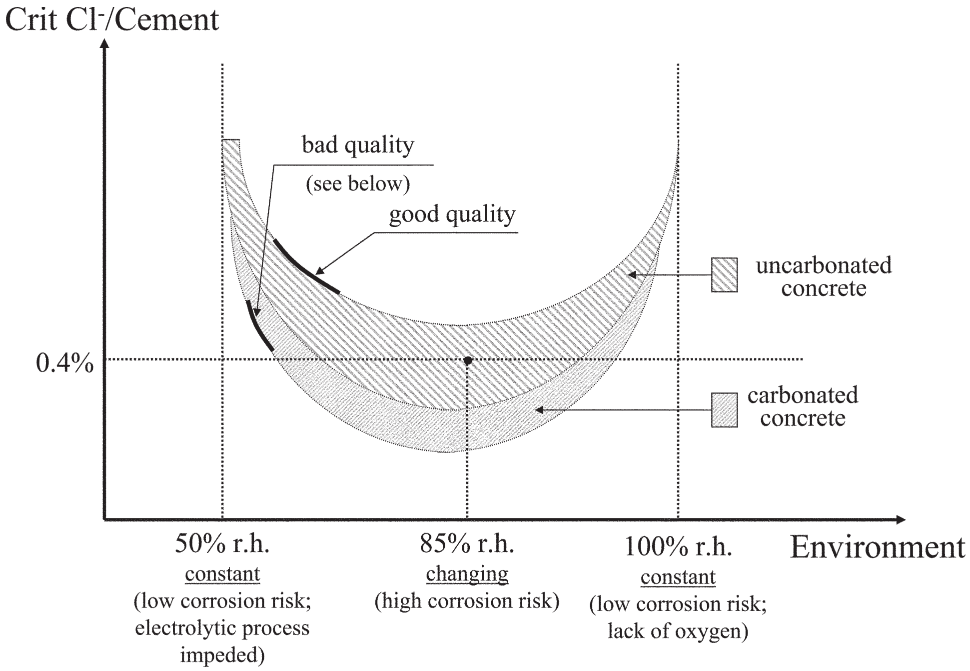

Fig. 3.2.4.1.3—Critical chloride content according to CEB-FIB (1992) recommendations (Goto and Roy 1981).

ride concentration, but also on the OH^- concentration and, specifically, the chloride-to-hydroxyl ion ratio (Cl^-/OH^-) (Venu et al. 1965; Hausmann 1967; Gouda 1970; Thangavel and Rengaswamy 1998; Alonso et al. 2000; Li and Sagüés 2001). Angst et al. (2009) provide an overview of Cl^-/OH^- values published in the literature and reported Cl^-/OH^- values for corrosion activation ranging from 0.01 to 6.0 for steel embedded in cementitious materials. CEB-FIP (1992) reported the maximum Cl^-/OH^- value that can be tolerated without breakdown of the passive film to be 0.29 at a pH of 12.6 and 0.30 at a pH of 13.3. Care should be taken when considering appropriate chloride values for the use in predictive models or estimation of life expectancy of concrete structures. Higher values, such as that provided by CEB-FIB (1992), may result in significantly longer predicted structural lives than use of the lower values typically adopted in North America.

3.2.4.2 *Initiation of corrosion by carbonation*—Carbonation is the general term given to the neutralization of concrete by reaction between the alkaline components of the cement paste and carbon dioxide (CO_2) in the atmosphere. Because the reaction proceeds in solution, the first indication of carbonation is a decrease in pH of the pore solution when the pH level decreases to a level as low as 8.5, at which level the passive film on steel is not stable. Carbonation generally proceeds in concrete as a front, beyond which the concrete is not affected and the pH is not reduced. When the carbonation front reaches the steel reinforcement, general depassivation over large areas or over the entire steel surface can occur and general corrosion can be initiated (Roberts 1981; Papadakis et al. 1991; Saetta and Vitaliani 2004; Isgor and Razaqpur 2004). The depth of carbonation can be predicted using numerical techniques (Papadakis et al. 1991; Saetta and Vitaliani 2004; Isgor and Razaqpur 2004), but it is also possible to approximate the increase in the depth of carbonation, d_c, in proportion to the square root of time, t_y (years), as (Currie 1986)

$$d_c = kt_y^{0.5} \tag{3.2.4.2a}$$

Fortunately, carbonation rates in sound concrete with low *w/cm* values are generally low. However, concrete in or near industrial areas may experience higher carbonation rates due to increased CO_2 concentrations in these environments. Under natural conditions, the atmospheric concentration of CO_2 is 0.03 percent. In cities, this is typically increased to 10 times that value and in industrial sites, it can be as high as 100 times naturally occurring levels (EPA 2016).

The ingress of gases is higher at low relative humidities, but the reaction between the gas and the cement paste takes place in solution and is higher at high humidities. Therefore, the most aggressive environment for concrete neutralization is that of alternate wet and dry cycles and high temperatures (Pfeifer and Scali 1981). Under constant conditions, an ambient relative humidity of approximately 60 percent has been the most favorable for carbonation (Tuutti 1982; Houst and Wittmann 2002). Three other major factors that influence corrosion initiation times for carbonation-induced corrosion are: thin concrete cover, the presence of cracks (Beeby 1978a,b, 1983), and high porosity associated with a low cement factor and high *w/cm*.

3.2.4.3 *Synergistic effects of carbonation and chlorides*—Research by Tuutti (1982) has shown that chloride content at the carbonation front reaches higher levels than in uncarbonated concrete and can be higher than levels measured just below the concrete surface. This increases the risk of corrosion initiation when the carbonation front reaches the steel reinforcement. The decrease in pH of the carbonated concrete also increases the risk of corrosion because the concentration of chlorides necessary to initiate corrosion—the threshold value—decreases with decreasing pH (Papadakis 2000; Puatatsananon and Saouma 2005). This is because the chloroaluminates break down, freeing the bound chlorides as the pH drops.

3.2.5 *Corrosion rates after initiation*—Depassivation, either local or general, is necessary but not sufficient for active corrosion to occur. The presence of moisture and oxygen is essential for corrosion to proceed at a significant rate. Corrosion can occur in wet, de-aerated concrete with hydrogen evolution as the cathodic reaction, but the rates of corrosion are much lower than in chloride-contaminated aerated concrete (Hansson 1986).

Chlorides are directly responsible for initiating corrosion, but they appear to play only an indirect role in determining the rate of corrosion after initiation. The primary rate-controlling factors are the availability of oxygen, the electrical resistivity, moisture content, pH, and temperature. However, the chloride compounds can influence the pH; electrical conductivity; porosity; and, because salts are hygroscopic, the moisture content. Similarly, carbonation destroys the passive film but does not influence the rate of corrosion. After corrosion initiation, corrosion rates may also be reduced by using corrosion inhibitors (3.4.5).

Drying of hardened concrete requires transport of water vapor to the concrete surface and subsequent evaporation. Wetting dry concrete, on the other hand, occurs by capillary suction and is, therefore, considerably faster than the drying process (Volkwein 1993; Baroghel-Bouny 2007). Consequently, concrete rarely dries out completely except for a thin layer at the surface (Hu and Stroeven 2003; Jiang et al. 2006). Below this surface layer, there will usually be a film of moisture on the walls of the capillaries and the bottlenecks in the pore system will be filled. Because the diffusion of dissolved oxygen is approximately four orders of magnitude slower than that of gaseous oxygen (Gjorv et al. 1976; Kobayashi and Shuttoh 1991; Sercombe et al. 2007), diffusion of dissolved oxygen through the bottlenecks will be the rate-controlling process in concrete at normal relative humidity levels. Laboratory studies suggest there is a threshold value within concrete, in the range of 70 to 85 percent relative humidity, below which active corrosion cannot take place (Enevoldsen et al. 1994). Similarly, a high electrical resistivity can inhibit the passage of the corrosion current through the concrete. This is particularly important in the case of macrocell corrosion where there is a significant separation between the anodic and cathodic reaction sites.

Fully submerged concrete structures tend to be protected from higher corrosion rates by lack of oxygen. Therefore, despite being contaminated by high concentrations of chlorides, structures continuously submerged below sea water may not be subject to significant corrosion. The part of a structure in the splash zone, however, experiences particularly aggressive conditions. It is generally water-saturated, contains high concentrations of salts, and is sufficiently close to the exposed parts of the structure that macrocells can easily be established. High salt levels are the result of salt water being transported by capillary action upward through the concrete cover and evaporation of water from the surface, leaving behind the salts.

3.3—Reinforcing bar

3.3.1 *Uncoated bars*—Conventional steel reinforcement is typically melted from scrap products; cast into billets; stored; reheated; rolled to size; and then cooled, stored, and distributed. Conventional bars are manufactured in accordance with ASTM A615/A615M or ASTM A706/A706M with the primary goal of meeting specified mechanical properties (tensile characteristics and bending). ASTM A615/A615M specifies four grades: 280, 420, 520, and 550 MPa (40, 60, 75, and 80 ksi) and limits the phosphorus to 0.06 percent. ASTM A706/A706M specifies two grades: 420 and 520 MPa (60 and 80 ksi) and also places limits on the carbon (maximum 0.30 percent), manganese (maximum 1.50 percent), phosphorous (maximum 0.035 percent), sulfur (maximum 0.045 percent), and silicon (maximum 0.50 percent). Although ASTM A615/A615M does not prohibit welding of the product, the specification cautions the user from welding reinforcement meeting the specification. The composition limits specified in ASTM A706/A706M, in addition to the maximum carbon equivalent of 0.55 percent, enhance weldability of this product. In nonaggressive environments, products meeting both ASTM A706/A706M and ASTM A615/A615M can provide long-term performance. In aggressive environments where corrosion could be an issue, coated or corrosion-resistant reinforcement may be required. Alternatives to uncoated conventional steel reinforcement are epoxy-coated reinforcement, galvanized steel reinforcement, corrosion-resistant reinforcement, or stainless steel reinforcement. More information on steel reinforcement can be found in ACI 439.4R.

3.3.2 *Epoxy-coated steel reinforcement*—Epoxy-coated steel reinforcement has been widely used in many jurisdictions in aggressive environments for decades and has generally been successful in delaying corrosion due to the ingress of chlorides (Kilareski 1977; McDonald 2010). ASTM A775/A775M and ASTM A934/A934M are standard specifications that address coating application, fabrication, and testing of epoxy-coated reinforcement. In addition, ASTM A1055/A1055M prescribes requirements for a relatively new product for deformed and plain steel reinforcing bars. These bars have a dual zinc alloy and epoxy coating.

The widespread use has been based on a many laboratory and field studies (Bentur et al. 1997; Sagüés and Zayed 1989; Zayed and Sagüés 1990; McDonald et al. 1995; Manning 1996; Weyers et al. 1998; Samples and Ramirez 1999; Lee and Krauss 2004; Saeed et al. 2004; Al-Amoudi et al. 2004; Cui et al. 2007; Pincheira et al. 2008; Lau and Sagüés 2009; Lawler et al. 2011). To provide long-term corrosion resistance of epoxy-coated steel reinforcement, the coating should be:

a) Relatively free of coating breaks and defects
b) Maintain high electrical resistance
c) Confine corrosion to bare areas
d) Resist undercutting

e) Resist the movement of ions, oxygen, and water

These issues are addressed by ASTM A775/A755M, which has the following requirements:

a) The coating thickness should be 7 to 12 mils (175 to 300 μm) for bar sizes No. 3 to 5 (No. 10 to 16) and 7 to 16 mils (175 to 400 μm) for bar sizes No. 6 to 18 (No. 19 to 57).

b) Bending of the coated bar around a standard mandrel should not lead to formation of cracks.

c) Surface roughness should have an average blast profile roughness depth of 1.5 to 4 mils (0.04 to 0.10 mm).

d) The surface should be free from salt contamination prior to coating.

e) The number of pinhole defects should be no more than six per meter.

f) All damage should be repaired prior to placement into concrete.

Similar requirements are presented in ASTM A934/A934M. Since 1991, an improvement in the quality of epoxy-coated bars and understanding of adhesion of coatings to steel have developed, primarily because of additional research and plant certification programs. In 1991, the Concrete Reinforcing Steel Institute (CRSI) began a program of voluntary certification of plants that apply epoxy coatings to reinforcement (CRSI 1991). Other researchers have demonstrated that even damaged epoxy-coated reinforcement has improved corrosion performance when compared with uncoated carbon steel reinforcement (McDonald et al. 1995; O'Reilly et al. 2011; Lee and Krauss 2004).

Poor field performance of epoxy-coated bars has been documented in structures in the Florida Keys; Virginia; and Ontario, Canada (Manning 1996; Pyc et al. 2000; Pianca et al. 2005; Weyers et al. 2006; Sprinkel at al. 2010). Some researchers reported that the primary causes of corrosion were inattention to preparation of the bars before coating and debonding of the coating before placement in the structures. In addition, Hansson et al. (2000) reported that some jurisdictions have observed failures of epoxy-coated reinforcement and have stopped specifying the coated steel because: 1) there is no guarantee that the reinforcement will be placed with the coating in pristine condition; and 2) epoxy will absorb moisture from the concrete over time and could become debonded from the steel, allowing corrosion to take place.

Epoxy-coated welded wire reinforcement is also available, meeting ASTM A884/A884M. Protective epoxy coatings are available in two types based on the coating flexibility (Type I and Type II) and two thicknesses after curing as Class A or Class B. Type I coatings meet ASTM A775/A775M and Type II coatings meet ASTM A934/A934M. Class A materials have a minimum thickness of 175 micrometers (7 mils) while Class B coatings have a minimum thickness of 450 micrometers (18 mil) for use in mechanically stabilized earth applications. ASTM A884/A884M specifies that a Class A minimum coating thickness is required for wire and welded wire reinforcement intended for use in concrete and masonry.

3.3.3 *Zinc-coated (galvanized) steel reinforcement*—Zinc–coated (galvanized) steel reinforcement has been used in concrete for over 50 years. This reinforcement type has been reported to be particularly appropriate for protecting concrete subjected to carbonation because zinc remains passivated to much lower levels of pH than conventional uncoated steel does (Yeomans 2004). However, zinc dissolves in a high-pH solution with the evolution of hydrogen (H_2) as the cathodic reaction but develops a protective layer of calcium hydroxyzincate after some hours (Tan and Hansson 2008). When galvanized steel reinforcing bars are used in concrete, a porous layer of concrete can form around the bar before the protective film is formed, if steps are not taken to prevent it. A small amount of chromate salt may be added to the fresh concrete to prevent hydrogen evolution (Yeomans 2004), and calcium nitrite has been used to prevent hydrogen evolution of galvanized precast concrete forms (Berke et al. 1990; Yeomans 2004).

Zinc-coated or galvanized welded wire reinforcement meeting the requirements of ASTM A1060/A1060M is also available. ASTM A1060/A1060M is intended to be applicable to cold-worked wire, drawn or rolled, plain or deformed, coated with zinc in a continuous process or alternatively through a hot-dip process of the welded wire reinforcement after fabrication.

3.3.4 *Stainless steel*—Stainless-steel reinforcement is being used more often as a reinforcing material for structures in particularly aggressive environments (Neuhart 2000; IMOA 2007). ASTM A955/A955M covers deformed and plain stainless steel bars for concrete reinforcement for applications requiring corrosion resistance or controlled magnetic permeability and requires demonstration of a minimum level of corrosion resistance. AASHTO MP 18M/MP 18 also prescribes requirements for uncoated concrete steel reinforcement where corrosion-resistant performance is needed.

Stainless reinforcing steels were initially used in the 1930s and the continued performance of the oldest significant installation, the Progresso Pier, Yucatan, Mexico (constructed 1937-1941), has been the subject of ongoing research (Madrid et al. 2007). Several researchers reported that stainless steels exhibit higher corrosion resistance than conventional uncoated steel reinforcement (McDonald et al. 1995; Neuhart 2000; Trejo and Pillai 2004; Bautista et al. 2006; Hartt et al. 2007, 2009; IMOA 2007; Kouřil et al. 2010). Higher corrosion resistance can result in longer service lives.

In some applications, stainless steels have been used in combination with other types of reinforcement as a cost-effective design for applications in aggressive environments. These designs typically use stainless steel for the outer layers of a structure that are expected to eventually have chloride exposure in combination with carbon steel for inner parts that are not likely to be exposed to chlorides. Researchers have reported that the use of stainless steel in combination with carbon steel does not increase the risk of corrosion of passive carbon steel (Bertolini et al. 2002, 2013; Knudsen et al. 1999; Knudsen and Skovsgaard 2001; Pérez-Quiroz et al. 2008; Klinghoffer et al. 2000).

ASTM A941 reports that stainless steel is a steel that contains a minimum of 10.5 percent chromium and a maximum carbon content of 1.2 percent. The stainless steels used for reinforcement meeting the ASTM A955/A955M corrosion requirements have higher chromium contents, generally 16 percent or greater. Nickel, molybdenum, and nitrogen, typically used in lower percentages than chromium, are other alloying elements that increase corrosion resistance. In general, stainless steels are grouped by their metallurgical microstructure (ferritic, austenitic, martensitic, and duplex [austenic and ferritic]). For aggressive concrete environments, molybdenum and nitrogen, in addition to chromium, have been reported to be influencing factors for corrosion resistance.

When referring to steel reinforcement for concrete, it has been common to classify stainless steels by the numbering system originally developed by the American Iron and Steel Institute (AISI): 300 series steels are austenitic and 2000 series are duplex. In addition, Unified Numbering System (UNS) designations provide an internationally recognized identification for specifying stainless steels. Common stainless steel reinforcement types include the austenitic steels 304LN (UNS S30453), 316LN (UNS S31653), and XM-28 (UNS S24100); and the duplexes 2304 (UNS S32304), and 2205 (UNS S32205), although other alloys are available. Letters following the ANSI grade number refer to the limitation or presence of other elements (for example, L indicates low-carbon and N indicates the addition of nitrogen). Series 300 austenitic stainless steels are non-ferromagnetic, relatively ductile, and contain chromium and nickel as the primary alloys. Chromium provides corrosion resistance and nickel acts to stabilize the austenite phase. Some alloys also include molybdenum for enhanced corrosion resistance. Duplex stainless steels contain chromium as the primary alloying additions with sufficient austenite stabilizers to produce a discontinuous ferrite phase within a continuous austenite phase in roughly equal proportions. This creates a matrix with properties that are different from those with individual phases. Duplex steels are generally much stronger than austenitic steels with similar pitting resistance. It should be noted that different types of stainless steel are commercially available in each specific grade, and the corrosion resistance of these can vary (McDonald et al. 1995; Sykes 1995; Hansson et al. 2000; García-Alonso et al. 2007; Yamaji et al. 2004; Trejo and Pillai 2004; Islam et al. 2013; Van Niejenhuis et al. 2016).

Stainless steel welded wire reinforcement is also available, meeting ASTM A1022/A1022A. This specification covers stainless steel wire and welded wire reinforcement from hot-rolled stainless steel rod to be used as concrete reinforcement with corrosion-resistant and magnetic permeability properties. The material may be cold-worked, drawn or rolled, and plain or deformed.

3.3.5 *Other corrosion-resistant steel*—Although there is no consensus of what defines a steel reinforcement as being corrosion-resistant, for the purposes of this document a corrosion resistant steel will be one that exhibits better corrosion resistance than conventional uncoated steel reinforcement (ASTM A615/A615M; A706/A706M) and resists corrosion in the environment in which it will be exposed. ASTM A1035/A1035M is a standard specification for a low-carbon chromium steel reinforcement. Several researchers report that low-carbon chromium steel reinforcement exhibits higher resistance to corrosion than conventional uncoated steel reinforcement (Trejo and Pillai 2004; Hartt et al. 2007, 2009; Darwin et al. 2009). Higher resistance to corrosion can result in longer service lives.

3.4—Concrete environment

Concrete is the primary barrier against external aggressive agents. Good concrete cover limits the ingress of aggressive chemicals and provides a medium with high electrical resistivity. In this section, main parameters of the concrete environment that affect these properties are described.

3.4.1 *Cement and pozzolans*—The composition and availability of the pore solution, rather than the concrete itself, are the controlling factors of active corrosion of steel reinforcement. Therefore, those components of the concrete that determine the pH of the pore solution, the total porosity, and the pore-size distribution are of importance for the corrosion process.

When portland cement hydrates, the calcium silicates react to form calcium silicate hydrates and calcium hydroxide ($Ca(OH)_2$). The $Ca(OH)_2$ provides a buffer for the pore solution, maintaining the pH level at approximately 12.6. The pH is generally higher than this value (typically 13.5) because of the presence of potassium and sodium hydroxides (KOH and NaOH), which are considerably more soluble than $Ca(OH)_2$. KOH and NaOH are present in limited quantities, however, and any carbonation or pozzolanic reaction rapidly reduces the pH to that of the saturated $Ca(OH)_2$ solution. Thus, concerning corrosion, the higher the total alkali content of the cement, the better the corrosion protection. However, increasing the alkalis can result in other deterioration—reactive aggregates in the presence of alkalis can lead to expansive and destructive alkali-aggregate reactions. During the initiation and propagation stages of corrosion, concrete cover acts as a barrier against the transport of chlorides, carbon dioxide, oxygen, moisture, and other aggressive agents. The importance of concrete cover depth and quality on the corrosion process has been reported (Alonso et al. 1988; Smith and Virmani 2000; Russell 2004; Zhang and Lounis 2009). If the concrete cover is cracked, a faster path for transport of these agents to the steel reinforcement is provided (Marcotte and Hansson 2003; Fanous et al. 2000; Bertolini et al. 2013; Poursaee and Hansson 2008; Pacheco and Polder 2010). Where cracking is a concern, the effects of the amounts and types of cementitious materials, as well as necessary construction practices and precaution for the particular application should be considered. Cracking has been reported to be particularly pronounced in materials that rapidly gain strength (Krauss and Rogalla 1996).

For a given *w/cm*, the fineness of the cement and the pozzolanic components determine the porosity and pore-size distribution (Hansson and Sørensen 1987; Chindaprasirt et al. 2004). Generally, supplementary cementitious materials such as fly ash, slag, and silica fume reduce and refine the porosity and decrease the transport rate of aggressive chemicals into the concrete toward the steel reinforcement (Scali et al. 1987; Ampadu et al. 1999; Bermúdez and Alaejos 2010). Concretes containing these supplementary materials exhibit enhanced resistance to penetration of chlorides from the environment (Hooton 2000; Bouzoubaâ et al. 2000; Thomas et al. 2008). If too much pozzolan is used, however, all of the $Ca(OH)_2$ may be used in the pozzolanic reaction, effectively destroying the pH buffer and allowing the pH to drop to levels at which the steel reinforcement is no longer passivated (Wiens et al. 1995; Fraaij and Bijen 2004).

Traditionally, the binding capacity of cement for chlorides has been considered to be directly related to the C_3A content of the cement. The chlorides can react to form insoluble chloroaluminates, which removes them from the pore solution. There is a limit to the binding capacity of these aluminate phases, however, and equilibrium is always established between the bound and the free chlorides, so that even with high C_3A contents, there will always be some free chlorides in solution.

Evidence shows that a reaction with C_3A is only one of several mechanisms for effectively removing chlorides from solution. In ordinary portland cement, there is no direct relationship between the concentration of bound chlorides and the C_3A content. Thus, the total aluminate phases are considered to be involved in the binding (Rasheeduzzafar and Al-Saadoun 1993; Delagrave et al. 1997a; Zibara et al. 2008). Mehta (1977) reported a qualitative relationship with both the ($C_3A + C_4AF$) content and pH of the pore solution. The literature contains contradictory results on the effect of chloride binding for concretes containing supplementary cementitious materials (Ozyildirim 1987), but it has been well established that supplementary cementitious materials are beneficial in providing resistance to chloride-induced corrosion, primarily by reducing the transport rate of chlorides into the concrete (Li et al. 1999; Zhang et al. 1999; Thomas et al. 1999; Papadakis 2000; Obla et al. 2003). Some adsorption of chlorides on the walls of the pores, or in the interlayer spaces, and some trapping in unconnected pores may account for the higher chloride binding in blended cements with very fine pore structures (Sellevold et al. 1985).

Research indicates that supplementary cementitious materials, particularly fly ash, increase the rate of carbonation (Papadakis et al. 1992; Thomas and Matthews 1993; Malhotra et al. 2000; Sisomphon and Franke 2007). It appears that the decrease in buffer capacity by the pozzolanic reaction can allow the neutralization of the cement paste by atmospheric gases to proceed at a higher rate than in ordinary portland cement concretes.

3.4.2 *Water-cementitious materials ratio (*w/cm*)*—The porosity and the rate of penetration of deleterious materials are directly related to the *w/cm*. In general, a reduced *w/cm* results in improved corrosion resistance, provided the increase in stiffness, and reduced creep of the concrete does not result in increased cracking (Krauss and Rogalla 1996; Schmitt and Darwin 1999; Darwin et al. 2004).

3.4.3 *Aggregate*—Unless it is porous, contaminated by chlorides, or both, the aggregate generally has little influence on the corrosion of steel reinforcement in concrete. Free moisture on aggregate will contribute to the water content of a concrete mixture and effectively increase the *w/cm* if it is not accounted for by adjusting the batch water accordingly. The porosity of the paste surrounding the aggregate is usually higher than that of the paste, resulting in a weaker transition zone at the interface of the aggregate and hardened paste (Bentz et al. 1996; Delagrave et al. 1997b; Asbridge et al. 2001; Yang and Su 2002; Zheng et al. 2009; Angst et al. 2017). Therefore, if the size of the aggregate is nearly equivalent to the concrete cover over the reinforcement, the ability of the chlorides to reach the reinforcement is enhanced. If reactive aggregates are used and alkalis are present in the binder, alkali-silica reactions may take place. This can damage the concrete cover and potentially accelerate the corrosion process in certain environments.

3.4.4 *Curing conditions*—Increased curing times and longer times before being exposed to aggressive media can result in better resistance to penetration by chlorides or CO_2 (Fattuhi 1988; Bentur and Jaegermann 1991; Mangat and Limbachiya 1999). This is particularly important for concrete containing SCMs, especially those containing fly ash, in which the pozzolanic reaction is much slower than the portland cement hydration reactions. At early ages, fly ash concrete usually exhibits lower resistance to penetration of chlorides than an ordinary portland cement concrete, whereas at greater maturity, the fly ash concrete may have superior properties (Feldman 1981; Marsh et al. 1985; Alhoizaimy et al. 1996).

3.4.5 *Corrosion inhibitors*—A corrosion inhibitor for metal in concrete is a substance that reduces the corrosion of the metal without reducing the concentration of the corrosive agent. This is a paraphrase from the ISO definition (ISO 8044) of a corrosion inhibitor and is used to distinguish between a corrosion inhibitor and other additions to concrete that improve corrosion resistance by reducing chloride ingress into concrete. Corrosion inhibitors are not a substitute for sound concrete. They can work either as anodic or cathodic inhibitors, or both, or as oxygen scavengers. A significant reduction in the rate of either anodic or cathodic reactions will result in a significant reduction in the corrosion rate and an increase the chloride-induced corrosion threshold level. There is an enhanced effect when an anodic inhibitor is used. Adding an anodic inhibitor promotes the formation of limonite, a hydrous gamma ferric oxide, γ-FeOOH, which is a passive oxide at typical concrete pH levels. Adding a cathodic inhibitor or oxygen scavenger stifles the reaction in Eq. (3.2.1b), reducing corrosive oxidation as shown in Eq. (3.2.1a).

Numerous chemical admixtures, both organic and inorganic, have been shown to be specific inhibitors of steel corrosion in concrete (Berke and El-Jazairi 1990; Nmai et al. 1992; Pyc et al. 1999; Bolzoni et al. 2006). Among the inorganic corrosion inhibitors are potassium dichromate, stannous chloride, sodium molydbate, zinc and lead chromates, calcium hypophosphite, sodium nitrite, and calcium nitrite. Sodium nitrite has been investigated and used with apparent effectiveness in Europe (Corbo and Farzam 1989; Gu et al. 1997). Calcium nitrite is the most widely used inorganic corrosion inhibitor in concrete (Hope and Ip 1989; Ramasubramanian et al. 2001; Bola and Newtson 2005; Ann et al. 2006), and it has the advantage of not having the side effects of sodium nitrite—namely, low compressive strength, erratic setting times, efflorescence, and enhanced susceptibility to alkali-silica reaction. Organic inhibitors investigated have included sodium benzoate, ethyl aniline, morpholine, amines, and mercaptobenzothiazole (Sagoe-Crentsil et al. 1993; Hansson et al. 1998; Monticelli et al. 2000; Saraswathy et al. 2001; Trabanelli et al. 2005).

As with other admixtures, corrosion inhibitors can affect plastic and hardened concrete characteristics. Before using them, their effects on concrete properties should be understood and, where necessary, appropriate steps should be taken in consultation with the inhibitor manufacturer to overcome or minimize detrimental interactions. Because corrosion-inhibiting admixtures are water-soluble, there is concern that leaching from the concrete can occur, particularly of inorganic salts, effectively reducing the concentration of the inhibitor at the level of the reinforcement. When used in sound concrete with *w*/*cm* values less than or equal to 0.4 and with adequate concrete covers, the effects of leaching are significantly reduced (Berke et al. 1994).

CHAPTER 4—PROTECTION AGAINST CORROSION IN NEW CONSTRUCTION

4.1—Introduction

Corrosion in reinforced concrete structures can result in significant damage (Fig. 4.1) and often requires extensive repairs or replacement. Generally, these actions are expensive and, therefore, it is prudent to take economically justifiable measures for new structures to minimize corrosion during the anticipated life of the structure.

Measures to protect reinforcing steel against corrosion in new structures include:

a) Design approaches and choices

b) Methods of excluding external sources of chloride from concrete

c) Corrosion control methods—This chapter addresses each measure with the objective of assisting the reader in producing a durable and corrosion-resistant reinforced concrete structure.

4.2—Design approaches and choices

Every design process begins with setting expectations and performance criteria with the owner and stakeholders. As a design approach, corrosion resistance and economy over a defined time period, or design service life (ACI 562; ACI 365.1R), is most easily implemented when it is set as a goal in the early developmental stage of a project. Design considerations that can enhance the corrosion resistance of reinforced concrete structures include achieving the minimum concrete cover prescribed by ACI 318, selecting the reinforcement type, developing requirements for the detailing of the structure, and achieving proper drainage of the structure. It is critical that the designer know and understand the impact of the overall environmental conditions in which the structure will be exposed and the potential local environmental conditions surrounding the structure that could influence corrosion activity. Knowing the environmental conditions and understanding how these conditions influence corrosion of the reinforcement will assist in ensuring long-term and durable performance.

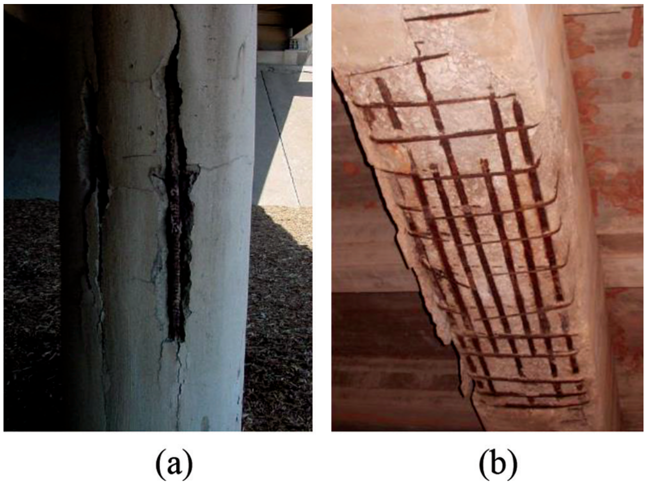
(a) (b)

Fig. 4.1—(a) Damaged bridge column and (b) damaged bridge beam as a result of reinforcement corrosion (photos courtesy of D. Trejo).

4.2.1 *Consideration of environmental conditions*—The rate at which a structure is damaged by corrosion is dependent on the global and local environments in which the structure is constructed. Chapter 3 provided information on the influence of chlorides and carbonation on corrosion of steel in concrete. For an actively corroding structure, temperature is also a significant environmental parameter. Environments in which chlorides are present and CO_2 levels are high will require corrosion prevention strategies during design. Sohanghpurwala (2006) defines corrosivity zones for the United States based on climatic conditions and the use of deicing salts. However, the author also reported significant variation in input variables used to define the zones and cautioned the reader on using these zones for design purposes. The designer should assess exposure conditions using generally available resources, but also exercise judgment regarding local environments and standards of practice. ACI 365.1R describes approaches to modeling the design service life and can assist in setting owner expectations.

In addition to global environmental conditions, local environmental conditions can influence the corrosion rate of the steel reinforcement (Lindvall 2000). For example, in concrete that is continuously submerged, the rate of corrosion is often low, limited by the rate of oxygen diffusion (Gjørv et al. 1976; Kobayashi and Shuttoh 1991; Hussain et al. 2012; Guo et al. 2015). Alternatively, splash zones or concrete embedded in soil, where deicing salt runoff occurs, are areas where high concentrations of chlorides can accumulate, which can lead to high rates of corrosion (Hansson et al. 2006). Thus, it is possible that different portions of the same structure might require different corrosion resistance strategies to achieve the design service life for the entire structure.

4.2.2 *Consideration of structural design parameters*—The detailing of a reinforced concrete structure is critical to its durability characteristics and service life. Adequate drainage and a method of removing drainage water from the structure are particularly important. Pockets that hold water or even horizontal surfaces that allow water to stand can lead to increased absorption of water containing aggressive ions (Hartt et al. 2004). Similarly, even vertical surfaces that stay damp for long periods may be subjected to a greater ingress of water and chlorides. For reinforced concrete structural members exposed to chlorides and subjected to intermittent wetting, the degree of protection against corrosion is determined primarily by the depth of concrete cover, the presence and width of cracks, and the transport rate of aggressive ions through the concrete (ACI 201.2R; Liu and Weyers 1998; Weyers 1998; Trejo and Reinschmidt 2007; Williamson et al. 2009; Hussain et al. 2012). Estimates of the increase in corrosion protection provided by an increase in concrete cover depth can be determined through the interrelationship with the rate of chloride ingress via apparent diffusion. Trejo and Reinschmidt (2007) showed that a 50 percent increase in cover depth can lead to an almost 200 percent increase in time to corrosion. Although larger cover can be beneficial, it can also lead to larger crack widths, increasing the risk of corrosion as discussed in ACI 224R. Therefore, selecting a cover that can result in sufficient service life and minimal costs during construction is important.

In addition to selecting proper cover, the specified reinforcement layout, size, and type can influence the corrosion performance of a structure. Reinforcement size and spacing directly impact the clear spacing between the reinforcing bars. This clear spacing dictates the maximum-size aggregate that can be used in the structure, and the aggregate size influences the characteristics of the interfacial transition zone (ITZ), particularly its size, and the tortuosity of the ionic transport path, both of which influence the chloride transport rate in concrete (Zheng and Zhou 2007). In addition, the time to cracking and spalling after the initiation of corrosion is a function of the concrete cover, bar diameter, the reinforcement spacing, and the concrete strength (Kulicki et al. 1990; El Maaddawy and Soudki 2007; Jang and Oh 2010; Guzmán et al. 2011; Mullard and Stewart 2011).

When chlorides are transported into hardened concrete, highly variable chloride concentrations can exist around the reinforcing steel. This variation is caused by differences in the concentration of chlorides on the concrete surface, local differences in transport rates, the effect of large aggregate pieces (Hartt and Nam 2008), and variations in the depth of concrete cover to the reinforcing steel, including the spacing between the top and bottom mats. These factors can promote differences in the oxygen, moisture, and chloride contents in the environment surrounding the steel reinforcement. Furthermore, most reinforced concrete structural members contain steel reinforcement at different depths that are usually connected electrically. Constructors commonly position and secure the reinforcing steel with bent bars, chairs, tie wires, or a combination of these, which permit metal-to-metal contact between the different reinforcement layers. Therefore, when chlorides penetrate the concrete, some of the reinforcing steel can be in contact with chloride-contaminated concrete while other reinforcing steel is in relatively chloride-free concrete. The difference in chloride concentrations within the concrete can create corrosion cells that can create a large driving potential and a large cathode-to-anode ratio that accelerates the rate of corrosion.

4.2.3 *Consideration of constituent materials*—Chlorides at sufficient quantities cause corrosion of reinforcing steel in concrete. Chlorides can be naturally present in constituent materials used to make concrete or from external contamination. Therefore, there is a need to define allowable limits of admixed chloride in concrete for new construction. However, there is debate as to what limits should be specified (Trejo and Weyers 2013). There is a wide range of allowable admixed chloride limits, depending on structure type and structure exposure. A limit of no chlorides in concrete is unrealistic because trace amounts of chlorides are naturally present in many concrete-making materials. In some cases, for example, when producing concrete under cold-weather conditions, producers of chloride-bearing aggregates and some admixture companies would prefer higher and fewer restrictive limits. The risk of corrosion, however, increases as the chloride content increases. When the chloride content exceeds a certain value, termed the critical chloride threshold, corrosion can occur if oxygen and moisture exist to support the corrosion reactions. It is impossible to establish a single chloride content below which the risk of corrosion is negligible for all mixture constituents, systems, and exposure conditions. In practice, the allowable chlorides in a new concrete should be significantly lower than the chloride concentration required to initiate corrosion if water and oxygen are available.

Because the allowable admixed chloride content is dependent on the critical chloride threshold, some background information on critical chloride threshold is provided herein. Significant research has been performed to assess the critical chloride threshold level required to initiate corrosion of steel reinforcement in concrete (Lewis 1962; Gouda 1970; Stratfull et al. 1975; Arup 1982; Hope and Ip 1987; Hansson and Sørensen 1990; Schiessl and Raupach 1997; Hussain et al. 1995; Thomas 1996; Glass and Buenfeld 1997; Breit 1998; Alonso et al. 2000; Trejo and Pillai 2003, 2004; Trejo and Monteiro 2005; Ann and Song 2007; Darwin et al. 2009; Ghods et al. 2010). Early research by Stratfull et al. (1975) indicated that the critical threshold value for steel reinforcement embedded in concrete ranged from 0.2 to 1.4 percent of total chlorides by mass of binder. Research at the Federal Highway Administration (FHWA) laboratories (Clear 1976) showed that hardened concrete subjected to external chlorides had a corrosion threshold of 0.20 percent by mass of cement (acid-soluble chlorides). A later study by Pfeifer et al. (1987) reported the threshold to be 0.21 percent by mass of cement. These corrosion threshold values were also confirmed with field studies of actual bridge decks, including several in California (Stratfull et al. 1975) and New York (Chamberlin et al. 1977). These field studies led to the common value of 1 lb Cl^-/yd^3 (0.59 kg Cl^-/m^3) of concrete used by many state highway agencies for predicting service life of reinforced concrete structures. Reported values in the literature for critical chloride threshold vary significantly. This can be attributed to the lack of a standard test for critical chloride threshold testing, the lack of a standardized definition for corrosion activation, lack of one standard chloride test method, and other factors.

Alonso et al. (2000) and Angst et al. (2009) performed reviews of the literature and identified several parameters that influence the critical chloride threshold. Angst et al. (2009) reported that the following influence the critical chloride threshold value:

a) Steel-concrete interface
b) pH of the pore solution
c) Electrochemical potential of the embedded metal
d) Surface condition of the embedded metal
e) Binder type
f) Moisture content of concrete
g) Oxygen availability
h) Water-binder ratio
i) Electrical resistivity of concrete
j) Degree of hydration of the cementitious material
k) Chemical composition of the embedded metal
l) Temperature
m) Source of chlorides (premixed or penetrated)
n) Type of cation accompanying chloride
o) Presence of other species

Other relevant factors, such as consolidation of concrete, irregularities on steel surface, reinforcement alloy composition, presence of admixtures (especially corrosion inhibitors), and others could also influence the measurement and value of the critical chloride threshold. Alonso et al. (2000) reported in the authors' experiments that the percent of total (acid-soluble) chlorides by mass of cement required to initiate corrosion ranged from 1.24 to 3.08 percent.

The Angst et al. (2009) review indicated that the literature reported values from near zero to 2.5 percent by mass of binder. This review also reported that mortar exhibited less scatter and lower upper limit values than concrete and that the test method for determining chloride threshold was a critical parameter. The authors also reported the following:

a) There is a need for a practice-related test method
b) The test method should use ribbed reinforcement in the as-received condition
c) The reinforcement should be embedded in mortar or concrete
d) Chlorides should be introduced by capillary suction and diffusion and should not be added to the fresh mixture
e) Corrosion activation (depassivation) should be detected by potential measurements, linear polarization, or electrochemical impedance spectroscopy

The critical chloride threshold depends on whether chlorides are present in the mixture constituents or penetrates the hardened concrete from external sources. When chlorides are added to the concrete, some will chemically combine with the hydrating cement paste, predominantly the aluminate phase (Enevoldsen et al. 1994). Chlorides can also be combined or bound when penetrating the concrete. The amount of chloride that forms calcium chloroaluminates is a function of the C_3A content of the cement (Roberts 1962; Rasheeduzzafar et al. 1991; Sakr 2005). As a result of binding, several researchers have reported elevated critical chloride threshold concentrations for admixed chlorides (Locke and Siman 1980; Browne 1980; Liu and Weyers 1998). Given the high solubility of chloride in water-based solutions, however, these bound chlorides may reenter the concrete pore solution over time, thereby increasing the available chlorides for corrosion (Azad and Isgor 2016). For example, bound chloride will be released during carbonation of the concrete creating a wave of increased chloride in front of the carbonation front (Broomfield 2007). The resulting decrease in pH and increase in chloride can initiate corrosion at very low chloride concentrations. Therefore, care should be taken when considering these elevated threshold values because the studies on the critical chloride threshold of steel reinforcement embedded in concrete with admixed chlorides were relatively short-term studies. The potential for later-age release of bound chlorides and how these influence service life was not assessed.

The condition of the reinforcement surface can also influence the critical chloride threshold and corrosion performance of a reinforced concrete system. Mammoliti et al. (1996), Pillai and Trejo (2005), Ghods et al. (2011b), Rossi and Elsener (2012), and Isgor et al. (2013) all showed that reinforcement surface condition can influence critical chloride threshold, corrosion activity, and performance. For example, Ghods et al. (2011b) and Isgor et al. (2013) reported that when millscale on steel reinforcement was removed by turning and polishing, the chloride thresholds increased by as much as an order of magnitude.

In addition to the wide range of critical chloride threshold values for conventional steel reinforcement, research has also focused on determining the threshold value for prestressing strands. Early research by Pfeifer et al. (1987) reported that for an unstressed prestressing strand, the chloride threshold was 1.2 percent by mass of cement, nearly six times that of conventional reinforcing steel. When stressed, the authors reported that the strand was more susceptible to corrosion but was still reported to be more resistant than mild conventional reinforcement. The authors concluded that commercially available strand wires were coated with zinc phosphate, calcium stearate, and other lubricants before drawing and these coatings may have provided corrosion protection to the tested strands. More recent research indicates that the threshold level of prestressing is much lower than the values reported for conventional reinforcement, ranging from 0.006 to 1.2 percent acid-soluble chlorides by mass of cement (Fernandez et al. 2013; Azuma et al. 2007; Trejo et al. 2009). Lee (2012) reported active corrosion of prestressing tendons in several bridges with chloride concentrations less than 0.003 percent by mass of cement. It should be noted, however, that these bridges also had other defects, including grout bleed voids, that could have exacerbated the observed corrosion of the tendons. Note that these are threshold levels and not necessarily limits on allowable chlorides in the cementitious mixture. Allowable limits should be much lower than the critical chloride threshold limit. ACI 318, AASHTO (2014), and PTI M55.1-12 limit the acid soluble chlorides in grout to 0.08 percent by mass of cement.

Corrosion of prestressing steel is generally a greater concern than corrosion of nonprestressed reinforcement because of the possibility that corrosion may result in a localized reduction in cross section and failure of the prestressing steel. The high stresses in the prestressing steel also render it more vulnerable to stress-corrosion cracking and, where the loading is cyclic, to corrosion fatigue. Also, prestressed structures can be less redundant than conventionally reinforced concrete structures. Li et al. (2011) reported that higher stresses in prestressing steel results in increased corrosion activity and that pitting corrosion is the main mechanism of attack on these steels. Most of the reported examples of failure of prestressing steel (CC Staff 1976; Peterson 1980; Goins 2000; Harries 2009) have resulted from corrosion, reducing the load-carrying area of the steel strands. Because of the potentially greater vulnerability and the consequences of corrosion of prestressing steel, chloride limits for prestressed concrete are lower than those for reinforced concrete. A review of corrosion of prestressed steels is provided in ACI 222.2R.

In determining a limit on the allowable admixed chloride content of the mixture constituents, several factors should be considered. The first factor is the test method used to assess the chloride concentration. Chloride concentrations can be determined with different tests, and these tests result in different concentrations. Two methods are typically used to determine the chloride content of hardened concrete: acid-soluble (ASTM C1152/C1152M) and water-soluble (ASTM C1218/C1218M) methods. The acid-soluble method measures chloride that is soluble in nitric acid and can include chloride bound within the hydrated cement paste and aggregate. The water-soluble chloride content method measures the chloride extractable in water under defined extraction conditions. Usually, chloride testing of concrete free of ingressed chloride is performed to determine the background chloride content, which can later be used to determine the amount of ingressed chloride content. The acid-soluble chloride test generally results in higher chloride concentrations than water-soluble chloride testing, and the ratio between the two quantities depends on the amount of chlorides present and cement chemistry (Mohammed and Hamada 2003). Typically, the ratio between acid-soluble and water-soluble chloride contents decreases with increasing chloride contents. This guide assumes that, on average, the water-soluble chloride concentration determined following ASTM C1218/C1218M is 20 to 25 percent lower than the acid-soluble chloride concentration determined by ASTM C1152/C1152M.

ASTM C1524 provides a method to determine the water-soluble chlorides in aggregate (that is, the Soxhlet method). This method cannot be used to determine the chloride concentration in grout, mortar, concrete, or other cementitious systems. Therefore, if aggregates contain chlorides and concern exists regarding the chloride concentration of the concrete containing these aggregates, it is recommended that the paste (cementitious materials, water, and admixtures) be assessed independently for chloride concentration following either the water- or acid-soluble testing and the aggregates be assessed independently following ASTM C1524. The chlorides from the water, cement, supplementary cementitious materials, admixtures, and any other constituent materials (using ASTM C1152/C1152M or ASTM C1218/C1218M) and the chlorides from the aggregates (using the Soxhlet method from ASTM C1524) should be summed to determine if the chlorides are lower than the required allowable limits.

In general, aggregates do not contain significant amounts of chloride, but there are exceptions. Aggregates from southern Ontario, Canada, have been reported to have an acid-soluble chloride content of more than 0.1 percent by mass, of which less than one-third is water-soluble, even when the aggregate is pulverized (Rogers and Woda 1977). Aggregates (both fine and coarse) exposed to seawater or brackish water could also contain high levels of chlorides. Some chlorides in aggregates are not soluble when unpulverized and when the aggregate is placed in water over extended periods. Rogers and Woda (1977) reported that there was limited difference in corrosion performance of aggregates containing bound chlorides and aggregates containing no chlorides when used in reinforced concrete structures. The authors concluded that the chlorides in the aggregate were bound within the aggregate and this binding was so significant that these chlorides would not contribute to corrosion. Although some aggregates have bound chlorides that may not contribute to corrosion, some aggregates containing chlorides could release their chlorides, causing later corrosion. Aggregates, particularly those from arid areas or dredged from the sea, can contribute sufficient chlorides to the concrete and these can initiate early corrosion. Because of this, care should be taken when using aggregates containing chlorides. ASTM C1524 can be used to assess water-soluble chlorides that could be freed from the aggregate.

Allowable admixed chloride limits in building codes vary widely. ACI 318 allows a maximum water-soluble chloride content by mass of cement of 0.06 percent in prestressed concrete, 0.15 percent for reinforced concrete exposed to moisture and an external source of chlorides, 0.30 percent for concrete exposed to chlorides from external sources, and 1.00 percent for reinforced concrete that will be dry or protected from moisture in service. The British Code, BS 8110, limits the total chloride content by mass of cement to 0.1 percent for prestressed concrete, 0.2 percent for concrete exposed to chloride in service, and 0.4 percent for concrete that will be dry or protected from moisture in service. These values are largely based on an examination of several structures that had a low risk of corrosion with up to 0.4 percent chlorides added to the mixture (CC Staff 1976). The Norwegian Code, NS 3420-L, allows an acid-soluble chloride content of 0.6 percent for reinforced concrete made with normal portland cement, but only 0.002 percent chloride for prestressed concrete. Other codes have different limits, although their rationale for establishing these limits is not well documented.

ACI publishes a wide range of allowable admixed chloride limits in their various committee documents and these limits seem to be dependent on structure type and exposure conditions. No documents address the allowable chloride limits for corrosion-resistant reinforcing steels. The allowable admixed chloride limits in Table 4.2.3 for concrete used in new construction, expressed as a percentage by mass of cementitious materials (with limits), are recommended to minimize the risk of chloride-induced corrosion. These values are the consensus of the committee.

Concrete materials that meet the requirements given in either of the relevant columns in Table 4.2.3 should be acceptable. If the concrete materials do not meet the relevant limits given in the table, alternate materials should be used. The committee emphasizes that these are recommended limits for new construction and not threshold values for corrosion initiation. These allowable admixed chloride limits should result in low risk of corrosion. However, there is no specific chloride concentration, other than zero, that will not have an associated level of risk of initiating corrosion of reinforcing steel.

Note that the limits published in Table 4.2.3 are based on cementitious materials. ACI 318 bases chloride limits on portland cement content. The reader is encouraged to review the papers in Tepke et al. (2016): *Chloride Thresholds and Limits for New Construction*. The SP provides information that indicates that the addition of SCMs, up to about 50% replacement (Azad and Isgor 2016), can provide similar resistance to chloride-induced corrosion; therefore, the limits in Table 4.2.3 are based on cementitious materials content, and not just portland cement content.

The maximum chloride limits recommended in Table 4.2.3 for reinforced concrete differ from those published in ACI 318. As noted previously, ACI Committee 222 has taken a more conservative approach than most other ACI committees because of the serious consequences of corrosion, the conflicting data on corrosion-threshold values, and the difficulty of defining the service environment throughout the life of a structure. Even so, the literature indicates that corrosion can occur at these limits and even below these limits. Potentially, some or all the bound chlorides in concrete, such as those combined with C_3A, may become unbound or free at later ages. This is due to reactions with carbonates or sulfates that displace or release the chloride in the insoluble compound of the concrete and free it into the pore solution during the corrosion process. Trejo and Weyers (2013) recommended basing chloride limits on exposure class (C0, C1, and C2 as in ACI 318), importance of structure, and on reinforcement type (conventional or prestressed). Structures of high importance would require lower chloride concentrations, structures exposed to moisture and chlorides would require lower chloride limits, and materials for prestressed or post-tensioned concrete would require the lowest limits. Limits should also consider steel reinforcement type (for example, conventional reinforcement, stainless steel reinforcement).

Table 4.2.3—Allowable admixed chloride limits for new construction

	Chloride limit for new construction (percent by mass of cementitious material[*])	
	Test method	
	Acid-soluble	Water-soluble
Category	ASTM C1152/C1152M	ASTM C1218/C1218M
Prestressed concrete	0.08	0.06
Reinforced concrete in wet conditions	0.20	0.15
Reinforced concrete in dry conditions[†]	0.30	0.25

[*] Portland-cement-based systems only. Total cementitious material includes portland cement and SCM; however, for determining allowable admixed chloride level, the SCM content cannot exceed the portland cement content.

[†] Typically interior concrete protected from moisture, high humidity, or both.

For prestressed and reinforced concrete exposed to chlorides while in service, the lowest possible chloride levels should be maintained in the concrete mixture to maximize the service life of the concrete. Consequently, chlorides should not be intentionally added to the concrete mixture or its constituent materials even if the chloride content of the materials is less than the stated allowable limits. In many exposure conditions, such as highway and parking structures, marine environments, and industrial plants where chlorides are present, additional protection against corrosion of embedded reinforcing steel may be necessary.

Because moisture content and oxygen concentration around reinforcement are critical for the corrosion process, there are some exposure conditions where corrosion will not occur at significant rates even though chloride levels may exceed recommended values. For example, as mentioned in 4.2.1, reinforced concrete continuously submerged in deep seawater rarely exhibits corrosion-induced distress because low oxygen levels slow the corrosion rate. If a portion of a reinforced concrete member is above the water level and a portion below water level, the portion above can exhibit significant corrosion compared to the lower portion due to formation of an oxygen-concentration cell. Similarly, where concrete is continuously dry, there is little risk of corrosion from chloride in the hardened concrete because transport of ions necessary to sustain corrosion is limited by the lack of moisture. Interior locations that are wetted occasionally, such as kitchens, bathrooms, and laundry rooms; buildings constructed with pumped lightweight concrete that is subsequently sealed before the concrete dries (for example, with vinyl tiles); and internal locations with high humidity can be susceptible to corrosion damage. Note that the designer has little control over the change in use or the service environment of a building, but the chloride content of the concrete mixture constituents should be specified.

Estimates of whether a specific environment will be dry can be misleading. Stratfull (1984) reported case studies of approximately 20 bridge decks containing 2 percent calcium chloride built by the California Department of Transportation (DOT). The bridges were located in an arid area where the annual rainfall was approximately 5 in. (125 mm), most of which fell during a short period. Within 5 years of construction, many of the bridge decks were showing signs of corrosion-induced spalling and most were removed from service within 10 years. For these reasons, a conservative approach to allowable chloride limits is needed.

4.2.4 *Consideration of mixture proportions*—Where concrete will be exposed to chlorides, the concrete should be made with the lowest *w/cm* (a design parameter) consistent with achieving adequate consolidation and curing (construction variables) and limiting cracking due to restrained shrinkage (Schmitt and Darwin 1999; Darwin et al. 2004, 2010; Lindquist et al. 2006). Although a low *w/c* (or *w/cm*) can reduce the chloride transport rate, cracking due to restrained shrinkage of concrete, such as occurs on bridge decks, is exacerbated by the increased concrete strength and lower tensile creep that result from a lower *w/cm* (Schmitt and Darwin 1999; Darwin et al. 2004, 2010). The cracks that form, typically directly over transverse reinforcing bars, result in greatly increased chloride contents at the level of the bar when the bridge deck is exposed to chloride-bearing deicing chemicals (Lindquist et al. 2006).

The effect of *w/cm* on time-to-corrosion initiation has been well documented in the literature (Jaegermann 1990; Balabanić et al. 1996; Frederiksen et al. 1997; Sharp and Mokarem 2014). Bentz and Thomas (2014) reported that the 28-day apparent diffusion coefficient ($D_{a,28}$ in m^2/s) is a function of *w/c* as follows

$$D_{a,28} = 10^{(-12.06+2.40 \cdot w/c)} \tag{4.2.4a}$$

Page et al. (1986) reported the influence of *w/c* and cover depth on chloride penetration, and these results are shown in Fig. 4.2.4a. The authors also reported on the degree of consolidation versus rate of ingress of chloride—these results are shown in Fig. 4.2.4b. Concrete with a *w/c* of 0.40 was found to resist chloride penetration significantly better than concretes with *w/c* values of 0.50 and 0.60. However, a low *w/c* alone is not sufficient to ensure low chloride transport rates. As shown in Fig. 4.2.4b, concrete with a *w/c* of 0.32 with poor consolidation is less resistant to chloride penetration than well-consolidated concrete with a *w/c* of 0.60. The combined effect of *w/c* and depth of concrete cover is shown in Fig. 4.2.4c, which illustrates the number of daily applications of salt before the chloride content reached a presumed critical value of 0.20 percent by mass of cement (acid-soluble) at the various depths. Thus, 1.5 in. (40 mm) of 0.40 *w/c* concrete was sufficient to protect embedded reinforcing steel against corrosion for 800 salt applications. Equivalent protection was provided by 2.75 in. (70 mm) of concrete cover with a *w/c* of 0.50, or 3.5 in. (90 mm) of 0.60 *w/c* concrete. Strategies to reduce cracking, larger concrete cover, or the provision of additional corrosion protection treatments may be required in some environments.

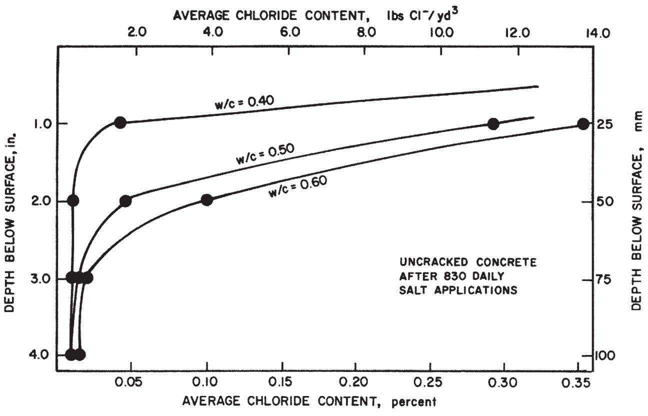

Fig. 4.2.4a—*Effect of* w/c *on salt penetration (Page et al. 1981).*

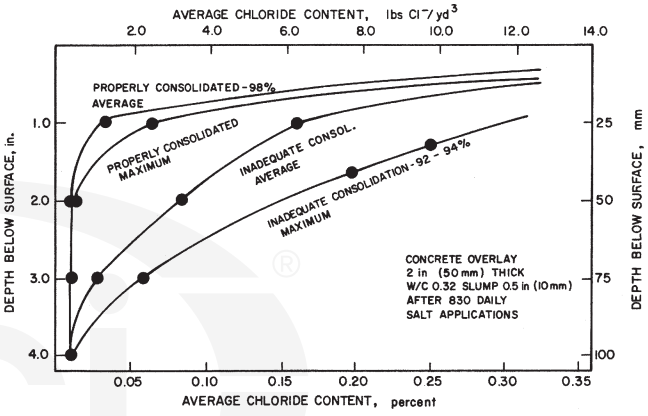

Fig. 4.2.4b—*Effect of inadequate consolidation on salt penetration (Page et al. 1981).*

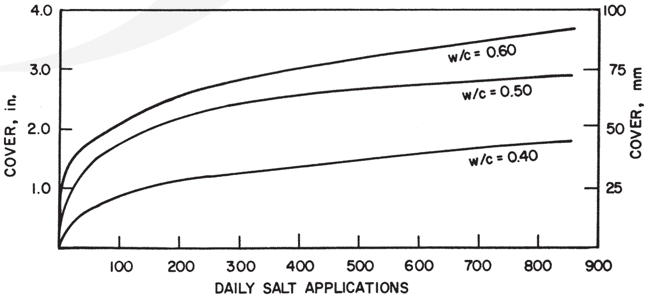

Fig. 4.2.4c—*Effect of* w/c *and depth of concrete cover on relative time to corrosion (Page et al. 1981).*

Bentz et al. (2000) reported that the initial diffusion coefficient of concrete with up to 15 percent (mass) silica fume replacement as

$$D_{SF} = D_{OPC}e^{-0.165 \times SF} \tag{4.2.4b}$$

where D_{SF} is the chloride diffusion coefficient of concrete with silica fume replacement, and D_{OPC} is the chloride diffusion coefficient of concrete of the corresponding 100 percent ordinary portland cement (OPC) concrete. This leads to a prediction of initial diffusion coefficient of approximately 43 percent of an OPC concrete for a concrete with 5 percent mass replacement of cement with silica fume. Thomas and Bamforth (1999) reported higher reduction in diffusion coefficient over time for slag cement concrete and fly ash concrete when compared to the OPC mixtures, and used a time-dependent diffusion coefficient equation to predict a decreasing coefficient at ages up to 25 years for concrete containing up to 50 percent fly ash and 70 percent slag. Their model predicts diffusion coefficients for 30 percent slag cement and 40 percent fly ash replacements of approximately 43 and 21 percent when compared to that of the 100 percent OPC concrete at 10 years, respectively, and approximately 37 and 16 percent, respectively, at 25 years.

In addition to *w/c*, *w/cm*, and the use of supplementary cementitious materials, Kirkpatrick et al. (2002a,b) reported that in bridge decks, a reduction in *w/cm* from 0.47 to 0.45 was predicted to result in a service life increase from 34 to 47 years. The service life with a *w/cm* of 0.45 using either slag or fly ash was estimated to be 62 years (Kirkpatrick et al. 2002a,b). In addition to *w/c* and *w/cm*, chemical admixtures have been shown to increase the critical chloride threshold and reduce the transport rate of chlorides in concrete (Aldred 1988; Nmai et al. 1992; Goodwin et al. 2000; Trépanier et al. 2001; Ormellese et al. 2006; McCarter et al. 2006; Manoharan et al. 2009).

Numerous corrosion inhibitors are available and the effectiveness of these vary. The user should validate performance. Other mixture variables could also influence the time to corrosion of a reinforced concrete structure. The fine-to-coarse aggregate ratio, cement content, aggregate-to-cement content, amounts of fine aggregate, coarse aggregate type, cement type, SCM type, and quantity can all influence the workability, placeability, and durability of a concrete mixture. As shown in Fig. 4.2.4b, workability can be a critical factor influencing time to corrosion and service life.

4.2.5 *Consideration of construction practices*—Although construction practices can have a significant influence on long-term performance of reinforced concrete structures, little is published on the quantitative influence of construction practices on time to corrosion. It is well known that if reinforcement is not placed with adequate cover, a significant reduction in time to corrosion could occur (Vu et al. 2005; Oh and Jang 2003; Oh et al. 2009; Williamson et al. 2009; Hussain et al. 2012; Mullard and Stewart 2011; Yu et al. 2015). Placing tolerances for reinforcing steel, the method of construction, and the level of inspection should be considered in ensuring that the specified concrete cover is achieved. The minimum depth of cover should be incorporated with expected construction tolerances (accounting for typical construction variance) to achieve a rational depth of cover specification (Weed 1974; Tikalsky et al. 2005).

The role of cracks perpendicular to reinforcing steel on the corrosion of reinforcing steel has been controversial and the evolution of the thinking on the impact of cracks has changed. In earlier studies, Tremper (1947), Martin and Schiessel (1969), and Raphael and Shalon (1971) reported that there is no relationship between crack width and corrosion. However, ACI 224.1R reported that the role of cracks perpendicular to reinforcing steel on corrosion is a function of concrete quality, cover, and crack size. In 1971, ACI 318 limited the maximum specified crack width to 0.016 and 0.013 in. (0.41 and 0.33 mm) for interior and exterior exposure conditions, respectively. Atimay and Ferguson (1974) confirmed that cracks perpendicular to reinforcing steel with widths less than approximately 0.012 in. (0.3 mm) wide have little influence on the corrosion of the reinforcing steel. This implies that allowable cracks greater than 0.012 in. (0.3 mm) could influence corrosion performance. Later, the CEB-FIP (1992) code reported that although there were issues with the relationship between crack width and corrosion, a maximum crack width of 0.012 in. (0.3 mm) was satisfactory for long-term durability. Later, Darwin et al. (1985) and Oesterle (1997) reported that crack width cannot be correlated with the corrosion rate and resulting damage. Darwin et al. (1985) also reported little difference between the amount of corrosion of cracked and uncracked concrete specimen after exposure. The literature (Atimay and Ferguson 1974; CEB-FIB 1992; Darwin et al. 1985; Oesterle 1997) possesses a wide range of findings on the influence of crack width on corrosion activity. Several researchers have found that crack widths less than 0.012 in. (0.3 mm) have little, if any, influence on corrosion of the embedded reinforcement and larger crack opening could negatively influence the corrosion performance of a reinforced concrete structure. Larger cracks and longitudinal cracks can negatively influence the corrosion performance of reinforced concrete structures and actions should be taken to prevent, minimize, or seal these cracks.

Darwin et al. (2004) and Lindquist et al. (2006) hypothesized that cracks in the concrete cover could accelerate the corrosion, but crack width is not related to corrosion activity. Darwin et al. (2004) reported that crack density—that is, the summation of crack length on the concrete surface in the given unit area—is a more significant parameter than the crack width. However, a study conducted by Vidal et al. (2007) revealed the importance of flexural cracks on corrosion activity. The authors reported that crack widths less than 0.012 in. (0.3 mm) did not influence the corrosion process. Other researchers found that the reaction product of the corrosion process can slow the ingress of water and chlorides, resulting in reduced corrosion rates (Jacobsen et al. 1996; Li 2000). However, Jacobsen et al. (1998) and Sahmaran and Yaman (2008) reported that the size of the surface crack opening directly influences the corrosion activity. Cracks resulting from plastic shrinkage, settlement, or an overload condition can cause concern. Cracks that follow the line of a reinforcing steel (often referred to as longitudinal cracking), such as settlement cracks, have been reported to be more damaging because the exposure and resulting corroded length of the reinforcement is greater (Poursaee and Hansson 2008).

4.3—Methods of excluding external sources of chloride from concrete

4.3.1 *Waterproof membranes*—Waterproof membranes are used to minimize the ingress of moisture and chloride into concrete, as discussed in detail by ACI 515.2R. These membranes are generally in the form of preformed membrane systems, constructed-in-place, or liquid-applied systems (Kepler et al. 2000; Russell 2012). Built-up membranes that became popular in 1960s are not widely used in current practice (Manning 1995). Preformed membrane systems, which are most often used in the United States, include sheets that are bonded to the concrete surface using pressure-sensitive abrasives or heat. Liquid-applied systems include the application of the membrane in the form of hot or cold liquids and may include a layer of reinforcing fabric. Hot-applied rubberized asphalt has been the most widely used liquid membrane in North America (Russell 2012).

Waterproof membrane systems may also include other components. Primers are applied to the concrete surface to increase the bond between the membrane and the concrete. Membranes are sometimes covered with a protective overlay, as in the case of concrete bridge decks, and a tack coat is used between the membrane and the protective overlay to increase the bond between the two materials (Russell 2012). In Europe, ventilating layers, made up of either a thin lift of sand asphalt or a perforated sheet made of felt or other nonwoven fabric, are sometimes used to dissipate vapor pressures and reduce consequent blistering. However, most agencies in North America do not recommend ventilating layers because they reduce the bond of the membrane to the deck (Manning 1995; Kepler et al. 2000).

The overall performance of a waterproof membrane system depends on the performance and compatibility of its components. Several standards and specifications exist for the application methods, material specifications, and test methods involving these systems. For example, the performance of a membrane system can be evaluated by measuring its resistance to a hydrostatic head over a preformed concrete crack in accordance with ASTM D5385/D5385M or by detecting water penetration through the membrane using electrical resistance measurements as per ASTM D3633/D3633M. The continuity of waterproofing membranes on concrete surfaces can be verified in accordance with ASTM D4787. The application of waterproofing membrane systems in concrete bridge decks is standardized in ASTM D4071 and D6153, which cover both preformed and liquid-applied systems. Waterproofing of concrete bridge decks is also addressed as part of Section 21 of the AASHTO LFRD “Bridge Construction Specifications” (2014). State specifications generally follow AASHTO specifications; however, some states impose additional requirements (Russell 2012).

As described by Sohanghpurwala (2006), the ideal waterproofing membrane system should:

a) Be easy to install

b) Have good bond to the substrate and the protective overlay (where used)

c) Be compatible with all the system components, including the substrate, prime coat, adhesives, and overlay (where used)

d) Maintain impermeability to chloride and moisture under service conditions, especially temperature extremes, crack movements, aging, and superimposed loads

e) Be safe to apply and have low volatile emissions

Additionally, the system should be abrasion-resistant, have good frictional characteristics for slip resistance where exposed to foot traffic, have good UV resistance when exposed to sunlight, and should be easy to repair. Depending on time of application and expected moisture conditions within the slab during service, consideration may be required for breathability of the membrane or use of vapor suppression systems to mitigate moisture-related coating failures. Field performance of waterproof membranes depends not only on the type of waterproofing material used, but also on the workmanship, weather conditions during installation, design details, and the service environment. The preformed sheets are formed under factory conditions but are often difficult and labor-intensive to install, usually requiring adhesives, and are highly vulnerable to the quality of the workmanship at critical locations in the installation, such as at slab penetrations and expansion joints. Although it is more difficult to control the quality of the workmanship with the liquid-applied systems, these systems are easier to apply and tend to be less expensive (Kepler et al. 2000). Although earlier studies have indicated that waterproof membranes may be able to provide up to 50 years of service before reinforcement corrosion becomes a problem (Frascoia 1984), typical service lives in North America vary between 10 to 30 years (Manning 1995; Wojakowski and Hossain 1995; Xi et al. 2004; Russell 2004, 2012; Hearn and Xi 2007; Liang et al. 2010).

An NCHRP survey of state highway agencies in the United States and provincial ministries of transportation in Canada indicated that waterproofing membranes are mostly used in new bridge decks, but defects and failures are more likely to occur when membranes are used on existing bridge decks than on new bridge decks (Russell 2012). In this survey, 34 state agencies reported the installation of waterproof membranes on concrete bridge decks from 1994 to 2012. During this period, three agencies discontinued their use, four specify them only for new bridge decks, 11 specify them on existing bridge decks, and 16 continue to use them both for existing and new bridge decks.

The main reason of reported defects and failures in the NCHRP study (Russell 2012) was the lack of adhesion between the membrane and the concrete surface and between the membrane and the overlay. Moisture penetration through the membrane due to unknown reasons was also reported. Membrane blistering, punctures, and voids under the membrane were reported as additional causes of membrane failures; however, the incidence of these defects has been reduced significantly since the 1970s due to improved materials and application protocols (Manning 1995). For example, membranes can be installed without blistering when proper curing procedures are followed. Blistering is caused by the expansion of entrapped gases, solvents, or moisture in the concrete after application of the membrane. Once cured, the adhesion of the membrane to the concrete is usually sufficient to resist blister formation. To ensure good adhesion, the concrete surface should be carefully prepared; dried; and free from curing membranes, laitance, and contaminants such as oil drippings (Weyers et al. 1993).

4.3.2 *Polymer concrete overlays*—Polymer concrete overlays consist of aggregate in a monomer or polymer binder. The binders that are commonly used are epoxies, polyester-styrenes, methacrylates, and epoxy-urethanes (Fowler and Whitney 2011). Polymer concretes are rapid-setting, can be formulated for a wide variety of strengths and flexibility, are highly abrasion-resistant, and are resistant to water and chloride penetration. However, because they are relatively expensive and difficult to place, their applications are generally restricted to the rehabilitation of concrete bridge decks (Sprinkel 2003). Polymer overlays can be placed either by spreading the resin over the concrete deck and broadcasting the aggregate into the resin, or by premixing all the constituents and placing the polymer concrete with a screed (Fowler and Whitney 2011). Although thicker applications have been reported, they are generally used as thin overlays that are less than 1 in. (25 mm) thick (Kepler et al. 2000). Carter (1993) reported that polymer concrete overlays that are less than 0.40 in. (10 mm) thick may be economically competitive with other repair materials. The use of thin polymer overlays tripled from 1990 to 1999 and more than quadrupled from 1999 to 2008 (Fowler and Whitney 2011). As these are general guidelines, the specific instructions from the manufacturer's data guides should always be followed where they differ.

High shrinkage and high coefficients of thermal expansion can make some resins incompatible with concrete decks; therefore, careful selection of the polymer binder and aggregate gradation is necessary. Most monomers do not perform well if they are applied on a substrate with high moisture content and if the temperature is low; therefore, the substrate should be dry and at a temperature higher than 40°F (4°C). Improper mixing of the components of the polymer has been a common source of challenges in the field. The concrete substrate and aggregates should be dry so as not to inhibit the polymerization (Sherman et al. 1993; Harper 2007; Kepler et al. 2000). When constructed properly on new decks, thin polymer overlays can provide a service life of 10 to 25 years, depending on the average daily traffic (Weyers et al. 1993; Carter 1993; Fowler and Whitney 2011).

4.3.3 *Portland cement concrete overlays*—Portland cement concrete overlays consist of low-slump (dense) concrete and reduce chloride and moisture ingress into the underlying concrete (Sherman et al. 1993). These overlays have high cement contents (typically over 800 lb/yd^3 [470 kg/m^3]) and low *w/c* (as low as 0.30) (Kepler et al. 2000). Due to their low slump and high density, these overlays can be difficult to place, expensive, and prone to surface cracking. Some of these problems can be mitigated with the use of high-range water-reducing admixtures, but overworking or the addition of water in the field might also reduce the performance of the overlay. Proper curing is important to prevent cracking (Babaei and Hawkins 1990).

Low-slump dense concrete overlays were first placed in the early 1960s in Iowa and Kansas and their applications expanded quickly in other states. In 1999, over 44,600 yd^3 (37,300 m^2) of overlays were placed in Iowa alone. The first low-slump dense overlay in Iowa lasted for 23 years before it was replaced; overlays that were cast later had service lives over 25 years (Kepler et al. 2000).

In 1991, 153 bridge decks with low-slump dense concrete overlays were evaluated for a strategic highway research project (Weyers et al. 1993). Performance of the overlays was measured by quantifying the percent of the deck area that was damaged by delamination or spalling. It was shown that the performance of the overlays was less dependent on the type of overlay than on the methods used to prepare the deck. Overlays performed best when concrete was removed from areas that had showed high probability of corrosion with half-cell potential measurements rather than only from damaged areas, when concrete was removed to below the reinforcement, and when the exposed surface was sandblasted. Chamberlin and Weyers (1994) estimated that when these procedures are followed, low-slump dense concrete overlays have service life potentials of 30 to 50 years, assuming that the end of service life occurs when 40 percent of the total deck area is damaged.

4.3.4 *Silica fume (microsilica) concrete overlays*—Silica fume is a highly effective pozzolan, with fine particle size and large surface area. In concrete, the silica fume reacts with excess calcium hydroxide to form calcium silicate hydrate binder, which results in the material being stronger and more resistant to chloride transport. Due to the small size of silica fume relative to cement and other SCMs, improved particle packing can be obtained, resulting in a denser and less permeable microstructure. As a result, silica fume can reduce the ingress of chlorides into concrete exposed to chloride-containing solutions (Kepler et al. 2000; Bentz et al. 2000; Ramezanianpour et al. 2015). Silica fume concrete overlays can be used to mitigate corrosion challenges in reinforced concrete structures, particularly bridge decks. However, experiences with these overlays vary in different applications and studies.

Silica fume concrete overlays were placed on bridge decks in Virginia as part of two separate studies to evaluate the characteristics of silica fume concrete as thin overlays for corrosion protection and to determine the minimum amount of silica fume needed to reduce the transport rate of chlorides to low levels (Ozyildirim 1992; Sprinkel 2000). Ozyildirim (1992) investigated two concrete types with 7 and 10 percent silica fume cement replacement and concluded that silica fume concrete could be used effectively in thin overlays for bridge decks to reduce chloride ingress, but that plastic shrinkage was a concern and proper placing and curing procedures needed to be followed. Whiting et al. (2000) confirmed the findings regarding cracking susceptibility in their study investigating cracking tendency and drying shrinkage of silica fume concrete overlays for bridge deck applications. It was reported that silica fume concrete overlays need to include a provision for 7-day continuous moist curing to prevent premature cracking. Sprinkel (2000), who investigated six bridge decks with 7 percent silica fume overlays, reported that these overlays should continue to be used as deck protective systems, and even when cracks might form due to short curing times, they cause minor reductions in the life of the deck.

Despite the potential of silica fume concrete as an overlay material, testing by Wee et al. (1999) showed that the penetration of chloride into silica-fume-modified concrete was only slightly less than the penetration into high-quality ordinary concrete. Miller and Darwin (2000) and Lindquist et al. (2005) also reported that the use of silica fume in bridge deck overlays do not provide advantages over conventional concrete overlays; therefore, the extra cost and construction requirements of silica fume overlays may not be justified. A field and laboratory investigation in Kansas did not show a large difference in chloride penetration among good conventional concrete; low-slump, high-density overlays; and silica fume overlays (Lindquist et al. 2005; Browning and Darwin 2007).

4.3.5 *Latex-modified concrete (LMC) overlays*—LMC consists of a conventional portland cement concrete supplemented by a polymeric latex emulsion, mostly in the form of styrene-butadiene latex. Typically, 10 to 15 percent of water is replaced with latex, which provides additional binding while the water in the emulsion hydrates the cement. As a result, low-*w*/*c* concretes with high resistance to chloride ingress are achievable. Because LMC overlays are generally easier to apply than low-slump dense concrete overlays, despite their higher cost, they are widely used in the United States. In Virginia, LMC overlays consistently outperformed conventional and low-slump dense concrete overlays (Sprinkel 1992, 2000). LMC overlays can be expected to last up to 25 years, after which they usually need to be replaced due to rutting and general wear (Babaei and Hawkins 1990). Hot-weather conditions can cause rapid drying of LMC, which can make finishing difficult. Similar to silica fume concrete, latex can reduce bleeding and promote plastic shrinkage cracking; night placement of LMC overlays helps reduce these problems. Refer to ACI 548.3R and 548.4 for additional information about LMC and its applications.

4.4—Corrosion control methods

To prevent corrosion of the reinforcing steel in a corrosive environment, either the reinforcement should be made of a corrosion-resistant material or the reinforcing steel should be protected from contact with oxygen, moisture, and chlorides (or other aggressive ions). Corrosion of the reinforcement may also be mitigated by using corrosion inhibitors or applying cathodic protection. The selection of a corrosion-resistant material should be based on the environmental exposure condition using a service life model. For additional information on service life modeling and method analysis, refer to ACI 365.1R.

4.4.1 *Corrosion-resistant steels*—Chapter 3 provided a review of available reinforcement for concrete. In addition to conventional reinforcement, a wide variety of corrosion-resistant steels are available for use as concrete reinforcement. These types of reinforcement include solid low-alloy steel reinforcement as well as solid and clad stainless steel reinforcement. The literature includes a significant body of knowledge on the corrosion performance of these solid reinforcing steels embedded in cementitious materials, and corrosion resistance has been reported to be better than conventional reinforcement (Gu et al. 1996; Clemeña 2003; Clemeña and Vermani 2004; Trejo and Pillai 2003, 2004; Phares et al. 2006; García-Alonso et al. 2007; Scully and Hurley 2007; Kouřil et al. 2010; Serdar et al. 2013; Van Niejenhuis et al. 2016). Clad reinforcement has also been evaluated and has been reported to reduce the frequency of corrosion-induced cracking compared with uncoated carbon steel in test slabs, but did not prevent corrosion (McDonald et al. 1998; Darwin et al. 2002).

Available stainless steel reinforcement includes 304LN, 316LN, 2101, 2205, 2304, and others (McDonald et al. 1998; Pedeferri et al. 1998). Trejo and Pillai (2004) reported critical chloride threshold values of 0.82, 1.23, and 1.91 percent by mass of cement for 304, ASTM A1035/A1035M, and 316LN steel reinforcing bars, respectively. Darwin et al. (2009) reported a critical chloride threshold of 1.06 percent by mass of cement (0.17 percent by mass of concrete) for reinforcement meeting ASTM A1035/A1035M and a lower-bound value of 3.2 percent by mass of cement (0.51 percent by mass of concrete) for 316LN stainless steel. Hartt et al. (2009) reported a critical chloride threshold value of 1.44 percent by mass of cement for reinforcement meeting ASTM A1035/A1035M and higher values for 316, 304, and 2304 (specimens with these reinforcement types did not initiate during the research program). ACI 365.1R recommends a chloride limit range of 0.2 to 0.5 percent by mass of concrete for 304 reinforcement and 0.5 to 0.8 percent by mass of concrete for 316 reinforcement for probabilistic corrosion models.

4.4.2 *Fiber-reinforced polymer (FRP) reinforcement*—FRP used as concrete reinforcement include both carbon FRP (CFRP), glass FRP (GFRP), and more recently, basalt fibers (Hassan et al. 2000; Bradberry 2000). A variety of resin types and sizings are used in the manufacture of FRP reinforcement. The polymer matrix, which binds the fibers into a defined shape and provides inter-laminar shear strength, may be epoxy, vinyl ester, or blends (Bradberry 2000). Fiber sizing is typically used to improve the interface between the polymer matrix and the fiber for strength and durability considerations. Combinations of fiber, sizing, and polymer matrix influence the short- and long-term mechanical properties of FRP reinforcement in concrete.

The primary value of FRP reinforcement is its electrochemical inertness and resistance to classical corrosion in chloride-contaminated concrete. However, the designer should consider the long-term residual strength of FRP reinforcement. Although GFRP reinforcing bars do not exhibit classical corrosion, researchers (Tannous and Saadatmanesh 1999; Micelli and Nanni 2004; Trejo et al. 2005; Mukherjee and Arwikar 2005; Debaiky et al. 2006; Gardoni et al. 2012) have reported that there is a significant reduction in the tensile capacity of GFRP reinforcement when exposed to aggressive solutions and exposure conditions (for example, high pH, salt water, high temperature, freezing-and-thawing cycles, and wet/dry cycles). Based on many accelerated exposure tests, models have been developed to predict the long-term performance of these GFRP reinforcing bars. Trejo et al. (2005) performed tests on the residual tensile strength of GFRP bars embedded in concrete and reported that the tensile capacity could drop below the design capacity after just 7 years. Kim et al. (2012) reported that the probability of failure of the decks containing both No. 4 (12 mm diameter) and No. 6 (19 mm diameter) GFRP bars is higher than the failure probability generally accepted in the AASHTO load and resistance factor rating (LRFD) specifications. CFRP reinforcement does not exhibit the same reduction in capacity (Bradberry 2000). Care should be used when selecting FRP reinforcement for concrete, especially when considering GFRP.

4.4.3 *Metallic reinforcement coatings*—Metallic coatings for steel reinforcement fall into two categories: sacrificial or noble (nonsacrificial). In general, metals with a more negative corrosion potential (less noble) than steel, such as zinc and cadmium, give sacrificial protection to the steel. If the coating is damaged, a galvanic couple is formed in which the coating is the anode. Noble coatings, such as copper and nickel, can protect the steel as long as the coating is unbroken because any exposed steel is anodic to the coating. Even where steel is not exposed, corrosion of the coating may occur in concrete through a mechanism similar to the corrosion of uncoated steel.

Nickel (Tripler et al. 1966; Baker et al. 1977), cadmium (Bird and Strauss 1967), and zinc (Tripler et al. 1966; Cornet et al. 1968; Cook and Radtke 1977; Darwin et al. 2009) have all been shown to be capable of delaying, and in some cases preventing, the corrosion of reinforcing steel in concrete, but only zinc-coated (galvanized) reinforcing bars are commonly available.

Results of the performance of galvanized reinforcing bars have been conflicting, in some cases, exhibiting improved performance (Cornet and Bresler 1966; Gowripalan and Mohamed 1998; Bellezze et al. 2006; Darwin et al. 2009), in others, exhibiting relatively similar performance as conventional reinforcement (Griffin 1969; Bautista and Gonzalez 1996; Andrade and Alonso 2004), and sometimes giving mixed results (Hill et al. 1976). Zinc will corrode in concrete (Baker et al. 1977; Pourbaix 1974) and pitting can occur under conditions of nonuniform exposure in the presence of high chloride concentrations (Unz 1978; Andrade and Alonso 2004). Field studies of embedded galvanized bars in service for many years in a marine environment or exposed to deicing salts have also shown mixed results (Cook and Radtke 1977; Bhuyan and Tracy 1984; Phares et al. 2014). In general, it appears that there can be a modest increase in service life in severe chloride environments (Clear 1981). When galvanized reinforcing bars are used, all bars and hardware in the exposed portions of the structure should be coated with zinc to prevent galvanic coupling between coated and uncoated steel (Clear 1981).

4.4.4 *Organic reinforcement coatings*—Epoxy-coated reinforcement (ECR) is the most commonly used corrosion-resistant reinforcement for concrete structures. It was introduced to protect concrete structures based on research conducted by the National Bureau of Science (Clifton et al. 1974) and adoption was rapid to prevent deterioration of concrete bridge decks due to the application of deicing salts.

The early history of epoxy-coated reinforcing steel was marred by several failures, including corrosion of coated reinforcing steel in piers of bridges in the Florida Keys (Sagüés et al. 2010). This corrosion led to many research programs questioning the value of the epoxy-coating system. Other examples of corrosion damage of structures built with ECR were reported with service life corrosion protection periods of less than bare reinforcing to 7 years more than bare reinforcing (Smith et al. 1993; Clear 1998).

One research program initiated in the 1980s was conducted to better understand the cases of poor performance of epoxy-coated reinforcing steel (Pfeifer et al. 1992). This program involved testing of bent coated reinforcing steel from seven manufacturers. After approximately 2 years of testing, five of the systems exhibited poor performance whereas two of the systems exhibited good performance. It was concluded that the performance was directly based on the application of the coatings and manufacturing procedures. Critical performance criteria were identified during the research, including the need for a low number of holiday and maintenance of minimum damage throughout construction, as well as the need for improved steel surface roughness and steel cleanliness prior to coating. In 1991, the Concrete Reinforcing Steel Institute (CRSI) introduced a voluntary certification program to address these issues. In general, the industry has made significant improvements in the production of ECR, and future performance may be improved.

There has been significant interest in predicting the life extension of epoxy-coated reinforcing steel. Studies conducted in the 1990s appear to estimate significantly lower design lives than those estimated from research in the 2000s. Field studies from 1997 to 2000 estimated the increase in corrosion period of ECR compared with carbon steel at only 1 to 7 years (Weyers et al. 1997; Clear 1998; Covino et al. 2000; Brown and Weyers 2003). McDonald et al. (1998) estimated from laboratory studies that the corrosion life extension was approximately 24 years.

In 2010, Sagüés et al. researched the performance of the nearly 300 structures containing epoxy-coated bars in Florida. The work grouped the performance of the structures into three main groups. Only five were considered to be undergoing significant early-age distress and only 4 percent showed distress after 30 years of service. The majority of bridges (290) were predicted to provide 100-year design lives with minimal damage.

In a study for FHWA involving multiple types of epoxy-coated reinforcement in laboratory tests and in simulated bridge decks in the field, with chloride exposure matching decks in northeast Kansas, Darwin et al. (2011, 2014) observed that damaged epoxy-coated reinforcement exhibited more disbondment than the same reinforcement without damage. Even when damaged, however, the various epoxy coatings provided significant corrosion protection. The effective critical chloride corrosion thresholds were reported to be several times those of uncoated reinforcement and the predicted times to first repair equaled or exceeded 55 years for all systems in the study.

Reducing the damage to the coating during transportation and handling helps maximize the life of epoxy-coated bars. Specifically, damage can result from poor storage methods, rough installation, impact from hand tools, and contact with unprotected immersion vibrators. ACI 301 and ASTM D3963/D3963M require padded bundling bands and nonmetallic slings to reduce damage during transportation, handling, and storage at the job site. Coated tie wires, coated wire, or precast concrete block bar supports are also required to minimize damage to the bar coating during placing. All visible coating damage should be repaired, and if the total amount of damaged coating exceeds the limit in project specifications, the coated bar should be considered unacceptable and be replaced.

While there remains some controversy regarding life prediction of epoxy-coated reinforcing steel, during the past 40 years several studies have shown that ECR can provide increased service life (McDonald et al. 1998; Sagüés et al. 2010; Darwin et al. 2011, 2014).

The condition of the epoxy coating is critical to the performance, and while several studies have shown good performance, several agencies are still concerned about the use of ECR in aggressive environments.

4.4.5 *Chemical inhibitors*—Corrosion inhibitors are admixtures that either extend the time to corrosion initiation, reduce the corrosion rate of embedded metal, or both, in concrete containing chlorides in excess of the accepted corrosion threshold value for the metal in untreated concrete. The mechanism of inhibition is complex, and no general theory is applicable to all situations.

There have been many studies on the effectiveness of various chemicals as corrosion inhibitors for reinforcing steel in concrete (Verbeck 1975; Rosenberg et al. 1977; Tomosawa et al. 1990; Nmai et al. 1992; Al-Qadi et al. 1992; Nmai and Krauss 1994; Mäder 1995; Gu et al. 1997; Trépanier et al. 2001; Ann et al. 2006; Ormellese et al. 2006; Halmen and Trejo 2012; O'Reilly et al. 2013). The compound groups that have been investigated included primarily chromates, phosphates, hypophosphites, alkalies, nitrites, fluorides, and amines. Some of these chemicals have been reported to be effective, but others have produced conflicting results in laboratory tests. Some inhibitors that appear to be chemically effective may have adverse effects on the physical properties of the concrete (O'Reilly et al. 2013). All inhibitors should be tested in concrete before use.

Corrosion-inhibiting properties of an anodic admixture should be tested in accordance with ASTM C1582/C1582M, which references ASTM G109, that takes at least 1 year but often longer (Halmen and Trejo 2012). Some corrosion inhibitors may not show the initiation of corrosion within the maximum time of 5 years specified by ASTM C1582/C1582M. Cathodic or mixed corrosion-inhibiting admixtures may be tested by the method described in O'Reilly et al. (2013), or a similar method, to determine the effect on both corrosion threshold and corrosion rate. When applicable, ASTM G180, which is performed in cementitious slurries, could be used to evaluate corrosion-inhibiting properties of admixtures in an expedited way.

Calcium nitrite has been documented as an effective inhibitor (Tomosawa et al. 1990; Berke and Weil 1992; Berke et al. 1994; Montes et al. 2005; Halmen and Trejo 2012) and calcium nitrite is commonly used in the field. In addition, amine derivatives have been used commercially as corrosion inhibitors in the United States since the mid-1980s and have an extensive track record both in laboratory and field studies. The latest version of such inhibitors is referred to as amine carboxylates. For more information on chemical admixtures, refer to ACI 212.3R.

4.4.6 *Cathodic protection*—Cathodic protection was first used on reinforced concrete bridges in the 1970s (Stratfull 1974). Cathodic protection systems, both galvanic and impressed current, have been used to extend the service life of reinforced concrete structures, but Sohanghpurwala (2009) reported that the use of impressed current cathodic protection was declining. Sohanghpurwala (2009) reported that the primary disadvantages of impressed current cathodic protection systems are initial costs, required monitoring, and required maintenance, but noted that galvanic systems are becoming more attractive and more widely used than impressed current systems. Section 6.3.3 provides description of cathodic protection systems.

The cathodic current density necessary to maintain a passive layer on the reinforcing steel before the reinforced concrete is contaminated with chlorides is relatively low. Typical operating current densities range from 0.02 to 0.2 mA/ft^2 (0.2 to 2.0 mA/m^2) for cathodic protection of new reinforced concrete structures, compared with 0.2 to 2 mA/ft^2 (2 to 20 mA/m^2) for existing salt-contaminated structures (Daily and Kendall 1998). Cathodic protection can be used by itself or in conjunction with other methods of corrosion control.

4.5—Summary

Corrosion in reinforced concrete structures can result in significant damage. Corrosion in reinforced concrete structures is expensive and it is imperative that designers take economically justifiable measures to design long-lasting structures. This chapter identified key factors that engineers and designers should consider when designing new structures. Although the chapter covers a wide range of topics, every situation that could lead to early corrosion has not been addressed. When questions arise regarding corrosion durability of new structures, the engineer, designer, or both, is encouraged to consult an expert in corrosion and durability.

CHAPTER 5—PROCEDURES FOR IDENTIFYING CORROSIVE ENVIRONMENTS AND ACTIVE CORROSION IN REINFROCED CONCRETE STRUCTURES

5.1—Introduction

This chapter focuses on technologies and instrumentation used for conducting condition evaluations of reinforced concrete structures to identify corrosive environments and active areas of corrosion.

Corrosion-induced damage in reinforced concrete structures such as bridges, parking garages, and buildings, and the related cost for maintaining such structures in a serviceable condition, is a major concern for the owners of these structures. There are many examples of severe corrosion-induced damage of reinforced concrete structures. In extreme cases, corrosion-induced damage has led to structural failures in the form of partial or total collapse. NACE IMPACT (2016) reported that the total annual estimated direct cost of corrosion in the United States is approximately 3.4 percent of the nation's gross domestic product (GDP), and reinforcement corrosion in concrete shares a large portion of this cost. The corrosion problem, which is primarily caused by chloride intrusion into concrete, is particularly acute in snowbelt areas where deicing salts are used and in coastal marine environments. Detecting corrosion in its early stages and developing repair, rehabilitation, and long-term protection strategies to extend the service life of structures are challenging tasks. Effective survey techniques are necessary to evaluate the corrosion status of structures and to facilitate implementation of appropriate and timely remedial measures while allocating available resources in the most efficient manner.

Selecting the most technically viable and cost-effective remedial measure for deteriorated structural concrete in a corrosive environment is a formidable task. The alternatives span the extremes of inaction to complete replacement of the structure, but most often, some type of corrosion prevention or rehabilitation measure is deemed appropriate. In any case the specific approach to each situation to be used needs to be made. Historically, this process has been arduous, with no standards or guidelines available to assist in the analysis, but a step-by-step process has evolved for selecting a technically viable and cost-effective solution for a given structure in a corrosive environment. This methodology has been successfully applied to bridge structures and can be applied to any reinforced concrete structure in a corrosive environment (Scannell et al. 1999). The methodology includes the following steps:

a) Obtain information on the structure and its environment
b) Apply engineering analysis to the information and define a scope of work
c) Conduct a thorough condition evaluation of the structure
d) Analyze the condition evaluation data
e) Develop a deterioration model for the subject structure
f) Identify rehabilitation options that are viable for that particular structure
g) Perform life-cycle cost analysis (LCCA)
h) Define the most cost-effective alternative for rehabilitating the structure

The first step in the methodology involves reviewing structural drawings (including shop drawings and subsequent modification drawings, if any), reports of previous condition surveys, and available information on the environmental conditions at the site. Acquired information should include the following:

a) Location, size, type, and age of the structure
b) Any unusual design or as-built features
c) Environmental conditions, such as temperature, relative humidity, and precipitation and their variations
d) Exposure conditions—for example, pollutants, acids, marine or deicing and anti-icing applications
e) Reinforcing steel details
f) Construction history
g) Type of steel reinforcement (uncoated, epoxy-coated, galvanized, and prestressing steel)
h) Drainage details
i) Maintenance and repair history
j) Presence of any corrosion-protection systems
k) Time since previous survey

The second step entails engineering analysis of the obtained information to develop a specific scope of work that is followed by the third step in the process, which is to conduct a thorough condition survey of the structure. The condition survey involves performing appropriate field and laboratory tests to quantify the deterioration of the subject structure and as-built conditions. This step should be balanced by funds available versus data needed for assessment. More time spent during evaluation can improve the knowledge base of the effects and quantity of corrosion damage. Delaying further evaluation until the repair construction phase runs the high risk of finding conditions that were not found in the evaluation phase or anticipated in the repair design phase.

The fourth step focuses on analyses of the field and laboratory test results and facilitates the next step in the process, which is the development of a deterioration model. Deterioration models are a set of mathematical relationships between corrosion condition data and remaining service life, future condition of the structure, or estimated future damage. Several models have been proposed that predict remaining service life using different definitions of end of life (Purvis et al. 1994; Sagüés et al. 1998a,b; Gulikers and Raupach 2006; Isgor and Razaqpur 2006a,b; Samson and Marchand 2007; Pour-Ghaz et al. 2009; Ehlen et al. 2009). Some of these models predict the initiation stage of corrosion of reinforcement by simulating the transport of ionic species in concrete (Samson and Marchand 2007; Ehlen et al. 2009), whereas others predict the propagation stage (Sagüés et al. 1998a,b; Gulikers and Raupach 2006; Isgor and Razaqpur 2006a,b; Pour-Ghaz et al. 2009). For any of these models to be useful, they should be correlated with actual field conditions or a sufficiently large database. A deterioration model also provides information on the optimal time to repair or rehabilitate a given structure, and can assist in managing the overall cost to achieve the desired service life. Refer to ACI 365.1R for information on the service life prediction of concrete structures.

The condition survey data, the output from the deterioration model, and the amount of corrosion-induced damage that can exist on a particular structure before it should be repaired are used in identifying rehabilitation options that are viable for that particular structure. In this step, a number of options for rehabilitation are defined based on technical viability and desired service life of the structure.

The second-to-last step in the methodology is the LCCA, which compares and evaluates the total cost of competing rehabilitation options to satisfy similar functions based on the anticipated life of the rehabilitated structure (Purvis et al. 1994; Daigle and Lounis 2006). The value of a particular rehabilitation option includes not only its initial cost, but also the cost of using that option for the desired period. To perform the LCCA, estimates of the initial cost, maintenance cost, service life for each rehabilitation strategy being considered, and the cost of lost revenue during repairs are needed. Finally, based on the LCCA results, the most cost-effective rehabilitation strategy can be selected. It should be noted that the LCCA can include a multi-step approach of various options applied at different times.

5.2—Condition evaluation of reinforced concrete structures

Since the late 1960s, techniques and procedures have been developed to facilitate a proper condition assessment of reinforced concrete structures. Judicious use of these techniques and proper data interpretation are required before arriving at a conclusion and implementing corrective action.

Several nondestructive test (NDT) methods are available for assessing, either indirectly or directly, the corrosion activity of reinforcing steel in concrete or future propensity for corrosion. Other test methods are also available for assessing the condition of the concrete. A typical condition survey, therefore, involves two interrelated aspects: corrosion of the reinforcing steel and concrete evaluation. ACI 228.2R provides details on the underlying principles of most of the NDT methods discussed in this chapter.

The objective of the condition survey is to determine the cause and extent of the reinforcing steel corrosion and what can be expected in the future with regard to continued deterioration. Based on the specific scope developed for the target structure, some or all the procedures listed in the following are used in the condition survey.

Methods for evaluating the corrosion of reinforcing steel include:

a) Visual inspection (5.3.1.1)
b) Delamination survey (5.3.1.2)
c) Concrete cover measurements (5.3.1.3)
d) Chloride content analyses (5.3.1.4)
e) Depth-of-carbonation testing (5.3.1.5)
f) Electrical continuity testing (5.3.1.6)
g) Concrete moisture and resistivity measurements (5.3.1.7)
h) Corrosion potential mapping (5.3.1.8)
i) Corrosion rate measurements (5.3.1.9)
j) Determination of cross section loss by removing concrete to access the reinforcing steel (5.3.1.10)

Moisture, resistivity, corrosion rate, and potential mapping provide instantaneous results that will vary according to climatic conditions.

Concrete evaluation test methods:

a) Visual inspection (5.4.1)
b) Core extraction and compressive strength testing (on cores or in place) (5.4.2)
c) Chemical testing for chloride content and estimation of chloride diffusion coefficients (5.4.3)
d) Rapid chloride permeability testing (5.4.4)
e) Petrographic analysis (5.4.5)

Various methods that can be used for conducting a condition assessment of concrete structures and these are summarized by Poston et al. (1995) and ACI 228.2R.

5.3—Corrosion evaluation methods

Corrosion evaluation methods in this section are primarily for reinforced and pretensioned concrete structures. Some methods, particularly those that directly measure corrosion, are not directly applicable to post-tensioned structures for reasons that are discussed in 5.3.2.

5.3.1 *Nonprestressed reinforced concrete structures*—The different test methods that can be used to identify corrosive environments and active corrosion in structures with nonprestressed reinforcement are discussed in the following sections.

5.3.1.1 *Visual inspection*—A visual inspection or condition survey is the first step in the evaluation of a structure for assessing the extent of corrosion-induced damage and the general condition of the concrete. A visual survey includes documentation of cracks, spalls, rust stains, popouts, scaling, construction joints, coatings, and other visual evidences of concrete deterioration. Naturally, not all this visual evidence will be a result of corrosion. The size and visual condition of any previous patch repairs should be also documented. In addition, the condition of any existing corrosion-protection systems or materials and drainage conditions—in particular, evidence of poor drainage—should be recorded.

The visual survey information is recorded on a drawing of the structure. A visual inspection is a vital part of the evaluation because the use of subsequent test procedures depends on the visual assessment of the structure. The inspection should follow an orderly progression over the structure so that no sections of the structure are overlooked. For more information on conducting visual inspections of reinforced concrete structures, refer to ACI 201.1R.

5.3.1.2 *Delamination survey*—Delamination of the concrete is one of the most important forms of deterioration induced by corrosion of reinforcing steel. A delamination is a separation of concrete planes, generally parallel to the reinforcement, resulting from the expansive forces of corrosion products. Depending on the ratio of concrete cover to bar spacing, the fracture planes will form either V-shaped trenches, corner cracks, or a delamination at the level of the reinforcing steel parallel to the concrete surface. The extent of delaminations increases with time due to continuation of the corrosion process, cycles of freezing and thawing, impact of traffic, or all of these. Upon attainment of critical size, a delamination will result in a spall. The extent of concrete delamination influences the selection of cost-effective repair (refer to ACI 364.1R), rehabilitation, and long-term protection strategies. More information on repair techniques can be found in ACI 546R and ICRI 310.1R.

Several techniques, based on mechanical, electromagnetic, or thermal principles, are available to detect delaminations. Sounding techniques, such as striking the concrete with a chain, rod, or hammer, use low-frequency sound waves discernible to the human ear, whereas impact-echo, impulse response, and pulse velocity use high-frequency stress waves. Short-pulse, ground-penetrating radar (GPR) is an electromagnetic energy-based system and infrared (IR) thermography is a thermal energy-based system. Khan (2003) summarized various conventional and emerging NDT methods.

The most commonly used and least expensive method for determining the existence and extent of delaminations is sounding with a chain, hammer, or steel rod. Depending on the orientation and accessibility of the concrete surface, the concrete is struck with a hammer or rod, or a chain is dragged across the surface. Concrete with no delaminations produces a sharp, ringing sound. Delaminated areas emit a dull, hollow sound. ASTM D4580/D4580M describes this test method. For large horizontal areas, such as highway bridge decks, a chain is dragged along the concrete surface to locate delaminations. The edges of delaminations are then defined using a steel rod or hammer. Vertical surfaces and the bottom surfaces of slabs or other overhead areas are tested with a hammer or steel rod. Delaminated areas are outlined on the concrete surface and subsequently transferred to survey drawings with reference to the survey grid coordinates. Delaminated areas are often approximated as rectangles to minimize the number of sawcuts during preparation and reentrant corners in the finished patch repair.

The effectiveness of sounding depends on the depth of delamination: the deeper the delamination, the more difficult it is to detect it. In addition, the sounding technique depends on operator judgment and is prone to operator errors. Operator fatigue and high background noise levels can also reduce the accuracy and speed of the survey. The use of automated sounding techniques and software to reduce operational errors has been very limited (ACI 201.1R).

Other mechanical energy-based devices, such as pulse velocity, impact-echo, and impulse response methods, have been used for detecting delaminations, but have not gained widespread use because they need experienced operators. In addition, these techniques are point-by-point techniques requiring many test points to define areas of deterioration. The ultrasonic pulse velocity method is a proven technique for detecting flaws, such as voids and cracks, in concrete as well as determining concrete properties, such as the modulus of elasticity and density (ASTM C597). This technique has been demonstrated to accurately detect delaminations if through-transmission of the ultrasonic pulse is possible. Many tests are required, however, because measurements have to be conducted on a fine grid.

The impact-echo technique can detect internal concrete defects, such as voids, cracks, or delaminations in concrete structures (Henriksen 1995; Sansalone and Carino 1989). In this method, a broadband displacement transducer measures surface displacements resulting from the propagation of stress waves generated by an external impact. Differences in the characteristics of the reflected signals are used to locate internal defects in the concrete. Interpreting impact-echo data requires expert knowledge and experience. Additionally, many tests are required because measurements conducted on a fine grid are necessary to obtain meaningful results. The impact-echo method can also be effectively used to determine the thickness of in-place concrete slabs (ASTM C1383).

Commercial GPR and IR thermography are additional systems for detecting delamination. Short-pulse GPR is a unique type of radar design based on the necessary tradeoff between propagation depth through solid, nonmetallic materials, and resolution in the concrete. IR thermography relies on contrast in material emissivity that results in thermal differentials in the medium to detect defects.

5.3.1.2.1 *GPR survey*—The use of GPR as a nonintrusive method of detecting deterioration in concrete bridge decks was first reported in 1977 (Cantor and Kneeter 1977), and additional research resulted in improvements in the accuracy of the technique (Alongi et al. 1982; Cantor and Kneeter 1982; Clemeña 1980; Laurens et al. 2005; Dérobert et al. 2008). GPR technology was studied in depth under the research efforts of the Strategic Highway Research Program (SHRP) and is considered a viable technique for detecting deterioration in reinforced concrete (Alongi et al. 1992). Based on the SHRP research, AASHTO developed a provisional standard for evaluating asphalt-covered bridge decks using GPR (AASHTO TP 36). The use of GPR to detect delaminations is also described in ASTM D6087.

GPR is analogous to the impact-echo or pulse-echo methods used for delamination detection, except that GPR uses electromagnetic waves instead of stress waves. Echoes and reflections of radio frequency waves originating at the interface between materials with different dielectric or conductive characteristics are analyzed to detect delaminations.

A short-pulse GPR typically emits precisely timed, very short pulses of low-power, radio-frequency energy. Each pulse lasts approximately 1 nanosecond and occurs at a rate greater than 1 MHz (typically 500 MHz to 3000 MHz, depending on the depth range of the measurement). The transmitted pulse is radiated down toward the concrete surface by an antenna. As the transmitted pulse encounters a difference in dielectric property or conductivity, a portion of the radio frequency wave is reflected and the remaining portion propagates through the underlying medium. The antenna picks up the reflected waves, feeds them to a receiver, and processes them for display and analysis.

GPR can be used on bare and asphalt-covered concrete to identify areas that are obviously or likely deteriorated. Depending on the concrete quality, moisture content, and thickness of the asphalt overlay, GPR will either directly identify delaminations or detect moisture and chlorides in cracks. In dry, low-permeability concrete (a low-loss medium), radar has difficulty identifying delaminations with accuracy. Accuracy can be further reduced if either the concrete cover, the asphalt overlay thickness, or both, is small. GPR is more accurate in detecting delaminations that are filled with moisture and chlorides (Alongi et al. 1992). ACI 228.2R provides information regarding the effect of moisture and chlorides on the attenuation of GPR readings.

Advances in computer hardware and software have had a tremendous impact on GPR technology. Non-contact or air-horn antennae have been optimized for use in rapid scanning. Data acquisition, processing, and interpretation have become more efficient and relatively simpler. Expert knowledge, however, is still needed. GPR vehicles with multiple radar antennas have been developed. GPR surveys can be conducted at highway speeds, but for accurate results, surveys are best carried out at speeds of 15 to 20 mph (24 to 32 kph). Additionally, GPR surveys using air-horn antennae require minimal traffic control, which makes the technique advantageous for application on bridge structures.

5.3.1.2.2 *IR survey*—IR thermography was initially developed as a pavement inspection tool in the late 1970s and early 1980s (Holt and Manning 1978). The IR technique can be used to identify delaminations in reinforced concrete structures by observing the effects of temperature differential between delaminated and sound reinforced concrete under certain environmental conditions (Benz and Ulrikson 1990). Use of IR thermography as a viable nondestructive technique for detecting delaminations in concrete bridge decks was initiated in the 1980s and was applied successfully to some bridge structures (Love 1986; Manning and Holt 1982; Winters and Frascoia 1988; Sirieix et al. 2007; Kurita et al. 2009). ASTM D4788 is a test method for detecting delaminations in bridge decks using IR thermography.

Anomalies in the emission of thermal radiation (surface radiance) from a concrete surface are picked up in IR thermography and analyzed to detect delaminations. A delamination is marked by a separation of concrete planes. These separations are usually filled with air or moisture, both of which have different thermal emissivity compared with concrete. The difference in thermal properties affects temperature gradients within the concrete and thermal radiation, particularly during cool-down and warm-up of the structure. With proper calibration, the thermal radiation can be converted to temperature, and variations in the surface temperatures detected by IR on the basis for identifying probable delaminations. However, several handicaps exist. Differences in thermal gradients can be created by the sun shining directly on some sections of the structure and not on others and various other adverse climatic conditions. Interpretation of data under such conditions becomes difficult, and accuracy is reduced. Usually the early morning is an ideal time to detect thermal gradients. Because IR can detect a 0.15°F (0.08°C) difference in temperature, even the outlines of a human hand placed on the concrete for 1 minute can be detected. Such sensitivity makes interpretation of data even more complicated and prone to error. Limitations of the IR technique have also been researched and documented (Khan et al. 1998; Sirieix et al. 2007).

5.3.1.3 *Concrete cover measurements*—The thickness of concrete cover over the reinforcing steel has a great influence on the time to corrosion initiation of the reinforcing steel. A shallow concrete cover allows easier ingress of deleterious substances, which leads to more rapid corrosion of the reinforcing steel and subsequent deterioration of the structure if other environmental conditions are conducive. Locating reinforcing steel is also essential in conducting corrosion condition surveys. The location of the reinforcing steel and the thickness of concrete cover can be determined nondestructively using a covermeter, pachometer, or reinforcing steel locator. Alternatively, GPR can be used to locate reinforcing steel, and small-diameter holes can be drilled to expose reinforcing steel for direct measurement of concrete cover. Concrete cover information is valuable in assessing the corrosion susceptibility of reinforcing steel and deviations from original contract documents, particularly the project or as-built drawings for the reinforcement.

A covermeter measures variations in either magnetic flux or magnetic fields induced by eddy currents, due to the presence of steel, to locate reinforcement and determine the thickness of concrete cover. The accuracy of a covermeter varies, but generally, it is very accurate. A few covermeters can estimate the size of reinforcement within two bar sizes, and some can store measurements and transmit them to a computer. Cover measurements have less error when the structure is lightly reinforced. For accurate cover measurements, prior knowledge of the size of the reinforcing steel is necessary. Commercially available covermeters are compact, with single-element, hand-held probes, and are useful for locating and determining the concrete cover over individual reinforcing bars. Obtaining cover measurements over large areas of a structure, however, is time-consuming and tedious.

Single-point covermeters can be used to develop depth-of-cover maps in the same way as half-cell potential maps are produced. The covermeter is used to determine the depth of cover at individual grid points on the structure. These readings are then recorded on a standard data form with reference to the grid coordinates. Results can then be entered manually into a computer or transmitted directly to generate concrete cover maps. There is no standard test procedure for conducting cover measurements. For additional information on covermeters, refer to ACI 228.2R.

5.3.1.4 *Chloride content analysis and estimation of chloride diffusion coefficients*—Chlorides are a major contributing factor in the corrosion of steel in concrete, provided sufficient moisture and oxygen are present. Chloride sampling and analysis methods for laboratory and field determinations are discussed in 5.3.1.4.1 through 5.3.1.4.3.

5.3.1.4.1 *Chloride sampling*—The chloride content in concrete is determined through analysis of powdered concrete samples. Samples can be collected on site at different depths up to and beyond the level of the reinforcing steel using a grinder or hammer drill (AASHTO T 260). Care should be exercised to avoid contamination of the samples. Alternatively, cores can be collected and powdered samples can be obtained at different depths in the laboratory. The latter method provides better control on sample depths and greatly reduces the risk of contamination.

5.3.1.4.2 *Chloride analysis: laboratory method*—Usually, the chloride content of concrete is measured in the laboratory using wet chemical analysis—for example, AASHTO T 260, ASTM C1152/C1152M, and ASTM C1218/C1218M. Separate procedures are available for determining water-soluble and acid-soluble chloride content and free and bulk chloride diffusion coefficients (ASTM C1556). Determination of chloride concentration in hardened concrete most often involves acid-soluble chloride content analysis, which is achieved through the standardized acid extraction test given in AASHTO T 260 and ASTM C1152/C1152M. Acid chloride content analysis in concrete is typically performed because bound chlorides in the concrete can become unbound due to chemical reactions within the concrete over a period of time

(Sopler 1973). For example, relatively insoluble chloroaluminate, which is formed when chlorides are present in fresh concrete, may convert with time and exposure to sulfoaluminate and carboaluminate, releasing free chlorides (Erlin and Hime 1985). Because the acid extraction test is easier to reproduce and less time-consuming than water-soluble chloride analysis procedures, it has become more accepted (Gaynor 1985; Broomfield 2007; Angst et al. 2009).

Chloride content results are reported in percent chloride by mass of concrete, parts per million (ppm) chloride, percent chloride by mass of cement, or pounds of chloride per cubic yard (kilograms per cubic meter) of concrete. The results can be easily converted from one unit to another using appropriate conversion factors (refer to Gaynor [1985] and Scannell et al. [1994a,b]).

5.3.1.4.3 *Chloride analysis: field method*—Although laboratory testing is most accurate, it is also time-consuming, often taking several days before results are available. As a result, field test kits have been developed. Field test kits allow rapid determination of chloride levels to be made on site. Commercial units that use a specific ion electrode are available. Because these commercial kits do not follow existing standards, however, some precautions need to be taken. Jackson et al. (1995) evaluated the accuracy of the two chloride test kits against the AASHTO T 260 laboratory method. The primary conclusion was that both test kits correlate well with (but did not precisely match) the AASHTO method at chloride concentrations between approximately 0.40 and 13.70 lb/yd^3 (0.20 and 8.10 kg/m^3) (0.010 and 0.350 percent by mass of concrete); the two kits gave results that represented approximately 57 to 62 percent of the AASHTO values. Because these field techniques depend on the particular field kit used, applying a correction factor is likely necessary to obtain comparable results to standardized laboratory test methods. Although a simple multiplicative factor could be used, further research is needed.

5.3.1.5 *Depth of carbonation testing*—Carbonation testing can be performed on-site or at a later time using core samples that have been carefully preserved or during petrographic analysis. Carbonation depth is measured by exposing a fresh concrete surface and applying a solution of phenolphthalein in ethanol. Phenolphthalein is a colorless pH indicator that turns magenta or pink at or above a pH of approximately 9. Therefore, when applied to a freshly exposed concrete surface, the solution will indicate areas of reduced alkalinity. The magenta areas indicate uncarbonated concrete. The colorless areas indicate carbonated concrete. Because of the presence of porous aggregates, voids, and cracks, the carbonation front only approximates a line parallel to the concrete surface. RILEM CPC-18 is a common test used to assess carbonation depth. The depth-of-carbonation test is most important for older reinforced concrete structures. If carbonation is a contributing factor to the deterioration of a given structure and it is not accounted for, one can expect future premature damage after repairs are completed. Steel reinforcement in a carbonation zone is susceptible to corrosion.

5.3.1.6 *Electrical continuity testing*—This test is performed to determine if various embedded metallic elements are in electrical contact with each other. The test has three purposes:

1. Results of this test are needed before conducting corrosion potential surveys (corrosion potential mapping) and rate of corrosion tests on the reinforcing steel.
2. Direct contact between reinforcing steel and other metals can lead to accelerated corrosion of the steel if the steel is more anodic with respect to the metal; for example, aluminum.
3. The state of electrical continuity of all embedded metals should be known when considering electrochemical options for protection against corrosion.

The corrosion potential survey is particularly sensitive to continuity because all the reinforcing steel within a given potential survey area should be electrically continuous if data are to be collected in a grid pattern. If the ground connection is made to a reinforcing bar or other metallic element that is electrically isolated from the reinforcing steel in the survey area, the readings will essentially be remote corrosion-potential measurements of the isolated ground and will be meaningless. The same is true for rate-of-corrosion testing. If reinforcing steel within a survey area is electrically discontinuous, separate ground connections should be made to each reinforcing bar where corrosion measurements will be made.

Uncoated wire reinforcement supports, direct contact at crossings, and uncoated wire ties normally provide good electrical continuity throughout reinforced concrete members. Electrical continuity, however, should always be verified during a condition survey. Continuity across expansion joints, between scuppers and reinforcing steel, and between railings and reinforcing steel is suspect and requires verification. Any metallic element can be used as the ground location for testing if it is electrically continuous to the reinforcing steel being tested. During the survey planning stage, proposed potential grid map locations should be laid out to avoid spanning obvious discontinuities.

Theoretically, when epoxy-coated reinforcing bars are used, every bar should be electrically isolated or electrically discontinuous. Testing on structures with epoxy-coated reinforcing bars, however, has shown that the degree of electrical continuity can range from none to complete, depending on the structure. Therefore, before conducting electrical tests on epoxy-coated reinforcing bars, each bar should be tested for electrical continuity. Results of electrochemical tests on epoxy-coated reinforcement should be analyzed with care because of the dielectric nature of the epoxy (ASTM C876).

Reinforcing steel should be exposed so that electrical contact to individual bars can be made. If the reinforcing bars are not exposed, a pachometer (or other appropriate equipment) should be used to locate them (5.3.1.3). Once located, the reinforcing bars can be exposed by coring or rotary hammer. There are several test methods for checking electrical continuity:

a) DC resistance—The resistance between two metallic elements is measured with a high-impedance multimeter with lead polarity: normal and reversed. Resistance values greater than 1 ohm indicate discontinuity.

b) DC voltage difference—The potential difference between two metallic elements is measured with a high-impedance multimeter. Potential differences greater than 1 mV indicate discontinuity.

c) AC resistance—The AC resistance between two metallic elements is measured with an AC bridge null resistance meter. AC resistance values greater than 1 ohm indicate discontinuity.

d) Half-cell potentials—The potential of several metallic elements to be tested is measured against a reference cell placed at a fixed location on the concrete surface. Potential measurements greater than 3 mV indicate electrical discontinuity.

None of these methods ensures continuity. Each test requires a degree of interpretation and experience to confirm electrical continuity because the cutoff level of acceptable results is not a definite point. All four test methods, however, will provide some indication of discontinuity. The DC resistance and DC voltage difference methods are the most commonly used for electrical continuity testing. No consensus standard is available for these test methods.

5.3.1.7 *Concrete moisture and resistivity measurements*—The moisture content in concrete has a significant impact on many deterioration processes, including corrosion of reinforcement, alkali-silica reaction, freezing and thawing, and sulfate attack. The resistivity of the concrete, which is a function of the moisture and electrolyte content, has an important bearing on the rate of corrosion of embedded reinforcing steel. Consequently, it is sometimes desirable to measure concrete moisture content and resistivity. It is not common practice, however, to determine these parameters. ASTM F2170 describes a standard procedure for measuring relative humidity in concrete slabs with in place probes. ASTM WK37880 describes the test method for measuring the surface resistivity of hardened concrete using the Wenner four-electrode method.

One method of determining the moisture content of concrete is to measure the relative humidity in the concrete. Several different probes are available that use the dependence of electrical resistivity of certain materials on the relative humidity of the surrounding environment. To measure humidity, a probe is sealed in a hole in the concrete. A portable meter is then used to measure the relative humidity inside the hole. This method can monitor relative humidity changes over time and provide insight into moisture cycling in a reinforced concrete structural member.

A relationship between electrical resistivity of concrete and rate of corrosion of embedded reinforcing steel is widely acknowledged. Studies have been conducted to directly relate concrete resistivity with corrosion rate of reinforcing steel (Alonso et al. 1988; Flis et al. 1992; Fraczek 1987; Hope et al. 1986). Under field conditions, Flis et al. (1992) showed that there is a direct correlation between concrete resistivity and rate of corrosion of reinforcing steel. Conditions such as high pore-water content and the presence of electrolyte salts that lead to low resistivity usually favor active corrosion. Conversely, high concrete resistivity implies a high electrolyte resistance, which limits the rate of corrosion over larger areas as maximum permitted distance between anodes and cathodes is reduced. In general, significant corrosion is not likely when the resistivity exceeds ~200 to ~4800 ohm-in. (~500 to ~12,000 ohm-cm) (Marcotte and Hansson 2003). If cracks are present in the cover of such high-resistivity concretes, however, highly localized corrosion can result. Significant corrosion is not likely when the resistivity exceeds ~3400 to ~4800 ohm-in. (~8500 to ~12,000 ohm-cm) (Hope et al. 1985).

Concrete resistivity can be measured using a modification of the Wenner four-electrode technique commonly used for measuring soil resistivity (ASTM G57). The modified procedure involves installing four equally spaced probes in a straight line on the concrete to be tested. The probe spacing is equal to the depth to which measurement of the average resistivity is desired. The average resistivity is a function of the voltage drop between the center pair of probes with current flowing between the two outside probes. The surface resistivity of concrete is correlated to bulk resistivity, which are typically obtained from cylindrical specimens, through geometric conversion factors that are provided by the manufacturers of Wenner probes (Ghosh and Tran 2015). Unlike resistivity measurements in soil, particular care has to be taken during measurements in concrete to overcome the high contact resistance between the probes and the concrete surface. This is achieved by using a conductive interface, such as a sponge or wooden plug, at the probe tips and by grinding the concrete surface before taking measurements at each location. It should be noted that moisture content and temperature of concrete affect resistivity measurements. Although the degree of saturation and temperature can be controlled in tests on specimens in the laboratory, they are not easily controllable in the field. In addition, surface resistivity measurements conducted near reinforcement or above discontinuities in concrete (for example, discrete cracks, joints, and delaminated zones) can produce erroneous measurements; therefore, these tests should be conducted away from reinforcement and discontinuities in concrete.

Less expensive and less accurate two-probe systems are available. In these systems, the electrical resistance of concrete is measured using two electrodes from the surface of concrete. The applied current and the potential response between the two electrodes are determined and the resistance of concrete is calculated using the Ohm's Law (Gowers and Millard 1999; Polder 2001).

Another approach for measuring resistivity is the disk method, which involves the use of a single electrode on the concrete surface that is connected to the reinforcing steel within the concrete. The advantage of this technique is that only the resistivity of the concrete cover is measured if the contact resistance between the disk electrode and the concrete can be eliminated (Broomfield 2007).

Table 5.3.1.7 provides guidelines for interpreting resistivity measurements from the Wenner four-probe system when referring to corroding reinforcing steel embedded in concrete (Broomfield 2007). Concrete resistivity is a useful

Table 5.3.1.7—Trend between concrete resistivity and corrosion rate (Broomfield 2007)

Resistivity, kΩ-cm	Corrosion rate
>100	Very low (cannot distinguish between active and passive steel)
20 to 100	Low
10 to 20	Low to moderate
5 to 10	High
Less than 5	Very high

complementary measurement for identifying problem areas or confirming concerns about low-quality concrete, although the data should be considered along with other measurements (Broomfield 2007; Ghods et al. 2007).

5.3.1.8 *Corrosion-potential mapping*—Corrosion is an electrochemical process and potential (voltage) is one of the parameters that can indicate the state of the process. Corrosion-potential measurements provide an indication of the state of active corrosion. They do not assess the rate of corrosion, but rather the probability that corrosion is occurring. The corrosion rate is a function of many parameters, such as temperature, equilibrium potential, concrete resistivity, ratio of anodic and cathodic areas, and rate of diffusion of oxygen to cathodic sites. ASTM C876 provides a standard test method for conducting corrosion potential surveys on uncoated reinforcing steel embedded in concrete.

Caution should be exercised in interpreting corrosion-potential data. Many factors can affect the measured potentials and lead to inaccurate assessment of the corrosion status of the embedded reinforcing steel. These factors include (Elsener 2002; Bertolini et al. 2013; Bohni 2005):

a) Carbonated concrete
b) Fully water-saturated concrete
c) Electrical discontinuity of the reinforcing steel grid
d) Presence of stray currents
e) Presence of epoxy-coated reinforcing steel
f) Presence of galvanized reinforcing steel
g) Presence of other embedded metals
h) Availability of oxygen
i) The effect of the contact medium used for the survey
j) Human error during testing (for example, not properly wetting the concrete surface or carrying out the test too quickly before the potentials reach equilibrium)

In addition, corrosion potentials vary with environmental conditions, test location, the presence of moisture, and temperature. Cold winter temperatures will result in more cathodic (negative) potentials than in hot, summer temperatures. Corrosion-potential measurements should not be taken in areas with delaminated concrete.

Corrosion-potential measurement is also a function of the type of corrosion process. In the case of uniform corrosion, the potential readings at the surface of concrete are generally close to the potential at the interface of steel and concrete (Sagüés et al. 1997b; Pour-Ghaz et al. 2009); however, in the case of nonuniform (pitting) corrosion, the measured potentials at the surface of concrete can be different from those of the steel/concrete interface. This potential difference is a function of cover thickness and concrete resistivity, and increases with both.

One of the most important applications of the corrosion-potential survey is to develop a history of the reinforced concrete structure. For example, if corrosion-potential surveys are conducted at regular intervals of time, trends of potential with time can indicate with confidence if reinforcing steel corrosion activity in undamaged concrete is increasing with time or if the total area of reinforcing steel showing active potentials is increasing. Such information can be valuable in making decisions regarding maintenance or repair. Corrosion potential mapping has been used extensively to determine the probability and extent of active corrosion of uncoated reinforcing steel in both field concrete structures and laboratory specimens (Arup 1985; Clemeña et al. 1992; Bazzoni and Lazzari 1994; Islam 1995; Elsener 2002; Bertolini et al. 2013; Bohni 2005).

5.3.1.8.1 *Procedure and instrumentation for corrosion potential measurement*—The voltage reading between a standard portable half-cell placed on the surface of the concrete and the reinforcing steel bar located below the surface is compared with values that have been empirically developed to indicate relative probabilities of corrosion activity. A portable copper-copper sulfate (CSE) half-cell electrode is normally used for field readings. A moist sponge is attached to the tip of the electrode to reduce the electrical resistance between the concrete surface and the electrode. A wetting solution is used for moistening the sponge. Other reference half-cells, such as silver-silver chloride (Ag-AgCl) or calomel ($Hg-Hg_2Cl_2$) can also be used. The CSE is popular because it is rugged and stable. Copper is easily maintained at a standard potential over a wide range of conditions if it is submerged in an electrolyte saturated with copper sulfate crystals.

A prerequisite for corrosion-potential surveys is to establish that the reinforcing steel in the structural member is electrically continuous. If the underlying reinforcing steel is not electrically continuous, each area to be surveyed should have a unique ground point. If electrical continuity exists, a common ground point can be used for several survey areas. The ground point can be established by exposing an area on the reinforcing steel and drilling a hole in the bar. A self-tapping screw is then driven into the hole and the test lead wire is clamped to the screw head. With this method, less concrete is removed and a good connection is achieved.

Corrosion potentials can be measured manually with any high-quality 3-1/2 digit, high-impedance (10 megaohms or greater) voltmeter, or with data loggers. Corrosion potential surveys should be performed on a regular interval grid. Depending on the size of the structure and the grid interval, the quantity of data collected can vary from a few to several thousand numbers. Large areas are usually mapped with an electrode spacing of 2 to 5 ft (0.6 to 1.5 m), whereas small areas are usually mapped with a spacing of 6 in. to 1 ft (0.15 to 0.30 m).

As with many other technical fields, advances in computer and electronic technology have enhanced data collection and processing. Several commercial instruments that record and store multiple readings are available. These units are equipped with a data logger to collect data in real time and store them for later processing. Data stored in the data logger are transferred to a computer for manipulation and creating equipotential maps.

Some systems use multicell arrays so that more than one potential reading can be recorded simultaneously. These units allow large areas to be surveyed thoroughly and efficiently. Another type of potential measurement device consists of wheel electrodes. One or more miniature reference electrodes are installed along the periphery of the wheels. Potentials are recorded in a data logger as the wheels are rolled along the surface of the concrete. Some degree of familiarity and experience is required to use the computerized equipment. Several systems for mapping corrosion potential have been evaluated competitively (Sohanghpurwala et al. 1994).

5.3.1.8.2 *Corrosion potential data interpretation*—The corrosion potential of reinforcing steel indicates whether or not the steel is likely to be actively corroding in the area of measure at the time the measurement is obtained. The following guidelines are given in a nonmandatory appendix of ASTM C876 for interpreting corrosion-potential data of uncoated reinforcing steel in concrete.

a) If potentials over an area are more positive than –0.20 V CSE, there is a greater than 90 percent probability that no reinforcing steel corrosion is occurring in that area at the time of measurement.

b) If potentials over an area are in the range of –0.20 to –0.35 V CSE, corrosion activity of the reinforcing steel in that area is uncertain.

c) If potentials over an area are more negative than –0.35 V CSE, there is a greater than 90 percent probability that reinforcing steel corrosion is occurring in that area at the time of measurement.

These guidelines should be used only for uncoated conventional reinforcing steel embedded in concrete; data interpretation guidelines have not been developed for epoxy-coated, galvanized, or stainless steel reinforcement and prestressing steel.

Differences in corrosion potentials across a structure or in an area of a particular reinforced concrete member are better indicators of the level of corrosion activity than the absolute potential values. For example, a 5 ft (1.5 m) square section of slab with potentials that vary 100 mV is more active than a similar section with a 30 mV variation. The chloride level at the surface of the reinforcing steel and other factors such as temperature should also be considered when evaluating corrosion potential readings. ASTM C876 stipulates a temperature correction if the temperature during the corrosion-potential survey is outside the range of 72 ± 10°F (22.0 ± 5.5°C).

When the average potential at the surface of concrete is relatively high—that is, more positive—the probability of corrosion is low, as per ASTM C876. The rate of such corrosion, however, if it occurs in the form of localized corrosion, can be high. On the other hand, if the average potential at the surface of concrete is more negative, the probability of the corrosion may be high, but such corrosion may proceed uniformly at slow rates. In fact, accurate detection of localized corrosion may not be feasible with corrosion potential mapping unless supplementary information is available. Mathematical and numerical models have been developed to relate potential readings on the surface of concrete to the rate of probable localized reinforcement corrosion through concrete resistivity, cover thickness, and temperature (Ghods et al. 2009).

5.3.1.9 *Corrosion rate measurements*—Rate-of-corrosion tests provide information on the rate at which reinforcing steel is being oxidized. The higher the rate, the sooner concrete cracking and spalling will appear. Therefore, this information can be useful in estimating the time to additional damage and in selecting cost-effective repair and long-term corrosion-protection systems.

Corrosion rate measurement techniques are based on the assumption that there is a linear relationship between a small polarization ($\Delta E < 20$ mV) and the corresponding current (ΔI) around the corrosion potential such that

$$R_p = \Delta E/\Delta I \tag{5.3.1.9a}$$

where R_p is called the polarization resistance.

Once R_p is determined, the corrosion current I_{corr} can be calculated using the Stern-Geary equation (Stern and Geary 1957; Stern 1958)

$$I_{corr} = B/R_p \tag{5.3.1.9b}$$

B is often assumed to be a constant that depends on the anodic and cathodic Tafel slopes

$$B = \beta_a\beta_c/2.3(\beta_a + \beta_c) \tag{5.3.1.9c}$$

where β_a and β_c are the anodic and cathodic Tafel constants, respectively. The corrosion rate can be calculated in terms of corrosion current density by dividing the corrosion current by the polarized area of the reinforcement. If uniform corrosion is assumed at a constant rate, corrosion rate of steel can be converted to the rate of thickness loss or corrosion penetration. The corrosion rate of existing reinforced concrete structures, however, is seldom uniform or constant; therefore, calculating thickness losses with time can be misleading.

The use of an anodic Tafel slope of 150 mV/decade and a cathodic Tafel slope of 250 mV/decade has been suggested for reinforced concrete by Clear (1989). Al-Tayyib and Khan (1988) found that these Tafel slopes depend on the active and passive corrosion states of the reinforcing steel (Al-Tayyib and Khan 1988). Most commercial field devices use B-values of 26 mV for active steel and 52 mV for passive steel. Critiques of these assumptions for conventional reinforcement can be found in the literature (Gepraegs and Hansson 2004; Elsener 2005; Garcés et al. 2005; Nygaard et al. 2005, 2009; Ghods et al. 2008).

Corrosion rate can be monitored with electrochemical techniques. Several options are available and can be broadly classified as either transient or steady-state techniques. Tafel extrapolation (E-log I), and linear polarization resistance (LPR) are examples of steady-state techniques. Potential step, small-amplitude cyclic voltammetry, electrochemical noise, and AC impedance measurements are classified as transient methods (Flis et al. 1992). In the context of reinforcing steel in concrete, Tafel extrapolation, LPR, AC impedance, and electrochemical noise techniques have been used for corrosion rate measurements. There is no standard procedure for rate-of-corrosion measurements. LPR is the most widely used method for field investigations. The AC impedance technique is considered one of the most accurate methods of corrosion rate measurement of steel in concrete; however, it is not the most practical one for field studies.

The LPR technique provides a simple method for determining instantaneous corrosion rates. It has been used in electrochemical laboratories for decades for measuring corrosion rates of metals in aqueous environments. The three-electrode LPR technique is based on the assumption that small changes in the potential of a freely corroding metal—that is, a metal at a potential close to its corrosion potential—have a linear relationship with applied current. The linear relationship can be captured using either potentiostatic sweep or galvanostatic sweep. In the potentiostatic sweep, the voltage is increased with a constant sweep rate, and the response current is recorded. In the galvanostatic sweep, the current is increased with a constant sweep rate, and the response potential is recorded. The ratio of the change in potential to the change in current gives the polarization resistance R_p.

AC impedance methods record the impedance response offered by the system to AC signals over a range of frequencies, ω (Jones 1996). The real (Z') and imaginary (Z'') components of the impedance are plotted. Such a plot, as shown in Fig. 5.3.1.9, should be a semicircle with polarization resistance R_p as its diameter. The high-frequency intercept on the real axis is the solution (concrete) resistance R_Ω, whereas the low-frequency intercept (approaching DC signals) on the real axis is the total impedance of the system, $R_p + R_\Omega$. Therefore, subtracting the high-frequency intercept from the low-frequency intercept results in R_p, the polarization resistance that then can be used in corrosion rate calculations.

In practice, the plot of the real and imaginary components of impedance deviates greatly from the semicircular shape. From a field application viewpoint, a complete frequency scan is simply too time-consuming and equipment is overly expensive and bulky. Because only the very low- and very high-frequency response data are needed to derive R_p, some devices scan only these two regions to measure this parameter, thereby reducing the time required to conduct the test. For every system under measurement, however, the low frequency that defines total impedance and the high frequency that defines solution resistance are the fundamental characteristics of that particular system, and a complete frequency scan is required to define them. Several equipment manufacturers have introduced portable equipment to conduct AC impedance tests in the field but have not been able to reduce the time required to conduct the test over the entire frequency range. In addition, interpretation of the AC impedance spectra can be difficult and subjective for someone without significant experience with this type of testing and knowledge of the implications of the test results.

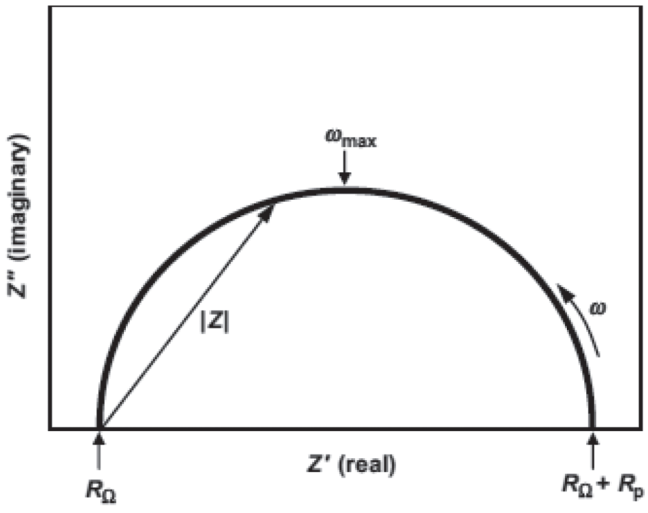

Fig. 5.3.1.9—Nyquist plot of idealized AC impedance data of corroding reinforcement in concrete (Poursaee 2016).

Several attempts have been made to correlate corrosion rate to the remaining service life of structures or time to damage. Based on experience, one manufacturer of a corrosion-rate device provides guidelines for interpreting the data in terms of time to damage. Others attempt to mathematically model remaining life based on corrosion rate information (Gepraegs and Hansson 2004; Elsener 2005; Garcés et al. 2005; Nygaard et al. 2005, 2009; Ghods et al. 2008).

5.3.1.9.1 *Corrosion rate field measurements*—Due to the natural heterogeneous nature of concrete, environmental variables such as temperature and humidity, and the inability to position the corrosion rate probe sufficiently close to the reinforcing steel, corrosion-rate measurements have inherent error. Moreover, a nondestructive assessment of the actual area that is corroding cannot be made. Critical reviews about the inherent problems of these techniques have been given by many researchers (Hansson 1984; Wojtas 2004; Elsener 2005; Gulikers and Raupach 2006; Poursaee and Hansson 2008).

It has been argued, however, that due to large differences in corrosion rates (several orders of magnitude) between passive and active reinforcing steel, the measurement errors do not significantly affect interpretation of results from an engineering point of view (Andrade and Gonzalez 1978).

In a field test, one difficulty is to determine the area of steel tested. Knowing the steel area is important because the corrosion rate is defined in terms of the corrosion current per unit area of steel (for example, mA/ft^2 [μA/cm^2]). The magnitude of the corrosion current measured is a direct indication of how fast corrosion is occurring on the steel surface. High currents indicate a high corrosion rate and low currents indicate a low corrosion rate.

Rate of corrosion measurements are usually conducted at the most active corrosion sites identified during the corrosion-potential survey. The rate of corrosion measurement is a point-in-time indicator. Temperature and concrete moisture content may change in a matter of days or even hours and result in corresponding changes in the corrosion rate. Therefore, predicting future corrosion activity should include evaluation of dynamic environmental factors. The most active corrosion or highest corrosion rates may not be occurring at the time of the field survey. Continuous or intermittent monitoring over a period of time gives a more accurate appraisal.

Flis et al. (1992) identified several parameters that significantly affect corrosion rate measurements:

a) Proper electrical contact between the probe and the concrete
b) Symmetrical positioning of the probe over the reinforcing steel
c) Presence of a stable open-circuit (corrosion) potential
d) A guard ring of appropriate size and spacing to define the polarized area
e) Measurements that are carried out over the active parts of macrocells

5.3.1.9.2 *Rate-of-corrosion equipment and data interpretation*—Typical rate-of-corrosion equipment consists of a measuring device and a suitable probe. Most commercial instruments use the potentiostatic LPR technique; however, some also incorporate the galvanostatic pulse technique. These devices mostly use microprocessor control and are fully automated, but manually operated instruments also exist. The automated devices use the guard-ring concept to measure the rate of corrosion (Feliu et al. 1990). Measurements made using the guard ring have been reported to be more accurate by up to two orders of magnitude where corrosion rates are low (Flis et al. 1992). However, the effectiveness of guard rings in containing the polarizing current is a subject of debate (Poursaee and Hansson 2008).

Data interpretation guidelines range from projecting actual time to damage to representing corrosion rate in terms of a passive condition—low, moderate, or high corrosion, depending on the magnitude of the corrosion-current density. Interpreting rate-of-corrosion data in terms of a passive, active, or partially active corrosion condition has also been suggested (Cady and Gannon 1992).

5.3.1.10 *Cross section loss of reinforcing steel bars*—This test is used to quantify the amount of corrosion that has occurred on reinforcing steel bars by directly measuring cross-sectional loss with a caliper (ACI 364.14T). To make measurements, the reinforcing steel bar has to be exposed in spalled areas or carefully excavated. Additionally, the reinforcing steel bar should be cleaned of all corrosion products before taking measurements. This is not a standardized test but is sometimes used to aid in determining structural integrity.

5.3.2 *Prestressed reinforcement*—Identifying corrosion in a structural concrete member containing prestressing steel is typically more difficult than in structural concrete containing nonprestressed reinforcement. Local corrosion of prestressed reinforcement can occur without any outward signs of concrete damage. Stress-corrosion cracking and intergranular corrosion due to hydrogen embrittlement can also occur without local accumulation of corrosion products. All test methods used for corrosion assessment of reinforced concrete structures containing nonprestressed reinforcement can be used for pretensioned prestressed steel. On post-tensioned structures, some of the test methods, such as corrosion potential and corrosion rate, may be applicable only in localized areas such as in the anchorage zones (Wang et al. 2005). Although corrosion potential and corrosion rate readings may indicate corrosion of a metallic sheath (duct) or an anchorage, data interpretation is difficult if corrosion of nonprestressed reinforcement is occurring simultaneously. It is not possible to detect corrosion of the prestressing steel using corrosion potential or corrosion-rate measurements when the sheath shields the steel. The only method of determining corrosion of a post-tensioned tendon at a particular location is to remove the concrete around the tendon and observe the prestressing steel locally. This, however, must be done carefully, as restraint can be released. Note that removing concrete and assessing corrosion in one location does not provide any information regarding corrosion at other locations along the tendon. Concrete repairs must be performed.

Continuous acoustic monitoring (CAM) is a nondestructive technique used for detecting and monitoring damage in unbonded prestressed structures (Elliot 1996) and in grouted systems. When installed in a prestressed structure, the CAM system uses an array of sensors (one every 1000 ft^2 [93 m^2] in a typical building) to record acoustic signals of tendon element failures. Proprietary software is used to analyze the recorded signals to filter out extraneous noise so that the time and location of failures can be identified. The system permits efficient continuous monitoring of large and small structures. Knowing the location and rate of prestressing steel failures after installation makes it possible to manage a known or suspected corrosion condition in a structure. However, the technique does not record any damage that has occurred before the installation of the system.

5.3.2.1 *Pretensioned structures*—Corrosion assessment techniques used for reinforced concrete can be applied to pretensioned concrete as well, but some tests should be used with caution. For pretensioned concrete structures, corrosion-potential readings are possible if corrosion of the prestressing steel is not masked by aggressive corrosion of the reinforcing steel. Pretensioned steel should be electrically interconnected to the reinforcing steel, but continuity testing is required to confirm interconnectivity. Some members have no stirrups, such as hollow core plank, and therefore, the prestressing strands are electrically isolated. Additionally, the ASTM C876 guidelines for interpreting corrosion potential readings are not directly applicable for prestressing steel.

5.3.2.2 *Unbonded post-tensioned structures*—Anchorage zones that show corrosion products and concrete distress are a sign of corrosion at the anchors and possibly corrosion of prestressing steel contiguous to the anchor in unbonded systems. If the sheath does not exclude water, then water ingress from the anchorage often results in corrosion of prestressing steel. Local removal of concrete from the anchorage area to observe prestressing steel corrosion is a limited sampling procedure that requires extreme caution.

Another limited sampling procedure involves removing concrete and a portion of the duct at a specific location away from anchorages to observe the condition of the prestressing steel. This type of direct visual testing is commonly done at tendon low points where water tends to accumulate.

Unlike nonprestressed reinforcement, corrosion products that form on unbonded single-strand tendons are contained within the sheathing and usually do not cause any cracking or spalling of the concrete. However, in older systems using metallic or paper ducts, cracking along the tendon trajectory may occur because of freezing water trapped inside the duct. This may indicate prestressing corrosion, particularly if chlorides are present. Because the tendon is unbonded, the strand can be removed and replaced in most cases. Strand removal is the only procedure that completely assesses the extent of corrosion along the tendon. For guidelines on strand removal and replacement, refer to ACI 423.4R and ACI 222.2R. ACI 423.4R also devotes a chapter to the evaluation of corrosion damage in unbonded tendons.

5.3.2.3 *Bonded post-tensioned structures*—In bonded post-tensioned structures, the prestressing steel is encased in a duct that is filled with grout (Schupack 2004). Corrosion can occur if there are voids in the grout, the steel is exposed, and water is available or becomes available. Im et al. (2010a) provided a method to inspect tendons in bonded post-tensioned structures for voids and the presence of moisture. The impact-echo technique has been used successfully to detect voids in post-tensioned ducts (Ghorbanpoor 1993; Jaeger et al. 1996, 1997). This technique is limited to accessible tendons and cannot be used for interior tendons in a multiple tendon configuration or if plastic ducts have been used. Further, entrapped water can freeze and cause cracking that follows the trajectory of the tendon. Steel sheath corrosion may also provide enough corrosion products to cause cracking along the tendon. For further guidance on evaluating and repairing corrosion-damaged bonded tendons, refer to Im et al. (2010a,b) and ACI 423.8R.

5.4—Concrete evaluation test methods

Various test methods for assessing the condition of concrete on in-service structures are listed in 5.2. Except for the visual inspection test procedure and nondestructive testing, the other tests for conducting concrete condition evaluations are of a destructive nature. They involve the extraction of core samples from the structure for subsequent testing and analysis.

5.4.1 *Visual inspection*—Visual inspection purposes and procedures are the same as for the corrosion condition evaluation of the structure. The methodology was discussed in detail in 5.3.1.1.

5.4.2 *Core extraction and compressive-strength testing*—Compressive-strength tests are usually conducted on 4 in. (100 mm) diameter core samples in accordance with ASTM C42/C42M or AASHTO T 24M T24. Compressive-strength testing is conducted to verify that the concrete compressive strength meets the specified requirements of the mixture design and to determine if the strength has been compromised due to any deterioration process such as alkali-silica reaction or freezing-and-thawing damage. Wide variations in compressive strength indicate local areas of deterioration. Concrete damaged by freezing-and-thawing action, usually exhibited as horizontal cracks in the upper portion of the core, may register a high compressive strength, but still be of poor quality.

All cores collected from a structure should be identified by core number and grid location to the nearest 6 in. (150 mm). Each core should be photographed and its condition described. The cores should be arranged to show any significant deterioration; unusual features; and, where possible, embedded steel reinforcement. In some cases, wetting the cores may improve the contrast and emphasize defects such as cracks and voids. A sketch should be made to show the overall dimensions of each core, the location of any reinforcement, and significant defects. The sketch should illustrate the same view of the core as in the photograph.

5.4.3 *Chloride permeability testing*—The chloride permeability of concrete is best determined by test procedures that measure actual chloride ingress. The rapid chloride permeability test, an electrical procedure described in AASHTO T 277 and ASTM C1202, has gained acceptance as a means of evaluating the ability of concrete to resist chloride penetration. This accelerated laboratory test method consists of monitoring the amount of electrical current passed through a 2 in. (50 mm) thick slice of a 4 in. (100 mm) nominal diameter core or cylinder for 6 hours under a potential difference of 60 volts. One end of the specimen is exposed to a sodium chloride solution and the other end is exposed to a sodium hydroxide solution. The total charged passed, in coulombs, is a function of the initial conductivity of the concrete and the change in conductivity during the test. Because concrete conductivity and chloride penetration are both directly affected in part by the pore structure of the paste, the total charge passed provides a relative indication of the resistance to chloride penetration. Therefore, for a given set of concrete-making constituents, a high charge (greater than 4000 coulombs) indicates high chloride permeability, whereas a low charge (less than 100 coulombs) indicates negligible permeability.

The rapid chloride permeability test gives reasonable results for most concretes, but caution should be used when applying the test to concrete with relatively large quantities of admixtures that contain inorganic salts such as calcium nitrite because these salts increase the electrical conductivity of the concrete and make it appear more permeable. In addition, the test reflects the permeability of the concrete at the test age. For concrete that has been in place for many years, the test may indicate low permeability, yet chloride analyses may show high levels of chlorides present in the structure.

The rapid chloride permeability test has been reviewed and simpler alternatives for determining chloride permeability have been suggested (Arup 1985; Andrade and Sanjuan 1994; Snyder et al. 2000). According to these reviews, there are several errors in the test:

a) The test uses the total current and not that corresponding to the chloride flux.

b) When integrating the total current from the beginning of the experiment, the procedure does not distinguish between chloride flow plus reaction and simple flow.
c) The high voltage drop used induces heat that, in turn, changes the flow speed.

Therefore, a migration test, such as that used in the rapid chloride permeability test, cannot accurately quantify the transport of chlorides, much less the porosity or permeability of the concrete specimen (Andrade and Sanjuan 1994).

5.4.4 *Petrographic analysis*—The ASTM C856 consists of microscopic examination of a freshly fractured and polished concrete surface that is obtained from a 4 to 6 in. (100 to 150 mm) diameter drilled core. Collecting cores is prescribed by ASTM C42/C42M. A qualified petrographer should perform the analysis. Petrographic examination is often supplemented with a range of other evaluation techniques, including, but not limited to, chemical analysis, X-ray diffraction analysis, and scanning electron microscopy.

Information obtained during a petrographic analysis can include the following:
a) Condition of material
b) Causes of poor quality
c) Identification of distress or deterioration caused by chloride-induced corrosion, carbonation, alkali-aggregate reactions, and freezing-and-thawing cycles
d) Probable future performance
e) Compliance with project specifications
f) Degree of cement hydration
g) Estimation of *w/cm* and density
h) Extent of paste carbonation
i) Presence of fly ash and estimation of amount of fly ash
j) Evidence of sulfate and other chemical attack
k) Identification of potentially reactive aggregates
l) Evidence of improper finishing
m) Estimation of air content, spacing factor, and how much of the air voids are entrained versus entrapped
n) Evidence of early freezing
o) Assessment of the cause of cracking
p) Estimate of cement content

CHAPTER 6—REMEDIAL MEASURES

6.1—Introduction

This chapter discusses measures available to stop or minimize corrosion activity on the reinforcing steel of an existing, structurally adequate, reinforced concrete structure. Remedial measures for controlling corrosion of reinforcing steel in portland cement concrete use principles directed toward:
a) Insulating the concrete surfaces from the corrosive environment
b) Modifying the environment to make it less corrosive
c) Modifying the electrochemical reactions at the reinforcing steel

Several options are available for repair and rehabilitation of deteriorated reinforced concrete structures. Choosing an option depends on the observed deterioration, the environment, the availability of repair products, and the skill of the workforce used to implement the repair procedure. Some of the available options include:
a) Do nothing
b) Remove spalled and delaminated concrete and replace with a patch or overlay;
c) Remove all chloride contaminated concrete or carbonated concrete and patch with an overlay
d) Install cathodic protection to protect the reinforcing steel from further corrosion
e) Use electrochemical chloride extraction (ECE) to remove chloride from the surface of the reinforcing steel
f) Use realkalization to restore the concrete pH
g) Use migrating corrosion inhibitors on the surface of concrete, which are designed to penetrate to the reinforcing bars to reduce the corrosion rate of the steel

After repair is completed, it is important to prevent future deterioration to the structure. Reinforced concrete structures may be protected from corrosive environments by applying a variety of barrier systems between the structure and the corrosive environment. The barrier may be a coating or membrane applied to the surface of the concrete, formed as an integral part of the concrete matrix through polymer impregnation, or an overlay of polymer concrete, latex-modified concrete (LMC), silica fume concrete, low-slump dense concrete (LSDC), or internally sealed concrete (Chapter 4). Refer to ACI 546R for guidance on the selection and application of materials and techniques for the repair, protection, and strengthening of concrete structures.

The environment may be altered to reduce corrosion either by removing detrimental conditions such as chloride, oxygen, and moisture gradients, or by removing or neutralizing stray current sources. Corrosion can also be controlled by modifying the electrochemical reactions at the reinforcing steel, as done in cathodic protection, where the reinforcing steel is made a cathode with respect to an external anode or through application of corrosion inhibitors.

6.2—Applicability

Nearly all reinforced concrete structures are susceptible to corrosion. Although bridge decks are perhaps the most visible examples (Jones and Ellingwood 1993; Burke 1994; Weyers 1994; Williamson et al. 2008), the literature contains many references to other types of reinforced concrete elements that experience corrosion of the reinforcing steel (O'Connor and Kolf 1993; Khan et al. 1995; Bickley and Liscio 1997). These include buildings, caissons, foundations, parking garages, piers, piles, pipes, silos, tower footings, and water tanks. Some of these structures or elements may be buried totally or partially in soil. Marine structures, such as offshore platforms, piers and docks, waterfront structures, water and wastewater treatment plants, and water tanks are generally exposed to aqueous environments. Bridges, parking garages, and buildings are exposed to atmospheric conditions.

If the reinforced concrete structure or element is buried or permanently underwater so that the concrete surfaces are not accessible for treatment and it is impractical to expose them, surface treatments are not practical. Similarly, if the element is a buried pipeline or an offshore platform exposed in a large body of water, modifying the environment to make it less corrosive is not practical. Therefore, not all the remedies discussed herein are applicable to all types of reinforced concrete structures in various environments.

Cathodic protection has been used to prevent corrosion of reinforced concrete structures in corrosive environments. Care should be taken when considering using impressed current cathodic protection on structures containing prestressing strands due to the risk of hydrogen embrittlement. Information on hydrogen embrittlement can be found in ACI 222.2R.

6.3—Remedies and their limitations

6.3.1 *Surface treatments, coatings and isolation remedies (barrier systems)*—The methods used to isolate reinforced concrete structures from corrosive environments include surface coatings and membranes, polymer impregnation, overlays of polymer concrete, low-slump concrete, silica fume concrete, or latex-modified concrete (LMC). These barrier systems are suitable when the surfaces of the concrete structure are available for treatment and they reduce continued intrusion of oxygen; water; and corrosive agents, such as chlorides, that are required to sustain the corrosion reactions. Barrier systems used after active corrosion is initiated do not stop corrosion but can significantly slow the effects of the corrosion process. If the corrosive agents, particularly chlorides, are of sufficient quantity, the corrosion process will continue. Therefore, barrier systems should be considered as only temporary remedies, and routine maintenance should be required. All barrier systems will contain discontinuities such as pinholes, breaks, cracks, poor seams, or other defects that will allow intrusion of corrosive agents in localized areas. Nevertheless, barrier systems can reduce the rate of intrusion of corrosive agents and retard the corrosion process. In many cases, these barrier systems successfully extend the useful service life of a structure. A national survey of bridge decks overlaid with low-slump dense concrete (LSDC) or LMC has shown that the service life is extended between 20 and 30 years (Weyers et al. 1993; Kepler et al. 2000; Williamson 2007).

6.3.2 *Modification of the environment*—Methods available for rendering the environment less corrosive include the removal or elimination of substances and conditions that promote corrosion, such as chlorides, hydrogen sulfide, water, oxygen, and stray electrical currents.

Improving drainage and surface-applied materials, such as silane coatings, will reduce the penetration of water into the concrete. Gases such as oxygen and hydrogen sulfide can be stripped from the electrolyte by chemical processes that are applicable predominantly to structures exposed in aqueous solutions.

Chlorides can be removed from the vicinity of the reinforcing steel by electrochemical chloride extraction (ECE). The ECE process was investigated in the 1970s (Slater et al. 1976; Morrison et al. 1976), but it received attention during the Strategic Highway Research Program, which resulted in the development of several reports and an implementation guide (Bennett et al. 1993b; Bennett and Schue 1993). ECE is a process that involves placing an anode and electrolyte on the concrete surface and passing a direct current (DC) between the anode and the reinforcing steel, which acts as a cathode. In this electrochemical process, the chlorides migrate toward the anode, away from the reinforcing steel. An electrolyte, typically potable water or a calcium hydroxide solution (lime water), is circulated through the system. Calcium hydroxide provides a limited buffering capability and is used to maintain a basic pH during the process and prevent the etching of concrete and generation of gaseous chlorine. Lithium borate is an expensive electrolyte used only when the structure contains potentially alkali-silica reactive aggregates. The treatment time varies from 10 to 50 days, and the total charge varies from 60 to 150 A-h/ft^2 (650 to 1600 A-h/m^2).

The ECE process removes 45 to 95 percent of the chlorides present in the concrete (NACE 01101:2001). The amount of chlorides removed depends on several factors, including the amount of chlorides present, its distribution in the concrete, and the details of the reinforcing steel. After the treatment is complete, the chlorides remaining in the structure may be sufficient to reinitiate corrosion. These chlorides, however, are distributed well away from the reinforcing steel, and time is required for redistribution to occur. The return to corrosive conditions is further delayed by the buildup of alkalinity at the surface of the reinforcing steel and development of a protective oxide film on the steel surface. Laboratory studies indicate that the ECE process will prevent corrosion for 10 or more years if contamination with new chlorides is prevented. ECE-treated field structures have remained passive (noncorroded) for over 20 years. ECE is particularly suited to reinforced concrete structures in which active corrosion is occurring but no significant damage has occurred. The application of ECE is limited to structures with only nonprestressed reinforcement.

Deep polymer impregnation (Al-Qadi et al. 1993; Gannon et al. 1992; Cady and Weyers 1986) is another technique for modifying the environment around reinforcing steel. In deep polymer impregnation, an electrically nonconducting material replaces the continuous concrete pore water and stops corrosion. In this process, grooves 0.75 in. (19 mm) wide, 1.5 in. (38 mm) deep, and 3.0 in. (75 mm) on center are cut into the deck surface. The concrete is dried to a depth of 0.5 in. (13 mm) below the concrete surface using propane-fired infrared heaters and allowed to cool slowly under an insulating mat to ambient temperature. The monomer (methyl methacrylate) is poured into the grooves and allowed to soak into the concrete. Heat is applied to polymerize the monomer in place. The grooves are backfilled with a latex-modified mortar by pouring the mortar on the deck surface and using a squeegee to place it into the grooves. The initial drying of the concrete may cause extensive cracking in the concrete surface. Generally, such polymer treatments are not in common use.

Stray current corrosion can also be mitigated. A method for mitigating this type of corrosion, implemented for many years in the buried pipeline industry, is the installation of resistance bonds. In resistance bonding, the structure being affected is electrically connected through a resistor to the source of the stray current. In this manner, the current returns to its source via a metallic path so that the affected structure has no loss of metal. Another method uses galvanic anodes to drain the collected current. Collected current is passed on to the electrolyte (groundwater) and back to its source from the surface of the anode. The galvanic anode corrodes rather than the reinforcing steel. The system should be properly designed, installed, and maintained to ensure that the current discharge electrode is the least resistive path to the source.

6.3.3 *Modifying electrochemical reactions at the surface of reinforcing steel*—The Pourbaix diagram for iron (Fig. 3.2.1c) shows that reinforcing steel in concrete is normally passivated due to the highly alkaline concrete environment. The diagram shows another area wherein no steel corrosion occurs. This area, at the lower portion of the diagram, is labeled as immunity. In this area, the potential of the steel is more negative than in any naturally occurring condition, regardless of pH.

The method of providing more negative steel potentials required for immunity is called cathodic protection. A 1988-1989 survey indicated that more than 275 atmospherically exposed bridge structures in the United States and Canada have been installed with cathodic protection systems (Han et al. 1989). The total cathodically protected concrete surface area was approximately 9 million ft^2 (840,000 m^2). Additionally, there are millions of undocumented square feet (meters) of surface area in parking garages.

When cathodically protecting a structure, a favorable electrochemical circuit is established by installing an anode in contact with the electrolyte and passing a low-voltage direct current from that anode through the electrolyte to the reinforcing steel. This current polarizes the surface of the reinforcing steel in the electro-negative direction. When this is accomplished, there is no current flow between the formerly anodic and cathodic steel surfaces, and corrosion is arrested. This represents a balanced or equilibrium condition. Figure 6.3.3 is a schematic representation of current flow in a cathodic protection system.

The protective electrochemical circuit can be established in two ways. One method uses an electrode made of a metal or alloy that is more electro-negative than the structure to be protected, such as magnesium, zinc, or aluminum. This method is known as the galvanic anode method of cathodic protection. The galvanic anodes corrode or sacrifice themselves as they pass current to the electrolyte. An FHWA study (Whiting et al. 1995) observed that aluminum and zinc anodes were the most promising galvanic anodes. Some anodes may not provide adequate protection where the concrete has a high resistivity. Anodes should also be sized in accordance with their respective consumption rates to provide the necessary design life.

Cathodic protection can also be established by inducing an electrical current from an external source. This method is termed "impressed current cathodic protection". An impressed current cathodic protection system includes the following basic components:

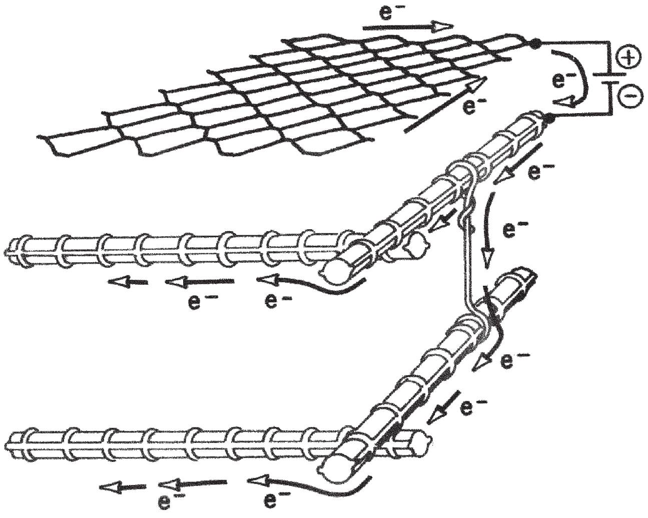

Fig. 6.3.3—Current flow in a cathodic protection system (Elgard Corporation 1990).

a) DC power source (transformer/rectifier)
b) Current distribution hardware (anode)
c) Conducting electrolyte (concrete)
d) Reinforcing steel to be protected (cathode)
e) Complete circuit (wiring)
f) Evaluation and control devices (probes, reference cells, controller)

A guide specification has been developed for cathodic protection of concrete bridge decks (Bennett et al. 1993b; Strategic Highway Research Program 1993). Similar specifications were developed for bridge substructure cathodic protection under SHRP. The specifications developed under SHRP are included in a manual of practice for cathodic protection of concrete bridges (Strategic Highway Research Program 1993). Cathodic protection systems included in the manual are briefly described as follows. These systems have performed successfully in the field to varying degrees (Clemeña and Jackson 2000; Sohanghpurwala 2009).

a) Coke-asphalt cathodic protection system: This system uses anodes of cast-iron alloy, a conductive asphaltic concrete overlay, and a conventional asphaltic concrete wearing surface.
b) Slotted-cathodic protection system: This system involves the insertion of an anode system, such as catalyzed titanium ribbon mesh, into slots cut into the concrete surface. The slots are then backfilled with a nonshrink, cementitious grout.
c) Distributed-anode cathodic protection system with concrete overlays: This system involves placing anodes on the concrete surface after repairs and cleaning, followed by encapsulation of the anodes in concrete. Encapsulation may consist of an overlay on horizontal surfaces or cast-in-place shotcrete on vertical surfaces. Anodes of catalyzed titanium mesh anodes are the most common.
d) Conductive coating: This system uses platinized wire and conductive carbon-based paints to cover the entire surface to be protected. This system is primarily used on nontraffic surfaces that are not exposed to continuous wetting during either construction or in service.

e) Sprayed zinc: In this system, molten zinc is sprayed onto the entire concrete surface using arc-spray or flame-spray equipment. The zinc coating can be used as a galvanic anode or as an impressed current anode if electrically isolated from the steel.

Whether a structure is a candidate for cathodic protection depends on several factors. These characteristics include:

a) Delaminated areas of less than 5 percent

b) Large percentage of potentials more negative than –350 mV CSE

c) An otherwise durable concrete without materials-related distress that impedes system function or desired service-life extension

d) Acid-soluble chloride contents of more than 0.20 percent by mass of cement at the level of reinforcing steel

e) Concrete cover greater than 0.5 in. (13 mm)

In addition, most of reinforcing steel in the structure should be electrically continuous, and the structure should be close to an electrical power source if impressed current cathodic protection is being considered.

Various parameters should be considered when estimating the required currents for impressed current cathodic protection. These parameters include the surface area of the concrete to be protected, size and spacing of reinforcing steel, and current density. The rectifier size is selected based on the current requirement. Cathodic protection anodes are usually segmented into zones that permit greater flexibility in control and facilitate troubleshooting in problem areas. Large zones can lead to problems such as undersupply or oversupply of the current to particular areas of the structure. Undersupply results in lack of adequate protection and increased risk of additional corrosion distress. Oversupply is inefficient and, depending on amount of excess current and conditions, can lead to reduced life of cathodic protection system components, adverse effects on concrete or other embedded materials, delivery of current to unintended components, stray current corrosion of embedded metals, hydrogen embrittlement of prestressing steels, potentially unsafe conditions, or some combination of these. Each zone typically requires a separate rectifier control circuit and power feed from the rectifier to the anode and system ground. Typical zone sizes range from 5000 to 7000 ft^2 (465 to 651 m^2). On bridges, the location and frequency of the expansion joints may determine the zone sizes.

The voltage drop and current attenuation along the anode and its connecting wire in the cathodic protection system should be evaluated. If the voltage drops are excessive, uneven and insufficient current distribution to the reinforcing steel may result. For even current distribution to a structure, the voltage drop should not exceed 300 mV from the current feed point to the farthest point in the anode circuit. The design should therefore optimize anode system length, anode spacing and size, and conductor size to achieve safe and efficient current distribution.

Redundancy of current output, circuitry, and monitoring devices (reference cells and probes) should be provided to minimize the area affected by malfunction or physical damage. The rectifier should be selected to provide adequate allowance for anticipated changes in current requirement with time. Note that rectifiers can be constant current or constant voltage and information on these can be found in Bennett et al. (1993c). The design output voltage should take into account the voltage required to drive the cathodic protection current from the anode to the reinforcing steel, the voltage loss in the positive and negative DC wiring, and should include a safety factor of 100 to 200 percent to accommodate future current demands.

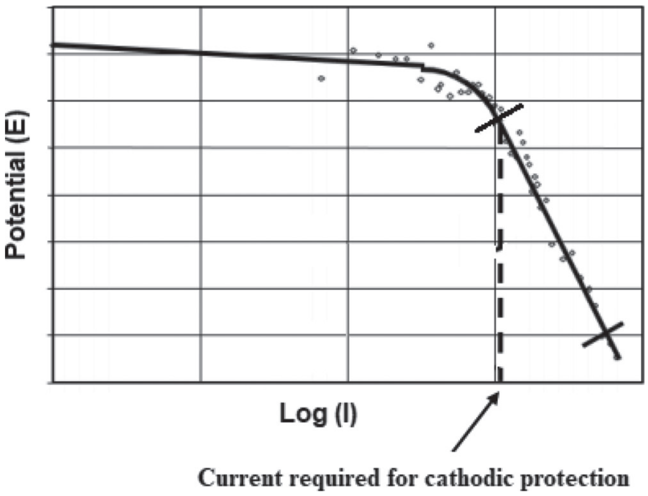

Fig. 6.3.4—E-Log(I) plot indicating the current required for cathodic protection (adapted from NACE CP4:2000).

Various criteria have been established to ensure adequate cathodic protection of reinforcing steel. The 100 mV depolarization (the change in electrochemical potential when cathodic protection current is interrupted for a defined time period) criterion is most commonly used. It is based on the theory that polarization of corroding reinforcing steel in the cathodic direction will inhibit anodic (corrosion) reactions. Polarization is usually estimated by measuring the polarization decay of the reinforcing steel that occurs after the protective current is turned off. Steel potentials, measured using half-cells, are plotted against time over a minimum 4-hour test period. If a 100 mV or greater polarization shift is measured, then the level of protection is judged sufficient to stop corrosion.

Another criterion is the E-Log(I) test performed by incrementally increasing the cathodic protection current and measuring the change in potential of the reinforcing steel. A plot of the potential versus the logarithm of the current is called the E-Log(I) plot, as shown in Fig. 6.3.4. The current required for cathodic protection is the value that occurs at the beginning of the linear portion of the cathodic polarization curve. The technique often requires special power supplies and instrumentation and it is difficult, time-consuming, and relatively complicated to analyze. Two criteria were developed under the Strategic Highway Research Program (Bennett et al. 1993a; Bennett and Turk 1994)—the corrosion null probe and constant current; however, these are seldom used due to difficulties in measurement.

After completing the initial inspection, the cathodic protection system should be adjusted for current requirements.

After adjustment, the system should be operated continuously for at least 1 month before conducting final acceptance testing. As a minimum, the rectifier voltage and current should be measured monthly for the first year and quarterly thereafter. Records should include a permanent log of the system output. Annual surveys should also be conducted to verify that the cathodic protection system is meeting protection criteria. The system should be adjusted and repaired as necessary to ensure continued, effective protection.

6.3.4 *Corrosion inhibitors*—Corrosion inhibitors 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 materials 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 and most widely used inorganic inhibitor (Ngala et al. 2002; Ormellese et al. 2006). Organic compounds used in admixtures to protect steel from chloride-induced corrosion include alkanolamines and an aqueous mixture of amines and fatty-acid esters (Nmai et al. 1992; Bobrowski and Youn 1993; Nmai and Krauss 1994; Mäder 1995). Organic amine-based compounds, such as some amine salts and alkanolamines, can be 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).

Numerous studies have also shown that surface-applied (also called penetrating or migrating) corrosion inhibitors have the potential to be used for corrosion control on existing reinforced concrete structures (Söylev et al. 2007; Stephenson and Kumar 2009; Krolikowski and Kuziak 2011). However, it was shown that beneficial effect of migrating corrosion inhibitors might decrease when initial chloride content in concrete is high (Morris and Vázquez 2002).

6.4—Summary

Remedies for controlling corrosion on reinforced concrete structures use sound corrosion engineering principles directed at isolating the reinforced concrete from the corrosive environment, alteration of the environment, or control of electrical current flow within the environment. Corrosive agents are already present within the concrete matrix when corrosion is detected in a reinforced concrete member. Though measures that isolate concrete minimize the rate of corrosion or the intrusion of additional corrosive agent, they trap the existing quantities of these corrosive agents. The effectiveness of isolation measures can be improved by removal of corrosive agents before sealing, such as by ECE. Many cathode protection systems are available that can control ongoing corrosion in existing reinforced concrete structures.

Various remedial measures have been proven capable of controlling corrosion on existing structures. Criteria and guidelines have been established for the selection, design, construction, and operation of these systems for the protection of reinforcing steel in atmospherically exposed concrete structures.

CHAPTER 7—REFERENCES

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38800 Country Club Drive
Farmington Hills, MI 48331
Phone: +1.248.848.3700
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+1.248.848.3700
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