

ACI 222R-19

Guide to Protection of Reinforcing Steel in Concrete against Corrosion

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Guide to Protection of Reinforcing Steel in Concrete against Corrosion

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Guide to Protection of Reinforcing Steel in Concrete against Corrosion

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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 REINFORCED 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 real-alkalization. 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



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



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



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 reinforcement.

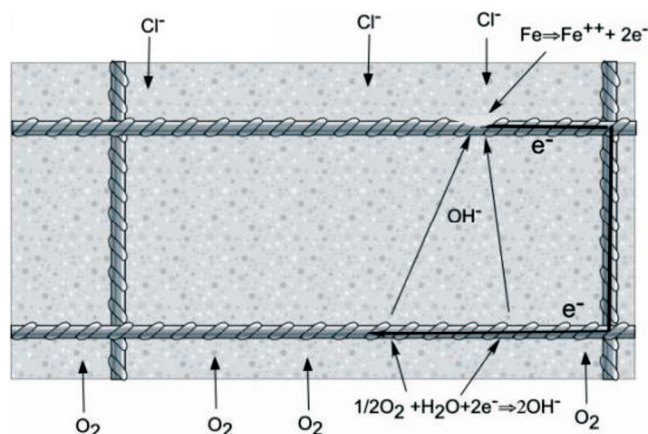


Fig. 3.2.1a—Macrocell corrosion (Hansson et al. 2006).

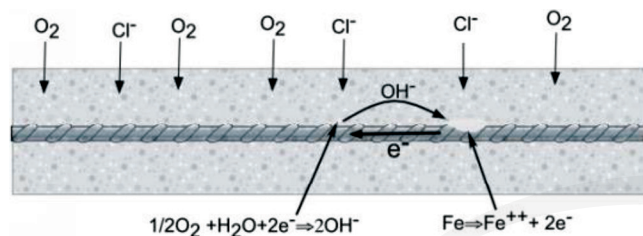


Fig. 3.2.1b—Microcell corrosion (Hansson et al. 2006).

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/CuSO₄ 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—Fe₃O₄ and Fe₂O₃ 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



where iron dissolves as HFeO₂⁻ (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 et 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 O₂). 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

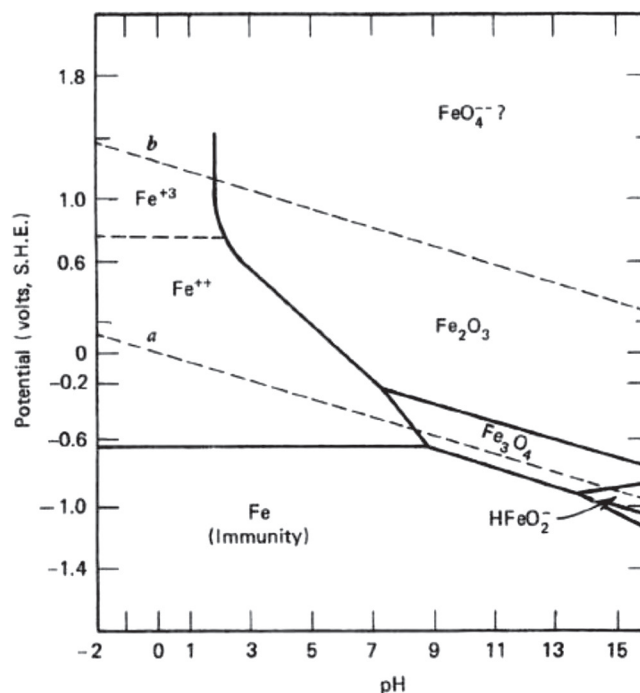


Fig. 3.2.1c—E-pH Pourbaix diagram 77°F (25°C) showing the potential pH ranges of stability of the different phases of iron in aqueous solutions (Revie and Uhlig 2008). The lines a and b represent the equilibrium potentials for the hydrogen evolution and oxygen reduction reactions, respectively.

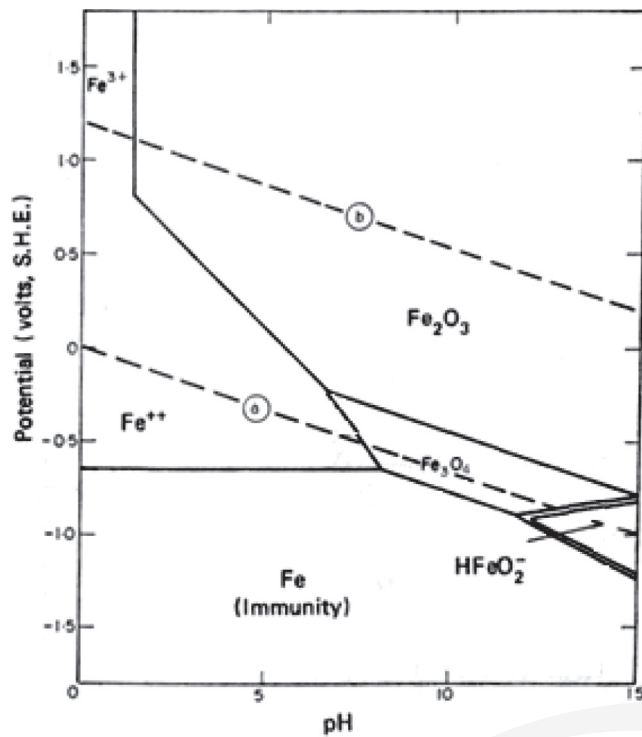


Fig. 3.2.1d—E-pH Pourbaix diagram 140°F (60°C) showing the potential pH ranges of stability of the different phases of iron in aqueous solutions (adapted from Townsend [1970]). The lines a and b represent the potentials for the hydrogen evolution and oxygen reduction reactions, respectively.

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 Fe²⁺ rich oxy-hydroxide (1 to 3 nm) and an outer Fe³⁺ 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

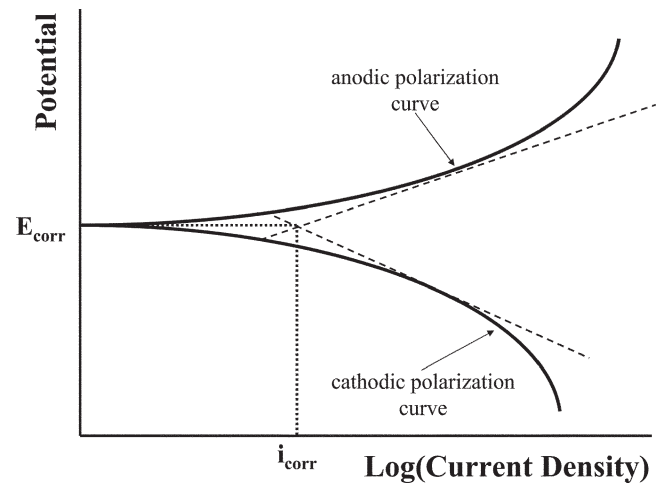


Fig. 3.2.3a—Schematic polarization curve for an actively corroding system (adapted from Jones [1996]).

$$M = \frac{itA_w}{nF} \quad (3.2.3a)$$

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):

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

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)

$$\eta = a + b \log(i) \quad (3.2.3c)$$



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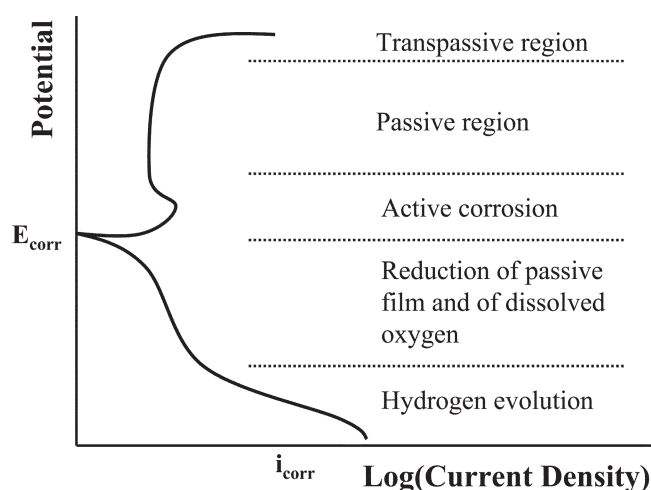
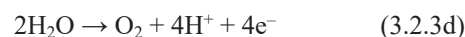


Fig. 3.2.3b—Schematic polarization curve for passive system with limited access of oxygen (adapted from Jones [1996]).

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



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 $3\text{Fe}(\text{OH})_2 \cdot \text{Fe}(\text{OH})_2\text{Cl} \cdot n\text{H}_2\text{O}$) is often found near the steel reinforcement that, upon exposure to air, turns brown/black (likely a combination of $\text{Fe}(\text{OH})_3$ and Fe_3O_4), and subsequently rust colored (likely a combination of $\text{Fe}(\text{OH})_3 \cdot 3\text{H}_2\text{O}$, $\alpha\text{-FeOOH}$, $\beta\text{-FeOOH}$ 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

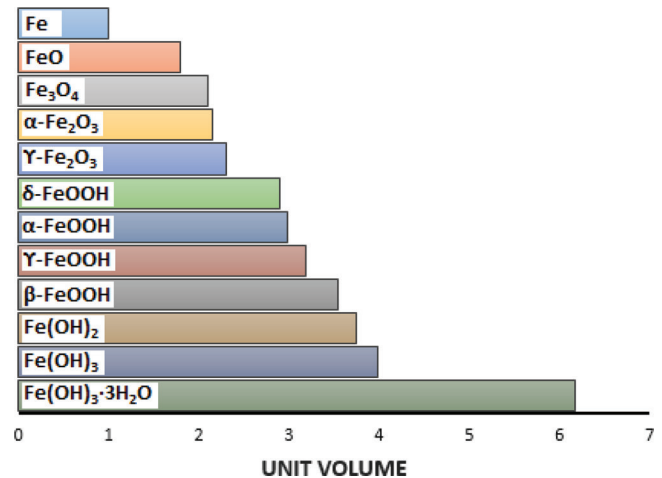


Fig. 3.2.4.1—The relative volumes of iron and its reaction product (adapted from Marcotte [2001]).

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).

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 Gjorv 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/yd³ [1.4 kg/m³] 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/yd³ (0.6 to 0.9 kg/m³) 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-

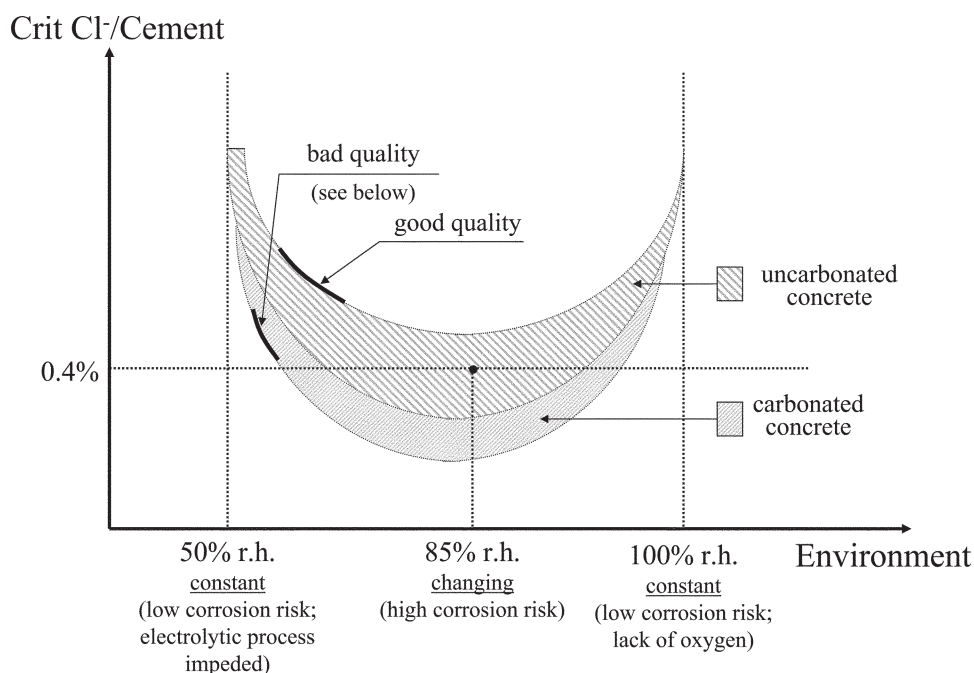


Fig. 3.2.4.1.3—Critical chloride content according to CEB-FIB (1992) recommendations (Goto and Roy 1981).

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; Saeetta and Vitaliani 2004; Isgor and Razaqpur 2004). The depth of carbonation can be predicted using numerical techniques (Papadakis et al. 1991; Saeetta 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)

$$d_c = kt_y^{0.5} \quad (3.2.4.2a)$$

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 carbon-

ated 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 Stroeve 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 (Kilaeski 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:

- Relatively free of coating breaks and defects
- Maintain high electrical resistance
- Confine corrosion to bare areas
- Resist undercutting



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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 et 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](#); [IMO 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 18/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](#); [IMO 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 in a steel bar 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 [austenitic 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 ($\text{Ca}(\text{OH})_2$). The $\text{Ca}(\text{OH})_2$ 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 $\text{Ca}(\text{OH})_2$. KOH and NaOH are present in limited quantities, however, and any carbonation or pozzolanic reaction rapidly reduces the pH to that of the saturated $\text{Ca}(\text{OH})_2$ 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; Poursaeed 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 the concrete porosity and decrease the transport rate of aggressive chemi-

cals 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 $\text{Ca}(\text{OH})_2$ 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 ($\text{C}_3\text{A} + \text{C}_4\text{AF}$) 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, $\gamma\text{-FeOOH}$, 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 molybdate, 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 Corbo 1998; Gu et al. 1997). Calcium nitrite is the most widely used inorganic

ganic corrosion inhibitor in concrete (Hope and Ip 1989; Ramasubramanian et al. 2001; Bola and Newton 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 struc-



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

ture 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.

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. Sohaghpurwala (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 (Gjorv et al. 1976; Kobayashi and Shattoh 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 found a common value of 1 lb Cl-/yd³ (0.59 g Cl-/m³) 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 (depasivation) 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 allowable limits for 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 the same

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

Category	Chloride limit for new construction (percent by mass of cementitious material [*])	
	Test method	
	Acid-soluble	Water-soluble
	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

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

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.

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

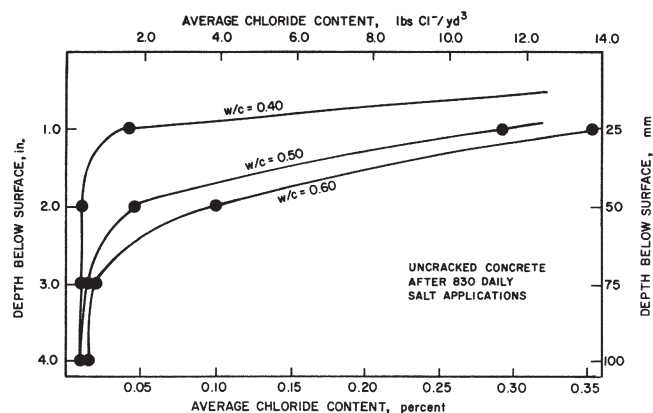


Fig. 4.2.4a—Effect of w/c on salt penetration (Page et al. 1981).

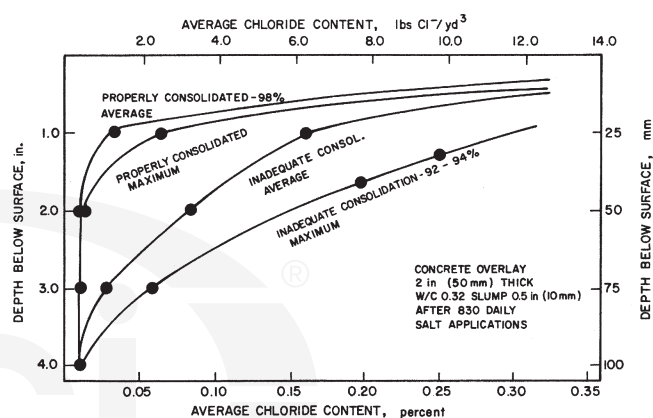


Fig. 4.2.4b—Effect of inadequate consolidation on salt penetration (Page et al. 1981).

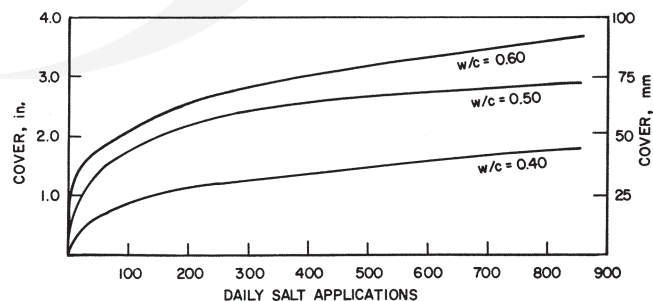


Fig. 4.2.4c—Effect of w/c and depth of concrete cover on relative time to corrosion (Page et al. 1981).

$$D_{SF} = D_{OPC} e^{-0.165 \times SF} \quad (4.2.4b)$$

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