Materials and Methods for Corrosion Control of Reinforced and Prestressed Concrete Structures in New Construction

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6300 Georgetown Pike
McLean, VA 22101-2296

FOREWORD

Salt-induced reinforcing steel corrosion in concrete bridges has undoubtedly become a considerable economic burden to many State and local transportation agencies. Since the iron in the steel has a natural tendency to revert eventually to its most stable oxide state, this problem will, unfortunately, still be with us, but to a much lesser degree due to the use of various corrosion protection strategies currently used in new construction. The adoption of corrosion protection measures in new construction, such as the use of good design and construction practices, adequate concrete cover depth, low-permeability concrete, corrosion inhibitors, and coated reinforcing steel, is significantly reducing the occurrence of reinforcing steel corrosion in new bridges. This report summarizes the results of various research investigations in developing and evaluating the performance of various corrosion protection systems. This report describes materials and measures that can be used for corrosion control in reinforced and prestressed concrete bridge structures in new construction.

This report will be of interest to materials and bridge engineers, reinforced concrete corrosion specialists, and those concerned with the performance of reinforced and prestressed concrete bridges.

T. Paul Teng, P.E.
Director, Office of Infrastructure
 Research and Development

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16. Abstract

Salt-induced reinforcing steel corrosion in concrete bridges has undoubtedly be come a considerable economic burden to many State and local transportation agencies. Since the iron in the steel has a nat ural tendency to revert eventually to its most stable oxide state, this problem will, unfortunately, still be with us, but to a much lesser degree due to the use of various corrosion protection strategies currently used in new construction. The adoption of corr osion protection measures in new construction, such as the use of good design and construction practices, adequate concrete co ver depth, low-permeability concrete, corrosion inhibitors, and coated reinforcing steel is significantly reducing the occurren ce of reinforcing steel corrosion in new bridges.

Because concrete has a tendency to crack, the use of good design and constructi on practices, adequate concrete cover depth, corrosion-inhibiting admixtures, and low-permeability concrete alone will not a bate the problem. Even corrosion-inhibiting admixtures for concrete would probably not be of use when the concrete is crack ed. This situation essentially leaves the reinforcing steel itself as the last line of defense against corrosion, and the use of a barrier system on the reinforcing steel, such as epoxy coating, another organic coating, or metallic coatings, is even more c ritical.

It is likely that there may never be any organic coating that can withstand the extreme combination of constant wetting and high temperature and high humidity that reinforcing steel is exposed to in some mari ne environments. Either steel bars coated with a sufficiently stable metallic coating or some type of corrosion-resistant solid metal bars would have to be used. There are some very convincing reports of good corrosion-resistance performance shown by epoxy -coated steel bars in concrete bridge decks where the concrete does not remain constantly wet and other exposure conditions are not as severe. Recent improvements to the epoxy coating specifications and the tightening of requirements on the prop er storage and handling of epoxy-coated reinforcing steel at construction sites will ensure good corrosion protection.

For construction of new prestressed concrete bridge members (where for structur al or other considerations epoxy-coated strands cannot be used), the use of a corrosion-inhibitor admixture in the conc rete or in the grout, in conjunction with good construction designs and practices, would provide adequate corrosion protection . However, the long-term effectiveness of all commercial inhibitor admixtures has not been fully verified.

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INTRODUCTION

The deterioration of reinforced concrete structures is a major problem. The cost of repairing or replacing deteriorated structures has become a major liability for highway agencies, estimated to be more than $20 billion and to be increasing at $500 million per year.[1] The primary cause of this deterioration (cracking, delamination, and spalling) is the corrosion of steel reinforcing bars due to chlorides. The two main sources of chlorides are deicing chemicals and seawater. The bare pavement policies of many highway agencies for winter snow and ice removal have resulted in extensive use of salt-based deicing chemicals. The most common chemical used has been sodium chloride. Many bridges have also been built in coastal areas and are exposed to seawater.

Figure 1. Corrosion-induced deterioration on a bridge column.

Bridges built with black reinforcing steel are showing progressive concrete deterioration as the concentration of chloride ions increases. According to a May 1997 report, *The Status of the Nation's Highway Bridges: Highway Bridge Replacement and Rehabilitation Program and National Bridge Inventory, Thirteenth Report to the United States Congress*, about 80,000 bridges on the Federal-aid system and 103,000 bridges off the Federal-aid system are deficient in some way. This includes both structurally deficient and functionally obsolete bridges. The average bridge deck located in a snow-belt State with black reinforcing steel and 40 mm (1.5 in) of concrete cover has shown spalling in about 7 to 10 years after construction and has required rehabilitation in about 20 years after construction.

The increase in the amount and severity of bridge deck deterioration in the late 1960's and early 1970's alarmed the State Highway Agencies and posed serious safety hazards to the traveling motorist. As a result several measures have been developed and implemented to prevent the chloride-induced corrosion of steel reinforcing bars and the resulting deterioration. Some of the early measures used included lowering the water-cement ratio of the concrete and increasing the concrete cover over the steel reinforcing bars. Concrete permeability can also be reduced by the use of admixtures. Corrosion inhibitors are also being used. Epoxy-coated reinforcing steel (ECR) was introduced in the mid 1970's as a protection system for new bridge decks. Another protection measure is the use of corrosion-resistant solid reinforcing bars or clad black reinforcing bars. The use of waterproof membranes in conjunction with asphaltic concrete overlays as a protective system has produced variable results. A multiple protection strategy is the simultaneous use of two or more protection measures, such as epoxy-coated reinforcing steel and a corrosion inhibitor.

The use of prestressed concrete members in bridges is a relatively new practice and consequently most bridge applications are relatively young. As a result, corrosion-induced deterioration of these members only became evident in the early 1980's. Although prestressed concrete members are generally manufactured with a higher strength concrete under better quality control, they are subject to the same effects of corrosion as is conventionally reinforced concrete. However, because of the high stresses in the prestressing strands, the corrosion process is accelerated. Even small corrosion pits could cause a strand to fracture, compared to conventional reinforcing that may rust to virtually nothing before breaking. There have been documented cases of prestressing strands breaking due to corrosion. This is a serious problem as prestressed concrete members rely on the tensile strength of the strands to resist applied loads.

Protective systems for prestressed concrete bridge superstructure members (girders and beams) and substructure members (piles, piers, etc.) are still under investigation and are being improved and developed further. These members are generally built with black prestressing steel. Current research shows promise for the use of epoxy-coated prestressing strands for prestressed concrete members. As of now, there is no generally accepted protective system for concrete bridge substructure members or prestressed concrete bridge members. However, because of the use of uncoated prestressing steel, the use of corrosion inhibitors in prestressed, concrete members has gained acceptance.

For most corrosion-protection measures, the basic principle is to prevent the chloride ions from reacting with the steel surface and also to increase the time needed for the chloride ions to penetrate through the concrete cover. While these measures generally do not stop corrosion from eventually initiating, they do increase the service life of reinforced concrete structures by slowing the corrosion process. Cathodic protection, however, has proven to be a successful corrosion-protection measure for conventionally reinforced and pretensioned, prestressed concrete bridge members.

CORROSION PROCESS

The corrosion of steel reinforcing bars is an electrochemical process that requires a flow of electric current and several chemical reactions. The three essential components of a galvanic corrosion cell are:

< Anode.
< Cathode.
< Electrolyte.

The general relationship between the components of a corrosion cell is illustrated in figure 2.[2]

The anode and cathode can be on the same steel reinforcing bar. Figure 3 illustrates a corrosion cell for a steel reinforcing bar embedded in concrete.[2] The anode is the location on a steel reinforcing bar where corrosion is taking place and metal is being lost. At the anode, iron atoms lose electrons to
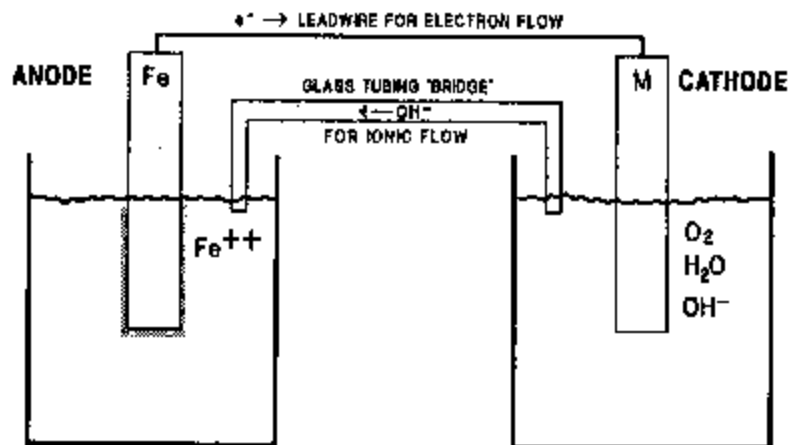

Figure 2. Electrochemical corrosion cell.

become iron ions (Fe^{+2}). This oxidation reaction is referred to as the anodic reaction. The cathode is the location on a steel reinforcing bar where metal is not consumed. At the cathode, oxygen, in the presence of water, accepts electrons to form hydroxyl ions (OH^-). This reduction reaction is referred to as the cathodic reaction.

The electrolyte is the medium that facilitates the flow of electrons (electric current) between the anode and the cathode. Concrete, when exposed to wet-dry cycles, has sufficient conductivity to serve as an electrolyte.

Both the anodic and cathodic reactions are necessary for the corrosion process to occur and they need to take place concurrently. The anode and cathode can be located next to each other or can be separated. When they are located immediately next to each other, i.e., on a
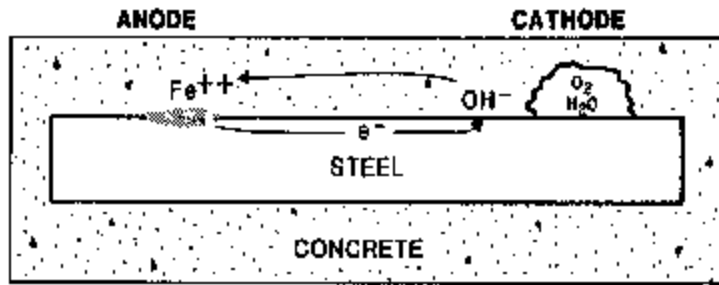
Figure 3. Corrosion cell in reinforced concrete.

microscopic scale, the resulting corrosion cell is referred to as a microcell. When they are separated by some finite distance, the resulting corrosion cell is referred to as a macrocell. Figure 4 shows examples of a microcell and a macrocell.[3] The corrosion of steel reinforcing bars embedded in concrete may be due to a combination of microcells and macrocells.
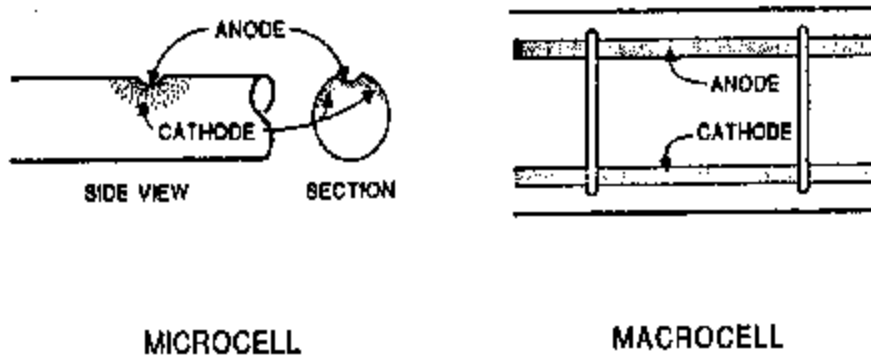
Figure 4. Microcell versus macrocell corrosion.

The initiation and continuation of the corrosion process are controlled by the environment in the concrete surrounding the steel reinforcing bars. The distribution of chlorides in a concrete bridge deck is not uniform. The chlorides typically enter the concrete from the top surface. The top mat of reinforcing steel is then exposed to higher concentrations of chlorides. The chlorides shift the potential of the top mat to a more negative (anodic) value. Since the potential of the bottom mat has a more positive (cathodic) value, the resulting difference in potentials sets up a galvanic type of corrosion cell called a macrocell. An electric circuit is established. The concrete serves as the electrolyte, and wire ties, metal chair supports, and steel bars serve as metallic conductors (figure 5).[3] Likewise, the concentration of chlorides at the top mat is not uniform along the length of the steel bars due to the heterogeneity of the concrete and uneven deicer application. These differences in chloride concentrations establish anodes

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and cathodes on individual steel bars in the top mat and result in the formation of microcells.
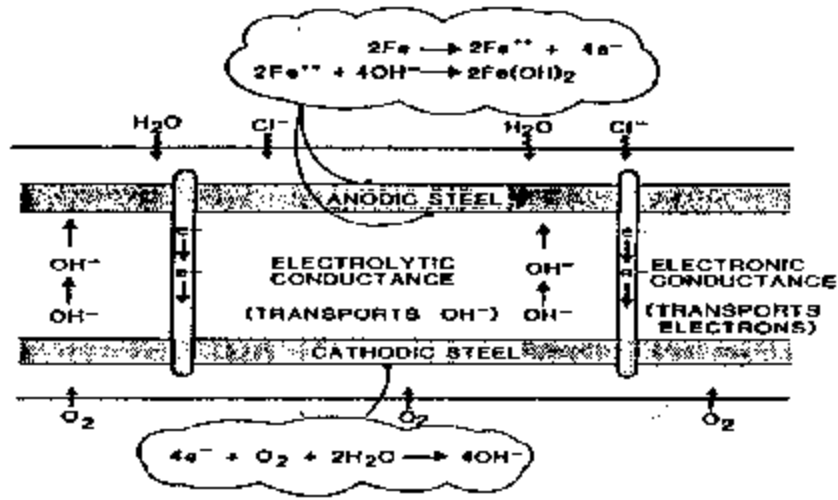
Figure 5. Electrolytic corrosion of reinforcement in concrete exposed to chloride and moisture.

Concrete is alkaline due to the presence of $Ca(OH)_2$, KOH, and NaOH and has an alkalinity typically between pH 12 and 13. The concrete pore solution consists primarily of KOH and NaOH. Due to the high alkalinity of the concrete porewater, the steel reinforcing bars are passivated by an iron oxide film ($ã @Fe_2O_3$) that protects the steel (figure 6).[2] The oxide film itself is a product of the initial corrosion of the steel reinforcing bar. In the initial stages of corrosion, a ferrous hydroxide $(Fe(OH)_2)$ compound is formed. Ferrous hydroxide has low solubility and, in the presence of oxygen and water, is oxidized to iron oxide (Fe_2O_3) to form the passivation film. As the film is being formed, the oxygen diffusion rate is reduced, which, in turn, reduces the corrosion rate.

Figure 6. Passivated steel in concrete.

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In order for corrosion to occur, the steel reinforcing bar needs to be depassivated. Oxygen, water, and an aggressive ion such as chloride need to be available, and the concrete needs to have low resistivity. In addition, all conditions need to be present simultaneously.

However, the intrusion of chloride ions is the most important factor in the corrosion of steel reinforcing bars embedded in concrete. Possible sources of chlorides include:
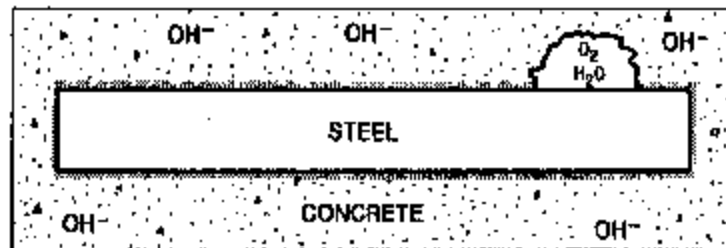
<   Aggregates.
<   Mix water.
<   Admixtures (in particular, accelerators).
<   Deicing chemicals.
<   Seawater.

The chloride content of portland cement, fly ash, and silica fume is typically very low. However, the chloride content of ground granulated blast-furnace slag is variable and depends on the water used in the quenching process. The chloride content can be significantly high if saltwater is used.

Aggregates may contain chlorides, especially if they are obtained from sites associated with seawater or with groundwater containing chlorides. Depending on the amount of chlorides present in the aggregates and the mix proportions, it is possible to produce a concrete that already has a chloride concentration at or above the limit for corrosion initiation.

Potable water can contain small amounts of chlorides (20 to 100 ppm). This amount of chlorides is generally considered to be insignificant. When used in concretes with typical mix proportions, the resulting concrete has a chloride concentration that is much lower than the threshold limit.

Besides admixtures based on calcium chloride ($CaCl_2$), some water reducers and setting admixtures contain chlorides. The amount is considered to be insignificant if the chloride content is less than 0.01 percent by mass of the cementitious material. The use of admixtures should be evaluated on a case-by-case basis for any impacts on the corrosion process. Calcium formate, sodium thiocyanate, calcium nitrate, and calcium nitrite are the commonly used chemicals. Calcium nitrite has been shown to be an effective corrosion inhibitor for steel embedded in concrete. Non-chloride accelerators should not be assumed to be non-corrosive. Sodium thiocyanate in high dosage rates has been reported to promote corrosion.

The process by which steel reinforcing bars are depassivated is not fully understood. Several theories have been presented to explain the role of chloride ions. Chloride ions reach the reinforcing steel by penetrating the concrete via the porewater and through cracks in the concrete. In the oxide film theory, the chloride ions break down the passive oxide film. At this point, the steel reinforcing bar becomes depassivated and corrosion may be initiated. In the adsorption theory, chloride ions are adsorbed into the surface of the steel reinforcing bar and attack the steel directly. In the transitory complex theory, chloride ions act as a catalyst. The chloride ions combine with the ferrous ions to form a soluble iron-chloride complex that diffuses away from the anode. Subsequent breakdown of the iron-chloride

complex frees the chloride ions for reuse when ferrous hydroxide is formed.

When carbon dioxide (CO_2) penetrates concrete and dissolves in the pore solution, carbonic acid is formed. This acid reacts with the alkali in the cement to form carbonates and to lower the pH of the concrete. When the alkalinity reaches a low enough level, the steel reinforcing bar becomes depassivated and, in the presence of sufficient water and oxygen, corrosion is initiated and proceeds. However, carbonation advances very slowly in sound concrete and is generally not a factor.

The corrosion of steel in concrete in the presence of oxygen, but without chlorides, takes place in several steps:

1.   At the anode, iron is oxidized to the ferrous state and releases electrons.

$$Fe \div Fe^{++} + 2e^-$$
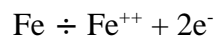
2.   These electrons migrate to the cathode where they combine with water and oxygen to form hydroxyl ions.

$$2e^- + H_2O + \tfrac{1}{2}O_2 \div 2OH^-$$
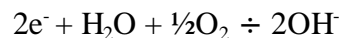
3.   The hydroxyl ions combine with the ferrous ions to form ferrous hydroxide.

$$Fe^{++} + 2OH^- \div Fe(OH)_2$$

4.   In the presence of water and oxygen, the ferrous hydroxide is further oxidized to form Fe_2O_3.

$$4Fe(OH)_2 + O_2 + H_2O \div 4Fe(OH)_3$$
$$2Fe(OH)_3 \div Fe_2O_3 @2H_2O$$
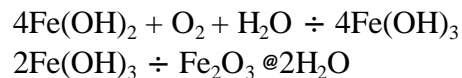
The corrosion of steel in concrete in the presence of chlorides, but with no oxygen (at the anode), takes place in several steps:

1.   At the anode, iron reacts with chloride ions to form an intermediate soluble iron-chloride complex.[4]

$$Fe + 2Cl^- \div (Fe^{++} + 2Cl^-) + 2e^-$$
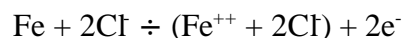
2.   When the iron-chloride complex diffuses away from the bar to an area with a higher pH and concentration of oxygen, it reacts with hydroxyl ions to form $Fe(OH)_2$. This complex reacts with water to form ferrous hydroxide.[5]

$$(Fe^{++} + 2Cl^-) + 2H_2O + 2e^- \div Fe(OH)_2 + 2H^+ + 2Cl^-$$
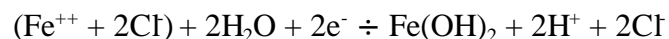
3. The hydrogen ions then combine with electrons to form hydrogen gas.

$$2H^+ + 2e^- \div H_28$$
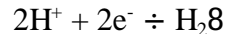
4. As in the case of the corrosion of steel without chlorides, the ferrous hydroxide, in the presence of water and oxygen, is further oxidized to form Fe_2O_3.

$$4Fe(OH)_2 + O_2 + H_2O \div 4Fe(OH)_3$$
$$2Fe(OH)_3 \div Fe_2O_3 @ 2H_2O$$
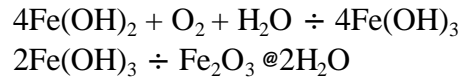
The corrosion products resulting from the corrosion of steel reinforcing bars occupy a volume equal to three to six times that of the original steel. This increase in volume induces stresses in the concrete that result in cracks, delaminations, and spalls. This accelerates the corrosion process by providing an easy pathway for the water and chlorides to reach the steel.

There are two main types of chloride contents that are tested for and reported – acid-soluble chlorides, sometimes referred to as total chlorides, and water-soluble chlorides. Acid-soluble chlorides are the chlorides extracted from a concrete sample using an acid. Water-soluble chlorides are those chlorides present that can dissolve in water. The amount of water-soluble chlorides is less than the total or acid-soluble amount of chlorides present in a concrete sample.

The minimum chloride ion concentration needed to initiate corrosion of steel reinforcing bars is also called the corrosion chloride threshold. Although the concept of a chloride threshold is generally accepted, there is little agreement on what the threshold value is. Several factors influence the chloride threshold value: the composition of the concrete (resistivity), the amount of moisture present, and the atmospheric conditions (temperature and humidity). The threshold concentration depends on the pH level and the concentration of oxygen. When chlorides are uniformly distributed, higher concentrations are needed to initiate corrosion. The amount of tricalcium aluminate (C_3A) present in the cement influences the threshold level. Regardless of what concentration of chloride ions is needed to initiate corrosion, an increase in the chloride ion concentration increases the probability that corrosion of the steel reinforcing bars will occur.

In general, the concentration of chloride ions needs to be more than 0.71 kg/m^3 (1.2 lb/yd^3). The ratio of chloride ions to hydroxyl ions also needs to be greater than 0.6.[5] However, the corrosion of steel reinforcing bars embedded in concrete is a complex process. The use of a single value or criteria for a chloride threshold may not be appropriate or accurate. This is illustrated in figure 7, which shows that the chloride threshold may be dependent on both pH and oxygen concentration.[6]

Whenever a difference in potentials on a metallic surface or between two metals is established, corrosion is initiated. This potential difference is referred to as the driving force. One area on the surface of the metal or one of the metals displays anodic behavior and the other displays cathodic behavior. Potential differences can be due to variations in the composition of the metal or in the environment surrounding the metal. Variations in the environment may be due to differences in pH,

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oxygen concentration, chloride concentration, moisture, or temperature. When the corrosion cell is created as a result of differences in the concentration of oxygen, chloride, or water, the cell is referred to as a concentration cell.
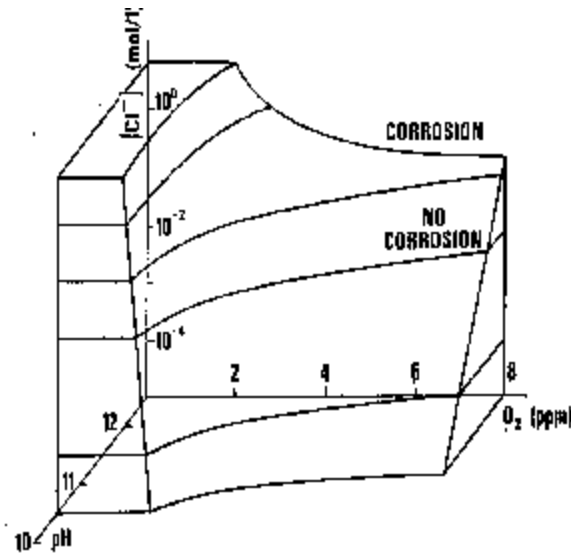
Figure 7. Three-dimensional plot of oxygen concentration, chloride concentration, and pH, illustrating regions of corrosion and no corrosion.

The four stages of chloride-induced deterioration of reinforced concrete are:

1. Chloride contamination and corrosion initiation.

2. Cracking – Occurs when the corrosion-induced tensile stresses exceed the tensile strength of the concrete (can be inclined or parallel to the roadway surface).

3. Delamination – Occurs when the cracks are parallel to the roadway surface and results in a fracture plane (often at the rebar level).

4. Spalling – When inclined cracks reach the roadway surface, freeze-thaw cycles and traffic loading cause the cracked delaminated portions to break away (accelerates the corrosion process).

A simple model for the corrosion of steel in concrete is shown in figure 8.[7] This service-life model for reinforced concrete structures has two stages – initiation and propagation. This model depicts the time to corrosion initiation and the subsequent deterioration rate. Some structures have been found to follow this model with reasonable accuracy. The initiation time is the length of time until depassivation of the steel reinforcing bars and the initiation of corrosion have occurred. The corrosion rate is controlled by

corrosion-process kinetics and may increase or decrease. At some point, cracking and spalling occur and the structure is either rehabilitated or has reached the end of its service life and is replaced. Several important factors are needed in order to quantify the deterioration rate: chloride profile, cover depth, carbonation depth, corrosion rate, concrete resistivity, and the environment.
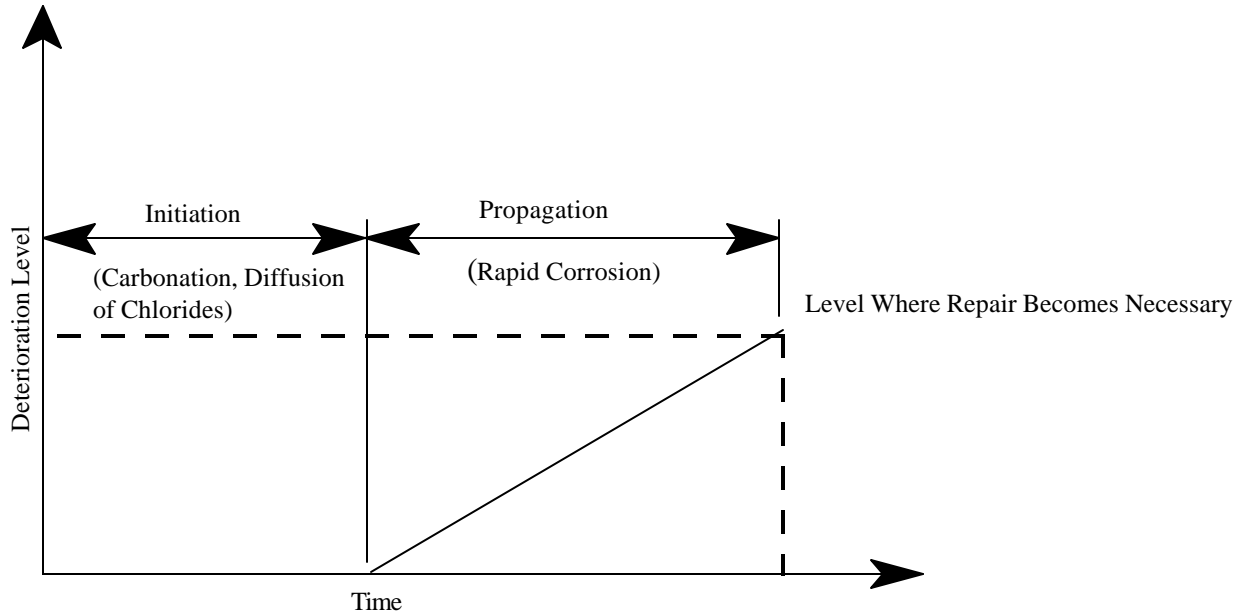
Figure 8. Simple deterioration model, corrosion of steel in concrete.

The amount of section loss after the initiation of corrosion can be estimated using Faraday's Law. An electrochemical equivalent is obtained by converting the corrosion current density to metal loss. A corrosion current density of 1 μA/cm^2 is equivalent to a metal loss of 12 μm/year (0.5 mils/year). This is generally considered to be a high rate and is likely to cause cracking and spalling within 1 year. Laboratory tests have shown that a 15- to 40-μm (0.6- to 1.6-mil) loss of metal has resulted in cracking in specimens containing rebars with a cover-to-diameter ratio between 2 and 4. This is equivalent to a current density of 0.5 μA/cm^2.

The three principal rate phenomena that control corrosion-induced deterioration of reinforced concrete bridge components are the chloride diffusion, corrosion, and deterioration rates. The chloride diffusion rate is the rate at which chloride ions diffuse through the concrete cover. The corrosion rate is the rate at which the corrosion process progresses after depassivation of the steel reinforcing bars has occurred. The deterioration rate is the rate at which concrete distress (cracking, delamination, and spalling) progresses. The deterioration rate will determine the length of time before repair or replacement of a deteriorated concrete bridge component is required.

FACTORS INFLUENCING THE CHLORIDE DIFFUSION RATE

The main factor that controls the diffusion of chloride ions in concrete is concrete permeability. Concrete permeability can be reduced by:

< Reducing the water-cement ratio of the concrete.
< Adding pozzolanic and pozzolanic/cementitious materials to the concrete.
< Adding polymer modifiers to the concrete.
< Aggregate gradation.

Some other factors influencing the diffusion of chloride ions in concrete include:[8]

< Surface charge on the hydrated cement paste.
< Formation of porous transition zones at the aggregate/cement paste interface.
< Microcracking.

An increase in microcracking can increase the rate of chloride ion permeability for structures subjected to cyclic loadings. Static compressive stresses do not appear to have any significant effect on chloride ion permeability. However, concrete exhibits a significant increase in permeability when loaded with cyclic compressive loads that are 60 to 80 percent of its ultimate strength. The rate of chloride ion permeability increases as residual strength decreases.

The prediction/calculation of chloride penetration into concrete is generally done using Fick's Second Law. However, the application of Fick's Second Law to predict chloride penetration yields results that are very conservative. This is mainly due to the description of concrete as a homogeneous medium to model the transport of dissolved ions (i.e., it is too simple a model and is not due to any fundamental problem with Fick's Law). In addition, the prediction of chloride ion penetration using diffusivity may be uncertain as the assumption of a constant chloride ion diffusivity is seldom seen in real structures.

FACTORS INFLUENCING CORROSION RATE

Once a sufficient amount of chlorides has reached the steel reinforcing bars to depassivate the bars and initiate corrosion, factors influencing the corrosion rate of steel reinforcing bars embedded in concrete include:[8]

< Availability of water and oxygen.
< Ratio of the steel surface area at the anode to that at the cathode.
< Amount of chloride ions in the porewater.
< Resistivity of the concrete.
< Temperature.
< Relative humidity (both internal and external).
< Concrete microstructure.

The availability of oxygen is a function of its rate of diffusion through the concrete, which is affected by how saturated the concrete is with water. When totally submerged, the diffusion rate is slowed because the oxygen must diffuse through the porewater. When the concrete is dry, the oxygen can freely move through the pores. Alternating wet-dry cycles accelerates the corrosion process. Wet concrete has a lower resistivity than dry concrete due to the presence of water as an electrolyte.

When the ratio of the area at the cathode to the area at the anode increases, the current density at the anode increases. The current density is the amount of electrical current passing through a unit area at the anode. An increase in current density results in an increase in the corrosion rate.

Other porewater chemistry parameters that may influence the corrosion rate and the nature of the resulting corrosion products include:[8]

< Ionic strength.
< pH.
< Redox potential.
< Cation composition.

Ionic strength affects the ionic exchange between the pore solution and the cement hydrate phases. When the pH is between 12.4 and 13.5, pore solutions have fairly high ionic strengths. The redox potential determines the oxidation state for those elements with multiple valances. Portland cement concrete pore solutions are generally oxidizing (positive redox potential), except when blast-furnace slag is added. The dominant cations in portland cement porewater solutions are sodium and potassium. The calcium content is significantly lower. This includes cements with slag, silica fume, and fly ash added. The alkali concentration in porewater solutions is generally not affected by the aggregates. However, alkali-reactive aggregates can affect the alkali concentration as the reaction removes alkalis from the pore solution.

The important chloride parameter is the amount of free chloride ions in the porewater. A sufficient amount of chlorides from external sources may overwhelm any benefits derived from chloride binding (high C_3A content in the cement) and any effects the porewater chemistry has on the corrosion process. An increase in chloride content increases the amount of free chlorides. A high level of free chlorides, together with a high Cl^-/OH^- ratio, leads to high corrosion currents, which, in turn, result in high corrosion rates.

The factors influencing the ionic conductivity/resistivity of concrete include:[8]

< Internal relative humidity.
< Amount of free water.
< Amount of connected porosity in cement paste.
< Ionic strength of the porewater.

An internal relative humidity of 70 to 80 percent is essential to maintain corrosion activity. Below this

level of relative humidity, active corrosion does not occur. The threshold varies with the concrete type and the ambient conditions.

When free water evaporates, electrical conductivity decreases to a low level. A high porewater content and the presence of electrolyte salts lead to lower resistivity. Lower resistivity generally increases the corrosion activity.

The controlling factors in the amount of connected porosity are the water-cement ratio and the use of mineral admixtures. Dense concretes have higher resistivity and inhibit ionic transport (i.e., they have low corrosion currents). Less dense concretes have lower resistivity and do not inhibit ionic transport (i.e., they have higher corrosion currents). This is particularly important when corrosion currents are between two layers of steel reinforcing bars (i.e., macrocell corrosion).

The heterogeneous nature of concrete, along with the non-uniform distribution of chlorides, results in localized depassivation and corrosion (both micro and macro) of the steel reinforcing bars. Corrosion is not uniform over the entire surface of steel reinforcing bars. The particle sizes in portland cement concrete range from sub-micron-sized hydrated cement phase crystallites to large coarse aggregates. The water-cement ratio and the use of mineral admixtures influence pore size distribution as well as total porosity. There is a difference in the porosity of the cement paste at the interface between the paste and the embedded items (steel reinforcing bars and aggregates), and away from these embedded items. The distribution of air voids may also be non-uniform. The air voids prefer to accumulate around the coarse aggregates and steel reinforcing bars.

FACTORS INFLUENCING DETERIORATION RATE

To date, not much research has been done in this area. The main focus has been on the depth of the concrete cover and permeability. High-strength concretes generally have low water-cement ratios, low porosity, and a relatively high modulus of elasticity. Because of its low porosity, high-strength concrete may have less ability to absorb corrosion products (i.e., they have fewer voids where corrosion products may accumulate without exerting any internal pressure on the concrete). In general, high-strength concretes have a higher modulus of elasticity and are less forgiving than concretes with a lower modulus. Concretes with a lower modulus may deflect without cracking, while in concretes with a higher modulus, stresses may build up and cause fracturing.[8] On the other hand, higher strength concretes generally have lower permeability and therefore it takes longer for chlorides to accumulate at the reinforcement level compared to lower strength concretes.

CORROSION-CONTROL MEASURES

Corrosion-induced deterioration of reinforced concrete structures occurs when the environmental loading on the structure is greater than the ability of the structure to resist the environmental loading (environmental resistance). One can either decrease the loading or increase the resistance or do a combination of both. The main deterioration mechanisms (chloride-induced corrosion of rebar) focus on the reinforcement and its protection.

Corrosion can also occur as a result of other deterioration processes: freeze-thaw cycles, expansive reactions, excessive deflections, fatigue, etc. These processes cause the concrete to crack, which subsequently allows water and chlorides easy access to the interior of the concrete and the steel reinforcing bars. These other deterioration mechanisms create conditions more conducive to the corrosion of the embedded steel reinforcing bars, which leads to further deterioration of the concrete.

The factors that influence the corrosion of steel reinforcing bars embedded in concrete are the amount of chloride ions at the steel level, the resistivity of the concrete, temperature, relative humidity (both internal and external), and the concrete microstructure. In general, by controlling these factors to an acceptable level, the corrosion of the steel reinforcing bars and resulting concrete deterioration can be minimized. This is the first step in most corrosion-control strategies in addition to other suitable corrosion-protection systems. Corrosion-control methods or systems are classified as mechanical or electrochemical.

Mechanical methods are physical barriers that prevent or delay the ingress of chlorides, oxygen, and moisture through the concrete cover to the reinforcing steel. They include admixtures, sealers and membranes, overlays, and coatings on steel reinforcing bars. Sealers and membranes made with materials such as resins, epoxies, emulsions, etc. are used to reduce the ingress of deleterious species. There are concerns about their effectiveness and durability on the traffic-bearing surfaces (bridge decks) due to the abrasion of applied sealers or the cracking of installed membranes. Portland cement concrete, low-slump dense concrete, latex-modified concrete, silica fume-modified concrete, and polymer concrete overlays are commonly used. Coatings used on steel reinforcing bars are either organic or metallic. Organic coatings include the non-metallic fusion-bonded epoxy coatings. Metallic coatings include materials such as nickel, stainless steel, and zinc. The nickel and stainless steel coatings protect steel by being a barrier system and more noble, i.e., have a lower potential than iron to corrode. The zinc coating protects steel by being sacrificial or more active (i.e., it has a greater potential than iron to corrode). Corrosion-resistant materials include austenitic stainless steels and fiber-reinforced polymer (FRP) rebars.

Electrochemical methods force the steel reinforcing bars to be cathodic. They include chloride extraction and cathodic protection. Chloride ion extraction and cathodic protection are typically used in the rehabilitation of reinforced concrete structures and not as a corrosion-control measure for new construction.

There are three categories of variables that influence the corrosion process and the extent of the

corrosion-induced deterioration of reinforced and prestressed concrete members – material, design, and environmental variables. Material variables for making durable concrete include cement type, admixtures, aggregate type and gradation, and the water-cement ratio. Design variables include the depth of concrete cover, physical properties of the hardened concrete, the size and spacing of the steel reinforcing bars, and the efficiency of drainage from the structure. Environmental variables include the source of chloride ions; temperature extremes; wet-dry cycles; relative humidity; and, to a certain extent, applied live loading.

Although little can be done to control environmental variables, material and design variables can be adjusted to build durable concrete structures that can resist corrosion-induced deterioration in environments conducive to the initiation and sustenance of the corrosion process.

SELECTION OF PROTECTION SYSTEMS

The proper corrosion-protection strategy will vary from structure to structure. Some factors to be considered during the design of a structure include:[9]

< Intended design life of the structure.
< Effects of corrosion and corrosion-induced deterioration – This includes the costs due to closure (either permanent or temporary) for repair. Bridges on major roads are more critical than bridges on local roads.
< Quality of workmanship in construction – The quality of construction entails good consolidation, proper rebar placement, sufficient concrete cover over the steel reinforcing bars, and other measures.
< Possible rehabilitation methods – The design of structures should include provisions for the possible future rehabilitation of corrosion-induced deterioration.
< Initial costs – May need to consider more than just initial costs (i.e., life-cycle costs). As the rehabilitation and replacement costs increase, corrosion-control measures become more cost-effective.

Multiple protection strategies may be cost-effective for long-term corrosion protection.[9] One such strategy is the use of epoxy-coated rebar in combination with a durable concrete containing corrosion inhibitors, having a low permeability, and adequate concrete cover. Silica fume and fly ash can be added to the concrete to reduce permeability and provide additional corrosion control. However, there is a need to balance the costs of the additional control measures against how much additional service life can be expected as a result of the added control measures. The additional costs can usually be justified based on a life-cycle cost analysis.

Some factors to be considered when choosing a corrosion-control measure include:[10]

< Reliability and effectiveness of the measure.
< Risk of unintended side effects.
< Possibility of future installation of other control measures.

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- <   Life expectancy of the measure.
- <   Any incremental costs over the "do nothing" option.
- <   Any impacts on the cost of other elements in the structure.
- <   How aggressive the environment is where the structure will be located.

Corrosion-protection strategies for steel reinforcing bars embedded in concrete can be grouped into four general categories: design, concrete, corrosion inhibitors, and reinforcement type.

The design category includes such items as:

- <   Concrete cover.
- <   Maximum allowable crack widths in service.
- <   Reinforcement distribution (crack control provisions).
- <   Rigid overlays (silica fume concrete, latex-modified concrete, dense concrete, polymer concrete).

The concrete category includes such items as:

- <   Water-cement ratio.
- <   Pozzolans (silica fume, fly ash, slag).
- <   Latex, epoxy, and polymer admixtures.
- <   Cement type.
- <   Aggregate gradation.

The inhibitor category includes such items as:

- <   Organic corrosion inhibitors.
- <   Inorganic corrosion inhibitors.
- <   Mixed corrosion inhibitors.

The reinforcement category includes such items as:

- <   Epoxy-coated bars.
- <   Galvanized bars.
- <   Nickel-clad bars.
- <   Copper-clad bars.
- <   Stainless steel-clad bars.
- <   Stainless steel bars.
- <   Corrosion-resistant alloyed bars.
- <   Non-metallic bars.

GENERAL DESIGN PROVISIONS

It is generally the design details that influence the overall performance and durability of bridge decks and other bridge components, both conventionally reinforced concrete and prestressed concrete. Some design factors that affect the durability of concrete structures include:

- Construction type.
- Expansion joints.
- Construction joints.
- Tendency of concrete to crack.
- Duct and anchorage layout in post-tensioned concrete.
- Drainage details.
- Access for inspection and maintenance.
- Proximity to seawater.
- Exposure to deicing chemicals.

The effectiveness and lifespan of expansion joints depend on how well they are installed. Whenever possible, construct continuous structures and integral abutments to eliminate expansion joints. However, when joints are used, provide adequate and proper drainage so that water does not reach the anchorages or bearings and does not pond. Include provisions for the inspection of the joints and structural components under the joint. Even well-constructed joints leak. Whenever possible, locate deck construction joints away from critical areas (prestressed anchorages in particular).

Cracks may be thermal or shrinkage. Cracking may also be due to creep or to the high modulus of elasticity of the hardened concrete. Exercise proper care in the layout and sequencing of concrete pours to minimize the risk of cracking. For post-tensioned structures, provide an adequate amount and distribution of reinforcement in the anchorage areas.

In post-tensioned concrete structures, the ease of grouting will influence the quality of the completed grouting operation. Both tendon profiles and duct size affect the ease of grouting. The location of anchorages affects the ease of stressing and inspection, as well as susceptibility to the ingress of water. Anchorages located in the top surfaces of decks are easy to construct, stress, and grout. However, due to their location, it is easier for chloride-contaminated water to penetrate and reach tendons.

Several design parameters can be adjusted as cost-effective corrosion-control measures. These include the use of adequate concrete cover, reinforcement distribution, the size and spacing of reinforcing steel for crack control, the use of rigid overlays, and provisions for good roadway drainage.

The use of well-consolidated, low-permeability adequate concrete cover is a cost-effective corrosion-control measure. The amount of concrete cover significantly influences the time-to-corrosion of the steel reinforcing bars and its quality influences the diffusion rate of chloride ions through the concrete. Since the diffusion of chloride ions in concrete is non-linear with increasing cover thickness, there is a significant increase in the time required for the chloride ions to reach the steel reinforcing bars.

However, with increased concrete cover, there is an increase in the potential for concrete cracking from shrinkage and thermal effects. The reinforcing steel bars become less effective for crack control with increasing cover thickness.

Chloride concentrations in the top 12 mm (0.5 in) of a concrete slab can be very high when compared to the concentrations at depths of 25 to 50 mm (2 to 3 in). A concrete cover of 25 mm (1 in) has been shown to be inadequate in severe environments, even with a water-cement ratio as low as 0.30. For moderate to severe environments, the amount of concrete cover should be at least 38 mm (1.5 in) and, preferably, 50 mm (2 in). Since 1974, the American Association of State Highway and Transportation Officials (AASHTO) Standard Specifications for Highway Bridges has required a minimum of 50 mm (2 in) of concrete cover over the top bars in bridge deck slabs.[9] The minimum cover for the main reinforcing steel with no positive corrosion protection in concrete deck slabs frequently exposed to deicing chemicals is 65 mm (2.5 in) for the top reinforcement and 25 mm (1 in) for the bottom reinforcement. The minimum concrete cover for reinforcing steel embedded in concrete with direct exposure to saltwater is 100 mm (4 in); it is 75 mm (3 in) for concrete cast against earth.[11]

The width of cracks in concrete is more of a concern than the number of cracks. The use of an increased number of well-distributed reinforcing steel bars is more effective in controlling crack widths than a smaller number of larger bars. The AASHTO Standard Specifications for Highway Bridges and the AASHTO Load and Resistance Factor Design Specifications both require reinforcement for shrinkage and temperature stresses.[11-12]

The minimum practical bridge deck thickness is 200 mm (8 in).[13] This is based on the typical reinforcement pattern in bridge decks (#4 and #5 bars), a 50-mm (2-in) concrete cover over the top bars, 25-mm (1-in) concrete cover over the bottom bars, consideration of the typical tolerances in placing the bars, and sufficient clearance between the two mats of steel to place and adequately consolidate the concrete. There is also more latitude in the placement of the reinforcing steel bars in thicker decks. Once deterioration of a concrete deck has started, it progresses more rapidly in thinner decks. Thinner decks also have more construction problems associated with increased reinforcement congestion and poor consolidation in particular.

There are some precautions that can be taken during the design of a structure to help minimize the potential for corrosion. The number of deck joints should be as few as possible and unnecessary joints should be eliminated. Open joints should be located as far as is practical from critical structural components. Place bearing devices on pedestals and use sloped surfaces on the tops of pier caps and abutment seats to reduce the ponding of salt-contaminated water and minimize the potential deterioration of these bridge elements. Gaps in railings also allow water and chlorides to reach beams. The coupling of dissimilar metals should be avoided to minimize galvanic corrosion.

Rigid overlay systems have been used to help prevent the penetration of salt-contaminated water into structural components. One such system involves the construction of a two-course deck. The first course (or sub-deck) contains the main load-carrying reinforcing steel. The concrete cover over the top mat of reinforcing steel is 35 mm (1.375 in) and the concrete cover over the bottom mat of reinforcing

steel is 30 mm (1.25 in). The second course consists of a 40-mm (1.5-in) silica fume overlay. Once the rigid overlay is in place, the completed deck system then has a total of 75 mm (3 in) of concrete cover over the top mat of reinforcing steel. With this system, the rigid overlay can be replaced when it begins to deteriorate and debond from the sub-deck and before chloride ions can begin to penetrate into the sub-deck. The condition of the rigid overlay should be periodically monitored. An adequate overlay maintenance program should either replace or repair deteriorated rigid overlays. This will extend the service life of the deck system (i.e., the time period before the sub-deck needs to be replaced).

With careful attention to details, proper deck slopes, and the size and spacing of deck drains, adequate deck drainage can be provided. This ensures that water will drain and not pond on the deck. Ponding prolongs the exposure to salt-contaminated water and allows water and chlorides more time to penetrate into the concrete. The use of fewer larger deck drains is generally more effective than more smaller drains. In addition, larger drains are not as apt to clog. Drains should be long enough and located so that salt-contaminated water is not discharged onto beams, pier columns, and abutments.

Some characteristics of a good drainage system are:

< Sufficient size of intake – a minimum of 200 mm by 200 mm (8 in by 8 in).
< Removable intake grates.
< Sufficient pipe diameter – a minimum of 150 mm (6 in).
< Adequate pipe slopes – a minimum of 60E.
< Properly located – away from expansion and contraction joints.

Inadequate drainage systems and leaky expansion joints allow water and chlorides to reach beam ends as well as piers and abutments. Expansion joints and drainage systems need to be properly maintained. Some common joint defects are deterioration of the joint sealer, lack of a joint sealer, and cracking of the concrete around the joints. No joint is perfect and traffic and environmental forces eventually result in joint deterioration and leakage. An adequate joint maintenance program should either replace or repair deteriorated joints.

GENERAL CONSTRUCTION PROVISIONS

There are several construction variables that influence the durability of concrete structures. These include concrete placing, consolidating, and curing; rebar placement; duct and tendon placement; and grouting procedures and materials. Poor construction practices can easily negate the best design provisions taken to produce a durable concrete structure.

Good consolidation practices help to avoid segregation and honeycombing, while yielding a uniform concrete with low permeability. A well-consolidated concrete can be achieved through the use of proper construction techniques and equipment. Poor consolidation results in concrete with higher permeability and voids, cavities, and poor bonding. Voids, cavities, and areas of poor bonding aid in the corrosion process. Poor procedures for grouting post-tensioning ducts can leave voids where moisture can accumulate and initiate corrosion of the prestressing tendons. In post-tensioned structures, certain grouts can cause severe corrosion if the excess mix water bleeds into the voided areas and is not absorbed into the grout during hardening. A recent example is the severe corrosion of all 19 seven-wire strands in the external post-tensioning ducts of the Niles Channel Bridge in Florida.

The proper and through consolidation of the concrete ensures that concrete is in intimate contact with the steel reinforcing bars. A good bond between the steel reinforcing bars and the surrounding concrete is critical for corrosion control. As a result of the intimate contact between the steel reinforcing bars and the concrete, the steel will be in the high-alkaline environment, necessary for the formation and maintenance of the passive oxide film. Exercise extra care when placing and consolidating concrete around embedded or partially embedded items so that water and chlorides do not have easy access to the steel reinforcing bars. When using epoxy-coated reinforcing steel, concrete consolidation should be done with a vibrator having a rubber-coated head.

Concrete curing procedures are an important part of workmanship. Proper and adequate curing provides durable concrete through increased cement hydration. A minimum of 7 days of uninterrupted moist cure is recommended. Whatever the curing method used, the surface of the concrete must be kept wet. Alternating wet-dry cycles promotes cracking in the concrete. There are three general categories of curing methods. A continuous water cure is done by a continuous spray, ponded water, or saturated surface coverings (burlap). Curing compounds seal the surface of the concrete. Moisture barrier materials, such as plastic sheets or waterproof paper, cover the surface of the concrete. A continuous water cure supplies sufficient water to prevent the surface of the concrete from drying. Both membranes and moisture barriers work by preventing evaporation of the mix water from the surface of the concrete.[14]

The accurate placement of steel reinforcing bars ensures that an adequate concrete cover over the bars will be obtained. Methods for placing and tying bars to ensure proper cover include the use of chairs, spacers, and form ties. Allowances for tolerances in bending bars may also be needed. Reinforcing steel should be adequately tied to prevent it from moving from the desired location during concrete placement and consolidation. Reinforcement support and ties should have adequate strength to carry construction loading before and during concrete placement and to avoid excessive deflection of the

reinforcing steel. The AASHTO Standard Specifications for Highway Bridges contains provisions for tying reinforcing bars.[11] All intersections around the perimeter of the reinforcing steel mat should be tied. Elsewhere within the reinforcing steel mat, the tie spacing should be not less than 0.6-m (2-ft) centers or every intersection, whichever is greater. Ties for epoxy-coated reinforcing steel should be plastic or epoxy-coated. Damage to the coating of epoxy-coated reinforcing steel should be properly repaired. Work platforms should be supported on the forms and not the reinforcing steel.

Mechanical finishing machines (screeds) are used to strike off the concrete to the desired profile grades. In order to not reduce the amount of concrete cover over the reinforcing steel bars, allowances for deflection, settlement, and camber need to be made. When the finishing machine is supported on rails, the rail supports need to be placed to minimize or eliminate any deflection of the rail between rail supports due to the weight of the finishing machine. A "dry run" is highly recommended to verify that the desired amount of concrete cover over the top layer of reinforcing steel is obtained. This will allow the contractor to make needed adjustments.

For post-tensioned concrete structures, grouting procedures are as important as the mix design. The grout needs to fully encapsulate tendons within the ducts in order to be an effective corrosion-control measure. Some common problems related to grouting procedures are line plugs, water and air voids, bleed water due to segregation, and shrinkage cracks. Line plugs can be due to duct damage, the presence of foreign material within the duct, and rapid stiffening of the grout.

CONCRETE

In new structures with good-quality concrete, the concrete can protect the steel reinforcing bars from corrosion for the service life of the structure. For steel in good-quality sound concrete – "uncontaminated" (little or no chlorides), uncarbonated, and uncracked – the steel is passivated and no corrosion, or a corrosion rate that is very low, can be expected. Any corrosion-induced concrete deterioration is not likely to reach a point where repair or rehabilitation will be required during the expected service life of the structure. However, the concrete quality can be violated by either chemical or mechanical means. Chemical means are chloride diffusion and carbonation, and the primary mechanical means is cracking. Cracks in concrete allow water, oxygen, and chlorides to enter the concrete at a faster rate and reach the reinforcing steel sooner than by the diffusion process alone.

In a recently completed research study, the effect of changes in independent concrete material variables on measured variables was evaluated and is summarized in table 1.[8, 15] This table shows where changes in the material components of concrete can improve its corrosion-control qualities.

The material variables included:

< Cement type (six types).
< Mineral admixture type (four types).
< Fine aggregate type (two types).
< Coarse aggregate type (two types).
< Water-cement ratio (three ratios).
< Air content (three values).

The measured variables included:

< Rapid chloride permeability.
< Compressive strength.
< Electrical resistivity.
< Corrosion rate.
< Corrosion potential.
< Final chloride concentration at the reinforcing steel surface resulting from diffusion through the concrete.

Table 1. Summary of the effects of material variables on concrete properties and corrosion behavior.									
Independent Variables	**Dependent Variables**								
	Rapid Chloride Permeability	**Resistivity**	**Compressive Strength**	**Corrosion Rate (Moderate)**	**Corrosion Rate (Aggressive)**	**Corrosion Potential* (Moderate)**	**Corrosion Potential* (Aggressive)**	**Chloride at Steel Surface (Moderate)**	**Chloride at Steel Surface (Aggressive)**
Water-Cement Ratio	>	?	?	>	>	>	>	=<	>
Air Content	=<	=<	?	=<	?	>	>	=<	=<
Coarse Aggregate	?	>	>	?	?	=<	>	=<	=<
Fine Aggregate	?	>	>	?	?	=<	=<	=<	=<
Mineral Admixture	?>	?>	?>	?>	?>	=<	?>	?>	?>
Cement Type	?>	?>	?>	?>	?>	?>	?>	?>	?>

? Decrease in dependent variable with an increase in independent variable.

> Increase in dependent variable with an increase in independent variable.

=< No trend in dependent variable with an increase in independent variable.

?> Significant change in dependent variable with change in independent variable.

* Increase in corrosion potential is an increasingly more negative potential.

** Increasing aggregate refers to increasing absorbent resistance

(going from limestone to quartz or glacial sand to quartz increases absorbent resistance).

Moderate Environment: 21EC (70EF), 75% Relative Humidity, 1.8 kg/m^3 (3 lb/yd^3) Cl$^-$

Aggressive Environment: 38EC (100EF), 98% Relative Humidity, 6 kg/m^3 (10 lb/yd^3) Cl$^-$

The key to long-term durability of reinforced concrete structures is the use of portland cement concrete with low permeability and adequate concrete cover. A concrete with low permeability has an improved resistance to chloride ion penetration or diffusion. This keeps chlorides, as well as water and oxygen, from reaching the steel reinforcing bars. An adequate concrete cover increases the amount of time required for any chlorides to reach the steel reinforcing bars.

A lower water-cement ratio generally makes concrete less permeable. Although a low water-cement ratio does not ensure that the concrete will have a low permeability, concretes with the proper gradation and type of fine and coarse aggregates and mineral admixtures that have a higher resistance to chloride penetration are those with a lower water-cement ratio. Concrete also needs to be properly proportioned and well-consolidated. A decrease in the water-cement ratio results in concrete with a reduced porosity and a reduced permeability. A reduction in water-cement ratio and the use of latex polymer modifiers or mineral admixtures, especially silica fume, are very effective strategies for reducing the permeability of the hardened concrete. With adequate cover, concrete with lower water-cement ratios perform better than those with higher water-cement ratios.

Changes in the water-cement ratio do not significantly influence resistivity at an earlier age. The electrical resistance of concretes at 28 days and with water-cement ratios varying from 0.30 to 0.50 have been shown in tests to be similar, but are significantly altered at 90 days. The improved performance of concretes with lower water-cement ratios is due to a reduction in concrete permeability and an increase in resistivity. The resistivity of concretes with a water-cement ratio of 0.3 is much higher than the resistivity of concretes with a water-cement ratio of 0.4 or 0.5 at 90 days.

Mineral admixtures can be used to enhance the corrosion-control potential of the concrete by reducing permeability. Some common admixtures used are fly ash, blast-furnace slag, and silica fume. For mineral additives, the additional calcium silicate hydrate that the mineral admixtures contribute lead to a reduction in permeability and a reduced chloride diffusion rate. The availability of hydroxyl ions is typically expected to decrease. The sources of ground granulated blast-furnace slag and fly ash should be evaluated for changes in their chemistry. Any changes can significantly affect the characteristics of the concrete and ultimately its performance.

Silica fume is a byproduct of silicon metal and ferrosilicon alloy production. Silica fume consists of fine glassy spheres with a specific surface area of 20,025 m^2/kg (97,650 ft^2/lb). The specific surface area of portland cement is 300 to 400 m^2/kg (1465 to 1950 ft^2/lb). The particle size of silica fume allows it to fit into the small spaces usually occupied by water, which results in a denser mix.

Concrete mixes containing silica fume are highly impermeable to chloride penetration and are resistant to the flow of corrosion currents due to their high electrical resistivity. Compressive strengths are also higher. Silica fume has been shown to offer the largest and most consistent reduction in penetration rates for chloride ions in concrete. However, these mixes are more susceptible to cracking. Silica fume mixes require more mix water in order to produce a mix with a workability comparable to a portland cement concrete mix without silica fume. A superplasticizer can be used to reduce the amount of mix water needed and to improve workability.

Latex-modified concrete (LMC) is made by incorporating a polymeric latex emulsion into fresh portland cement concrete. Latex consists of a polymer suspended colloidally in water. Specially formulated polymers are used in concrete. Styrene-butadiene latexes are most commonly used. Due to the high material cost, it is generally used only for bridge deck overlay. These overlays are usually not very thick, 40 to 50 mm (1.5 to 2 in), in order to minimize costs.

Latex-modified concretes exhibit improved durability. This is due to a reduced permeability, a high degree of resistance to chloride ion penetration, and an increase in resistance to tensile cracking. The use of a latex also allows for a reduction in the water-cement ratio since some of the mix water is replaced by latex. The reduced water-cement ratio leads to an increase in strength as well as a reduction in drying shrinkage cracking. The spherical polymer particles are very small (~0.01 to 1 µm in diameter). They act like entrained air and improve workability and decrease bleeding. In general, the latex also contains air and there may be a need to add an antifoaming agent to limit the entrained air content, usually 6.5 percent. Higher entrained air contents reduce the flexural, compressive, and bond strength of the LMC. If the air content is greater than 9 percent, the permeability to chloride ion penetration increases.

It is believed that the reduced permeability in latex-modified concrete is achieved through the formation of a continuous polymer film lining the pores. Film formation is aided by the removal of water through the hydration reaction. Therefore, there is a need to minimize the amount of continuous wet cure. It is generally recommended that a 2-day moist cure be followed by a dry cure for 72 h. Although this cure procedure does not allow the full strength of the paste to develop, the drying is needed for film development. In hot weather placement, rapid drying makes LMC more difficult to finish and promotes shrinkage cracking. As a result, nighttime placement has been done to compensate for this.

The aggregate permeability may be an important factor in the migration rate of chloride ions. For concretes with a normal pH (12.5 to 13.8), the typical coarse and fine aggregates used in bridge structures can be thought of as an "inert material."

Cement type appears to influence the diffusion of chloride ions through concrete. It is thought that this is accomplished through chloride binding. Chloride binding is the chemical reaction between chloride ions in solution and cement hydration products. It results in the formation of calcium chloroaluminates, an insoluble chloride phase. This removes chlorides from the porewater and reduces the amount of free chlorides available to participate in the depassivation and corrosion processes. The amount of free chloride ions in the porewater is more important than the amount of total chloride ions.

Some correlations between cement chemistry and the cement's ability to bind chlorides have been proposed. The key constituent is speculated to be the tricalcium aluminate (C_3A) content. Others have speculated that it is total alkalinity and not the C_3A content. Chlorides diffuse more in cements with low C_3A content. A calcium aluminate cement may bond a large amount of chlorides, while a magnesium phosphate cement binds little or no chlorides. Concrete mixes containing cements with high C_3A contents and ground granulated blast-furnace slag exhibit a significantly greater ability to bind chlorides. However, the amount of chlorides bound is low relative to chloride contents typically found in bridge

decks.

Although there is still a considerable amount of disagreement on the value of placing limits on chloride content in mix ingredients, some recommendations have been made by AASHTO and the American Concrete Institute (ACI). Since chlorides added in the concrete mix tend to be more uniformly distributed than chlorides from external sources, it is not as likely to lead to the creation of concentration cells. However, when concrete members are expected to be exposed to chlorides, it is advisable to keep any chlorides added to the concrete from the mix ingredients to a minimum. The AASHTO Standard Specifications for Highway Bridges recommends a maximum chloride ion concentration in the concrete mixing water of 1000 ppm.[11] The ACI committee report, ACI 222R-96, *Corrosion of Metals in Concrete*, contains recommended chloride limits in concrete for new construction to minimize the risk of chloride-induced corrosion.[16] These limits are summarized in table 2 and are expressed as a percent by weight of portland cement. A limit on the amount of chlorides in the fine and coarse aggregates is also presented. The amount of acid-soluble chlorides in the fine and coarse aggregates together should not exceed 0.06 percent by mass of aggregates.

Table 2. ACI-recommended chloride limits for new construction.		
	Acid Soluble (Performed by ASTM C1152)	**Water Soluble (Performed by ASTM C1218)**
Prestressed Concrete	0.08	0.06
Reinforced Concrete in Wet Conditions	0.10	0.08
Reinforced Concrete in Dry Conditions	0.20	0.15

A significant amount of research has shown that corrosion can initiate at chloride concentrations as low as 0.71 kg/m^3 (1.2 lb/yd^3) [approximately 0.15 percent for 341 kg (752 lb) of cement mix]. For wet conditions and 341 kg (752 lb) of cement mix, the ACI criteria allows for chloride concentrations up to 0.47 kg/m^3 (0.8 lb/yd^3). This leaves little room for the ingress of additional chlorides from the application of deicers or from exposure to seawater. In addition, ACI provisions are primarily used for building structures where the chloride exposure is significantly different than that for bridges.

PRESTRESSED CONCRETE

In prestressed concrete structures, high-strength prestressing steel is used to increase load capacity, improve crack control, and allow the construction of more slender components. There are two main types of prestressed concrete: pretensioned and post-tensioned.

In pretensioned concrete, the tendons (wires or strands) are tensioned before the concrete is placed and cured. After a predetermined required strength is achieved, the tendons are released. Prestressed concrete members are normally produced in a controlled environment. As a result, a higher quality concrete can be achieved. Standardized sections have been developed: I-beams, box beams, bulb-T, and modified bulb-T. Prestressed concrete deck panels are also produced.

In post-tensioned concrete, the tendons are tensioned after the concrete is placed and cured, and has achieved a predetermined required strength. After the tendons are stressed, they are anchored through the use of mechanical anchorages at each end of the member. There are two categories of post-tensioning tendons: bonded and unbonded. Bonded tendons are placed within ducts that were previously cast into the concrete. After the tendons are stressed and anchored, the ducts are filled with grout. Unbonded tendons are greased and then sheathed in plastic. Unbonded tendons are not used very much since it is difficult to ensure adequate corrosion protection. However, they have been used as external tendons to increase structural strength and integrity. In general, external tendons can be more easily inspected than internal tendons. Segmental bridges are a type of post-tensioned concrete structure where precast segments are joined together by post-tensioning tendons or bars.

The two main forms of corrosive attack of prestressing strands and tendons are pitting corrosion and stress corrosion (environmentally induced) cracking.

Pitting corrosion is a localized galvanic corrosion cell at weak points or ruptures in the passivation film. The resulting pits reduce the cross-sectional area of the tendons and are stress risers (i.e., they increase the magnitude of applied tensile stresses). This can lead to brittle fracture of individual wires in a strand or tendon and ultimately the failure of the tendon and prestressed concrete member. Pitting corrosion is also a source of atomic hydrogen that can contribute to possible hydrogen embrittlement of the high-strength steel strands.

Environmentally induced cracking (EIC) is when the combination of a corrosive environment and tensile stresses induces a brittle fracture in a normally ductile alloy. The tensile stress is static and there is a threshold stress below which EIC will not occur. The cracks can be either transgranular or intergranular and propagate normally to the direction of applied tensile stress. There are two main categories of EIC – stress corrosion cracking (SCC) and hydrogen-induced cracking (HIC). The cracking in HIC is predominately transgranular, usually unbranched, and very brittle and fast growing. The cracking in SCC is predominately intergranular, usually branched, and propagates at a slower rate.[17]

Stress corrosion cracking is when corrosion of the prestressing steel in combination with high tensile stresses in the prestressing steel lead to cracking perpendicular to the direction of the applied stress.

Hydrogen-induced cracking is commonly recognized as a form of stress corrosion and is due to hydrogen embrittlement. This is when atomic hydrogen diffuses into the steel and combines to form hydrogen gas. The hydrogen molecule exerts an internal pressure that when added to the applied tensile stress can lead to fracture of the steel wires in a strand or tendon.

Because of the high level of stress in prestressed concrete bridge members, corrosion is more of a concern for prestressed structures than for conventionally reinforced concrete structures. Because of the very high tensile stresses in prestressing steel strands, any reduction in cross-sectional area due to pitting may lead to fracture of individual wires. As a result, the ACI limit on chlorides in prestressed concrete members is half of that for conventionally reinforced concrete.

The main causes of failure for prestressing steel in bridges are the corrosion of the steel strands or tendons, deterioration of protective sheaths and ducts, and end anchorage failure. Although prestressing steel may fail due to manufacturing defects, this is not very common. Corrosion of the prestressing steel prior to its placement in concrete may be due to manufacturing defects or improper handling. The most common type of improper handling of steel prestressing tendons is not protecting the steel tendons from the environment (i.e., the weather).

Corrosion of pretensioning tendons may be due to one or more of the following:[18]

- < Voids under or next to the tendons.
- < Lack of passivation of the tendons due to a decrease in alkalinity.
- < A corrosive environment at the tendons due to the presence of chlorides, water, and oxygen.
- < Joints that are not sealed or not watertight.
- < Chlorides from the mix water or aggregates.
- < Inadequate concrete cover due to poor construction practices.
- < Concrete with a high permeability due to a high water-cement ratio and/or poor consolidation.

Corrosion of post-tensioning tendons may be due to one or more of the following:[18]

- < No protection between the time when the tendon is placed in the ducts and is stressed and grouted.
- < Poor-quality concrete and improper construction practices.
- < A corrosive environment at the tendons due to the presence of chlorides, water, and oxygen (for bonded or unbonded tendons).
- < Chlorides in the grout mix.
- < Contact between dissimilar metals, such as the aluminum casings at the anchorages and the steel strands.
- < Voids in the ducts, due to poor grouting procedures, leading to inadequate bonding between the tendon and the grout.
- < Inadequate sheathing (damaged during transportation and placement within the structure) leading to possible exposure of the steel tendons to a corrosive environment.
- < Excessive bleed water.

<   Chlorides from external sources penetrating the concrete and accumulating at the tendons through perforated ducts or sheathings and at anchorages.

Corrosion of prestressing steel should not be a problem if:

<   Uncracked good-quality concrete is used.
<   Adequate cover (AASHTO recommendations) is provided.
<   Adequate protection of prestressing steel during shipment and storage is provided.
<   Good grouting practices to minimize or eliminate voids in ducts are used.

A good-quality concrete with low permeability is a primary corrosion-control measure for both pretensioned and post-tensioned concrete. Low permeability can be achieved in a well-consolidated concrete with a low water-cement ratio. Admixtures such as silica fume and fly ash may also be used. The use of a corrosion inhibitor is also possible. In very corrosive environments, it may be necessary to use very high-quality concretes with an extremely low permeability. High-quality concrete with a low water-cement ratio and low permeability are typical characteristics of precast pretensioned concrete that is produced in a controlled environment.

There are some corrosion-protection measures that can be applied to the tendons themselves. Epoxy-coated strands are coated with an extra-thick coating of epoxy to allow for elongation, and often have a coarse grit embedded in the epoxy to aid in bonding. The epoxy coating provides a mechanical moisture barrier. The coating thickness is considerably more than that on epoxy-coated steel reinforcing bars. The performance of epoxy-coated strands in both pretensioned and post-tensioned specimens under severe exposure conditions has been excellent. However, there are a number of issues besides cost-effectiveness that need to be resolved before epoxy-coated strands are commonly used in either pretensioned or post-tensioned construction. An "intelligent strand" is a strand where a fiber-optic sensor is placed through the center wire. It can be used to monitor strains and for any breaks in the strands.

Precautions can be taken during the design of prestressed concrete beam structures that will enhance the corrosion performance. The exposure of the ends of prestressed concrete beams to deicing chemicals can be minimized by using continuous prestressed concrete beams or encasing the prestressed concrete beam ends in concrete diaphragms or abutments. The structural capacity can also be enhanced by making the structure continuous for live load. Minimize the number of deck joints to avoid leakage of water and chlorides to the bridge superstructure and substructure members. Bridges as long as 850 m (2800 ft) have been constructed without joints except at the abutments.

There are also some precautions that can be taken during the design of pretensioned box beam structures that will enhance the corrosion performance. These include the use of rigid wearing surfaces or composite deck slabs, stiffeners to prevent movement of beam faces, transverse post-tensioning to hold beams together, and adequate slope and drainage details.

POST-TENSIONED CONCRETE

Durable bonded post-tensioned concrete structures can be achieved through improved design details, specifications, and workmanship. Design details include anchorages, ducts, and provisions for the protection of the tendons. Specifications include materials, testing, and grouting processes. Workmanship includes construction practices, the grouting process in particular, and inspection and testing, both during and after construction. In post-tensioned concrete structures, the prestressing tendons are enclosed in a duct. A bonded post-tensioned concrete structure is when the duct is filled with grout.

Failures of corrosion-protection systems for bonded post-tensioned concrete structures are commonly due to ineffective grouting materials and methods, poor workmanship, construction defects, and poor design details. Poor design details provide easy access for chloride-contaminated water to reach tendons. Some design factors that affect the durability of post-tensioned concrete structures include:[19]

< Expansion joints.
< Construction joints.
< Concrete cracking.
< Duct and anchorage layout.
< Segmental construction and joint type.
< Deicing chemicals and drainage.
< Waterproofing systems.
< Access for inspection and maintenance.
< Proximity to seawater.

Both tendon profiles and duct size affect the ease of grouting. The ease of grouting will, in turn, influence the quality of the completed grouting operation. The location of anchorages affects the ease of stressing and inspection, as well as susceptibility for ingress of water. Anchorages located in the top surfaces of decks are easy to construct, stress, and grout. However, due to their location, it is easier for chloride-contaminated water to penetrate and reach tendons.

Although concrete provides a measure of corrosion protection for tendons, it makes it more difficult to inspect and monitor the tendons for possible corrosion, and in some locations, it is practically impossible. Therefore, a multi-strategy protection system is recommended. This consists of a number of protective measures applied to a post-tensioned concrete structure. Even if an individual measure becomes ineffective, the other remaining measures give adequate corrosion protection.

There are numerous corrosion-protection measures that can be used for post-tensioned concrete structures and some of these measures on their own can be sufficient to protect the tendons. However, as is usually the case, none are truly fully effective as a corrosion-control measure and the multi-strategy approach is appropriate. The designer needs to consider the risk of corrosion and the service life of the various measures, as well as future inspection needs. The use of a multi-strategy protection concept for the protection of anchorages is recommended. Some possible protective measures that are available

that can be combined to form a multi-strategy protection system for tendons include:[19]

- <   Use adequate concrete cover over the ducts.
- <   Completely fill the ducts with grout.
- <   Use durable and corrosion-resistant duct materials.
- <   Design the ducts to keep out water and associated contaminates.
- <   Pay careful attention to design details.
- <   Provide access for inspection, testing, and maintenance (especially at anchorages).
- <   Use corrosion-resistant or "intelligent" strands.

The use of a good-quality low-permeability concrete and an adequate concrete cover are good cost-effective corrosion-control measures for post-tensioned concrete structures. An increase in concrete cover is viable up to the point at which the added dead load requires additional prestressing force, resulting in increased cost and weight. The same steps that are taken to reduce concrete permeability for conventionally reinforced concrete structures can be used for post-tensioned concrete structures: a low water-cement ratio and the addition of silica fume, ground blast-furnace slag, or latex and other polymer modifiers.

The main benefit from grouting is that a suitable grout mix design creates an alkaline environment within the ducts. This benefit is highly dependant on satisfactorily filling the ducts. Voids do not necessarily cause corrosion if the ducts are sealed from chlorides as part of a multi-strategy protection system, along with good-quality concrete cover and corrosion-resistant non-metallic pressure-tested ducts. Suitability trials can be used to evaluate the pumpability and performance of a grout. Variations in age, chemical composition, fineness, and temperature of bagged cement can significantly influence grout performance.

In post-tensioned concrete structures, the penetration of chlorides to the steel strands is much more difficult if the ducts are intact and a good-quality grout is used. A good-quality concrete is also needed to protect the ducts and the anchorages. Field studies of existing post-tensioned concrete bridges have shown that strands inside ducts that were surrounded by grout were not corroding. Paper conduits filled with grease were found to have provided the least amount of corrosion protection. Bonded post-tensioned tendons rely on the concrete cover, ducts, and grout for corrosion protection. Unbonded post-tensioned tendons rely on special corrosion prevention greases and plastic coatings (sheaths) for corrosion protection.

The duct system (ducts and couplers), together with a good-quality grout, form a multi-strategy protection system for the prestressing tendons. A sealed-duct system forms an air- and water-resistant barrier. It should also be strong enough to retain its shape and resist damage during installation and concrete placement. When used as part of a multi-strategy protection system, if the duct is not completely filled with grout, then the sealed system is able to provide some measure of corrosion protection. However, in order for the ducts to be an integral part of a multi-layered strategy, the duct itself should not be subject to corrosion. It should not adversely react with the concrete or the grout. Several types of ducts are available: corrugated steel, smooth steel, and polyethylene or polypropylene.

Non-metallic ducts are made of high-density polyethylene or polypropylene. The main advantage of non-metallic ducts is that they will not corrode metallurgically; but the material will degrade with time. Non-metallic duct systems can also be pressure-tested before placing concrete to test for system integrity. It is also advisable to pressure test them after placing concrete to again test for system integrity. The minimum wall thickness for polyethylene or polypropylene ducts is 1.5 mm (0.06 in). The minimum wall thickness for polyvinyl chloride ducts is 1 mm (0.04 in).[20]

Metallic ducts are the type of ducts most commonly used in post-tensioned concrete bridges. Holes in ducts may allow concrete to enter and make it more difficult to place the strands (tendons) and pump the grout. Galvanized ducts offer better corrosion resistance than bare steel. Polyethylene ducts have demonstrated better performance. The minimum thickness of the strip steel used to fabricate corrugated steel ducts is 0.45 mm (26 gauge) for ducts less than 65 mm (2.625 in) in diameter and 0.6 mm (24 gauge) for ducts greater than 65 mm (2.625 in) in diameter.[20]

Adequately sealed and protected anchorages prevent water and chlorides from penetrating to the prestressing steel. The protection of anchorages depends on the performance of the surrounding concrete. A good-quality mortar is needed to protect the ends of anchorages (ends exposed during stressing operation). Non-shrink grout did not perform any better than conventional mortar in laboratory studies. Epoxy coating of anchorages and associated hardware has provided very good corrosion protection. It may be necessary to encapsulate anchorages in an impermeable material in order to protect them.

The inside duct diameter should be large enough to readily permit placement of the tendons and allow complete grouting of the tendons. The duct diameter depends on the size of the tendon, the curvature of the tendon profile, and the length of the tendon. The AASHTO minimum recommended duct size is:[12]

< Duct diameter should be 6 mm (0.25 in) larger than the nominal diameter of the single bar or strand.
< Cross-sectional area (internal) should be 2.0 times the net area of the prestressing strands for multiple bars or strands.
< Cross-sectional area (internal) should be 2.5 times the net area of the prestressing strands where the tendons will be placed by the pull-through method.
< Should not exceed 0.4 times the least gross concrete thickness at the duct.

The Post-Tensioning Institute's minimum recommended duct size is:[20]

< Duct diameter should be 6 mm (0.25 in) larger than the outside diameter (maximum dimension) of the prestressing bars.
< Net duct area (gross area minus the area of the prestressing bars or strands) should be 1.25 times the net area of the prestressing strands.

If the actual area varies from this, use grout tests to verify that adequate grouting can be accomplished.

GROUTS FOR BONDED POST-TENSIONED CONCRETE

Most corrosion problems with bonded post-tensioned concrete structures are associated with poor grouting practices and grouting materials. The primary purpose of the grout is to provide a non-corrosive environment and corrosion protection to the steel prestressing tendons. The grout also provides bonding between the prestressing steel and the concrete to transfer tensile stresses from the tendon to the concrete along the length of the prestressed concrete member. Grout consists of portland cement, water, and admixtures. The high alkalinity of the cement passivates the steel. As in conventional concrete mixes, the corrosion resistance for grouts is also related to its permeability to oxygen and chloride ions. The permeability of grouts can be reduced by lowering the water-cement ratio and adding silica fume, ground blast-furnace slag, or latex modifiers. When the chloride ion permeability is reduced, the time to initiation of corrosion is increased and subsequent corrosion rates are decreased. It is also important that potentially corrosive materials are not added to grouts.

Currently, there is no single American Society for Testing and Materials (ASTM) or industry specification covering grouts for post-tensioning tendons. The Post-Tensioning Institute is in the process of publishing a *Guide Specification for Grouting of Post-Tensioned Structures*.[20] Some ideal characteristics of grouts for bonded post-tensioned concrete are:

< Low permeability to chlorides.
< Ability to inhibit corrosion if chlorides reach the prestressing steel.
< Adequate fluidity and the ability to retain that fluidity during the time needed to fill the ducts.
< Volume stability – Ability to keep the duct filled over an extended period of time and not undergo a reduction in volume in the plastic or hardened state.
< Acceptable consistency to minimize potential for bleeding, segregation, or the creation of water lenses or voids when under pressure or in contact with steel prestressing strands.
< Adequate bonding and compressive strength.
< Resistance to vibration and shock.
< Minimal shrinkage to avoid shrinkage cracking.

Grout viscosity during grouting should be high enough to be effectively and adequately pumped, completely filling the post-tensioning duct, but low enough to expel air and any water from the duct. The grout must exhibit a suitable viscosity that can be maintained for the length of time needed to perform the grouting operation. The rheological properties of grouts can vary widely. The time period during which the grout can be pumped is referred to as the open time. Several methods are used to measure the flowability of grouts. For pourable grouts, the flow cone test can be used. With ASTM C939, the flow cone method, grout is allowed to flow due to its own weight. However, the fluidity of a thixotropic grout cannot be determined by using the flow cone. Thixotropic grouts that can be satisfactorily pumped may not pass through the flow cone. Most grout mixes used are thixotropic. In a thixotropic material, the apparent viscosity decreases when a shear stress is applied (shear thinning). There is a gradual recovery in viscosity after the stress is removed. The viscosity recovery is time-dependant. Thixotropic grouts may not exhibit flowability until a shear stress is applied, either by applying pressure or by agitation.

A common problem associated with the grouting of bonded post-tensioning tendons is the segregation of water from the grout mixture (pressure-induced bleeding). There are two forms of bleeding. In one form, water rises to top of the duct and the heavier cement and aggregates settle to the bottom due to differences in unit weights. In the other form, when the tendons are made up of strands, bleeding is due to the filtering action of the strands. In this form, water permeates through the spaces between the outer and center strands, but solid particles (cement) do not. In both instances, water accumulates near the top of the ducts. Air voids are created when water is reabsorbed into the hardened grout. Voids may allow air, water, and chlorides into the ducts, which may cause corrosion of the steel tendons.

The factors that influence the corrosion protection provided by the grout in post-tensioned concrete structures can be grouped into three main categories: mix design, grout properties in the fresh or plastic state, and grout properties in the hardened state.[21]

Mix design factors include:

<   Type and amount of portland cement.
<   Water-cement ratio.
<   Type and amount of mineral and other admixtures.
<   Type and amount of fine aggregate.

Grout properties in the fresh or plastic state include:

<   Grout fluidity (open time, the time period during which the grout has a measurable time of efflux).
<   Bleeding characteristics of the grout.
<   Expansion of the grout.

Grout properties in the hardened state include:

<   Grout strength.
<   Permeability of the grout.

GROUT MIX DESIGN FACTORS

The mix design components of grouts used in bonded post-tensioned construction are portland cement, mineral admixtures, chemical admixtures, aggregates, and water.

Use portland cement, Type I or Type II, that meets ASTM C150, Standard Specification for Portland Cement. Use Type II cement when a slower heat of hydration is desired. Blended cements (ASTM C595, Standard Specification for Blended Hydraulic Cements) may be used if they do not contain ground calcium carbonate, which will extend the setting time. This will allow this filler to rise with the bleed water, resulting in reduced pH and cementing qualities. Type K expansive cements (ASTM

C845, Standard Specification for Expansive Hydraulic Cement) can be used if special attention is paid to their interaction with admixtures and how the expansion properties may be influenced.

Mineral admixtures may be used to reduce the heat of hydration, increase long-term strength, decrease permeability, and control bleeding. The commonly used mineral admixtures are fly ash, ground granulated blast-furnace slag, and silica fume or microsilica. Silica fume and latex modifiers are added to reduce permeability for improved corrosion performance.

Fly ash (Class C and Class F) should meet ASTM C618, Standard Specification for Coal Fly Ash and Raw or Calcinated Natural Pozzolan for Use as a Mineral Admixture in Concrete. Use the same amount of fly ash as used in producing portland cement concrete, typically 10 to 25 percent by weight of the portland cement. Some fly ash may be susceptible to a reaction with admixtures, which may adversely affect setting time and expansion.

Ground granulated blast-furnace slag should meet ASTM C989, Standard Specification for Ground Granulated Blast-Furnace Slag for Use in Concrete and Mortars. The slag activity test classifies blast-furnace slags into three grades: Grades 80, 100, and 120. Only Grade 120 should be specified for use in grouts for post-tensioned concrete structures. The amount of slag used typically ranges from 30 to 55 percent by weight of the portland cement. The slag may delay strength development and extend setting times.

Silica fume is added to grouts to improve its corrosion-protection capability. Silica fume grouts have lower chloride permeability, lower porosity, and finer pore size distribution. They also have a higher compressive strength. An added benefit is that bleeding and segregation are significantly reduced. Silica fume grouts are thixotropic and have a much higher viscosity. Thixotropic grouts stiffen in a short time when at rest, but viscosity is lowered when the grout is mechanically agitated. A high-range water reducer can be used to obtain the desired fluidity (flowability). More high-range water reducer is needed as the silica fume content increases. Although the viscosity of silica fume grouts can increase very rapidly, relatively high fluidities can be maintained for several hours when the mix proportions are adjusted to achieve an initial low viscosity. Silica fume should meet ASTM C1240, Standard Specification for Silica Fume for Use as a Mineral Admixture in Hydraulic-Cement Concrete, Mortar, and Grout. The amount of silica fume used is typically 5 to 15 percent by weight of the portland cement.

Latex has been added to grouts to improve its corrosion performance. Grouts with latex have showed the least amount of shrinkage. However, the compressive strength is lowered. Latex and superplasticizers do not appear to be a good combination in grouts as bleeding and segregation become a problem. The equipment is also harder to clean. As a result, the use of latex modifiers has not been pursued any further.

Chemical admixtures are commonly added to grout mixtures to improve the corrosion performance of the grout, improve workability, stop bleeding, reduce shrinkage, control set time, entrain air, cause expansion, and aid in pumping. High-range water-reducing admixtures are added to improve

workability. Anti-bleed additives are used to reduce bleeding. The compatibility of all admixtures with the other components in the grout mix needs to be established through trial mixes. Admixtures are both the expanding and non-expanding type. They should not contain thiocyanate, nitrate, formate, chlorides, and sulphides.

Set-controlling and water-reducing admixtures are covered by ASTM C494, Standard Specification for Chemical Admixtures for Concrete. For normal water reducers (ASTM C494 Type A), typical dosage rates are 130 to 390 mL per 100 kg (2 to 6 oz per 100 lb) of cement. For high-range water reducers (also referred to as superplasticizers) (ASTM C494 Type F and Type G), dosage rates are up to 3000 mL per 100 kg (45 oz per 100 lb) of cement. Water reducers can be used to increase fluidity for a given water content or to maintain a desired fluidity while reducing the water content in the grout.

High-range water reducers can provide highly fluid grouts with water-cement ratios from 0.3 to 0.4. They have been used to provide up to a 20-percent water reduction and yet maintain adequate fluidity at reduced water-cement ratios in grouts with silica fume, fly ash, and polymer modifiers. The length of open time is controlled by the superplasticizer dosage rate. Styrene-butadiene latex modifiers, when used in conjunction with superplasticizers, produce fluid grouts with prolonged open times.

Air-entraining admixtures (ASTM C260, Standard Specification for Air-Entraining Admixtures for Concrete) are normally not used for bonded post-tensioned concrete construction. Historically, freeze-thaw damage has not been a problem.

Anti-bleeding admixtures are used to control the bleeding characteristics of the fresh grout. Although there is currently no specification for anti-bleeding admixtures, test method ASTM C940, Standard Test Method for Expansion and Bleeding of Freshly Mixed Grouts for Preplaced-Aggregate Concrete in the Laboratory, is an attempt to measure bleeding in grouts.
The use of an anti-bleeding admixture is important where strands are used in high vertical lifts or when bleed water may collect and form an air pocket. Their use should be permitted if the resulting grout mix satisfies the test requirements for setting time, strength, permeability, volume change, fluidity, bleed, and corrosion. When used in post-tensioning grouts, their performance should also meet the performance criteria for the modified ASTM C490 Test and the Pressure Bleed Test.

Several methods are available to control pressure-induced bleeding. Reducing the water-cement ratio of the grout has some effect on pressure-induced bleeding and, therefore, the water-cement ratio of the grout mixture should be less than 0.45. In addition to a low water-cement ratio, anti-bleed admixtures are also effective in reducing pressure-induced bleeding. Anti-bleeding admixtures thicken grouts by reducing its apparent fluidity or by making it thixotropic. The addition of styrene-butadiene latex in a relatively high dosage rate, 15 percent by weight of cement, also results in a significant reduction in pressure-induced bleeding. Other measures for improving the anti-bleed properties are the use of fine sand and silica fume. The addition of silica fume to the grout, in combination with a low water-cement ratio, results in a significant reduction in pressure-induced bleeding.

Expansion-causing admixtures are normally used to offset drying shrinkage by causing expansion while the grout is still in the plastic state. Two common gas-forming additives are aluminum powder and coke breeze. Although there is currently no specification for expansion-causing admixtures, test methods ASTM C937, Standard Specification for Grout Fluidizer for Preplaced-Aggregate Concrete, and ASTM C940 attempt to measure expansion. It is yet to be determined if gas-forming expansion admixtures improve the volume stability of the grout.

Expansive additives are added to the grout mixture in order to increase the volume of the grout and ensure the complete filling of the ducts without shrinkage. The U.S. Army Corps of Engineers has defined four categories of non-shrink grouts:[22]

< Gas-Liberating – These additives contain ingredients that generate or release hydrogen, oxygen, or nitrogen gases when they react. The expansion may continue until either the reactants are used up or the grout has hardened enough to resist further expansion.

< Metal-Oxidizing – These additives contain an oxidizable metal and an oxidation-promoting ingredient. The oxidation of the metal causes an increase in the volume of metal and the grout. The expansion may continue until the metal is completely oxidized or the metal is sealed off from its oxygen supply.

< Gypsum-Forming – With these additives, the non-shrink properties are due to the reaction of calcium sulfate hemihydrate (plaster of Paris) and water to form gypsum. The expansion may continue until all the calcium sulfate hemihydrate has been converted to gypsum or all the water has been used up.

< Expansive-Cement – The expansive agent normally used in expansive cement is aluminum powder, which reacts with the alkali in cement to form molecular hydrogen.

There are some problems with the use of expansive additives. For gas-forming additives, it is difficult to predict when the reaction will occur and it may take place before the grout has been placed in the duct. Expansive additives have been found to reduce compressive strengths and increase chloride ion permeability. For those grouts that expand in the plastic state, they may still be subject to normal drying shrinkage and a possible reduction in volume is possible. Recent research has shown that aluminum powder admixtures will provide an interconnected air-void system that will increase chloride permeability. Other gas-forming admixtures may behave similar to aluminum powder. Therefore, they should not be used in Class A or B grouts. [20]

Corrosion inhibitors have been used in conventionally reinforced concrete structures as a corrosion-control measure. There is currently no ASTM specification that covers inhibitors for grouts.

Aggregates normally have not been used in grouts for bonded post-tensioned concrete structures. Aggregates can be used in grouts to reduce permeability and improve volume stability (reduce drying shrinkage). If aggregates are used, the maximum size needs to be small enough (1 mm) to allow grout to

move through the duct and completely encapsulate the strands and should meet all other requirements in ASTM C33, Standard Specification for Concrete Aggregates.

Any potable water can be used to mix grouts. If a public water supply is not used, the water should have a chloride ion concentration of less than 500 ppm and contain no organics. If there is any doubt as to the suitability of a water source, testing should be done. The physical properties of the grout made with the actual water being used should not vary by more than ±5 percent from a mix made with potable water from a known source.

GROUT PROPERTIES

Performance specifications for the grout properties for grouts used in bonded post-tensioned concrete structures should include provisions for:

< Initial fluidity.
< Fluidity as a function of time and temperature (open time).
< Maximum amount of pressure-induced bleeding.
< Maximum chloride ion permeability.
< Minimum compressive strength.
< Rate of compressive strength gain.

The material specifications for the grout should consider:

< Type and amount of portland cement.
< Maximum water-cement ratio.
< Type and amount of anti-bleed additive (for pressure-induced bleeding).
< Type and amount of corrosion-protection additive.
< Type and amount of additive for initial fluidity requirement.
< Type and amount of additive for open-time requirement.

PERFORMANCE AND MIX DESIGN REQUIREMENTS

The Post-Tensioning Institute has identified four classes of grouts: Classes A, B, C, and D.[20] The choice of which grout to use is dependant on the severity of the anticipated exposure conditions (environment) that the structure will be in. Class A grouts are for a non-aggressive exposure. Class B grouts are for aggressive exposure. Class C grouts are prepackaged grouts suitable for aggressive exposure, but which can also be used in non-aggressive exposure. Class D grouts are specialized grouts for critical applications when properties and performance must be carefully controlled. Class A, B, and C grouts are applied under positive pressure grouting [pressure less than or equal to 1 MPa (145 lb/in^2)]. There are laboratory testing requirements for all classes except Class C, if the Class C grout has been demonstrated to already meet performance requirements. On-site testing of Class C grouts may still be performed.

The components of Class A, B, and C grouts include portland cement, Type I or Type II; potable water; and admixtures. Recommended mix proportions for Class A, B, and C grouts are given in table 3.[20] The allowable water-cement ratio for these grouts should be kept as low as possible, generally less than 0.44.

Table 3. Grout mix proportions for Classes A, B, and C.	
Water-Cementitious Material Ratio	0.35 to 0.45
Silica Fume	0- to 15-percent replacement by weight of portland cement
Fly Ash	0- to 25-percent replacement by weight of portland cement
Slag	0- to 55-percent replacement by weight of portland cement
Other Additives	Tests must show that there will be no adverse effects with respect to corrosion of the prestressing steel

For Class C grouts, the manufacturer is responsible for determining the mix proportions needed in order to meet the minimum performance requirements. The manufacturer is also responsible for performing and reporting the results of all required testing.

The components of Class D, Special Grouts, include portland cement, potable water, cementitious admixtures, and chemical admixtures. Class D grouts also need to meet all performance requirements for Class B and C grouts.

GROUT MATERIAL PROPERTIES TESTING

Laboratory testing is performed to evaluate the fluidity, bleeding characteristics, volume stability, strength, permeability, and set time of grout mixes prior to their use in a post-tensioned concrete structure. All laboratory mixing and testing should be done at the same temperature and humidity as will be expected at the structure site. However, the temperature and humidity should also be within acceptable limits.

The U.S. Army Corps of Engineers (COE) Specification CRD-C-611 is equivalent to ASTM C939. It has been found that some grouts passing the COE specification, but having efflux times of less than 30 s, may be susceptible to segregation. The flow table, ASTM C230, can be used to measure fluidity of thixotropic grouts. There is currently no correlation between the flow table and the flow cone measurements. In addition, no correlation has been established between the flow cone and flow table measurements and grout pumpability.

For non-thixotropic grouts, use the fluidity tests in ASTM C939, Standard Test Method for Flow of

Grout for Preplaced-Aggregate Concrete (Flow Cone Method). The efflux time immediately after thorough mixing should be between 11 and 20 s. After the grout has rested for 30 min without agitation, the efflux time should not be more than 30 s. Remix the grout for 30 s prior to the final flow measurement.

For thixotropic grouts, use a modified version of ASTM C939. Thixotropic grouts should not be tested by the flow cone method. In the modified C939 test, the flow cone is completely filled with grout and the efflux time is the time taken to fill a 1-L container placed under the cone orifice. The efflux time immediately after thorough mixing should be between 11 and 20 s. After the grout has rested for 30 min without agitation, the efflux time should not be more than 30 s. Remix the grout for 30 s prior to the final flow measurement.

Other tests for the pumpability of thixotropic grouts are the pressurized flow method that is described in Report No. FHWA-RD-92-095, *Performance of Grouts for Post-Tensioned Bridge Structures*.[23] In the pressurized flow method, an orifice plate is placed in the bottom cap of a cylindrical brass container. The time between the opening of the orifice until the first break in the continuous flow of grout is measured and compared with an acceptance criteria. For pressure-grouting operations, the initial flow and the flow 30 min after mixing the grout, as determined using the test method described in Report No. FHWA-RD-90-102, *Grouting Technology for Bonded Tendons in Post-Tensioned Bridge Structures*, have an acceptable range of 10 to 15 s.[22]

Two bleed tests are used to determine the bleeding characteristics of grout. They are the Wick Induced Bleed Test and the Pressure Bleed Test. The Wick Induced Bleed Test is a modification of ASTM C940 and should be performed for Class B, C, and D grouts. The maximum permissible bleed after 4 h is 0 percent. The Pressure Bleed Test requires the use of the Gelman Pressure Filtration Funnel to determine the bleeding characteristics of grouts used in pressure grouting for all four grout classes. For Class A, B, and C grouts, bleeding should be less than 2 percent. For Class D grouts, bleeding should be less than 1 percent. The grout should reabsorb water during the 24 h after mixing.

The volume change testing of grout should be performed according to ASTM C1090, Standard Test Method for Measuring Changes in the Height of Cylindrical Specimens From Hydraulic-Cement Grout. The total volume change of the grout should be between 0.0 and 0.3 percent at 28 days. The expansion and shrinkage tests described in Report No. FHWA-RD-92-095 may also be performed when specified.

Compressive strength of the grout is determined using ASTM C942, Standard Test Method for Compressive Strength of Grouts for Preplaced-Aggregate Concrete in the Laboratory. The compressive strength of restrained cubes should be at least 21 MPa (3000 lb/in^2) at 7 days and 35 MPa (5000 lb/in^2) at 28 days.

Permeability of the grout as determined by ASTM C1202, Standard Test Method for Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration, should be less than 3000 coulombs after 6 h when tested at 30V and when testing the grout at 28 days.

The setting time as determined by ASTM C953, Standard Test Method for Time of Setting of Grouts for Preplaced-Aggregate Concrete in the Laboratory, should be greater than 3 h and less than 12 h.

The acid-soluble chloride ion content in the grout as determined by ASTM C1152, Standard Test Method for Acid-Soluble Chloride in Mortar and Concrete, should be less than or equal to 0.08 percent by weight of portland cement. The acid-soluble chloride ion content in dry mix ingredients as determined by ASTM C114, Standard Test Methods for Chemical Analysis of Hydraulic Cement, should be less than or equal to 0.08 percent. One should not intentionally add chlorides to the grout mix.

Corrosion tests may be required by either the engineer or the contract documents. When a corrosion inhibitor is used, the Accelerated Corrosion Test (ACT) procedure described in Report No. FHWA-RD-91-092, *Improved Grouts for Bonded Tendons in Post-Tensioned Bridge Structures*, should be performed.

SUITABILITY AND ACCEPTANCE TESTING

Suitability testing assesses the materials prior to their use. Acceptance and production testing evaluates the actual materials being used and whether or not they meet performance requirements. Field testing uses the same testing procedures as in laboratory testing. A production test frequency recommended by the Post Tensioning Institute and the minimum testing requirements for suitability and acceptance testing proposed by the Concrete Society in the United Kingdom are summarized in table 4.[19-20] The Post Tensioning Institute recommends which production tests should be performed based on the class of the grout:

< Class A grouts – Bleed, strength, and fluidity tests.
< Classes B and D, and thixotropic grouts – Bleed, volume change, strength, and fluidity tests.
< Class C grouts – Fluidity.

		Table 4. Minimum testing requirements.	
Test	**Grout Property**	**Concrete Society, United Kingdom**	**Post Tensioning In**
Suitability	**Fluidity**	**Sampled immediately after mixing, one test. Common grout: after estimated time to grout duct or minimum of 30 min. Special grout: after 90 min. Two tests averaged (both cases).**	
	Bleed Volume Change Strength	**Each sampled immediately after mixing, three tests averaged.**	
Production Acceptance	**Bleed**	**Two tests per day, one sample taken from grout after flow through duct, at the end anchorage outlet, and the other from mixer.**	**Two tests per 1.5 m³ (2 One sample taken at th duct outlet when the g satisfactory fluidity.**
	Volume Change	**Two tests per day, one sample taken from grout after flow through duct, at the end anchorage outlet, and the other from mixer.**	**One test per day or pe grout.**
	Strength	**Two tests per day, one sample taken from grout after flow through duct, at the end anchorage outlet, and the other from mixer.**	**A minimum of one str yd³) of grout.**
	Fluidity	**One test immediately after mixing, one sample from mixer. One test after flow through duct, one sample from each anchorage outlet. One test on completion of the grout job, one sample from mixer.**	**A minimum of one test operations or per 1.5 m**

TEST METHODS FOR BONDED POST-TENSIONED CONCRETE

Prestressed concrete structures – pretensioned and post-tensioned – rely on the integrity of high-strength bars and strands for their primary safe load-carrying capability. The deterioration and ultimate failure of one or more of these high-strength bars and strands could lead to the catastrophic failure of part or all of a structure. In post-tensioned structures, the prestressing tendons are normally enclosed in ducts. It is desirable to have these tendons fully grouted as a corrosion-protection measure. If they are not fully grouted (i.e., voids are present), moisture and aggressive chemicals could penetrate the duct, resulting in corrosion of the prestressing steel.

The test methods should be able to demonstrate that the structure was built according to specifications and the tendons are fully grouted and no voids are present. Testing should preferably be done before acceptance to ensure that a durable structure was constructed. Testing can be done later in the service life of the structure to evaluate its long-term performance.

TESTS BEFORE AND DURING GROUTING

Tests performed during the initial stage of the construction of a bonded post-tensioned concrete structure have the most potential for preventing the formation of voids. Perform a pressure test to evaluate the sealing of the duct system before grouting. Perform a grout stiffness test during grouting. In this test, apply pressure to the grout in the duct while it is still in a plastic state and measure its response. This test is based on the principle that any gas in the grout will compress and the grout will not. Its stiffness, a "spongy" response, could then indicate the presence of voids (trapped air). If inadequate grouting is detected (voids), stop grouting and flush the ducts or make adjustments to the grouting operation and then continue. The "Spongeometer" Testing Device has been able to detect voids in full-scale ducts 60 m (197 ft) from where the measurement was taken.[19] However, entrained air could influence the results and there still remains some uncertainty regarding its accuracy and reliability. The equipment is still under development and should not yet be used for acceptance testing. A pressure test done after concreting could be used to determine if any major damage has occurred during concreting and to evaluate the duct in its final condition. This more closely approximates actual service conditions.

TESTS FOR VOIDS IN DUCT – HARDENED GROUT

Several different methods of evaluating the extent of grouting in post-tensioning ducts have been developed over the years.[24] Although it is more difficult to correct defects after the grout has hardened, an evaluation at this time can reveal any future potential corrosion problems due to the presence of voids in the ducts. At this point, the bridge owner can decide if corrective measures are needed.

The pressure method is used to check for small voids in locations not accessible by other methods. A pathway to the void is needed – a vent pipe, tube, or drilled hole. However, the likely location of the voids needs to be known. Holes should not routinely be drilled as a method for searching for voids. The pressure method requires specialized equipment. Although the hole can be used to re-grout any

detected voids, vacuum-grouting is needed to effectively fill these small voids.

Radiography uses gamma rays or x-rays to detect voids in the ducts. This test shows changes in density that are easy to see and understand. It can be used in concrete sections up to 800 mm (31.5 in) thick. Most equipment used for radiography requires the development of film. New versions now provide real-time images. The equipment is expensive. Radiography is also time-consuming and requires special safety considerations due to the high-energy output. It is not practical for very thick sections that contain heavy reinforcement and overlapping ducts.

Impulse radar is cheaper and safer. Access from only one side of the section being evaluated is needed. It can be used on thicker sections. The results, however, are less clear and some interpretation is required. It is also currently not precise enough to detect voids inside post-tensioning ducts.

The drilling and monitoring of the air flow method is a destructive method. There is a chance that the prestressing steel may be damaged or cut. The drilled hole, even after being filled, becomes an avenue for future corrosion. Since the holes are drilled at suspected locations, one needs to know where to test.

The boroscope method is also a destructive method. A fiber-optic cable is used to view the inside of the post-tensioning duct. The test is performed at the locations of suspected voids. It typically uses a camera to record the images. It has the same disadvantages as the drilling and monitoring of the air flow method.

A recent development in the nondestructive evaluation of concrete structures that has good potential for application to concrete bridge structures is the impact-echo (IE) method.[25] It has been successfully used to detect honeycombing and delaminations in the laboratory and in the field.

A mechanical impactor is applied to the concrete surface that introduces a transient stress pulse into the concrete member. The stress pulse propagates into the member along spherical wavefronts. Dilatational (or compression) waves (P-waves) are parallel to the direction of propagation. Distortional (or shear) waves (S-waves) are perpendicular to the direction of propagation. There are also surface waves, referred to as Rayleigh waves (R-waves), that travel along the surface away from the impact point. The amplitude of these waves decreases exponentially with depth into the member.

The P- and S-waves travel through the material and are reflected back by internal or external interfaces (i.e., internal defects or external boundaries). When the reflected waves arrive back at the surface point where initial impact was generated, the resulting surface particle displacement is converted into an electrical voltage by a receiving transducer. When this transducer is placed close to the impact point, P-wave arrivals dominate the displacement. In order to evaluate the displacement waveform, the travel time (Ät) between pulse initiation and the arrival of the first reflected P-wave is determined. When the P-wave propagation velocity (C_1) is known, the distance to the reflecting interface (d) can be calculated.

The time-domain data interpretation is both time-consuming and tedious. A more effective approach is to perform a frequency analysis on the displacement waveform. This involves the construction of the frequency spectrum time-domain data versus amplitude plot. As the stress pulse propagates between the top surface and the internal or boundary interfaces, it produces a characteristic displacement each time it arrives at the top surface. This displacement is periodic and can be recorded as a waveform with a period (t), a travel path (2d), and a frequency (f_0). The frequency is given by the following:

$$f_0 = \frac{C_1}{2d}$$

With a known velocity and frequency, either the slab thickness or the distance to an internal interface (defect) can be calculated using:

$$d = \frac{C_1}{2f_0}$$

The P-wave propagation velocity is measured experimentally using a simple calibration method. The Fast Fourier Transformation (FFT) algorithm is then used to obtain the frequency content of the recorded waveform.

An impact-echo method consists of three major elements – an impact source, a displacement transducer, and a data-acquisition system. Loading durations range from 10 to 55 ms and depend on the size of the steel ball. The impact source is a small steel ball dropped on the concrete surface or applied using a spring-loaded impactor. The transducer is normally a displacement sensor that is free of strong characteristic resonances, especially in the frequency range used in impact-echo testing. The data-acquisition system should be capable of performing waveform analysis in both the time and frequency domains.

The impact duration affects both the propagating waves wavelength and the size of the flaws or discontinuities that can be detected. Only flaws with lateral dimensions greater than the wavelength can be detected. Therefore, it is important to apply an impact load of an appropriate duration in order to detect the desired flaw sizes. While shorter contact times can detect smaller flaws, the ability of a stress wave to penetrate concrete is reduced as the contact time decreases.

Studies have shown that with the impact-echo method, it is possible to detect and examine the extent of voids in post-tensioning ducts. It may also detect honeycombing, delaminations, voids, and low-quality concrete. The studies performed included laboratory and field evaluations.

A series of laboratory tests were performed to determine the effects of various parameters on the receiving signals: planar flaws, grouted and ungrouted metal ducts, grouted and ungrouted polyvinyl

chloride ducts, honeycombing, and various steel reinforcing bar sizes. Concrete samples with known defects were fabricated and tested.

A field evaluation of the grouting status of the ducts in an existing post-tensioned concrete box-girder bridge in Milwaukee, Wisconsin, was performed.[24] The bridge is a two-span continuous structure with 24.38-m (80-ft) spans. Each girder web contains two post-tensioning tendons. The upper tendon consists of 31 seven-wire strands and the lower tendon consists of 19 seven-wire strands. The field results agreed with the laboratory experimental and numerical studies.

In another field study, 14 post-tensioning ducts were tested using the impact-echo method.[26] The location of the structure was not reported. Of these ducts, 11 were found to be fully grouted and 3 were found to contain voids. The results were verified by opening and visually inspecting the ducts; the results were accurate. The impact-echo method was able to determine the extent of the grouting, even though test complications were present – cracking along the centerline of the duct, the presence of multiple ducts in close proximity to each other, and a varying concrete cover thickness.

CORROSION INHIBITORS

Corrosion inhibitors are chemical admixtures added to portland cement concrete mixes during batching – usually in very small concentrations – as a corrosion-protection measure. Corrosion inhibitors are a viable corrosion-protection measure for the long-term durability of both conventionally reinforced and prestressed concrete bridge structures. When used as part of a multiple-strategy corrosion-protection system, they are promising materials to delay the onset of reinforcing and prestressing steel corrosion.

Inhibitors are often used in combination with low-permeability concrete and usually they have the effect of increasing the threshold chloride concentration needed to initiate corrosion. Inhibitors play an important role in protecting uncoated high-strength steel strands used in prestressed concrete bridge members and stays used in cable-stayed bridges. They are also used in cementitious grouts for filling the ducts of bonded post-tensioned bridges. Inhibitors may also reduce the subsequent corrosion rate after the initiation of corrosion, which ultimately leads to less corrosion-induced concrete deterioration.

There are three major concerns regarding the use of corrosion inhibitors. The first one is the long-term stability and performance of the inhibitor. The second is the inhibitor's effect on corrosion propagation after corrosion initiation. The third is the inhibitor's effect on the concrete's physical properties over the service life of the structure.

In order for a corrosion inhibitor to be an effective long-term corrosion-protection measure, it needs to be able to maintain long-term stability. It should be chemically intact and physically present (not leaching or evaporating) to retain its effectiveness.

Inhibitors may have an effect on the corrosion process after corrosion initiation. An insufficient dosage will have a negative impact on corrosion progression. Some inhibitors will have an effect on chloride transport and can reduce the rate of chloride ion migration.

Inhibitors should not have any negative effects on the concrete properties. The use of an inhibitor should not cause an undue increase in the amount of any concrete cracking. Some inhibitors decrease the concrete resistivity, which has a tendency to increase the corrosion rate. This effect is offset due to the inhibition provided by a suitable corrosion inhibitor.

Corrosion inhibitors are either inorganic or organic and, in general, are classified based on their protection mechanism. They can protect by affecting the anodic reaction, the cathodic reaction, or both reactions (mixed). An active type of inhibitor (anodic) facilitates the formation of an oxide film on the surface of the steel reinforcing bars. Passive systems protect by reducing the rate of chloride ion migration. Calcium nitrite is an inorganic inhibitor. It protects the steel reinforcing bars through oxidation-reduction reactions at the steel surface. Organic inhibitors consist primarily of amines and esters. They form a protective film on the surface of steel reinforcing bars and sometimes delay the arrival of chloride ions at the steel reinforcing bars.

There are four major commercially available corrosion inhibitors:

< DCI (Darex Corrosion Inhibitor) and DCI-S.
< Rheocrete 222 and Rheocrete 222$^+$.
< Armatec 2000, Ferrogard 901, and MCI 2000.
< Catexol 1000 Cl.

DCI (Darex Corrosion Inhibitor) and DCI-S[27]

DCI (calcium nitrite) is an inorganic corrosion-inhibiting concrete admixture. It has been commercially available since 1978 and up to now it was only marketed by W.R. Grace and Company. Recently, Master Builders, Inc. also started marketing a calcium nitrite inhibitor similar to DCI. DCI has approximately a 30-percent concentration of calcium nitrite. DCI-S is the same as DCI, but also includes a set retarder. When silica fume is used in combination with calcium nitrite, the silica fume reduces the rate of chloride ion penetration (migration) and the calcium nitrite increases the threshold for corrosion initiation. Calcium nitrite reduces the resistivity values of concrete, but the addition of silica fume offsets this reduction and, in fact, there is a net gain in resistivity.

DCI is an anodic type of inhibitor (active) and functions by passivating the anode. The oxidation-reduction protection mechanism is electro-chemical in nature and, as a result, its effectiveness is understandable and quantitatively measurable. The chloride and nitrite ions are involved in simultaneous, complicated, competing reactions at the surface of the steel reinforcing bars: corrosion vs. passivation. Chloride ions accelerate corrosion through the formation of Fe^{++} ions. Nitrite ions inhibit corrosion through the formation of passive iron oxide (Fe_2O_3).

Calcium nitrite inhibits or interferes with the removal of Fe^{++} ions through the following reaction:

$$2Fe^{++} + 2OH^- + 2NO_2^- \ 6 \ 2NO8 + Fe_2O_3 + H_2O$$
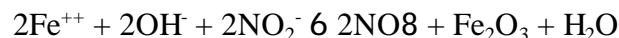
The relative rates of the two processes (corrosion and passivation) are unknown.

This prevents further corrosion. However, nitrite ions are reduced to nitric oxide gas. This reduces the nitrite concentration in the concrete in the immediate vicinity of the reinforcing steel. As the chloride ion content increases, the ability to maintain passivity is reduced (with a constant amount of nitrite).

DCI appears to be primarily effective because it does not allow the development of large electrical potentials between areas that otherwise would be anodic and cathodic – adjacent steel areas in the top mat of reinforcing steel bars and between top and bottom mats of reinforcing steel bars in a bridge deck. Electrical potentials are maintained at similar values that are also at or near the passive range. Large electrical potential between otherwise anodic and cathodic areas allows the flow of a corrosion current and subsequent dissolution of iron in the top mat.

The relative concentrations of nitrite and chloride ions are important. For a low Cl^-/NO_2^- ratio (less than

1), the potentially anodic rebar is passivated. For a high Cl^-/NO_2^- ratio, the eventual corrosion of the steel reinforcing bar is certain.

In general, the corrosion rate increases as the Cl^-/NO_2^- ratio increases. A long-term study on the effectiveness of calcium nitrite as a corrosion inhibitor was performed by the Federal Highway Administration.[28] Earlier results showed that with a Cl^-/NO_2^- ratio up to 1.25, the corrosion rate was reduced by at least an order of magnitude. In other words, it would take more than 10 years to consume an equal amount of iron as in 1 year for a slab with an equal chloride ion content, uncoated steel, and no calcium nitrite.

Subsequent data showed that in order for calcium nitrite to be an effective corrosion inhibitor, the Cl^- /NO_2^- ratio should be less than or equal to 1.1. For ratios between 0.3 and 1.1, the corrosion rate is reduced by a factor of approximately 10. When the Cl^-/NO_2^- ratio was above 1.1, the corrosion rates were reduced by a factor of about 2 and test slabs had cracks, major rust stains, and hollow and spalled areas.

The amount of calcium nitrite that should be added to the concrete during mixing depends on expected chloride ion concentration at the rebar for the desired corrosion-free service life. There is no significant benefit in using calcium nitrite as an inhibitor for Cl^-/NO_2^- ratios greater than 1. Therefore, the recommended dosage is that which will yield a Cl^-/NO_2^- ratio at the rebar level that is less than or equal to 1 during the service life of the structure. This conclusion is based on research where the chloride ions were mixed into the fresh concrete along with DCI, which is not the normal practice in the field. Therefore, the inhibitor has a better chance of performing under normal circumstances when chloride ions diffuse through the concrete. With time, the chloride ion level is the expected accumulative content at the surface of the steel reinforcing bars due to deicer applications or marine exposure. Long-term stability is an issue as the inhibitor is not needed until a sufficient amount of chloride ions reach the steel and corrosion is initiated. Recommended dosage rates for different expected chloride ion contents for a corrosion-free service life are shown in table 5.[29]

Table 5. Calcium nitrite dosage rates.	
Calcium Nitrite **[L/m³ (gal/yd³)]**	**Chloride Ion Content** **[kg/m³ (lb/yd³)]**
10 (2)	3.6 (6.0)
15 (3)	5.9 (9.9)
20 (4)	7.7 (13.0)
25 (5)	8.9 (15.0)
30 (6)	9.5 (16.0)

Rheocrete 222 and Rheocrete 222[+(27)]

Rheocrete 222[+] is an organic corrosion-inhibiting concrete admixture. It is manufactured and marketed by Master Builders, Inc. It is an improved formulation of Rheocrete 222 and contains amines and esters in a water medium. Rheocrete 222 is a combination inhibitor, both anodic and cathodic (passive-active mixed type). It protects the steel reinforcing bars in two ways: First, it forms a corrosion-resistant organic film that is adsorbed on the steel surface. Secondly, it also coats the pores of the concrete matrix, which slows the migration of chloride ions.

Both Rheocrete 222 and Rheocrete 222[+] have not been available for a very long time, compared to calcium nitrite. As a result, there is less published data on their performance. Rheocrete 222 has been used in a number of parking garages, and in a few bridges and marine structures. The recommended dosage rate is 5 L/m^3 (1 gal/yd^3). It is typically added to the concrete batch water. The dosage rate is not adjusted for the anticipated corrosiveness of the expected service environment.

Armatec 2000, Ferrogard 901, and MCI 2000[(27,30)]

These inhibitors are a blend of surfactants and amine salts [specifically dimethylethanolamine (DMEA), also referred to as alkanolamines or amino-alcohols (AMA)] in a water medium. Armatec 2000 and MCI 2000 are manufactured and marketed by Cortec Corporation. Armatec is reported to have a slightly different formulation from MCI. Ferrogard 901 is manufactured and marketed by Sika Corporation.

According to the manufacturers, they protect the steel reinforcing bars by forming a continuous mono-molecular film on the steel surface and they cover both the anodic and cathodic sites (mixed type). This film consists of an adsorbed layer of amino-alcohol that leads to the formation of insoluble iron oxide complexes. These stabilize the oxide surface and inhibit further corrosion. The film is typically 10^{-8} m thick and also acts as a barrier to aggressive ions migrating through the concrete. It has been reported that incoming chloride ions on the steel surface can be displaced by amino alcohols. The recommended dosage rate for Ferrogard 901 is 10 L/m^3 (2 gal/yd^3). The dosage is not adjusted for the corrosiveness of the anticipated service environment.

Catexol 1000 Cl[(30)]

This inhibitor is a water-based formulation of amine derivatives. Catexol 1000 Cl is manufactured and marketed by the Axim Concrete Technologies, Inc. The manufacture claims that protection is provided by formation of a protective barrier. This barrier reportedly stabilizes the passive iron oxide layer. This inhibitor appears to exhibit the characteristics of an organic film-forming inhibitor, as well as those of a nitrite-based inhibitor.

FIELD PERFORMANCE OF EPOXY-COATED REINFORCING STEEL

The primary cause of the deterioration of reinforced concrete structures is the corrosion of steel reinforcing bars due to chlorides. Epoxy-coated reinforcing steel (ECR) was developed and implemented in the mid-1970's to minimize concrete deterioration caused by corrosion of the reinforcing steel and to extend the useful life of highway structures. The epoxy coating is a barrier system intended to prevent moisture and chlorides from reaching the surface of the reinforcing steel and to electrically insulate the steel to minimize the flow of corrosion current.

Epoxy coatings (sometimes referred to as powders or fusion-bonded coatings) are 100-percent solid, dry powders. These dry epoxy powders are electrostatically sprayed over cleaned and preheated steel reinforcing bars. The coatings achieve their toughness and adhesion to the steel substrate as a result of a chemical reaction initiated by heat. These epoxy powders are thermosetting materials and their physical properties do not change readily with changes in temperature.

The coating process consists of several steps:[31]

1. Steel reinforcing bars between 6.0 and 18.3 m (20 and 60 ft) in length and of various diameters move along powered rollers at speeds of 6 m/min to more than 15 m/min (20 to 50 ft/min). Typically, eight bars move parallel to each other on a set of rollers.

2. The bars are blast-cleaned with grit or shot in a grit-blasting booth to a near-white blast as specified in the Society for Protective Coatings Specification SSPC-SP-10.

3. In some plants, the bars pass through a pretreatment application unit where a pretreatment solution is applied, the excess is removed, and the bars are dried prior to leaving the unit. To date, there is not enough data to verify the usefulness of this process.

4. The bars are then heated very rapidly (within 1 to 3 s) to 246EC (475EF) as they pass through an induction coil.

5. As the bars enter the spray booth, epoxy powder is electrostatically sprayed onto the bars by stationary automatic spray guns. When the powder hits the hot bars, it melts and becomes fluid to form a smooth coating.

6. After the bars leave the powder-coating booth, the ambient air begins to cool them. The coating continues to cure due to the heat retained within the bars. The powder coating is formulated to be tough enough not to be marked by the rollers while it is curing. The line speed is adjusted so that the coating has enough time to fully cure.

7. The bars are then either sprayed with cold water or completely immersed in water to cool the bars so that they can be handled.

8.	When the cooled bars reach the end of the line, they are automatically lifted off of the rollers and stacked.

Numerous studies on the corrosion performance of ECR have been conducted. The initial study was funded by the Federal Highway Administration (FHWA) and performed by the National Bureau of Standards (now called the National Institute of Standards and Technology). The results were contained in the 1974 FHWA Report, *Non-Metallic Coatings for Concrete Reinforcing Bars* (Report No. FHWA-RD-74-018). The optimum epoxy coating thickness was found to be 0.18 ±0.05 mm when considering corrosion protection, bond strength, creep characteristics, and flexibility.[32] The coating process was adapted from the method used for epoxy coating small-diameter pipes used by utility companies and the petroleum industry. Epoxy-coated reinforcing bars for a bridge deck were first used in 1973 in Pennsylvania. The bars were coated in a pipe-coating facility. The main problems influencing the corrosion performance at the time were considered to be damage to the coating during transportation and handling, and the process of bending the bars, which cracks the coating.

In the 1980's, FHWA initiated outdoor exposure tests. The purpose was to determine the benefits of coating both mats of reinforcing steel in bridge decks. The study intentionally used bars with excessive holidays, damage, and bare areas. Three different combinations for the top and bottom mats of reinforcing steel were investigated: black-black, epoxy-epoxy, and epoxy-black. Even poor-quality epoxy-coated reinforcing steel was found to have reduced corrosion by 11.5 times when only the top mat was coated and by 41 times when both mats were coated. The reduced corrosion rate was attributed to the high electrical resistance of the coating and a reduced cathodic area.[33]

As a result of observed corrosion in the Florida Keys bridges, the Florida Department of Transportation (DOT) initiated several studies to determine the extent and cause of the corrosion. The substructure showed signs of corrosion in only 5 to 7 years after construction in the area 0.6 to 2.4 m (2 to 8 ft) above the mean high-water mark (splash zone). The corrosion affected both straight and bent bars. The early studies showed that corrosion was aggravated by bending, coating defects, and corrosion macrocells. Coating disbondment could be due to exposure to saltwater, a mild level of cathodic polarization, and also to the anodic conditions present while the bar was corroding. Coating disbondment could also occur in chloride-free concrete. In an investigation of 30 bridges, most did not show visible signs of corrosion. The coating thicknesses were generally within the specification limits in effect at the time of construction. The median amount of coating damage was found to be 0.4 percent of the bar surface. There was a reduction in adhesion in the extracted coated steel specimens from most of the bridges compared to the coated steel at the time of construction. No evidence of rebar corrosion was found except for the Keys bridges. It appears that a sequence of events may have led to the premature corrosion in the Keys bridges.

PERFORMANCE OF EPOXY-COATED REBARS IN BRIDGE DECKS

In response to reports of poor performance of ECR, most notably in the substructure units in the Florida Keys, FHWA recommended that States evaluate the performance of ECR in existing bridge decks. This is because FHWA supported the use of ECR in bridge decks alone; however, due to its

good performance, States started to use ECR in reinforced concrete bridge substructures. As a result, several States initiated investigations and prepared reports documenting their findings and results.

The FHWA report, *Performance of Epoxy-Coated Rebars in Bridge Decks* (Report No. FHWA-RD-96-092), summarizes the results of those investigations, as well as others that have been performed by highway agencies in the United States and Canada, by academia, and by the Canadian Strategic Highway Research Program (C-SHRP).[34] A total of 92 bridge decks, 2 bridge barrier walls, and 1 noise barrier wall located in California, Indiana, Kansas, Michigan, Minnesota, New York, Ohio, Pennsylvania, Virginia, West Virginia, and Wisconsin, and the provinces of Alberta, Nova Scotia, and Ontario were evaluated. At the time of the investigations, the ECR had been in service for 3 to 20 years. For the majority of the bridges, the ECR had been in service for about 10 years.

The investigations typically included field and laboratory evaluation phases. The field evaluation phases consisted of some or all of the following:

< Visual examination of the deck concrete for cracking, spalling, and patches.
< Chain drag to locate areas of delamination.
< Use of a pachometer to determine the amount of concrete cover and to locate the top mat of reinforcing steel for concrete coring.
< Drilling for concrete powder samples for chloride content.
< Concrete coring to evaluate the quality of the concrete and for chloride content.
< Overall deck condition ratings.
< Half-cell potentials.
< Resistivity readings.
< Three-electrode linear polarization resistances to determine the rate of corrosion.

The laboratory evaluation phases consisted of some or all of the following:

< Visual examination of the concrete in the extracted cores.
< Measurement of concrete cover over the ECR in the extracted cores.
< Evaluation of the extracted ECR segments.
< Measurement of the epoxy-coating thickness on the extracted ECR segments.
< Determination of total or water-soluble chloride ion content in the concrete using the extracted cores or the concrete powder samples.
< Permeability of the concrete in the extracted cores.
< Determination of pH in the concrete adjacent to the ECR in the extracted cores.
< Compressive strength of the concrete in the extracted cores.
< Unit weight of concrete using the extracted cores.

The summary of the findings and discussion are based on the results of field evaluations of the structures and the laboratory evaluations of concrete cores taken from the various structures as documented in reports by the various departments of transportation. A total of 92 bridge decks, 2 bridge barrier walls (parapets), and 1 noise barrier wall were evaluated in the field for cracking, delaminations, and spalls.

Overall, the structures were generally found to be in good condition. Concrete deterioration was generally in isolated areas and often not related to corrosion of the ECR.

The extent of deck cracking ranged from very little or none, to extensive. Cracking, when present, was generally transverse in nature. Deck cracking was not thought to be the result of any corrosion of ECR. The cracking in the bridge barrier walls consisted of scattered pattern cracking with some vertical cracks. The noise barrier wall consists of precast concrete panels and the panels that are closest to the roadway surface were cracked the most and exhibited rust staining and spalling.

Very few spalls or delaminations were found in the bridge decks. Delaminations were detected in only 10 of the bridge decks. Approximately half of these delaminations were small (0.1 m^2 (1 ft^2) in size). The others varied from 0.3 m^2 (3 ft^2) to approximately 2.8 m^2 (30 ft^2) in size. Several other detected delaminations were associated with expansion devices (uncoated metal) and were not due to any corrosion of ECR.

The depth of concrete cover over the top rebar was measured in each of the cores. Average concrete cover was generally found to be adequate, at least 51 mm (2 in). However, some instances of inadequate concrete cover were found.

Most investigators determined the total chloride (acid-soluble) content at the rebar level or chloride profiles. The concrete samples were either obtained from the concrete cores or from holes drilled into the concrete. In most cases the average chloride concentrations at the rebar level were at or above the threshold level to initiate corrosion in black steel.

The total chloride concentration at the rebar level was determined in 40 bridge decks. The average chloride concentration of all these decks was 2.2 kg/m^3 (3.7 lb/yd^3). The chloride concentration was greater than or equal to 0.6 kg/m^3 (1.0 lb/yd^3) in 33 (83 percent) of these decks and was greater than or equal to 1.2 kg/m^3 (2.0 lb/yd^3) in 24 (60 percent) of these decks. In addition, the chloride concentration was greater than or equal to 3.0 kg/m^3 (5.0 lb/yd^3) in 11 (28 percent) decks, with the highest concentration being 6.8 kg/m^3 (11.5 lb/yd^3).

The water-soluble chloride concentration at the rebar level was determined in 16 other bridge decks. The average chloride concentration of all these decks was 0.7 kg/m^3 (1.1 lb/yd^3). The chloride concentration was greater than or equal to 0.6 kg/m^3 (1.0 lb/yd^3) in five (31 percent) of these decks and was greater than or equal to 1.2 kg/m^3 (2.0 lb/yd^3) in two (13 percent) of these decks. None of these decks had chloride concentrations greater than 3.0 kg/m^3 (5.0 lb/yd^3), with the highest concentration being 2.6 kg/m^3 (4.3 lb/yd^3).

Some of the ECR segments extracted from the cores were examined for visual defects in the coating (holidays), thickness of epoxy coating, and the blast profile on selected bars. Most, if not all, of the segments that were examined contained holidays or bare areas. The thickness of the coatings was generally within the limits specified at the time of construction. In most of the instances when the coating thickness did not meet specifications, it exceeded the upper limit. The blast profiles that were evaluated

were found to have met applicable specifications.

The same ECR segments extracted from the cores were examined to determine the condition of the steel surface under the coating. Approximately 212 different ECR segments were examined. This total does not include ECR segments evaluated in the C-SHRP study from the five bridge decks and one barrier wall that were previously evaluated by others. It also does not include ECR segments from Pennsylvania as that report did not indicate how many segments were examined. For the majority of ECR segments, no corrosion was present.

Approximately 202 ECR segments were extracted from bridge decks. Out of these segments, 162 (81 percent) did not have any corrosion present. For some of the remaining segments that exhibited evidence of corrosion, the corrosion may have been present at the time of construction since chloride contents at the time of the evaluation were below the initiation threshold. Only four ECR segments (2 percent) were reported as having experienced significant corrosion. The areas of corrosion were typically at locations of visible holidays or bare areas. The more heavily corroded ECR segments were also from locations of relatively shallow concrete cover with high chloride concentrations.

Ten ECR segments were extracted from the barrier and noise walls. Out of these segments, eight (80 percent) did not have any corrosion present. Only one ECR segment (10 percent) was reported as having experienced significant corrosion. The areas of corrosion were typically at locations of visible holidays or bare areas. The more heavily corroded ECR segment was also from a location of very shallow concrete cover, highly permeable concrete, and with high chloride concentration.

Some ECR segments extracted from the cores were also examined for any coating disbondment. The extent of coating disbondment varied and was found in both corroded and non-corroded areas. Visible holidays were generally present on ECR segments that experienced coating disbondment. In most cases, the coating was generally still bonded to the steel surface.
California reported coating disbondment on 12 ECR segments (out of 32 total) in both corroded and non-corroded areas. Except for one segment, visible holidays were present on all ECR segments that experienced coating disbondment. The extent of coating disbondment varied from 3 to 100 percent of the rebar surface, with six segments having coating disbondment of more than 75 percent of its surface. Indiana reported that no ECR segments showed any signs of debondment of the epoxy coating. The coatings were difficult to strip with a knife. Some segments were mechanically stripped of their coating in order to examine the underside of the film. Michigan reported that the epoxy coatings on ECR segments extracted from the experimental decks and with moist concrete were easily removed by hand with the use of a fingernail. Virginia reported that the epoxy coatings remained tightly bonded to the steel and could only be removed with a knife.

The results of two separate investigations done in Ontario were reported. In the first investigation, two barrier walls were evaluated. None of the ECR segments in one of the barrier walls showed evidence of any coating disbondment. There was evidence of isolated locations in the second wall with poor bonding between the epoxy coating and the ribs of the rebars where the coating could be removed with a knife. However, the coating on the body of the bar could not be removed by a knife after scoring a

cross into the coating.

In a second investigation, ECR segments were extracted from 12 bridges in exposed concrete components – barrier walls, end dams, sidewalks, and decks built without waterproofing. The extracted ECR segments were tested for adhesion of the epoxy coating to the steel surface. Of the ECR segments extracted from structures built in 1979 and 1980, 27 percent had a well-adhered coating that could not be lifted from the steel substrate. Of the ECR segments extracted from structures built between 1982 and 1985, 60 percent had a well-adhered coating. Of the ECR segments extracted from structures built in 1990, 88 percent had a well-adhered coating. It appears that adhesion of the epoxy coating decreases with time as ECR segments extracted from bridges with the longest service life exhibited the most adhesion loss.

In the C-SHRP study, the extent of coating disbondment was determined using the dry knife adhesion test. Of the 44 tests performed on ECR segments from the 19 structures, 54 percent had a very well-bonded coating, 14 percent had a coating that was somewhat easy to remove, and 32 percent had a coating that was easy to remove or totally disbonded. The coatings on slightly more than half of the ECR segments still had good adhesion.

The following conclusions are based on the results and findings from these evaluations of the performance of ECR in bridge decks, bridge barrier walls (parapets), and a noise barrier wall:

< The overall condition of the bridge decks was considered to be good. Even though deck cracking was prevalent, it did not appear to be corrosion-related. Very few of the decks had any delaminations and/or spalls. Most of the delamination was not associated with the ECR. The maximum extent of delamination reported was less than 1 percent of the deck area. However, the actual extent of delamination was not reported.

< A bridge in West Virginia had a total of approximately 3.7 m^2 (40 ft^2) of delaminated area out of a total deck area of 1653.6 m^2 (17,800 ft^2), approximately 0.25 percent of the deck area, after 19 years of service life. The largest of these delaminations was centered on a construction joint and was most likely not corrosion-related. Chloride contents are not available for this deck and the report does not indicate whether the delaminations were corrosion-induced. The State of West Virginia indicated in their report that based on their previous experience, a typical deck of the same design, but with black steel, would have more delaminations (5 to 20 percent of the deck area is common).

< The chloride concentrations at the rebar level for most bridges were at or above the corrosion threshold for black steel. However, the chloride levels in some others were still below the threshold.

< Corrosion on the extracted ECR segments was determined to be minor in most of the extracted cores. No evidence of corrosion was found on 81 percent of the extracted ECR segments, even though chloride concentrations up to 3.8 kg/m^3 (6.4 lb/yd^3) were well above the chloride

threshold level for initiating corrosion in black steel.

< ECR did not appear to perform as well when the concrete was cracked as when the concrete was not cracked. There was more corrosion activity on ECR segments extracted from cores taken at locations where the deck was cracked. Even with high chloride concentrations up to 7.6 kg/m^3 (12.8 lb/yd^3), no visible or negligible corrosion was found on ECR segments extracted from cores taken in uncracked locations. The cracks give both chlorides and moisture easy and direct access to the ECR, which appears to accelerate the corrosion process.

< In California, corrosion on the extracted ECR segments was more severe at locations of heavy cracking, shallow concrete cover (15 to 25 mm [0.6 to 1.0 in]), and high chloride concentrations (9.7 to 15.0 kg/m^3 [16.4 to 25.3 lb/yd^3]). Moisture/water and a high chloride content present at the rebar level for a considerable length of time are responsible for the observed corrosion.

< The Ontario Ministry of Transportation reported that corrosion on the extracted ECR segments was more severe at a location of heavy cracking, shallow concrete cover (15 mm [0.58 in]), and a high chloride concentration (9.4 kg/m^3 [15.8 lb/yd^3]). This ECR segment was extracted from a noise barrier wall panel that had significant corrosion-induced concrete distress. Moisture/water and a high chloride concentration at the rebar level are once again responsible for the corrosion observed. The concrete in this barrier wall was also very permeable (21,293 and 22,722 coulombs). A typical bridge deck does not have such a low concrete cover and/or highly permeable concrete.

< Reduction in adhesion and softening occurred as a result of prolonged exposure to a moist environment. In California, a reduction in adhesion occurred at both corroded and non-corroded areas and was generally detected at visible holidays. In Indiana, the ECR segments showed no signs of a reduction in coating adhesion. In Michigan, coatings on ECR segments extracted from moist concrete could easily be removed. In New York, coating deterioration was not found on any of the ECR segments. Tests performed in Ontario showed that adhesion of the epoxy coating to the steel substrate decreases with time. Approximately 54 percent of the ECR segments evaluated under the Canadian Strategic Highway Research Program (C-SHRP) still had good adhesion of the epoxy coating.

< The number of defects in the epoxy coating and the amount of disbondment influence the performance of ECR. Many of the extracted ECR segments contained defects – holidays, bare areas, mashed areas, or a combination of one or more of these. In California, high chloride concentrations up to 4.6 kg/m^3 (7.7 lb/yd^3) did not initiate corrosion when there were no defects (holidays) in the coating, indicating that undamaged epoxy coatings provide an adequate barrier to chlorides. In Virginia, there were no indications of significant corrosion, even though the initial condition of the coating was poor and numerous holidays and bare areas were present.

< A comparison of the performance of ECR in decks with only the top mat of reinforcing steel epoxy-coated, and decks with both the top and bottom mat of reinforcing steel epoxy-coated, suggests superior performance when both mats are epoxy-coated.

< The bridges evaluated in California were originally constructed with black steel. Based on the dates of original construction and first redecking, it appears that the use of black steel only provided 10 to 12 years of service life. However, it is possible that there were other contributing factors besides the use of black steel, including shallow cover and a lower quality of concrete.

< The use of an adequate good-quality concrete cover, adequate inspection, finishing, and curing of the concrete, and the proper manufacturing and handling of ECR complement the use of ECR in providing effective corrosion protection for concrete bridge decks.

< ECR has provided effective corrosion protection for up to 20 years of service. Corrosion was not a significant problem in any of the decks evaluated. No signs of distress were found in the first bridge decks built with ECR. There was no evidence of any significant premature concrete deterioration that could be attributed to corrosion of the ECR. Some of the cores were intentionally taken at locations representing a worst-case scenerio. Therefore, these cores may not be representative or indicative of the overall performance that can be obtained from ECR. Little or no maintenance or repair work has been done on most of the decks.

As a result of renewed concerns about the effectiveness of ECR as a corrosion-protection measure, the State of Virginia performed another evaluation of three additional bridges in 1996. The results of this evaluation are presented in a 1997 report entitled *Field Investigation of the Corrosion Protection Performance of Bridge Decks Constructed With Epoxy-Coated Reinforcing Steel in Virginia.*[35] The three bridge decks are located in a high deicer application area and are among the oldest bridge decks built with ECR in Virginia. All three were constructed with ECR in the upper mat of reinforcing steel and black steel in the lower mat of reinforcing steel. The decks were 17 years old at time of the investigation. The evaluation used the results of NCHRP Project 10-37B, *A Protocol for the Evaluation of Existing Bridges Containing Epoxy-Coated Reinforcing Steel*, which provides measurable performance indicators, investigative procedures and plans, and methods of analysis.

These are some comments on the results and findings of this evaluation of three additional bridge decks in Virginia:

< The same corrosion mechanism applies to substructures and bridge decks.
< Coating disbondment occurs at a faster rate in lower quality concrete.
< It takes longer for ECR bars in bridge decks to disbond compared to ECR bars in piles in marine environments.
< Epoxy coatings are expected to disbond in humid environments.
< If sufficient chlorides reach the ECR before the coating disbonds, the epoxy coating provides additional service life.

<   Based on a statistical analysis of bridge deck service life, it is estimated that for 95 percent of bridge decks in Virginia, the coating will disbond before chlorides reach the ECR and the coating will not provide any additional service life.

<   The authors conclude that ECR is not a cost-effective corrosion-protection measure for bridge decks in Virginia.

However, the actual field performance and condition of the decks as reported (minimal cracking and delamination, as well as no visual signs of corrosion-induced deterioration) does not seem to support this conclusion.

ECR, especially when used in bridge decks, has demonstrated excellent performance in resisting corrosion and avoiding any corrosion-induced concrete deterioration. However, when used in substructures, ECR has not performed as expected in a few bridges that were exposed to a severely corrosive environment. ECR performs better than black bars and can extend the time to corrosion-induced deterioration in reinforced concrete structures. Since its introduction coincided with the use of improved concrete quality and increased concrete cover over the reinforcing steel, both of which also impact the time to corrosion initiation, it may be too soon to tell how much ECR contributes to improved performance. The factors that would accelerate ECR corrosion, such as damaged coating, poor concrete quality, low concrete cover, and severe exposure, all need to be addressed. In general, the conclusions on the performance of ECR as a corrosion-control measure vary from satisfactory performance in decks to poor performance in substructures exposed to marine environments.

The Ontario Ministry of Transportation has implemented several measures to address some of these factors. One is time limits on the storage of ECR bars – a 30-day maximum for unprotected bars and a 120-day maximum total for on-site storage. Bars with more than 1-percent surface area damaged are rejected and repair is not allowed. For those bars that are accepted, any visually detected damage is to be repaired. The use of plastic-coated concrete vibrators when consolidating the concrete is strongly recommended. Additional requirements for cathodic disbondment and salt-spray testing have been implemented.

CORROSION-RESISTANT REINFORCING BARS

Steel reinforcing bars are subject to various severe environments before being placed in concrete (during transportation, storage, and installation) and in service while embedded in the concrete. Coated reinforcing bars are subject to abrasion or other damage during fabrication, transport, handling, installation, and concrete placement. During storage, reinforcing bars can be exposed to condensation, rain, deicing chemicals, and seawater. After the reinforcing bars are embedded in concrete, they are initially exposed to a high pH and moist environment, which, over time, is changed to high and low pH areas, high chloride, and moist environment. The coatings need to be stable in all these environments and should not deteriorate.

Coating defects are a major factor in coating deterioration. Holidays and coating defects provide chloride ions direct access to the steel surface and also reduce the overall electrical resistance and, hence, the durability of the coating. Typical coating defects are due to manufacturing, handling, bending, and abrasion, and are often found at bar marks. The typical specification limit is a maximum of 3.3 holidays per meter (1.0 holidays per foot). Holiday counts are made using a 67.5V, 80,000-ohm holiday detector in accordance with ASTM G62. Additional new holidays may be introduced as a result of the bending operation. Coating failure modes include crushing failure (cold flow), cracking, and holidays.

Coating thickness is covered by both AASHTO and ASTM specifications. Older specifications called for 90 percent of the coating thickness population to fall between 0.125 and 0.30 mm (5 and 12 mils). Present specifications call for 90 percent of the coating thickness population to fall between 0.175 and 0.30 mm (7 and 12 mils). This results in a more stringent coating variability requirement than for the older specifications using an assumed normal distribution of coating thickness. The coefficient of variation is a measure of the variability of coating thickness and is defined as the standard deviation divided by the average. Low coefficients are an indication of uniform rates of coating application. In order to meet the 90-percent confidence limits under the older ASTM/AASHTO thickness specification of 0.125 to 0.30 mm (5 to 12 mils), a coefficient of variation of 25 percent was needed. In order to meet the 90-percent confidence limits under the newer ASTM/AASHTO thickness specification of 0.175 to 0.30 mm (7 to 12 mils), a coefficient of variation of less than 16 percent is needed.

The maintenance of coating adhesion is a factor in the long-term effectiveness of a coating system as a corrosion-protection mechanism. Although a reduction in coating adhesion by itself may not necessarily result in corrosion under the coating, a reduction in adhesion does provide a pathway for the migration of chloride ions, water, and oxygen through the defective coating to the steel surface.

The cathodic delamination mechanism causes a disbondment of coatings from the steel surface. Water, ions, and oxygen must be present at the steel surface in order for cathodic disbondment to occur. In the cathodic disbondment mechanism, the cathodic reaction produces OH⁻ and locally increases the pH at the coating/steel surface interface. The pH may increase to 14 or greater. The organic polar bonds between the coating and steel are significantly reduced. In order for cathodic reaction to proceed,

cations (Na^+ or K^+) are needed at the cathode to neutralize the negative charge produced by the OH^-. The cations move either through coating or along the coating/steel interface. It is generally thought that cations move along the interface (unless a potential is applied) and the rate of cation transport to the cathode is the limiting step in the cathodic disbondment mechanism.

The use of highly corrosion-resistant reinforcing bars can provide an additional layer of corrosion protection for reinforced concrete structures. Although there is a significant initial cost for corrosion-resistant reinforcing bars, the increase in overall initial structure cost may be justified. An extended service life decreases life-cycle costs. When considering the consequences of unintended low concrete cover, poor curing, permeable concrete, concrete cracking, and harsh service environments, the use of corrosion-resistant materials may be very cost-effective, especially when repair of corrosion-induced deterioration is costly and/or hard to do.

A 75- to 100-year design life can be achieved by extending the corrosion initiation period and reducing corrosion rates. The use of reinforcing material that is less sensitive to depassivation can extend the corrosion initiation period. A reduced corrosion rate results in a decreased amount of metal loss and extends the time period until subsequent cracking. Due to the controversy on the effectiveness of epoxy-coated reinforcing bars, a study was initiated to evaluate new corrosion-resistant reinforcing bars and new reinforcing bar coatings to find the most cost-effective system to achieve a desired 75- to 100-year design life. The reinforcing bars evaluated included organic coatings and inorganic-, ceramic-, and metallic-claddings on conventional bars and solid metallic rebars.[36-39]

In the first phase of the study, prescreening tests were performed on 22 bendable and 11 non-bendable organic coatings on steel reinforcing bars. Bendable coatings are those coatings that are considered to be bendable after they are applied to the steel reinforcing bar (i.e., the reinforcing bars are bent to their required shape after being coated). Non-bendable coatings are those coatings that are not considered to be bendable and are applied after the reinforcing bar is bent to its final shape. Some of these coatings used new cleaning and chemical pretreatments on the steel surface to enhance adhesion (17 of the 33 coating systems). Prior to testing, straight sections of holiday-free bars coated with bendable coatings were bent 180E around a mandrel with a diameter four times the bar diameter (4D) and examined for holidays, cracks, and crushing or cold flow of the coating.

The prescreening solution tests were selected to represent four exposure conditions – rain, seawater, chloride-free concrete, and chloride-contaminated concrete. The adhesion of the coatings was evaluated on the straight and bent sections of the bars.

Cathodic disbondment (CD) tests were conducted on organic-coated reinforcing bars bent to 4D. The tests were conducted at a potential of -1000 mV versus the rest potential over a period of 28 days at 23EC (73EF) in a 0.3N KOH + 0.05N NaOH solution at pH 13.3. During the testing, the specimens were monitored using ac impedance techniques. Adhesion tests were also performed.

In the prescreening tests, excellent adhesion was observed for both bendable and non-bendable coatings on straight bars following the severe immersion tests. Excellent adhesion was also obtained for

the prebent bars using non-bendable coatings. As poor adhesion was obtained on bent bars using bendable coatings, it was necessary to initiate additional screening tests to determine the extent of bendability of the bendable coatings. This research was carried out in Phase II. In this phase, seven of the best-performing coatings from Phase I and three new coatings were vigorously screened. The adhesion of these 10 coating systems on straight, 4D, 6D, and 8D bars was tested after solution immersion and cathodic disbondment tests.

In addition, screening tests were conducted on 14 different ceramic-, inorganic-, and metallic-clad bar types. Clad bars included the following:

< Hot-dipped galvanized bars.
< Bars coated with zinc using the Delot process.
< Nickel-clad bars.
< Inorganic zinc silicate-clad bars.
< Ceramic-clad bars.
< Several bars with proprietary zinc-rich claddings.
< Copper-clad bars.
< Type 304 stainless steel-clad bars.
< Copper-based alloy-clad bars.
< Bars clad with reactive copper in an organic coating.
< Galvalum (aluminum and zinc)-coated bars.

Screening tests were also conducted on 10 different solid metallic bar types. These included the following bar types:

< Normal A615 reinforcing bars.
< Type 304 stainless steel bars.
< Type 304 stainless steel bars (European).
< Type XM-19 stainless steel bars.
< Nitronic 33 stainless steel bars.
< Type 316 stainless steel bars.
< Type 317 stainless steel bars.
< Corrosion-resistant steel alloy bars.
< Type C613000 aluminum bronze bars.
< Titanium bars.

For each type of clad bar, bars were tested in three different conditions in two different test solutions. The bent and straight bars were tested as received, with a hole drilled through the cladding, and after abrasion. Corrosion rates of the bars were evaluated using polarization resistance (PR) measurements.

Two solutions were used for the tests – a 3 percent NaCl solution and a 0.3N KOH + 0.05N NaOH + 3 percent NaCl solution. The tests used specially constructed machines that dipped the bent and straight specimens into the solutions (1.25 h of dipping in specified solutions and 4.75 h of drying in air)

for a specified period. At the end of the testing, the bars were visually assessed.

Initially, the clad-bars that performed well in the screening tests were the zinc-rich clad, the Type 304 stainless steel-clad, the copper-clad, and the ceramic-clad bars. These four clad-bar types were selected for further testing along side the solid metallic bars in longer, more aggressive solutions. The data from the stainless steel-clad, solid stainless steels, and solid titanium bars suggested that a significant corrosion-free life can be obtained.

Finally, the two best bendable and the two best non-bendable epoxies from the screening tests were selected for the in-concrete tests. Since Scotchkote 213 was used almost exclusively in concrete structures until 1993, it was also selected for the in-concrete tests. A post-baked epoxy was also chosen based on the prescreening tests.

Three metallic-clad and two solid metallic bar types were also selected for the in-concrete evaluation. Types 304 and 316 stainless steel bars were chosen as they also exhibited excellent durability in previous long-term concrete test programs. Due to inconsistent corrosion performance in some research tests, galvanized bars were also selected. It was found that a newer zinc alloy-clad bar performed significantly better than the galvanized bars. Researchers wanted to determine whether the advances shown through the use of a newer zinc alloy cladding would be exhibited during the in-concrete tests. Copper-clad bars were also found to have good performance and a limited study of those bars was included.

The in-concrete testing phase used both uncracked and precracked concrete slabs. The 12 bar types selected for the in-concrete tests were:

- < ASTM A615 black reinforcing bars.
- < Epoxy-coated bars coated with 3M Scotchkote 213.
- < Two bendable epoxy-coated bar types.
- < Two non-bendable epoxy-coated bar types.
- < One post-baked non-bendable epoxy-coated bar type.
- < ASTM A767 galvanized bars.
- < Zinc alloy-clad bars.
- < ASTM A955 Type 304 stainless steel bars.
- < ASTM A955 Type 316 stainless steel bars.
- < Copper-clad bars.

Three of the epoxies utilized a steel surface pretreatment prior to coating and three did not. Two of the epoxies used a chromate pretreatment. For the third epoxy, the manufacturer of the product did not reveal the chemical used for the pretreatment.

The concrete slabs used in the tests measured 300 x 300 x 175 mm (12 x 12 x 7 in). Each slab contained two mats of 16-mm (No. 5) reinforcing bars. The top mat contained two straight or two bent reinforcing bars. The bottom mat contained four straight reinforcing bars. Each top-mat bar was

connected to two bottom-mat bars using a 10-ohm resistor. Nine specimen configurations were used for the tests for each bar type, including uncracked and precracked concretes. The clear concrete cover used was 25 mm (1 in). In order to simulate damage from shipping, handling, installation, and concreting, all bars were intentionally damaged by drilling holes through the coating or cladding, or into the bar surface to achieve two levels of damage (0.5 percent and 0.004 percent of the bar surface area). A concrete with a nominal water-cement ratio of 0.47 was used. The slabs were cured with wet burlap and polyethylene film for 3 days to simulate field curing.

Fifty-nine days after casting the slabs, a wetting and drying cycle to accelerate the corrosion process of the steel reinforcing bars embedded in the concrete slabs was started. The wet-dry cycle consisted of 3 days of drying at 38EC (100EF) at 60- to 80-percent relative humidity, followed by 4 days of ponding with a 15 percent NaCl solution at 16 to 27EC (80EF) and 60- to 80-percent relative humidity. This wet-dry cycle was repeated for 12 weeks. After 12 weeks, the slabs were continuously ponded with a 15 percent NaCl solution. This 24-week schedule was repeated four times for a total of 96 weeks of accelerated exposure.

Measurements were made to enable the corrosion rates of the reinforcing bars to be determined, including macrocell currents, linear polarization, and ac impedance. These measurements were used to evaluate the different corrosion-resistant bar systems and are based on the results of a very accelerated and aggressive testing procedure. A concrete with a moderate diffusion rate was purposely used to fabricate the concrete test slabs. The 15-percent saltwater ponding solution had a very high chloride concentration, about five times the concentration for normal seawater. However, almost all test specimens had corrosion rates that were relatively uniform over time. No rapidly increasing rates were observed that would indicate a catastrophic and rapid failure of the bar system.

CONCLUSIONS

The Type 316 stainless steel bars should be considered during the design stage as a potential corrosion-protection measure to achieve a 75- to 100-year crack-free design life. The additional cost may be justified by a life-cycle cost analysis. This is especially true for structures that carry a significant amount of traffic and where repairs are difficult and/or costly and closures are a problem. An alternative to solid bars may be a stainless steel-clad bar. Methods for manufacturing stainless steel-clad bars at a competitive cost are currently being investigated. At this time, one manufacturer is producing Type 316 stainless steel-clad reinforcing bars using a patented process at a cost slightly higher than some of the other conventional corrosion-protection systems currently in use.

The Type 304 stainless steel bars had excellent performance when straight bars were cast in uncracked concrete. However, when bent bars were used with a black bar cathode, moderate corrosion resulted. Therefore, Type 304 stainless steel bars are not recommended, particularly in combination with a black bar cathode.

The copper-clad bars performed relatively well with low corrosion rates. It is known that lead, copper, and zinc salts can retard the portland cement hydration process. Any possible structural effects due to

the retardation of the cement paste adjacent to the surface of copper-clad bars need to be investigated (bond strength in particular).

Hot-dipped zinc (galvanized) coatings have been used since the early 1940's. They are currently covered by ASTM A767, Standard Specification for Zinc-Coated (Galvanized) Steel Bars for Concrete Reinforcement. ASTM A767 requires a chromate treatment after coating to preclude any adverse reaction between the zinc and the fresh portland cement paste. Corrosion protection is provided by a protective zinc coating. However, zinc will corrode in concrete when exposed to chloride ions. The coating is applied by dipping the properly prepared steel reinforcing bars into a molten zinc bath.

Galvanized bars, when used in both mats in uncracked concrete, had relatively good performance. When used in precracked concrete, their performance decreased considerably. When used with a black bar cathode, their performance was not much better than when using all black bars. If used, care should be exercised to eliminate any electrical contact between the galvanized steel and other metals.

The zinc alloy-clad bars had a better performance than black steel in almost all cases. However, the improvements in corrosion protection were not considered to be sufficient enough to warrant their use in concrete.

The straight epoxy-coated bars, when used in both mats in uncracked concrete, had macrocell currents almost as low as those for the stainless steel bars. However, when used in combination with a black bar cathode, the corrosion rates significantly increased. The bars that received a pretreatment did not perform significantly better than those that did not receive a pretreatment. Based on this latest research, epoxy-coated reinforcing steel still appears to be a viable corrosion-protection measure for reinforced concrete bridge decks. In addition, when epoxy bars are used:

< Use them in both mats or for all reinforcing in a component.
< Minimize coating damage in shipment and placement.
< Repair coating damage on site.
< Repair concrete cracks.

SUMMARY

Salt-induced reinforcing steel corrosion in concrete bridges has undoubtedly become a considerable economic burden to many State and local transportation agencies. Since the iron in the steel has a natural tendency to revert eventually to its most stable oxide state, this problem will, unfortunately, still be with us for the next few decades, although probably to a lesser extent because of some of the corrosion-protection measures that came into practice in the last 2 decades in building new concrete bridges. There is no doubt that adoption of corrosion-protection measures, such as use of good design and construction practices, adequate concrete cover depth, low-permeability concrete, corrosion inhibitors, coated reinforcing steel, clad reinforcing steel, and corrosion-resistant alloy reinforcing steel in new construction, will help in significantly delaying the occurrence of reinforcing steel corrosion in reinforced concrete bridges.

The use of good design and construction practices, adequate concrete cover depth, corrosion-inhibiting admixtures, and low-permeability concrete alone will not abate the problem, because concrete has a tendency to crack inordinately. In fact, it has been observed lately that low permeability or high-performance concrete (made from partial substitution of portland cement with silica fume or fly ash) has an even more pronounced tendency than conventional concrete to crack, thereby potentially trading a normally slow intrusion of chloride ions into the concrete (by the diffusion process) for a potentially faster gravity-assisted flow of salt-laden water. Even corrosion-inhibiting admixtures for concrete would probably not be of use when the concrete is cracked. This situation essentially leaves the reinforcing steel itself as the last line of defense against corrosion. For this very reason, the use of a barrier system on the reinforcing steel, such as epoxy coating or other organic or even other possible metallic coatings or corrosion-resistant alloys, is even more critical in abating this costly corrosion problem.

It is likely that there may never be any organic coating that can hold up to the extreme combination of constant wetting and high temperature and high humidity that reinforcing steel is exposed to in the marine environment in Florida, especially in the splash zone. For these severe exposure applications, rebars clad with Type 316 stainless steel or a type of corrosion-resistant solid metal bars would have to be used in conjunction with the use of durable concrete and sound design and construction practices. However, there are some very convincing reports of good corrosion resistance performance shown by epoxy-coated steel bars in concrete bridge decks, where unlike bridge members in the tidal zones of coastal bridges in Florida, the concrete does not remain constantly wet and the other exposure conditions are not as severe. And, just recently, good performance by epoxy-coated bars has been observed in bridge decks surveyed in Pennsylvania and New York by researchers from a corrosion engineering firm in Virginia, and in cracked and uncracked concrete by researchers from the University of New Brunswick. It must also be mentioned that unfavorable performance by epoxy-coated bars has recently been claimed, albeit unconvincingly. In one latter case, the poor-performing epoxy-coated rebars were located mostly in noise walls, where the quality of the concrete was known to be poor, and in concrete expansion dams (beside expansion joints), where it is suspected that the coated rebars may have been cleaned by abrasive blasting before the pouring of the concrete.

The many successful performances of embedded epoxy-coated steel bars in places outside of Florida and possibly in other similar locations indicate that when used in exposure conditions that do not keep the concrete constantly wet, the epoxy coating will provide a certain degree of protection to the steel bars and, thereby, delay the initiation of corrosion. The recent claims of poor performance of epoxy-coated rebars serve, at most, to indicate that the corrosion protection provided by ECR (more accurately, the old generation of ECR) is not permanent, and also to raise the question – How long does the use of ECR in a particular exposure condition delay the initiation of steel corrosion in the concrete? And, for a prospective user, the next question is – Is the savings in maintenance and traffic control costs resulting from this extra time worth the initial extra cost of using ECR instead of black steel bars? Unfortunately, accurate determination of the actual field performance of ECR in a particular State or region or exposure condition is extremely difficult, if not impossible, since many contributing factors are involved and have to be accounted for. Needless to say, the recent improvement of specifications for ECR by the industry and the tightening of requirements on proper storage and handling of ECR at construction sites will ensure good corrosion protection. Some States are now using epoxy-coated reinforcing steel in combination with certain corrosion inhibitors as a multiple corrosion protection system for use in marine environments.

The ongoing research study on steel bars coated with new organic and metallic coatings and alternative solid metal bars should result in identification of more corrosion-resistant and, hopefully, more cost-effective alternative reinforcement for future use in concrete bridges.

For construction of new prestressed concrete bridge members, the use of a corrosion-inhibiting admixture in the concrete or the grout, in conjunction with the use of good construction designs and practices, would provide corrosion protection. However, the long-term effectiveness of various commercially available inhibitors is under evaluation.

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