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# What Every Engineer Should Know

## About Reinforcement Corrosion in Concrete Highway Bridges

by

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### **Introduction**

The current, deteriorated state of several areas of our physical infrastructure in the United States is not news to anyone. This is especially true for the many professional engineers that deal with these problems either directly or indirectly. A very visible and significant portion of the country's infrastructure is represented by highway bridges that have suffered from benign neglect for a variety of reasons. This course concerns the major role that corrosion plays in deterioration of bridges, how this damage occurs and what design, repair and remediation measures may apply to mitigate the corrosion issues.

Corrosion of rebar used in traditional concrete construction as well as corrosion of the high strength steel strands in prestressed concrete construction are considered. Many bridges primarily use coated structural steel exposed directly to the atmosphere and those structures are briefly reviewed.

Because of the importance and magnitude of these problems, there is a wide range of technical literature and standards plus government agency documents that have been produced on various parts of the issue. This course attempts to consolidate and summarize a small portion of those many sources. The references used are cited with superscripted parenthetical numbers in the text or in diagrams and are listed by number at the end so that those interested in finding more detailed information may do so. Many of the references are available online. References (7), (9), (13) and (14) are excellent and comprehensive sources of information for this subject. They were primary resources for developing this course and may be useful to others.

### **Broad Aspects of the Problem**

As of December 2014 the Federal Highway Administration (FHWA) in its annual National Bridge Inventory (NBI) reported that there were approximately 600,000 highway bridges (of a span of 20-feet or more) in the U.S.<sup>(1)</sup>. Of this total approximately 10% were judged to be in one of several classes of structurally deficient (SD). Corrosion has been confirmed as the largest source of this damage by multiple sources. Bridge deck areas are the components most affected.



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A 2001 FHWA report<sup>(2)</sup> on the cost of corrosion in all segments of the U.S. economy estimated that the direct, total cost in 1998 was \$ 276 billion. It was also estimated that the portion of this total for all areas of infrastructure, i.e., highway bridges, Hazmat storage, gas and liquid transmission pipelines plus waterways and ports, was \$22.6 billion. Of the infrastructure segment, \$ 8.3 billion was for highway bridges. The report also estimated that the indirect cost of bridge corrosion was up to ten times the direct costs. These indirect costs include expenses from traffic delays and detours due to bridge maintenance, greater fuel usage, delays in commercial product delivery plus loss of general productivity. Thus the combined 1998 costs would have been approximately \$91.3 billion.

These 1998 corrosion costs can be brought forward for a more current estimate that includes the effects of inflation using the federal government website [www.usinflationcalculator.com](http://www.usinflationcalculator.com). The 1998 value then becomes \$133.2 billion in 2015.

How did the country get to this point? It's primarily because for many years society has not demanded adequate funding for the needed actions compared to alternative, more popular government expenditures. Corrosion has continued on its normal destructive course while, it seems, many of our politicians have been followers of the voters rather than leaders. In recent years the poor economy coupled with the bitter political disputes over spending priorities at both the federal and state levels have made rational infrastructure decision making even more difficult.

Other important factors have been the lack of general knowledge among decision makers about the extent of the problems and the importance of these issues to the public's safety and the country's economic viability. Then there is often ignorance about what specific technical actions should be applied. Engineers generally support the value of assessing alternative actions based on their technical merits and also their life-cycle cost (LCC) but very often these criteria are not, or cannot, be followed. Sometimes little money is available so that the lower initial cost alternative that is technically inferior is chosen while the technically superior but higher initial cost, lower LCC, alternative is rejected. The combined effects of all these factors have led to our present situation.

### **Types of Bridges and Statistics of Damage Caused by Corrosion**

References <sup>(3)</sup> and <sup>(4)</sup> contain much statistical information. The average age of U.S bridges is approximately 40 years while the average age of those that are structurally deficient is



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approximately 62 years. It is understandable that corrosion has worked its negative effects over these periods. This is especially true since until relatively recently it was widely believed that steel enclosed in concrete would not corrode and that surface damage was caused by poor construction practices. Further, older types of coatings on exposed structural steel and application practices were less effective than those currently used. Many of the older SD bridges will need complete replacement while others maybe can be salvaged with significant repairs.

Apart from timber construction bridges, the most common structurally deficient types (in descending order) in the U.S. are structural steel exposed to the atmosphere, traditional reinforced concrete, and, finally, prestressed concrete bridges. Among the latter three, corrosion causes the most structural deficiencies (SD) among structural steel bridges while the least SD types are prestressed concrete.

It is well known that salt used to melt ice and snow on the highways is the source of most corrosion-influenced damage to the county's bridges. From this it is natural to find as reported <sup>(5)</sup> that geographic location and environment directly affect the numbers of SD among the types of construction. Among traditional concrete reinforced and prestressed concrete bridges, larger numbers of SD exist in the northeastern and Midwestern states than in southern states.

These results are as expected given the traditional greater use of road salts in those areas subject to more severe winter weather. What is somewhat surprising is that structural steel bridges, while experiencing the most numbers of SD of the three types, were generally independent of geographic location and therefore the use of road salts. The reported <sup>(6)</sup> reason is that structural steel bridges have less stiff superstructures compared to concrete superstructures. This lack of stiffness coupled with the vibrations of traffic causes more cracks in the concrete decks and thus a greater number of SD steel bridges.

Based on the several sources cited in reference <sup>(3)</sup>, the author summarizes that bridges with the following characteristics are most likely to be structurally deficient: age of 50 years or more; simply supported structural steel stringer/multi-beam superstructure; less than 50-foot span; cast-in-place reinforced concrete deck without any deck corrosion protection; use of bare reinforcing steel; bridges sited in non-highway system locations with low average daily traffic ( 0 to 10,000 vehicles) and in rural areas.

### **Basic Concepts and Terms Used in the Electrochemistry of Steel Corrosion**



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This is intended to be a practice oriented course so little emphasis is placed on the theoretical aspects of corrosion. However, it is useful to understand some fundamental relationships and the following **bolded terms** are used by corrosion engineers. It's important because these terms and relationships define the basis for the different ways that corrosion can be controlled.

Steel attack and damage by aqueous corrosion involves two types of electrochemical reactions on the metal surface: **oxidation** and **reduction**. These two reactions must occur simultaneously and at the same rate.

Oxidation is the mechanism by which metal damage occurs because it involves metal atoms, without an electrical charge, being converted into electrically charged positive **ions** as the metal is penetrated at **anodic** sites and moving away from the surface into the corrosive medium - the **electrolyte**. An electrolyte is the aggressive corrosive liquid that is able to be **ionized** and thus permit the conduction of electrical current. While the damaging oxidation reaction is occurring at anodic sites on the metal, electrons flow through the steel to **cathodic** sites where the required reduction reaction(s) occur. At the cathodic sites multiple types of reduction reactions are possible depending on what electrolyte is present. However, in the corrosion of steel rebar the most common cathodic site reaction is the reduction of oxygen from atmospheric air. In this reaction water from diffused-in or existing moisture in the concrete plus diffused-in oxygen react with the arriving electrons from the anodic sites to produce negatively charged **hydroxyl ions** (OH<sup>-</sup>). These ions move into the electrolyte and in doing so they carry a direct electrical current, **DC**. The rate of DC flowing from cathodic sites is directly proportional to the rate of corrosion damage because the anodic and the cathodic reactions must occur at the same rate. See Figure 1 and the related electrochemical reactions that typically occur for corrosion of bare steel in concrete.

Every metal exposed to a given electrolyte has a unique electrical characteristic that is its **free corrosion potential**. This potential is measured in volts or millivolts. The value indicates the metal's relative propensity to corrode in that particular electrolyte for the particular conditions during the measurement. Potential cannot be measured directly but must be measured in comparison to a second metal – a **reference electrode** – being simultaneously exposed to the electrolyte. An exposed metal can be changed or **polarized** from its free corrosion potential by applying an external direct current to make it either corrode at a faster rate (**anodic polarization**) or at a slower rate (**cathodic polarization**). Anodic polarization also occurs naturally when a metal begins active corrosion without any outside applied current. Field or laboratory measurements of the potential of steel in concrete when polarized either anodically or



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cathodically is its **corrosion potential**. This value is an indicator of the possibility of active corrosion occurring but other factors may lessen its importance as an indicator. Measuring corrosion potential does not indicate a **rate of corrosion**.

In summary, aqueous corrosion requires four “components” besides a susceptible metal for the given electrolyte. These are anodic sites on the metal (where the oxidation reaction occurs), cathodic sites (where a reduction reaction occurs), a metallic path between the anodic and cathodic sites (the steel rebar through which electrons move) and the electrolyte (through which direct current can move away from the surface via ions). Measurements of the value of the corrosion potential of steel in concrete may indicate whether corrosion is occurring due to anodic polarization but they do not indicate the rate of corrosion.

Later the various ways that inspection for and control of corrosion of steel in concrete will be reviewed using these concepts and terminology.

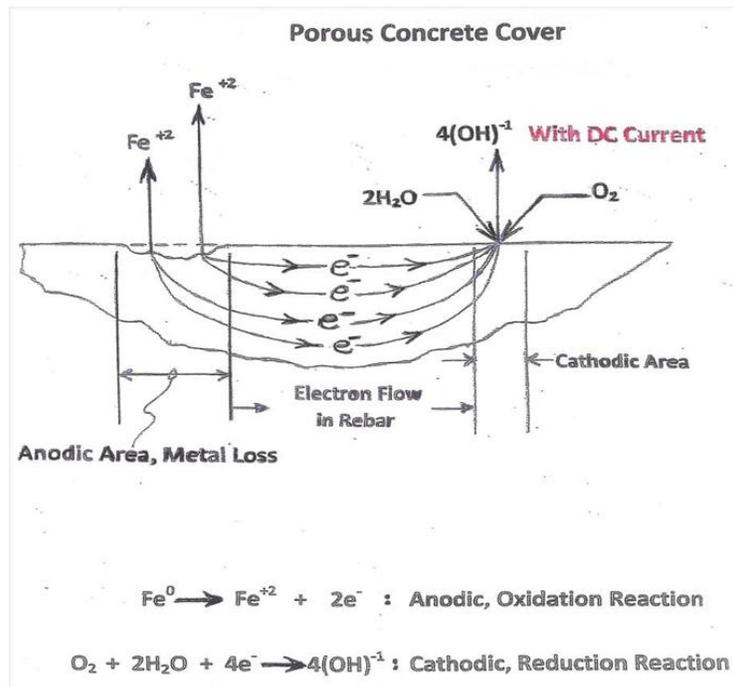


Figure 1

Schematic illustration of the chemical and electrical corrosion processes and the two electrochemical reactions that occur on a microscopic portion of bare steel reinforcement surface embedded in porous concrete. Voluminous, hydrated iron oxides that form on the surface as a result of corrosion are omitted for clarity.



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**Corrosion Damage Mechanisms by Type of Construction Method Used**

*Traditional Reinforced Concrete Construction*

As embedded rebar corrodes, hydrates of iron oxide form on the surface of the steel. These deposits are corrosion products, i.e., “rust” and related iron oxide compounds. The hydrate portion is water attached to the particular oxide of iron. The oxides’ volumes may be up to as much as 6 times the initial volume of the original iron from which they were formed. The resulting expansion creates a wedging action and local tensile stress that cracks the surrounding concrete. Over time sub-surface cracks that are initially small grow and may appear either as subsurface laminations or as spalling on the concrete’s outer surface. Eventually loose segments of surface concrete break or fall away, and without repairs, portions of the outer mat of rebar are revealed.

Normally concrete made with Portland cement is a very alkaline material with a pH of approximately 13. All concrete also has microscope-size pores. Embedded, bare steel rebar is immune to corrosion in a strongly alkaline environment that is free of oxygen from the air and especially certain ions. Under these benign conditions steel forms a surface film that acts as a barrier against the start of corrosion. Corrosion scientists and engineers refer to this corrosion-immune state as passivity and the film that forms as the passive film.

Chloride ions, typically from road deicing salts or exposure to marine waters, are the most corrosively aggressive of the ions that act on steel and breakdown the normal passive film. These ions plus oxygen from the air and external water enter and move through the concrete cover. This transport may be by diffusion (via Fick’s second law of diffusion); by capillary action via the tiny pores in the concrete; directly through surface cracks in the concrete or a combination of these. When a critical minimum, threshold concentration of chlorides reaches the rebar passivity is lost. Transported oxygen and water then initiate steel corrosion. The wedging action caused by the volume increase of the hydrated iron oxides generates new or added cracks, much larger than the concrete’s inherent pores that eventually reach the outer surface of the concrete. Surface cracks allow much easier movement of ions, oxygen and water through the concrete to the rebar. The corrosion process then becomes self-accelerating.



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Another important mechanism of rebar corrosion is produced during carbonation. This is caused by carbon dioxide (CO<sub>2</sub>) gas from atmospheric air and rain water containing CO<sub>2</sub> that diffuse through the concrete. Carbon dioxide mixed with water, which also enters the concrete, forms carbonic acid. Rain water with carbon dioxide can have a pH as low as about 5.6 <sup>(7),(8)and (9)</sup>. This low pH substance lowers the normally high pH alkaline condition around embedded rebar. That alkaline condition is needed to maintain the passive state of the rebar. Therefore corrosion may then start at the new, lowered pH. Normally carbonation as a cause of rebar corrosion is much slower than corrosion due to chlorides because CO<sub>2</sub> moves at a lower rate through the concrete. More carbonation occurs in large cities and industrial areas than in rural areas because of the larger concentration of CO<sub>2</sub> in the former areas.

Both chloride ion diffusion and CO<sub>2</sub> diffusion (along with water) through concrete are typically very slow processes<sup>(10)</sup>. The initiation of rebar corrosion may require several years. The rate of diffusion depends on several factors to be discussed later. If pre-existing cracks are present on the concrete's outer surface then, of course, the rate of diffusion is significantly increased. Once corrosion begins on the embedded steel there is another time delay, generally shorter than the diffusion period, before corrosion damage reaches the concrete's outer surface as obvious spalling. Software-based analytical models are reported<sup>(10)</sup> that are intended to predict the duration of each of these two periods based on the specific conditions in the given application.

### *Structural Steel Superstructure Bridges*

Resistance to corrosion of structural steel exposed to the atmosphere is generally provided by some type of surface coating. When intact, different coatings simply act as a barrier to corrosion. These are paints. Alternatively the selected coating may act as a barrier but also protect the steel by providing a sacrificial anode action. This is typically accomplished with galvanizing steel or by use of a zinc rich primer as the base coat of a painted surface.

For coated steel, corrosive conditions include deicing salts from splashed water and aggressive gases such as sulfur and nitrogen oxides in the air coupled with air humidity. Exposures to humid air and salt or brackish waters under or nearby steel bridges present particularly severe applications.

There is not a single mechanism by which coatings fail – there are several. Some of the most important factors include the following:



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Poor initial surface preparation prior to applying a paint coating “system”, i.e., a primer and maybe an intermediate coat plus a top coat, is a major if not the most important source of coating failures. Not completely removing salts, dirt and rust deposits before application of the coating system permits poor adhesion and provides the means for the initiation of corrosion of the substrate steel. No coating is completely free of pores and so water and oxygen can diffuse through it with time. If there is poor adhesion and salts are present on the underlying steel surface corrosion is encouraged. Eventually surface blisters and cracking of the coating will occur so that significant portions of the former intact barrier are destroyed. Voluminous corrosion products on the exposed steel then cause more coating damage, leading to more corrosion, etc. Proper surface preparation before applying a coating is essential.

Poor design details of the coated steel structure are significant causes of failures. Areas in corners and crevices between lapped plates and elsewhere that are not sealed closed can collect dirt and debris that retain rain and traffic-splashed water that will eventually diffuse through the coating. The same is true for structural angles and channels that are oriented so that they cannot easily drain. Some of these features are difficult to avoid on bridges with trusses and so they suffer more exposed coated steel corrosion than other types of construction.

UV exposure from the sun is a major reason <sup>(7)</sup> for failures of particular types of organic coatings. For those coatings ultraviolet light radiation acts to break or interfere with the chemical bonds in the resin of certain paints. Then the coating’s adherence to the steel substrate is weakened and failure may result. Water on the outer surface of the coating concentrates UV energy and thus the combination creates a more aggressive condition. Epoxy paints and particularly amine cross-linked epoxy resins are especially susceptible to UV damage. In contrast, other paint resins such as aliphatic polyesters, acrylics and polyurethanes provide good resistant to UV damage.

Zinc and zinc oxides in a paint primer also provide good resistance to UV. In addition, a zinc-rich primer as well as deposited zinc alone – as on galvanized steel – has the major advantage of acting as the anode in a galvanic corrosion cell with the substrate steel. The steel becomes the cathode in the electrochemical cell and is protected from corrosion while the more active zinc is sacrificially corroded. Apart from this use in coatings for atmospherically exposed structural steel, zinc coatings are widely used on traditional rebar embedded in concrete.

Acid gases in atmospheric air, e.g.,  $\text{SO}_2$ ,  $\text{SO}_3$ ,  $\text{NO}_x$  and even  $\text{CO}_2$ , react with moisture in the air to form dilute solutions of aggressive acids such as sulfuric, nitric and carbonic. These liquids can



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condense during cool nights on the coated steel superstructure. When the sun comes out during the day some of the water in the acid condensate evaporates making the acids more concentrated and more damaging to certain types of coatings. This type of attack is more likely if the bridge is near industrial plants that release acid gases to the atmosphere. Wind direction is also important.

Proximity to chloride-rich waters, high humidity and temperatures provide very aggressive conditions. This is the situation in marine atmospheric locations and even brackish water that also contains much higher concentrations of chloride ions than fresh water. The height of the lowest portions of structural steel above nearby water levels often is the important factor in the degree of coating damage that occurs.

Surface stress effects can accelerate coating failures. Vibrations induced by traffic (these vary with the type of construction and stiffness of the bridge) plus day-to-night expansion and contraction cause variable stresses on the coating system. These effects are especially damaging during very low temperatures that embrittle some coatings and make them more likely to form cracks that permit inward diffusion and then corrosion.

Inadequate coating system selection and/or field application are always possible sources of failures. There are many types of coatings – some better than others for particular local conditions – and so knowledgeable personnel are essential to specifying and procuring according to specific needs. Competent and conscious application contractors are very important and even then certified coating inspectors are needed to provide quality control for best results.

### *Weathering Steel*

This material is not an applied coating but it may be appropriate to consider the ways it can fail to perform in conjunction with the discussion of traditional coatings. These low-alloy steel products have the unique properties of relatively high strength and, when used properly, considerably higher corrosion resistance compared to traditional carbon steels. There are multiple variations of weathering steels and the alloys used for bridges in the U.S. are derived from the ASTM specifications A588 and A709.

These steels achieve enhanced corrosion resistance because they are alloyed with a minimum of 0.20% copper plus various, generally smaller percentages (depending on the specific alloy) of chromium, nickel and vanadium that affect both their corrosion resistance and their strength. After a period of exposure to ambient conditions they develop a patina on their surface, i.e., they



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form an iron oxide (rust) coating. Ideally this patina stops growing eventually and then acts to protect the unaffected substrate steel from further corrosion. The general aggressiveness of the environmental exposure determines how quickly the rust patina forms. It is quicker in more severe atmospheres and often takes years in more benign environments. Moderate environments are favorable to success but use these steels in the wrong environments and they will not perform. They are not a cure-all.

Very severe environments prevent the patina growth process from ever stopping. This occurs especially in humid marine applications because of the high atmospheric concentration of chloride ions and moisture. Weathering steel also fails in high traffic areas over extended winter periods where large amounts of deicing salt use is necessary. The presence of excessive salt on the steel prevents the proper forming and stability of the rust patina.

Geographic areas that are subject to extended periods of rain or to extended dry periods often cause failures. This is because the process of patina formation that eventually must stop depends on alternating cycles of wetting and drying of the steel surface. Probably the bigger problem is preventing the weathering steel from being continually wet. This is often caused by design defects. Some examples might include allowing pooled, non-drainable water, permitting a portion of the weathering steel to be almost continually wet from being below a drain or leaking expansion joint or due to inaccessible corners or crevices of the steel that retain water in dirt or debris next to the steel. Another problem is using the steel at low locations on the bridge superstructure where it is frequently receives splashed water from passing traffic.

*Prestressed Concrete Construction*

Prestressed steel reinforcements of concrete bridges are growing in favor. They include pre-tensioned and post-tensioned classes. Approximately 50% of new bridges use pre-tensioned construction<sup>(7)</sup>. Both of these types of construction provide increased resistance to tensile stresses by creating added compressive stresses in the concrete produced by initially applied prestressing. Those forces remain in place in the cured concrete. This often permits smaller and lighter structural members versus traditional rebar design. However different issues can be introduced so far as corrosion and its detection.

The corrosion that occurs with pre and post-tensioned concrete depends significantly on the particular installation configuration and methods used. Each type presents some specific circumstances that can either make corrosion and inspection for it easier or more difficult. Very



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high strength tensioned steel itself – often as multiple, small diameter wires twisted together to form strands – also is more susceptible to failure than traditional rebar. Treaded, high strength steel bars may also be used instead of twisted strands in some applications.

The pre-tensioning configuration usually consists of multiple wire strands that are placed in concrete formwork of the desired size of the finished concrete member. A single strand is usually six, small diameter high strength wires that have been spirally wound around a similar central wire to form a cable-like component. Tension is first applied to each strand and that force is held. Concrete is then poured in the formwork and allowed to cure to a specified strength while it contacts the strands directly. A bond develops between the strands and the concrete as curing progresses. Finally the applied tension in the strands is released and as they attempt to decrease in length compressive stress is transferred to the cured concrete. Structural members are produced off-site in a precast concrete vendor's plant and then brought to the job site.

The post-tensioned configuration includes a duct (in a loose sagging condition) that is placed from one end to the other of the desired size segment of concrete formwork. The duct material may be metal or a polymer but metal ducts are more commonly used. A stressing anchoring assembly is placed inside the formwork at each end and these are connected to the duct. Typically multiple strands are then pulled through a duct but less commonly a single strand or a threaded bar may instead be used in the duct. The steel reinforcing material together with the duct inside is known as a tendon.

Steel strands are connected and fixed to the stressing anchorages as shown in Figure 2. Concrete is poured into the formwork to embed the duct as well as the stressing anchorage assemblies. The concrete is allowed to cure to the specified strength. The wire strands in the duct are then pulled in tension through the embedded stressing anchorages and the applied force is fixed at each end. In this way compressive stress is transferred to the cured concrete member.



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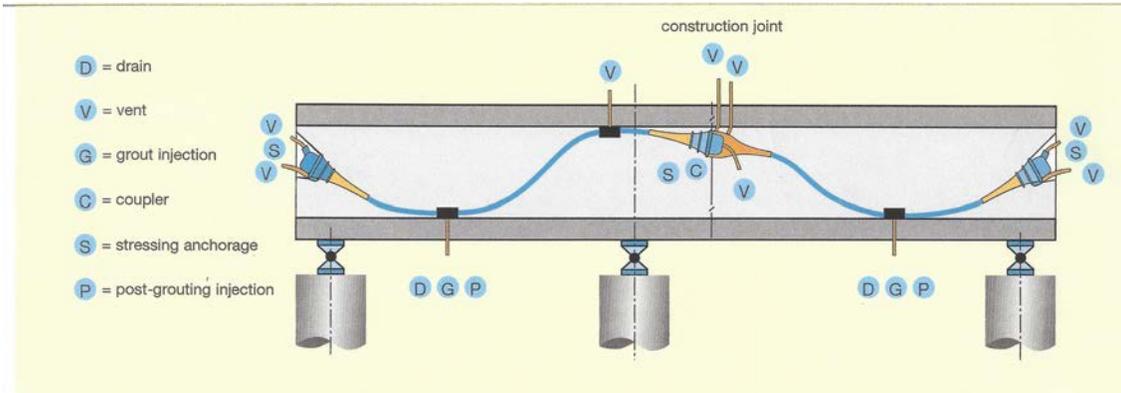


Figure 2

Schematic illustration of the cross-section of two horizontal bonded, post-tensioned concrete structural members with the identities and locations of the primary components of that prestressed configuration. (Used with the permission of Mr. David Martin of DYWIDAG – Systems International)

Figure 3 shows details of a stressing anchorage assembly.

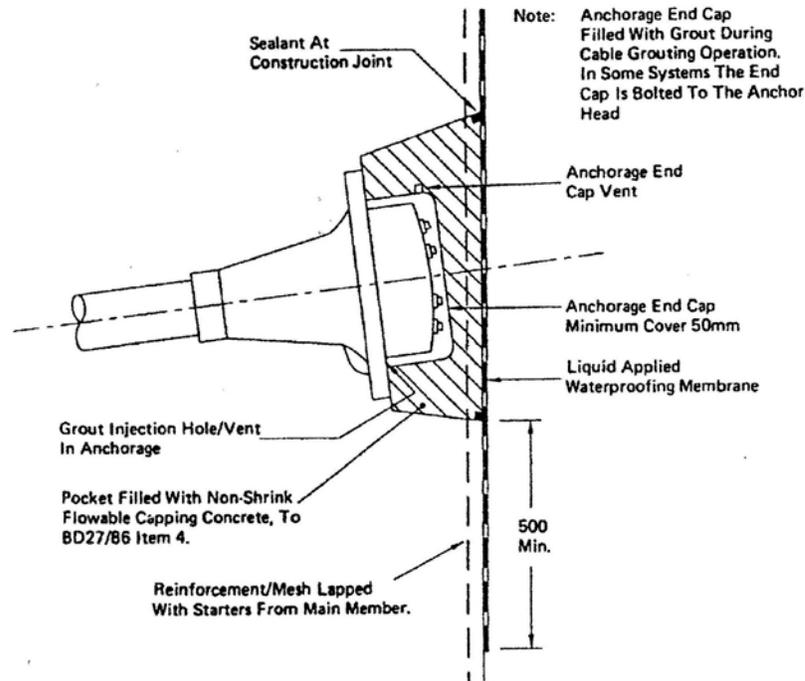


Figure 3

Cross-sectional diagram of a generic stressing anchorage assembly typically used in bonded, post-tensioned concrete structural members. (Authorized reprint from the American Concrete Institute [ACI] publication's Reference<sup>(13)</sup>, page 23)



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There are two alternative methods for final installation of the tendons in post-tensioned members. The strands may be unbonded or bonded. For the unbonded case grease containing chemical corrosion inhibitors is applied to the strands as well as injected into the duct so that the strands are surrounded. No bond is formed between the grease and the inside diameter of the duct. In the bonded method cement grout is pumped into the duct with needed vents placed as shown in Figure 2. The goal is to completely fill the annular space with quality grout between the inside diameter of the duct and the outside diameter of the strands and also to fill the inner spaces between strands. A bond develops between the grout and the inside of the duct.

The bonded method is more commonly used for concrete bridge members. All post-tensioned work is done at the job site so the quality control may not be as good as that for pre-tensioned members that are made in an off-site plant. The knowledge and quality controls provided by the installation contractor are critical to the probability of failure by corrosion in bonded post-tensioned construction.

The corrosion failure mechanism for pre-tensioned construction is similar to the mechanism previously described for traditional rebar because the outer surface of the high strength steel wire is in direct contact with concrete. However, there is one significant difference. The small diameter, high strength wires used for both pre and post-tensioned construction are initially pulled to about 60 % of their ultimate tensile strength (UTS). Standard rebar are much larger in diameter and the stress they experience in service is a much lower percentage of their UTS. If only one or two high strength wires in a spiral wound strand lose some of their thickness due to corrosion the applied stress becomes more than 60% of their UTS. If corrosion continues one or more of the partially thinned wires may eventually fail. When this occurs while the initial applied tensile force is still acting, higher stress is transferred to the remaining intact wires. That result makes those wires more susceptible to future failure due to even small metal penetrations of their thickness by corrosion.

In the post-tensioned case an intact duct typically provides a barrier against the factors necessary for corrosion. However, if there is a crack or small opening in the duct the barrier function is compromised. Most commonly this occurs at gaps in the necessary joint couplings that join segments of the duct or is due to corrosion on the commonly used metal ducts. Chloride ions plus water and oxygen that diffuse through the outer concrete may then enter any opening and travel to the wire strands. Instead of a metal duct a plastic duct material (usually polypropylene or polyethylene) may be used to provide a superior barrier but at higher cost.



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In the case of the unbonded, post-tensioned configuration corrosion protection depends, apart from other factors that also apply to traditional rebar, on the integrity of the duct and the chemical corrosion inhibitors used in the grease on and around the strands. When either of these restraints is deficient corrosion of the strands can initiate and eventually lead to failures.

In bonded, post-tensioned construction the cured grout inside the duct can provide a more substantial layer of protection for the strands but this depends on 1) the grout being of good quality and 2) the grout has to uniformly and completely fill the duct or anchorages.

Poor quality grout will produce excessive water generation or “bleeding” as it cures. Concrete used with traditional rebar also bleeds as it cures but that water can evaporate because the outer concrete surface is atmospherically exposed. However, the grout in post-tensioned, bonded usage is confined in the duct so the bleed water cannot evaporate. Without evaporation bleed water collects inside the duct at certain areas until it eventually is absorbed back into the grout. A void is then created at the point where the water was initially located. Voids permit more rapid transport of aggressive ions, oxygen and external water to the strands if there are openings in the duct.

Grout too often does not completely fill around and between the strands in bonded, post-tensioned concrete. This is usually because of lack of knowledge or quality control during the grout injection process. If there are openings in the duct(s) then large exposed steel areas may exist that permit easy entry of the components necessary for corrosion. This is similar to what occurs with excessive bleed water from poor quality grout but here the process occurs on a much larger and damaging scale. Corrosion is especially severe if there are adjacent freely exposed areas of strands adjacent to completely encased strands. In this situation an electrochemical concentration cell (essentially the same as in crevice corrosion) is generated between the freely exposed steel and those areas fully covered by grout<sup>(13)</sup>.

Often corrosion on wires in bonded, post-tensioned construction occurs at a stressing anchorage assembly shown in Figure 3. This occurs when chloride ions, water and oxygen diffuse through the outer concrete above or the grout in the anchorage and enter the anchorage or duct. These chemical components can then reach the strands if there are unfilled spaces around and between them. Entry is much more likely if the anchorage is located directly below or nearby a leaking expansion joint. If possible, expansion joints should never be located near anchorages. Recommendations for securely sealing stressing anchorages at all possible entry points and assuring that grout fills all possible cavities in the duct are not always achieved.



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### **Corrosion Control Methods for New Construction**

Mitigation of bridge corrosion can take many forms and is dependent on several variables. The biggest factor in choosing an approach is whether the engineer is dealing with new construction or how to best repair or remediate an existing bridge that is (or may be) experiencing active corrosion. There may be more practical options for new construction but many of these methods also are used during remedial work. In both cases control measures also differ with the type of bridge construction, i.e., traditional rebar in concrete, exposed structural steel in the superstructure and pre or post-tensioned concrete members.

In accordance with the ACI reference <sup>(9)</sup> corrosion control methods will be considered in terms of three areas: design and construction methods that improve the inherent resistance of the Portland cement concrete; surface treatments on the concrete that minimize the entry plus inward transport of the chemical components necessary for corrosion of reinforcement and techniques that directly minimize steel corrosion itself.

A vital step in logical repair and remedial work is to thoroughly evaluate the general conditions and extent of the existing corrosion damage before choosing appropriate repair, remedial and control methods. Therefore the range of inspection and condition evaluation techniques will be reviewed in a section to follow on assessment methods.

#### *Corrosion Control for New Construction with Traditional Rebar*

There are some fairly well-known actions for new construction that often may not be feasible during remediation. The first is to keep the water-to-cement volume ratio (w/c) during mixing of the concrete as low as practical. Less water in the mix will result in smaller capillary pore sizes and other beneficial characteristics of the pores in the finished concrete. The positive result is that it will be more difficult for chloride ions and oxygen to diffuse inward, reach the steel and start corrosion. However, with more cement and less water in the mix the concrete's flow ability will decrease and become more difficult to work. Adding plasticizers is one solution as will be discussed along with other admixtures that may be added to the initial concrete mix. Another beneficial action is to specify some minimum depth of concrete cover over the outer mat of rebar. This will result in a longer period of time between installation, possible first appearance of corrosion on the rebar and possible concrete surface damage.



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As general guidelines, the ACI standard 201.2R recommends a depth of concrete cover of 2-inches (50 mm) if the w/c ratio is 0.40 and a depth of cover of 2.5-inches (65 mm) if the w/c ratio is 0.45.

Another essential but often overlooked action during the design stage is to carefully specify details related to water retention in all areas of the bridge. These design details are vital to the incidence of corrosion. Pooled water should of course always be prevented by considering the locations and numbers of drains and scuppers that will be installed on the bridge deck and elsewhere. Further these components and downspouts should be generously sized not just to handle the expected water run off but also to minimize restriction of flow due to collected dirt and debris from passing traffic. Sharp angles in drain downspouts also clog more easily and should always be avoided.

Concave corners in concrete members and in attached steel members as well as partially open areas, i.e., crevices, between adjacent concrete and/ or steel components should be avoided where possible. These features will catch dirt and debris that in turn will retain water and promote its entry into the concrete. Where possible specify sloped surfaces so that these features can be fully drained. Specify that all crevices be sealed closed.

Leaking expansion joints on the deck of a bridge are a primary source of corrosion of both concrete members below and structural steel in the substructure. Deicing salts, sand and debris all can travel along with leaking water. Many types of sealed expansion joints are available. Each has advantages and disadvantages. A thorough search of the options should be completed to see which can meet their primary objective for the specific bridge but also provide the best life-cycle resistance to leaking. Ideally initial cost should not be the primary criterion for making a selection.

Another important factor to consider during the design phase of new construction is how inspection and maintenance will be done during the service life of the bridge. Consider what inspection and maintenance will be needed and, if possible, consider how to maximize accessibility to accomplish those tasks. Often this is overlooked and simple tasks that would extend the life of a bridge are not done because accessibility is impossible or too expensive for inspections to be done regularly.



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Many types of admixtures may be used in the concrete mix –some for corrosion control and some for other reasons. Some of the more commonly used types for corrosion control are briefly described as follows:

Chemical corrosion inhibitors – These are compounds that are added to the mix water prior to placement of the concrete. They interact in the cast concrete to beneficially alter the normal anodic or cathodic reactions or both so as to mitigate corrosion. They may also act to take up or scavenge oxygen and thus greatly reduce the cathodic oxygen reduction reaction on the steel. There are several types but calcium nitrite is very effective and is the most widely used.

Crack reducers – Concretes with very low w/c ratios are beneficial for corrosion control but they also may be prone to forming surface cracks that accelerate corrosion. This may be countered by adding nonmetallic fibers or microfibers to the concrete mix. Their presence tends to retard crack initiation and growth.

Superplasticizers – Also related to the beneficial use of low w/c ratios is the use of superplasticizers. Lower w/c ratios are desirable so far as corrosion control since they decrease pore size but they also lower the flow ability of the concrete. These additions significantly improve flow ability even when a low w/c ratio is used. Classes of compounds known as water-reducers serve the same function as superplasticizers but they are less effective.

Permeability reducers – Certain polymers may be added to the mix water to decrease the rate of diffusion of aggressive ions from the concrete's surface towards the rebar. The most common of these are latex polymers to result in latex modified concrete.

Mineral additions – These include fly ash, blast furnace slag and silica fume. These admixtures reduce and refine the porosity of the concrete and thus make chloride ion diffusion through it more difficult. Silica fume has another significant benefit – it greatly increases the electrical resistivity of the finished concrete. Higher values of electrical resistivity are associated with lower rates of corrosion because of the electrochemical nature of the necessary corrosion reactions. However, these admixtures must be used carefully. They have the propensity to lower the pH of pore water and the change may be enough to decrease the threshold concentration of chloride ions necessary to initiate corrosion on steel. Thus corrosion can start sooner.



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A second category of corrosion control for new construction of tradition rebar concrete bridges is surface sealers to minimize the diffusion of aggressive ions and water through the concrete to the embedded steel. The sealers act primary to minimize concrete porosity. They may form a visible surface film as a barrier or they may penetrate a distance into the concrete and become invisible on the surface. Sealers that penetrate have generally had more success. The following types of penetrating sealers have gained some acceptance:

Boiled linseed oil thinned with mineral spirits as a sealer – This option has been widely used for many years because it is relatively inexpensive and easily available. However, it can run off or leach out of the concrete and then be a source of environmental pollution.

Silicate-based sealers – The silicates react with lime, alkali and moisture in the concrete to seal capillary pores and thus minimize inward diffusion of ions.

In general, it has been found that penetrating sealers are most effective if they are applied within three to six months after final bridge construction. Then they should be reapplied every five years.

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The third category of control for new construction of rebar concrete bridges addresses what is used for the rebar itself and other measures related directly to the reinforcement. The most commonly used rebar options are uncoated steel (per ASTM A615), epoxy coated steel and galvanized steel. Stainless steel rebar may be considered for very severe environments. To a lesser extent cathodic protection (to be discussed) may be an option for new construction but is much more commonly used as a remedial measure.

Selection decisions in this area are not as straight forward as one might imagine. This is because other corrosion control practices in new bridge construction likely have at least an equal influence on the probable life of embedded rebar besides the type of material chosen or how it will be treated otherwise <sup>(7)</sup>. These other practices include widespread current recognition and use of low w/c ratios and specification of minimum depths of concrete cover over the rebar previously reviewed.



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The Virginia DOT did a study <sup>(11)</sup> of bridge decks in that state at a variety of locations and severities of corrosive environments in which low w/c ratios and at least minimum recommended depths of concrete cover were used. All the rebar used was uncoated steel. It was estimated that less than 25% of the total number of very differently exposed decks would require rehabilitation after 100 years of use. Included were bridges in marine environments, areas of high deicing salt usage, high traffic rates as well as rural areas with little use of deicers and low traffic volumes. It is unknown how universally applicable these results are but it does make the point that both w/c ratio and depth of cover are very important factors in achieving long-term resistance to corrosion of bare rebar.

Following are brief summaries of the most commonly used rebar materials:

[Note that the approximate initial cost information provided is 1999 data from reference <sup>(11)</sup>. It is expected that the current, relative differences in material costs are likely to be similar.]

Bare carbon steel as per ASTM A615 – no added information is provided.

Epoxy-coated steel as per ASTM A775 – This option, often known as ECR, epoxy-coated rebar, has had widespread usage and in many cases has been very successful. The most common problem has been damage to the coating during handling and placement where nicks of the coating have occurred. This is in spite of the fact that most specifications for ECR provide requirements to prevent coating damage as well as how to make field coating repairs prior to final placement. These requirements must be strictly followed. Unrepaired coating defects cause corrosion at those spots but also that corrosion leads to adjacent, larger damage because of loss of coating adhesion on adjacent ECR areas.

Another potential problem <sup>(7)</sup> is caused when the top or outermost mat is ERC and second mat deeper in the concrete is bare steel. Apparently the rationale in that design, in addition to being less expensive than making both mats ERC, is that the second mat has more than enough concrete cover so that a threshold concentration of chloride ions to start corrosion will never reach the second mat of bare steel during the anticipated life of the bridge.

The problem with using two types of reinforcement is that both mats will be electrically continuous at some points even if special chairs and wires between them are used to prevent this. When corrosion eventually starts at a coating nick on the ERC mat this becomes a very small anodic site in a galvanic corrosion cell (or couple) with the entire bare steel rebar at the site



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because they are connected electrically. An aggressive galvanic corrosion cell is formed. One characteristic of galvanic corrosion is that if the anodic metal's surface area is small relative to the area of the cathodic metal in electrical contact with it, then accelerated corrosion will occur on the anodic portion of the couple. The cost of rapid corrosion and then epoxy coating failure due to loss of adherence on the ECR will soon exceed any savings created by use of two types of reinforcement.

The initial cost of ERC itself, without considering other cost factors, is approximately 30% more than bare steel rebar.

Galvanized steel rebar as per ASTM A767 – This possibility has been used for many years. It has the advantage of offering both a barrier to corrosion by the coating itself and in that zinc also acts as a sacrificial anode to protect substrate steel should spots on the steel become exposed. A galvanic cell is then created with the steel substrate as the cathode in the cell. The galvanized coating is much more resistant to damage by handling versus the epoxy on ERC. However, best performance requires that cut ends and weld splices that remove the zinc be repaired using ASTM A780. Galvanized rebar's performance has been variable in different field applications. It is not particularly effective in very severe corrosive conditions particularly if there are outer cracks in the concrete.

Just as for ERC, galvanized rebar should not be used in a second rebar mat with bare steel in the same concrete to avoid a destructive galvanic corrosion cell.

The initial cost of this galvanized material itself, again as for ERC, not including other cost factors, is approximately two times that of bare steel rebar.

Solid stainless steel and carbon steel clad with stainless steel - These options provide resistance to chloride ion corrosion that exceeds by orders of magnitude the resistance of bare steel. They also have the major advantage of being essentially immune to handling damage that a coated rebar has. They are used rarely and only in the most severe marine exposures because of their very high costs relative to other products. The initial material cost itself is approximately 6.6 and 2.5 times greater, for solid stainless and clad stainless, respectively, than bare steel. This explains the rare use of these products as rebar other than for exceptional situations. However, stainless steels are often used in fully exposed bridge components such as hinged arches and related components in marine areas subject to splashed seawater. In those applications even more



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expensive and resistant stainless alloys are sometimes used rather than the common Types 304 or 316.

*Corrosion Control for New Construction with Exposed Structural Steel* <sup>(12)</sup>

Proper surface preparation of the steel prior to applying the protective coating is the best insurance against coating system failure. If contaminants remain on the surface prior to applying the first coat of the paint system, loss of adhesion to the substrate will eventually occur and then multiple types of coating damage may follow. Most structural steel specifications for bridge coating systems require preparation known as “near-white blast cleaning” as per the Society for Protective Coatings (SSPC) standard SSPC-SP 10. Dry abrasive blasting is preferred over wet blasting. SSPC –SP10 is applied to larger surface areas while power tool cleaning is used for smaller and especially corner or more confined areas as per SSPC-SP-11.

For many years a three layer coating system using an inorganic zinc (IOZ) primer as the first coat <sup>(12)</sup> has been used on steel exposed to the atmosphere. When properly specified and applied the IOZ acts as a protective barrier to corrosion and also as a sacrificial anode material to cathodically protect the steel substrate. After proper surface preparation, the typical three-coat system consists of IOZ with a dry film thickness (DFT) of 2.5 – 3.5 mils (one mil is one thousandth of an inch) equal to 65 -90 microns plus high-build polyamide epoxy at a DFT of 4 – 6 mils equal to 100 -150 microns and, finally, a topcoat of 1.5-2.5 mils equal to 38 -63 microns of polyurethane-acrylic. Uncovered epoxy is subject to chalking damage by UV light and the polyurethane top coat acts to prevent that damage as well as maintain gloss and color and abrasion resistance.

Small non-flat areas such as corners, sharp edges and difficult-to-reach areas on the steel superstructure are difficult to thoroughly coat using sprayed application. If possible, coating for these areas should be applied manually. The inability to do this likely is the reason for more frequent coating failures on bridges with truss and girder construction. Such bridges have more of these difficult to coat features.

It is very important that coating specifications explicitly detail all application requirements as specified by the coating manufacturer and general good practice. In addition use of a certified coating inspector during field application of the coating system is money well spent.

Weathering steel often can be another option versus traditional coated steel. However, the precautions previously reviewed need to be followed for successful use. In summary these steels



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should not be used in high chloride concentration applications. In addition, the patina can form, become stable and continue to be protective only if there are regular cycles of wet and dry conditions.

*Corrosion Control for New Construction with Prestressed Steel*

In general most of the methods of corrosion control that apply to traditional rebar in concrete also apply to both pre and post-tensioned prestressed construction. As previously reviewed these include w/c ratio and depth of concrete cover guidelines, design to maximize drainage and to address expansion joint leakage, use of admixtures in the concrete and treatments to the outer concrete surface. Both types of prestressed steel construction generally have more depth of concrete cover and thus they have this basic advantage over traditional rebar. Epoxy coated high strength steel wire (per ASTM A882) used for strands is also sometimes used in pre-tensioned construction. The A882 standard requires a much greater epoxy thickness compared to the value required by A 775 that applies to traditional ECR. This is because the pre-tensioned strands will be elongated and steel wedges used in the stressing anchorages bite into the surface of the strands. Both of these effects require a thicker and more ductile surface coating to prevent exposing the strand surface below.

The Federal Highway Administration (FHWA) prohibits the use of galvanized high strength steel strands for bridges for both pre and post-tensioned construction<sup>(13)</sup>. This is because atomic hydrogen H<sup>0</sup> (not molecular hydrogen, H<sub>2</sub>) is able to enter steel. When this occurs in high strength steels hydrogen embrittlement and eventual brittle failure is likely. Atomic hydrogen is created during the corrosion reactions occurring when high strength steel is cleaned by acid pickling before being galvanized and also it may be created during the galvanizing process itself without close manufacturing controls. Hydrogen embrittlement does not occur in the much lower strength steel that is used for traditional rebar so it can be galvanized.

Typically new construction corrosion control for post-tensioned, prestressed steel is inherently more robust than in the pre-tensioned case because of the differences in their configurations. That is due to the barrier provided by the duct in all post-tensioned construction and the additional barrier provided by internal grout in the bonded, post-tension class commonly used for bridges. All the measures reviewed to mitigate corrosion for traditional rebar construction also can and should, if possible, be applied to both pre and post-tensioned configurations.



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Stressing anchorage assemblies (shown in Figure 3) in bonded post-tensioned construction must be properly filled with grout and sealed at all possible entry points. Expansion joints should be kept away from anchorages if possible.

**Inspection and Assessment Techniques Prior to Repair or Remedial Work on Existing Bridges**

These measures apply to steel reinforcement contained in concrete, i.e., traditional rebar and prestressed steel structures.

Inspection and assessment areas are discussed in detail in the National Association of Corrosion Engineers (NACE International) Standard Practice SP0308-2008<sup>(14)</sup> plus ACI 222R<sup>(9)</sup> and ACI 222.2R<sup>(13)</sup>. Following are summaries of the classes of inspection and assessment methods plus specific techniques that may be used.

- Visual inspections & delamination surveys
- Location of reinforcement and concrete cover measurements
- Assessment of the nature of the concrete itself (several aspects)
- Corrosion potential and corrosion rate measurements

Visual inspection typically also includes delamination surveys. The items to note in the visual inspection include rust staining, surface cracks, spalling and scaling. The locations of partially or fully blocked drains and leaking expansion joints relative to the any deficient areas should be recorded along with all findings. Areas for removal of concrete cores, exposure of rebar or surface coatings or additional testing can then be correlated with the gathered visual information. Concrete delamination may be detected on relatively small areas, i.e., about 1000 square feet or less, by sounding with a hammer or by chain dragging. The validity of these traditional methods greatly depends on the experience and skill of the person using the techniques. Larger surface areas are typically surveyed much more efficiently with vehicle-mounted specialized equipment, e.g., ground penetrating radar or IR thermography, using knowledgeable contractors that gather, interpret and record delamination data.

The actual area of delamination often is found to be 40% or more larger than the area detected by the survey. This should be factored into making budget estimates for the cost of needed repairs. This difference between survey results and eventual reality is caused by inaccuracies of the



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detection technique used and the fact that corrosion will continue to do more damage during the typical extended period between the completion of the survey and when the repairs are made.

The locations of traditional rebar and their depth of concrete cover are generally determined non-destructively using a magnetic-based instrument known as a covermeter or pachometer. In using this instrument it is usually necessary to know the sizes of the specified rebar to get accurate results. However, some of these devices can provide size estimates as part of their function. The data collected should be checked against the specified cover depth and correlated to deficient areas detected during the visual inspection.

Multiple physical and chemical characteristics of the concrete itself may affect the extent of reinforcement corrosion that has or may soon occur. The primary areas that should be considered are the following:

- Laboratory petrographic analyses of cores
- Chloride ion concentration and concrete permeability via ion depth profile
- Concrete permeability
- Carbonation profile
- Electrical resistivity

The report of petrographic analyses by a qualified expert can indicate several factors important to corrosion. These include the presence of subsurface cracks, freeze/thaw damage, estimation of the w/c ratio, evidence of chloride or carbonation induced damage, the degree of consolidation, the presence of specific admixtures and probable future performance. The ASTM standard C 856 is applicable to these evaluations.

Measurements of chloride concentration at various depths into the cover thickness will indicate general trends in rates of corrosion that have or likely will occur and also the approximate permeability of the concrete. The NACE reference<sup>(14)</sup> refers to three AASHTO standards, i.e., T259 and T277, plus an ASTM standard, i.e., C 1202, that address how these analyses can be done.

Carbonation affects the alkalinity and thus the pH of affected concrete. As previously discussed, this may be a concern in certain atmospheric environments because if the normal pH of cured concrete, i.e., about 13, is reduced significantly near the rebar the normal passive condition of the steel can be lost. The possible presence and approximate depth of carbonation can be



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approximated in the field. Specific chemical compounds (acid-base indicators) are sprayed onto surfaces of freshly broken off samples of concrete that include the outer surface in profile and the area below. Interaction of the spray on the concrete causes color changes that can indicate the presence of and depth of carbonation. However, only pH values less alkaline than about 10 can be detected. Therefore smaller changes in pH, between 13 and about 10, cannot be detected.

DC electrical current must flow through the concrete electrolyte for corrosion of steel reinforcement to occur. Resistivity of the concrete is inversely proportional to the corrosion rate that may occur. Significant corrosion is said to be unlikely when the resistivity is 8,500 to 12,000 ohm-centimeters<sup>(9)</sup>. However, the source <sup>(15)</sup> below assesses the relative meaning of resistivity values somewhat differently. Thus it is important to be aware of the note of caution below the tabulated data.

Resistivity of concrete can be measured by several methods as described in the NACE SP 0308-2008<sup>(13)</sup> with advantages and disadvantages of each. Researchers <sup>(15)</sup> in the UK have developed approximate empirical relationships between the level of concrete resistivity (measured using the Wenner, 4-probe method normally used for soil resistivity but modified to make measurements on a concrete surface) and the estimated corrosion rate of embedded steel that is in a depassivated, possibly active corrosion condition.



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Three classes of concrete resistivity and their effects on rates are tabulated as follows:

<b>Level of Concrete Resistivity (Units of <math>k\Omega\text{-cm}</math>)</b>	<b>Approximate Corrosion Rate (Relative values)</b>
<b>&gt; 20</b>	<b>Low</b>
<b>10 to 20</b>	<b>Low to Moderate</b>
<b>5 to 10</b>	<b>High</b>
<b>&lt; 5</b>	<b>Very High</b>

\*(Work by J.P. Broomfield and P. Langford)

**Caution:** It is important to recognize that measured resistivity indicates only the capacity of a given concrete to allow corrosion. It does not mean that active corrosion has actually started or, if started, that the maximum possible corrosion rate is occurring. Other governing factors, e.g., the availability of a sufficient concentration of chloride ions, water and oxygen at the level of the rebar, are essential. Resistivity information should be used in conjunction with other assessment data and not used apart from other critical factors. As in all assessments, the overall knowledge and practical experience of a corrosion expert with experience in concrete is invaluable in coming to overall conclusions using all forms of evaluation employed.

A critically important method of assessing the possible existence of on-going corrosion of traditional bare rebar is to conduct corrosion potential surveys. These provide indications of specific areas that may have active corrosion versus those that do not. Making comparisons of potentials in different areas surveyed over time is the key to identifying areas of concern. In addition tracking changes in measured values in possibly “problem” areas is essential.

Using the simplest method, corrosion potential measurements are taken by electrically connecting the positive terminal of a high-impedance electric multimeter to embedded rebar while the negative, or common terminal of the meter is connected to a reference electrode (also called a half cell) on the outer surface of the concrete as shown in Figure 4. The magnitude of the



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reading obtained indicates whether or not active corrosion is probable on the rebar under and near the given location of the reference electrode. Regular, multiple surveys are vital for assessment to establish comparisons versus time. ASTM C876 applies to this procedure.

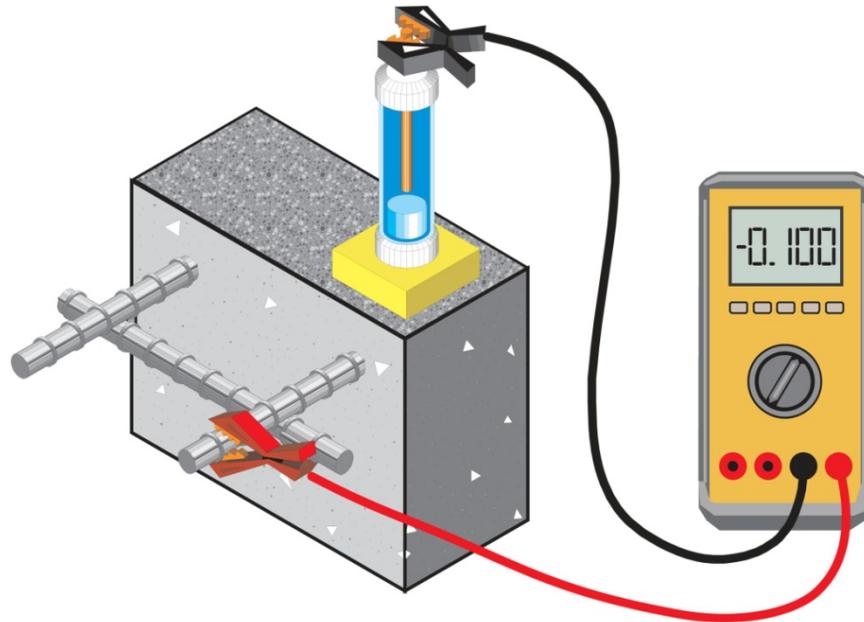


Figure 4

Schematic illustration of the measurement of corrosion potential of embedded, bare steel reinforcement on a concrete surface at one location. A copper-copper sulfate reference electrode is being used and a high impedance multimeter is reading DC voltage that indicates a low probability of corrosion. (After illustration from Reference<sup>(7)</sup>, page 573)

Important application limitations of potential surveys should be noted. They do not provide useful data if: 1) the concrete is coated with asphalt and 2) epoxy-coated rebar (ECR) are used. See also the effect of depth of concrete cover discussion starting at the bottom of page 31.

A corrosion potential survey, i.e., mapping, is best done on a grid pattern with changing placements of the reference electrode so as to obtain a potential (voltage) measurement at each reference location. The survey results can then provide a map of areas with low versus high probabilities of corrosion of the embedded steel below.



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The data from a given survey is not always easily interpreted. It is very important that persons that perform and, especially, that analyze the data obtained be fully knowledgeable of the corrosion of steel in concrete plus the uses and limitations of this technique. It is highly recommended that the personnel that perform these surveys and analyze the data be certified as competent for the specific tasks. Generally this will mean those personnel should be certified by NACE International (by both practical experience and examinations) as a Certified Cathodic Protection Technician who is supervised by a Certified Cathodic Protection (CP) Specialist having specific experience with corrosion of steel in concrete. The CP Specialist analyzes the data and develops conclusions. Use of the technique by unqualified personnel can, at best, generate misleading results.

It is essential that all embedded rebar be electrically continuous to provide useful corrosion potential data. The design locations of all rebar first must be established (via specifications or as-built drawings) and then continuity has to be confirmed by field measurements.

Another mandatory requirement in C876 that must be followed is often neglected. The reference electrode(s) must be calibrated just prior to use in each corrosion potential survey. The commonly used copper-copper sulfate references cannot provide consistent results unless this is done.

If the concrete at a given location is either dry or wet from one survey to the next (a common occurrence) then, the measured potential values likely will vary considerably. Temperature and rain or clear weather conditions in the period before each survey must be recorded to properly evaluate data from different surveys while looking for trends. This pre-survey weather recording period should be at least several days and not just the day of the present survey.

Figure 4 is intended to illustrate very simply what is involved in measuring corrosion potentials. Clearly this method that requires moving the reference electrode many times and manually recording data is laborious. It was done this way previously and it still is used for surveying small surface areas. However, for large areas the current method is to use specialized equipment consisting of multiple reference electrodes on a wheeled assembly that includes capacity to simultaneously measure multiple voltages. Data are automatically recorded and stored. Values obtained for large areas with 6-inches between grid detection points can be made fairly rapidly. Figure 5 illustrates the general set-up.



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Figure 5

Photograph of the performance of a rapid corrosion potential survey of embedded, bare steel reinforcement on a concrete surface at several locations. Multiple reference electrodes plus the multimeter function are contained in a wheeled assembly with automated data recording & storage. (Used with the permission of, Mr. Jack Tinnea, from his article, Reference<sup>(16)</sup>)

The ASTM standard test method C876 provides essential procedures for conducting corrosion potential surveys and using the data. The following approximate guidelines apply from that standard for interpreting data:



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<b>Rebar Potential Values (Units of Volts versus Cu-CuSO<sub>4</sub>)</b>	<b>Corrosion Status</b>
<b>&gt; - 0.20</b>	<b>&gt; 90 % probability of no corrosion</b>
<b>Between - 0.20 and - 0.35</b>	<b>Uncertain</b>
<b>&lt; - 0.35</b>	<b>&gt; 90% probability of corrosion</b>

**NOTE:** Use of a Cu-CuSO<sub>4</sub> reference electrode for measuring the corrosion potential of steel in concrete will result in negative voltage data with the rebar connected to the positive terminal of the multimeter as in Figure 4. However, it may be simpler to consider absolute values of corrosion potentials, i.e., small absolute values indicate a low probability of corrosion while large absolute values indicate a very high probability of corrosion. Potentials in the middle range are not definitive. As always, this assessment parameter should be used by a specialist, along with other assessment parameters, to gain the most meaningful conclusions.

It is also important to understand that corrosion potential values don't indicate corrosion rates but only the possibility that corrosion is occurring if the measured values fall in the middle or the bottom ranges provided in C876.

The best use of potential surveys is to complete them on a regular, repeating cycle. This allows analyses of the data for *trends* to determine if the status of the rebar in given areas are remaining generally stable OR are trending into more corrosive ranges. A one-time, independent survey without multiple evaluations will not provide very useful results. There will always be variations but the key is to see if the potentials of possible problem areas are remaining approximately stable like nearby “safe” areas OR are the possibly “hot” areas moving into dangerous directions with time.

In assessing corrosion potential survey results it is also important to realize that the depth of concrete cover over bare rebar affects the measured value of potential obtained at the surface of the concrete. However, it is the corrosion potential on the steel surface that is the significant factor in influencing whether a passive or active corrosion state exists. The concrete cover



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creates electrical resistance between the steel and the outer concrete surface so that the potential (voltage) measured on the concrete's surface is less negative than the most important value directly on the steel. Lateral movement of the reference electrode on the concrete and away from a position directly over a corroding area also affects corrosion potential measurements.

This is illustrated schematically by Figure 6. The lines shown in the concrete cover indicate potentials (and current flow) at various depths. If the concrete cover is at an "A" depth there is a smaller difference between the outer concrete potential and the value at the corroding rebar's surface but there is a larger difference if the cover is at the "B" level. This effect – at any depth of cover – is greater if the concrete is very dry because then its lack of moisture increases its electrical resistivity.

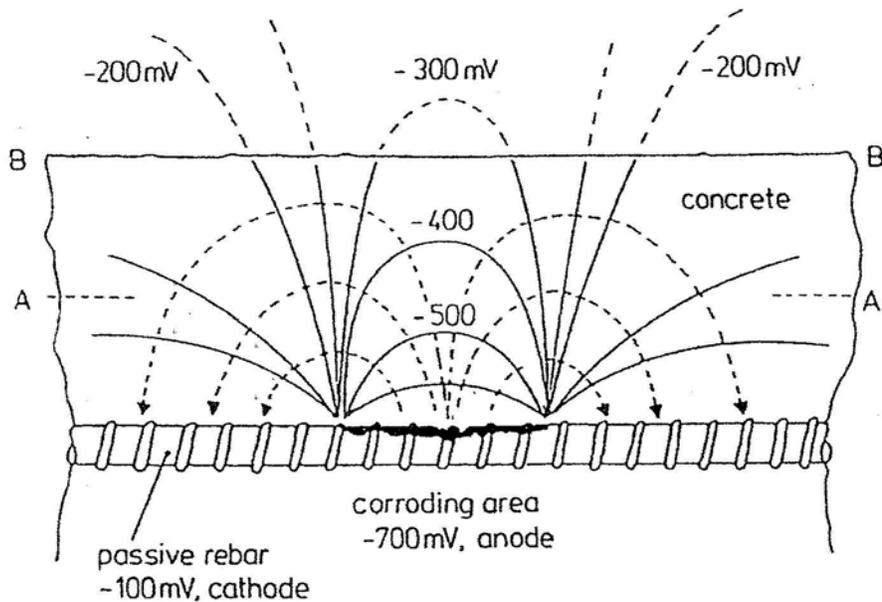


Figure 6

Schematic illustration of lines of constant corrosion potential of bare, steel reinforcement with two thicknesses of concrete cover. Also shown are the effects on surface measurements taken directly over or laterally away from the area of metal loss. (Authorized reprint from the American Concrete Institute [ACI] publication's Reference<sup>(13)</sup>, page 25)



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In traditional rebar construction the typical concrete cover is usually much less than the cover over bare, pre-tensioned strands. For this reason measured corrosion potentials for traditional rebar are generally reliable. This is true as long as the depth of cover is about 3-inches or less. However, pre-tensioned strands likely will have more than 3-inch of cover and the reliability of potential measurements then becomes doubtful due to the greater resistance created between the concrete surface and the embedded strand(s).

Potential measurements in post-tensioned reinforcement construction are not meaningful over most of the steel's length. That is because the strands are shielded from free exposure to the outer concrete by the duct which typically is metal. Potential data obtained over the length of the duct applies to the metal duct and not the strands inside. If a plastic duct material is used potential data are also meaningless.

Potential values could be measured at the anchorage assemblies on the outer ends of the overall length of a structural member after some concrete or grout is removed. However, the data obtained would then only apply to the steel immediately at the anchorage and for a short distance from it towards the other end of the concrete member.

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Still another area for possible assessment is to attempt to measure the approximate rate of corrosion in real time. Such measurements are not frequently made and very few state transportation departments attempt these measurements. The state of Oregon is one that attempts to measure rates. The linear polarization resistance (LPR) electrochemical method is typically used for corrosion rate measurements. The problem is that many practical factors enter into obtaining meaningful data in the field although laboratory results often are successful. One of the most difficult issues is finding the specific surface area of rebar that should be used for the necessary calculation that predicts corrosion rate in units of current per square area of rebar. Defining this total area for field conditions is not as simple as it may appear yet this is critical to deriving generally accurate rate predictions. The rebar surface areas in small concrete laboratory specimens are easily defined.

Rather than attempting to obtain meaningful corrosion rate data in the field the more useful method of assessment, along with those initially discussed here, is to do multiple corrosion potential surveys on a regular schedule during the life of a bridge. Periodic evaluation is especially important during the later years of the expected service life of a structure. Each survey should be done in strict accordance with ASTM C876. Further, as has been said, all assessment



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work should be done and interpreted by a specialist contractor or in-house personnel that have both knowledge and specific experience with corrosion of all types of reinforcement in concrete. Several qualified contracting firms are available if there are no suitable personnel in-house.

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Many of the assessment techniques discussed for traditional rebar also can, in general, be applied to the pre-tensioned class of prestressed construction. However, the corrosion potentials of deeply embedded pre-tensioned steel cannot be reliably assessed due to the greater resistance of the greater concrete cover thickness as previously noted.

Post-tensioned steel assessment is in a different category. The shielding provided by the duct and internal grout in bonded construction make evaluation of the strands inside at least difficult if not impossible. Unfortunately bonded, post-tensioned construction is more commonly used for bridges compared to the unbonded post-tensioned method.

Shielding by the duct and by the grout prevent the usefulness of visual inspection on the outer concrete surface. Corrosion potential measurements on the outer concrete over most of the length of the concrete member are also meaningless because the duct – either metal or a plastic – and internal grout prevent an electrical assessment of the steel strands.

Two possible problems with bonded, post-tensioned construction is that the grout in the duct(s) may not be uniformly distributed during initial injection or voids are created by bleeding from poor quality grout as previously discussed. If concrete removal is viable, voids may be detected from outside the duct(s) by the impact-echo, non-destructive evaluation technique<sup>(13)</sup>. This procedure depends on interpretation of the propagation of stress waves caused by impacts on the duct's exterior surface that transfer in and "echo" back from the grout or void inside. Specially trained personnel and specific equipment are required to successfully complete such evaluations.

If possible it is important to evaluate the degree of filling and condition of the grout inside accessible anchorages after opening sealed areas and removing small amounts of filling material. The degree of filling can be assessed visually on the outer-most areas of these anchorages. Removed grout can be assessed by one or more of the analysis techniques previously discussed for the condition of concrete used with standard rebar. During the time that the outer end of an anchorage is relatively open, corrosion potential measurements of the strand can be made. As



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previously mentioned this will provide information on the probability of corrosion for only a short distance into the strand's total length.

The most definitive assessments of embedded, bonded post-tensioned steel require more extensive invasive methods. Limited inspection of the condition of the duct can be accomplished by carefully coring through the covering concrete to near the duct level and finishing with manual concrete removal down to the duct. This likely will be a hit and miss operation with no assurance that areas of damage on a duct are detected. The different locations of coring over the duct may logically be selected based on surface conditions more likely to cause corrosion damage to the metal duct that is typically used, e.g., at an area susceptible to water pooling or at an area under a leaking expansion joint.

While the duct inspections are being done portions of the duct may be removed so that the extent of filling by the grout and its condition can be evaluated. This is done by cutting out a portion of the duct and checking the filling by grout at that location. This will need to be done at multiple core locations to get an approximate evaluation of the efficiency of filling by the grout and still it will be a hit and miss procedure. When (or if) grout is found at a given cored and opened spot in the duct then this can be removed so that a borescope can be inserted to examine the surface condition of the outer visible wires in the strand.

As may have been done at the anchorages that were inspected, the quality of the removed grout can be evaluated using many of the techniques previously reviewed for examining the condition of traditional concrete. Any test location found that is not completely filled with quality grout needs to have that added to fill cavities in and around the steel strand to the maximum possible extent.

Obviously duct openings must be repaired and outer concrete replaced at all invasive assessment locations along the length of the duct and/or at the anchorages. Each location should be refilled with quality concrete and securely resealed.

Clearly thorough corrosion assessments of bonded, post-tensioned strands used in bridge construction can be difficult and expensive. As always only fully knowledgeable persons that are experienced with the specific issues and techniques discussed should be selected to do the work. As stated earlier (second paragraph at top of page 4), it is fortunate that this type of construction has the best record and has suffered the least corrosion-induced, structurally deficient problems among the most popular types of bridge construction.



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## **Repair and Remedial Actions**

Responsible persons have to make an early decision about whether a repair or remediation will be done for a given bridge because of corrosion-induced damage. Here the repair option is defined as action(s) that will provide immediate, but likely only temporary, correction of structural deficiencies and/or lessen the effects of corrosion. The remedial option is defined as including the necessary repair(s) but it goes further to include actions selected to reduce or essentially halt future corrosion damage over a much longer term.

The key point is that there is no “right answer” to the choice to repair or remediate. The choice will likely be made after at least some portion of a current, physical assessment program that fits the particular structure has been completed. Every application includes different variables that can and should govern what action is taken. In the repair option there are likely to be alternatives to consider just as there are alternatives for remediation. This section is not intended to advocate any universally applicable choices but to provide information and suggest a logical approach to an overall plan of action. There is no universal standard that applies to making these decisions.

There are different approaches to decision making in this area but there are few widely accepted procedures. The one cited here<sup>(17)</sup> seems logical. The opinions stated are those of the author of this course. Besides a generic plan this reference gives case histories of bridges in Florida primarily exposed to marine environments but the general methodology might also be used in a variety of bridge applications. Somewhat modified steps from the cited reference are summarized as follows:

- Review historical information
- Complete appropriate assessment surveys
- Analyze and interpret the assessment data
- Develop well-defined, technically valid options
- Complete discounted life-cycle cost analyses for all valid options
- Make the decision and proceed as indicated.

There are diverse aspects to this plan and thus several different competences are needed to perform all areas well. A team of persons with both knowledge and practical experience in the following areas should be assembled to result in the best final decision(s):



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- Structural/ civil engineers
- Concrete material experts
- Corrosion engineers and technicians
- Persons skilled in completing life-cycle cost analyses

Persons with appropriate abilities should be called in to work on a given step in the plan as needed. Ideally multiple people will be involved simultaneously on some tasks to give best collective results.

The review of historical data for the bridge in question generally will include examination of specifications, design and as-built drawings, past assessment reports, reports of past repair and routine maintenance actions and records of local environmental conditions and any unique information. If appropriate this step will also include any anticipated projections of changes in traffic usage for the future. If they are not current in their knowledge of the overall condition of the bridge, personal visits to the structure by the ultimate decision maker(s) along with members of the work team can be beneficial.

The assessment surveys to be done will depend on many factors and especially the type of structure and reinforcement method(s) employed. Assessment will cover two major areas: 1) the current structural integrity and possibly related strength characteristics of concrete at critical locations and 2) the corrosion assessment techniques that have been discussed in this course. Structural engineers, civil engineers and materials laboratory personnel should handle the first area. Expertise in these aspects is outside of the limits of this course. It is also expected that persons with these skills will take active consultative roles in the corrosion assessment tasks that will be done by persons with thorough knowledge and experience with corrosion of reinforcement in concrete.

Detailed corrosion assessment activities may include some or many of the procedures discussed previously including visual inspections, delamination surveys, actual depth of concrete cover over reinforcement compared to specified values, coring for petrographic analyses of concrete, chloride ion concentration and/or carbonation analyses, concrete resistivity measurements and corrosion potential mapping surveys. In the case of the latter ideally there will be at least some past corrosion potential survey data – taken as part of a complete maintenance program for the bridge – so that trends in corrosion potentials versus time can be assessed by the corrosion engineer.



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Both structural and corrosion personnel should be involved in analyzing all the combined results from the structural, concrete and corrosion assessments. This likely will be the juncture in the activity at which the decision will be made as to whether a repair or a remediation will be pursued. Many factors will enter into this choice not the least of which may be the current availability of funding and the anticipated long-term role for the bridge in question.

If a repair is to be the chosen action then the next steps will be to define viable alternative repair options and have a knowledgeable person complete a discounted life-cycle cost analysis (more explanation later) for each action. Many repair options are temporary and thus will need to be repeated during the remaining life of the given bridge. The estimated costs of repeating anticipated short-term repairs during the expected total service life of the bridge should be included in the cost analyses.

If a repair is the choice but yet the assessments have indicated that reinforcement corrosion has begun, or likely soon will begin, it is highly recommended that regular, periodic corrosion potential surveys (with mandatory record keeping) be started if such a program does not now exist. The cost of these corrosion surveys should not be added to the other costs in the life-cycle analysis for each repair option. This is because every repair option done when corrosion is known to exist (or is likely soon) should be accompanied by corrosion monitoring to define when further action will be needed to assure safety.

**Note:** Reference (17) briefly discusses analytical models that are intended to predict the timing of the different phases of reinforcement corrosion and damage in bridges as a part of the repair/remediation plan recommended. These have to do with the times after initial construction for a minimum threshold concentration of chloride ions to diffuse through the concrete cover, reach embedded rebar, cause corrosion to initiate and then progress until significant structural damage occurs. Some references that discuss this modeling are cited but little detail is provided. If viable, such models would be very useful in determining when repairs or remediation should be done and especially in specifying the timing of future actions for use in the life-cycle cost analyses. Such models are beyond the scope of this course.



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If after careful analysis of all assessments it is concluded that a remediation action will be done it should be clear that very often some repairs or maybe a temporary structural addition may be needed before the final remediation action. For example if the concrete surface has significant spalling or there are areas of delamination those areas would have to have concrete removed and new material cast in before adding a surface treatment to minimize longer-term future corrosion. As previously stated, the sizes of delaminated areas due to corrosion often are significantly larger than initially indicated. If pre-remedial actions differ from one type of remediation to another then the different costs of the needed repair should be included in the life-cycle cost analysis for each complete alternative.

The alternative remediation actions for longer-term corrosion control might include one, two or a mix of the actions already discussed here in the new construction section, e.g., using quality concrete with a low w/c ratio or concrete that contains desirable admixtures such as corrosion inhibitors in repair areas where old concrete is removed, if feasible increasing the depth of concrete cover in those repaired areas, using a surface sealer on those new concrete areas or filling voids in bonded, post-tensioned ducts and at anchorages. The choice among these options, as always, will depend on the specific circumstances that exist.

Three other remediation options have not yet been discussed. These are all electrochemical techniques that can be useful in many applications. They are electrochemical chloride extraction (ECE), electrochemical realkalization (ER) and cathodic protection (CP). ECE and ER are applied over relative short periods of approximately four to eight weeks during remediation. Their corrosion mitigating effects are relatively quick acting but typically will be effective only for approximately five to ten years<sup>(7)</sup>. This effectiveness period for ECE assumes that parallel actions are taken during installation to prevent new chlorides from entering the concrete.

Installation times for CP take approximately the same time as ECE or ER but once installed, and if properly monitored and maintained, CP typically will be effective against corrosion for a significantly longer period. CP's effectiveness period depends on the type of anode used and other factors but has been estimated in an early source<sup>(18)</sup> to be five to forty years. This longer period is more likely with current technology. CP is most commonly first installed during remediation of existing bridges. However, there may be a life-cycle cost benefit if CP is installed as part of new construction because often the necessary initial repairs before installing it during remediation are avoided.



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ECE and ER are described in detail in NACE International Standard Practices SP0107<sup>(18)</sup> and SP0390<sup>(19)</sup>. CP is described in detail in reference<sup>(7)</sup>. Each procedure includes applying a DC current to the reinforcement at a given electrical potential level. In most cases the three procedures are not used for prestressed construction. For the post-tensioned category of prestressed construction the strands are inside ducts and the ducts create an electrical shield that prevents the desired protection of the steel. The pre-tension category is not suitable for application of one type of CP, i.e., ICCP, because of the danger of hydrogen embrittlement. These techniques also are not used for epoxy coated steel because the coating blocks the desired effects. Each method is briefly described as follows:

When chloride ions diffuse through concrete cover and reach a threshold concentration at the surface of bare steel reinforcement corrosion initiates. As previously described this is because the normal passive film on the steel breaks down. ECE functions by driving away a portion of the chloride ions, 20 to 50% according to reference<sup>(17)</sup>, on the steel surface so that the revised concentration becomes less than the threshold concentration necessary for corrosion. Passivity of the steel is thus substantially restored.

ECE acts to accomplish its desired effect by short-term application of a relatively high level of DC current to the steel (which becomes a cathode in a corrosion cell) that is connected electrically to a temporary anode material. Potable water or an alkaline water solution is used as the electrolyte on the outer surface of the concrete. On horizontal concrete surfaces such as a bridge deck, the electrolyte covers and surrounds the anodic material and is kept in place when a temporary dam is formed around the area to be treated. The procedure is described fully in the two NACE standards cited and also in the ACI reference<sup>(9)</sup>.

ER acts to minimize the corrosion-causing effect of carbonation. Specifically it raises the pH of concrete that has been reduced by the entry of CO<sub>2</sub> gas that reacts with water to form carbonic acid. The treated concrete's normal alkalinity (pH of about 13) and the passivity of the steel are restored by ER. This is accomplished using a procedure generally similar to that used for ECE but the mechanism by which pH increases is more complex than the mechanism in which chloride ion concentration reduction occurs. The ER procedure is described in the two NACE standards previously mentioned.

Cathodic protection (CP) provides a long-term defense against corrosion but its use involves additional considerations versus the shorter lasting procedures of ECE and ER.



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CP functions by supplying direct current to the steel from one of several possible types of permanently installed anodes. The current that comes off of the steel (shown in Figure 1) as a natural consequence of the corrosion reactions is reduced to near zero. Thus corrosion of the steel is mitigated to low, long-term practical rates. This occurs because the reinforcement is polarized in a more cathodic direction away from its active, anodic corrosion potential value.

There are two types of CP - sacrificial anode CP and impressed current cathodic protection (ICCP).

The first type of CP uses an active anode material, such as zinc or aluminum, which is inherently more susceptible to corrosion than the steel it is intended to protect. This is possible because the anode has a less negative corrosion potential than steel. When the anode is electrically connected to the steel a galvanic corrosion cell is created and the anodic material corrodes and sacrifices itself to provide DC current that travels to the steel surface. The steel is then polarized in a cathodic direction, i.e., it becomes more negative, and corrosion of the steel is greatly reduced as long as there is anodic material available to “feed” the process.

Sacrificial anode CP (also called galvanic anode CP) is relatively simple in terms of the initial components needed and minimal periodic monitoring needed to keep it functioning. Thus its initial cost and maintenance costs are much less than ICCP. However, it has limitations. The anode in sacrificial anode CP can only supply a constant level of voltage and current no matter if local conditions change and more current is needed to provide protection. For example if the concrete becomes dry, its resistance increases and a higher potential (voltage) is needed to drive sufficient DC to the steel. The sacrificial anode cannot do that so corrosion can reactivate. Another issue is that eventually all of the anodic material will be consumed and has to be replaced. These are not problems with use of ICCP.

ICCP can function with a wider variety of anode materials but all are essentially permanent and are not consumed in use. Instead the anodes continually receive DC current from a special on-site component – the rectifier – and transmit that current through the concrete to the steel being protected. The rectifier combines an AC-to-DC electrical rectifying function with a transformer that decreases the converted DC voltage to an adjustable level needed with local conditions to accomplish corrosion protection. The last feature is valuable because the rectifier can be reset to supply more or less current depending on local conditions versus time. However, this type of CP can only function properly when the rectifier is “ON”, i.e., in operation, and is set at a suitable adjustment to deliver the needed level of DC current. This requires regular monitoring to assure



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continued operation and readjustment as necessary by knowledgeable personnel. There is also the cost of continual AC power input to the rectifier throughout the life of the CP system. ICCP is definitely not an install it and forget it technique.

The next vital phase of a meaningful remediation analysis plan is to complete a discounted life-cycle cost analysis for each technically viable remediation option being considered.

These cost analyses use the principles taught in the undergraduate engineering economics university course that most engineers complete. The basic concept used is the time value of money or in other words – dollars spent at different points in the future have different current or present values. The present is when the cost comparisons are being done so those future dollars have to be adjusted or “brought back” to their present values to fairly compare the different remediation (or repair) options.

These adjustments are made by multiplying each identified cost in a given option by a discounting factor. The value of each discounting factor depends on the applicable interest rate used in the organization (owner of the bridge) and on the time period between the present and when the given future cost is estimated to occur. Typically the most knowledgeable technical personnel will provide information on the several costs associated with each option and when each future cost is estimated as most likely to occur. That second aspect is often difficult but estimates must be made. Usually a financial specialist then is assigned to complete the actual discounted life-cycle calculations using information supplied to him or her.

The reliability of life-cycle cost analyses depend on obtaining the best possible input information, i.e., all costs involved in each alternative being considered and when each is expected to occur in the future. There is no guarantee that results will be definitive. However, this approach is far better than basing decisions on only the sum of initial costs in each alternative without any accounting for what the future costs and their timing will be over the expected long service life of a given bridge.

Finally, when all aspects of the repair or remediate plan are available decisions can be made and implementation started.



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