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Corrosion induced deterioration in prestressed concrete structures

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CORROSION INDUCED DETERIORATION IN PRESTRESSED CONCRETE STRUCTURES

by

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(BSc, "Politehnica" University of Timisoara, Romania, 1996)

A project report
presented to Ryerson University
in partial fulfillment of the
requirements for the degree of
Master of Engineering in Civil Engineering

Toronto, Ontario, Canada, 2009

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A State-of-the-Art report on Corrosion in Prestressed Concrete Structures

Sorin Aslau , Graduate student, 2009

Master of Engineering in Civil Engineering, Ryerson University

Abstract

Corrosion is less frequent in prestressed concrete than in ordinary reinforced concrete, but when it does occur, the results can be more dangerous and more costly. In prestressed concrete structures the reinforcing steel elements are subjected to high mechanical stresses, therefore corrosion of the tendons can lead to catastrophic consequences, by the failure of the steel and consequently of structure, or part of it, with a great potential for life losses, life disruption, and a huge economical impact. One of the most dangerous aspects is that the corrosion related fracture of the steel might lead to a sudden collapse of the structure with little or no warning. The intention of this project report is to collect and synthesize information on the corrosion of the prestressing tendons as a mechanism of deterioration in prestressed concrete.

Acknowledgements

Particular thanks are due to Dr. Lamy Amleh for her guidance and help in preparing this project report. I would also like to acknowledge my wife Nancy Kapralos and my son Daniel for their support and encouragement while I worked on this project.

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The number of prestressed concrete structures around the world has been steadily increasing for a number of years (Crane and Wether, 1997). Prestressed structures are also considered to be durable, with relatively low maintenance costs. The structural integrity of prestressed concrete structures is dependent upon the condition of prestressed steel tendons/strands embedded in the concrete. Therefore, it is the primary concern to the structure owners to properly repair and rehabilitate deteriorated prestressed concrete elements in their structure. In particular, extensive repairs due to unexpected corrosion of prestressed tendons in prestressed concrete buildings (Rahman et al, 1995) and corrosion of steel cables encased in grouted pipe/duct on cable-stayed bridges (Weischedel, 1995) have raised concern about the possibility of corrosion-induced failure of prestressed tendons in prestressed concrete bridges. While the failures of prestressed concrete structures are relatively rare, a few have occurred with the failure attributed to corrosion or rupture of prestressed tendons. (Abraham and Cott, 2002; Woodward, 1989)

In prestressed concrete structures the reinforcing steel elements are subjected to high mechanical stresses, therefore corrosion of the tendons can lead to consequences far more serious than in the case of conventionally reinforced concrete structures. This may result in failure of the steel and consequently of substantial part of it, with a great potential for life losses, life disruption, and a huge economical impact. Significant mechanical weakening may occur in prestressed reinforcement even when the same extent of corrosion in conventional reinforcement would be considered negligible, which may result in the failure of the structure. The failure is more likely to occur in tendons of smaller diameters. One of the most dangerous aspects is that the corrosion-related fracture of the steel might lead to a sudden collapse of the structure with little or no warning. Therefore, investigations to

1. Introductions

Corrosion of reinforcement of reinforced concrete structures occurs by destroying the passivity of the steels when chloride ions from deicing salts or seawater penetrate into the concrete structures. Corrosion of prestressed concrete structures is highly detrimental since prestressing wires are under significant mechanical stress.

The number of prestressed concrete structures around the world has been steadily increasing for a number of years (Chase and Washer, 1997). Prestressed structures are also considered to be durable, with relatively low maintenance costs. The structural integrity of prestressed concrete structure is dependent upon the condition of prestressed steel tendons/strands embedded in the concrete. Therefore it is the primary concern to the structure owners to properly repair and rehabilitate deteriorated prestressed concrete elements in their structure. In particular, extensive repairs due to unexpected corrosion of prestressed tendons in prestressed concrete buildings (Rahman et al, 1996) and corrosion of steel cables encased in grouted pipe/conduit on cable-stayed bridges (Weischedel, 1995) have raised concern about the possibility of corrosion-induced failure of prestressed tendons in prestressed concrete bridges. While the failures of prestressed concrete structures are relatively rare, a few have occurred with the failure attributed to corrosion or rupture of prestressed tendons (Abraham and Cote, 2002; Woodward, 1989).

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determine the causes of corrosion are essential in maintaining the require serviceability, integrity and safety of prestressed concrete structure.

Prestressed concrete it can be used to produce beams, floors, tanks where pressure on the walls is of a main concern or bridges with a longer span than is practical with ordinary reinforced concrete. Prestressed concrete is a type of reinforced concrete in which the steel reinforcement has been tensioned against the concrete. This tensioning operation results in self-equilibrating system of internal stresses (tensile stress in the steel and compressive stresses in the concrete) that improves the response of the concrete to the external loads. While concrete is strong and ductile in compression it is weak and brittle in tension, and hence its response to external loads is improved by applying a precompression. Prestressing tendons (generally of high tensile steel cable or rods) are used to provide a clamping load that produces a compressive stress that offsets the tensile stress that the concrete compression member would otherwise experience due to a bending load.

Prestressing removes a number of design limitations conventional concrete places on span and load and permits the building of roofs, floors, bridges, and walls with longer unsupported spans. This allows architects and engineers to design and build lighter and shallower concrete structures without sacrificing strength.

1.1 Background and History

The concept of tensioning the steel reinforcement so as to place the concrete in a state of compression as to counteract the tensile stress resulting from service loads has be known for many years. Although a San Francisco engineer patented prestressed concrete in 1886, it did not emerge as an accepted building material until a half-century later. Early attempts of utilizing the concept were unsuccessful because only mild steel reinforcement was available at the time. The prestressed force that could be induced in the mild steel was relatively low and it could not be sustained in the structure because of creep and shrinkage of the concrete and stress relaxation of the reinforcement.

The development of modern prestressed concrete is generally attributed to Eugene Freyssinet, who began using high-strength steel wires for prestressing in 1928. "More than any other person it was the relentless pioneering efforts of this courageous French engineer-builder who converted the concept of prestressing into a practical reality." [1]

The use of prestressed concrete gradually increased in Europe but accelerated considerably after 1945, when the need for rapid postwar reconstruction coincided with shortages of materials, steel in particular. By the time of the First United States Conference on Prestressed Concrete in 1951, it was reported that about 175 prestressed concrete bridges and 50 prestressed concrete buildings frames had been constructed in Europe and about 700 prestressed concrete water tanks had been built in North America. [2]

There are two different procedures for prestressing concrete: post-tensioning and pretensioning. In the post-tensioning procedure, which was developed by Freyssinet, the steel is tensioned after the concrete is cast. The second procedure, in which the steel is tensioned before the concrete is cast, was developed by the German engineer E. Hoyer in 1938, as a practical technique. [3]

Since these early developments, prestressed concrete has grown to be a multibillion-dollar industry in North America. In 1997, about 200,000 tonnes of prestressing steel is used in North America each year, which is about one-quarter of the total world consumption. In North America, about two-thirds of the prestressing steel is used to manufacture precast, pretensioned products, with the remaining one-third being used for post-tensioning. About 65% of precast products are structural elements for buildings and bridges, and the rest being used for architectural precast elements. Also, in Canada and United States, about 59% of post-tensioning steel is used in the construction of buildings and about 26% is used in bridges. In the rest of the world, a different pattern exists, with about 66% of post-tensioning steel being used in bridge construction. [4]

The construction of the Walnut Lane Bridge built in the United States was a most significant event. It was the dramatic groundbreaking project that showed North American

engineers the practical value of combining the compressive strength of concrete with high tensile strength prestressing steel. The design of the bridge was based on European prestressing technology and design methods introduced to North America after the Second World War. Professor Gustave Magnel (1885-1955) of Belgium gave lectures to engineers in the US and Canada in the mid 1940's to the early 1950's. Magnel's book "Prestressed Concrete" attracted considerable interest in North America of the potential for prestressed concrete.

The following examples are a very brief history of prestressed constructions in Canada, and two examples from outside the country.

Walnut Lane Bridge – 1949 Philadelphia, PA USA.

The successful completion of the first bridge structure in the US with its impressive 160 ft main-span and 74 ft end span precast prestressed concrete girders inspired many engineers on both sides of the border to closely examine the properties, benefits and design methods of prestressed concrete. Over 300 engineers from 17 states and 5 countries witnessed the formal testing to destruction of an identical girder used in the main span of the bridge.

This structure was designed by Professor Gustave Magnel of Belgium. The design specifications were basically European. The anchorage hardware used was the Magnel system, a patented system developed by the professor himself, while the prestressing steel used was 0.276 in. (7 mm) diameter, stress-relieved wire furnished by Roebbling, a Swiss-American company famous for supplying the steel cables for the Brooklyn Bridge in New York City and other suspension bridges. Note that seven-wire strand was still in the experimental stage and in limited use.

The bridge was essentially a posttensioned concrete girder structure cast on site. The girder spans were 160 ft (49 m) long, which are fairly large even by today's standards.



Fig. 1-1: Walnut Lane Bridge – 1949 Philadelphia, PA USA [5]

Mosquito Creek Bridge – 1952 North Vancouver, BC

The Mosquito Creek Bridge in Vancouver has the distinction of being the first prestressed concrete bridge built in Canada. This bridge proved to be both economical and satisfactory from a structural viewpoint. The bridge is still in service, having been widened on both sides over the years.



Fig. 1-2: Mosquito Creek Bridge – 1952 North Vancouver, BC [5]

Champlain Bridge - 1959-1962 Montreal, QC

At the time, this was the largest application of prestressed concrete in Canada. Designed in concrete and steel, the precast concrete option used 53.6 m (176 ft) long precast pretensioned girders supported on T-shaped piers for 46 spans across the St. Lawrence River and the Seaway at Montreal.



Fig. 1-3: Champlain Bridge - 1959-1962 Montreal, QC [5]

CN Tower - 1973 to 1975 Toronto, ON

Truly a Canadian landmark, the tower at 1815 ft (553 m) is the world's tallest freestanding structure. Over 1000 tonnes of prestressing steel was used extensively in the superstructure of the tower and in the foundation.

The project contains 53,000 cy (40,500 m³) of concrete. The 1450 ft concrete shaft for the superstructure has a 3-legged cross section that tapers inward. This shaft was slip-formed in about 8 months under conditions of high winds and freezing weather.

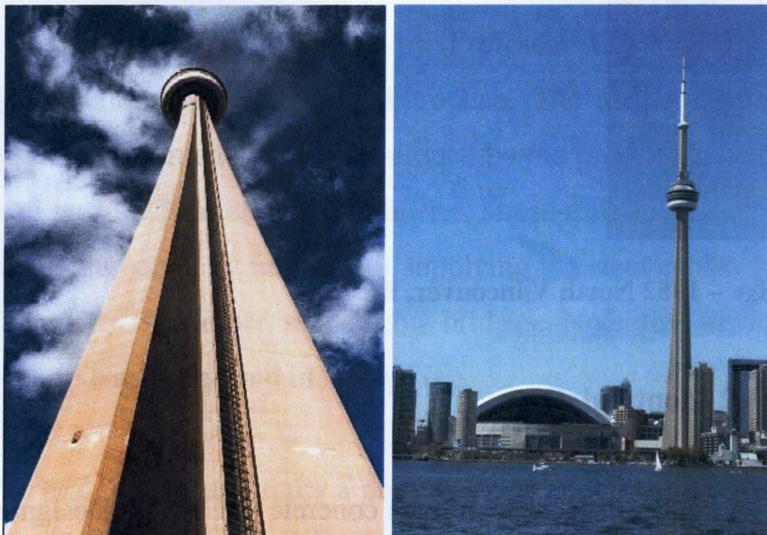


Fig. 1-4: CN Tower - 1973 to 1975 Toronto, ON [6]

Tendons were placed, secured, stressed and grouted vertically as the slip forming continued on a scale never before attempted anywhere. The tower prestressing is arranged concentrically and designed to disallow tensile stresses under all estimated dead and live loads and 50-year wind loads. The shear stresses due to wind are greatly reduced because of the tapering shape of the tower. [6]

Saddledome - 1981-1983 Calgary, AB

The stadium was built to host the 1998 Winter Olympics. The building form is a 67.7 m (222 ft) radius sphere, intersected by a hyperbolic paraboloid generating the roofline and a plane to delineate the base. This arrangement provided the absolute minimum building volume and unobstructed views of the playing surface.

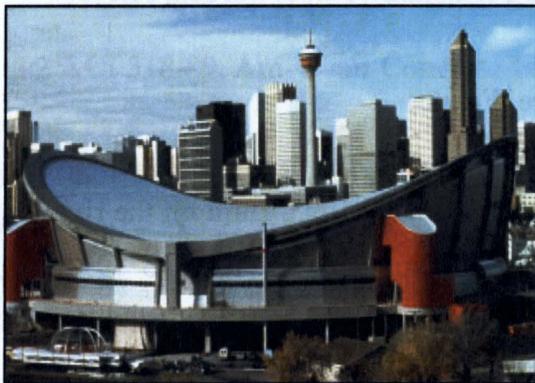


Fig. 1-5: Saddledome - 1981-1983 Calgary, AB [5]

Confederation Bridge - 1993-1997 Cape Tormentine, NB, to Borden, PEI

No accounting of the accomplishments in prestressed concrete in the 20th century would be complete without including the Confederation Bridge, a two-lane fixed-link 12.9 km (8 mile) bridge. The main bridge has 44 - 250 m spans that rise up to a navigation channel in the middle of the structure. This part of the bridge used 175 precast concrete components (hard point segments, pier bases and shafts, main and drop-in girders).

The bridge used the latest in high performance concrete technology and is designed to achieve a 100-year service life. The bridge was built in record time using massive precast concrete segments, the largest being the 160 m (525 ft) long pier sections that weigh 7500 tonnes.

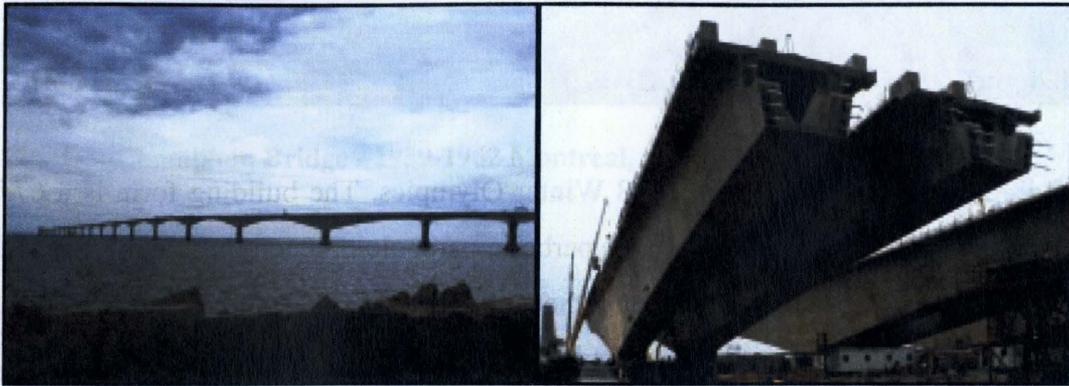


Fig. 1-6: Confederation Bridge - 1993-1997 Cape Tormentine, NB, to Borden, PEI [5]

The Normandy Bridge – 1988-1994 Le Havre, France

The Normandy Bridge has been inaugurated on January 20 1995. Built on the river Seine, near Le Havre, France, the construction period has spread from 1988 to 1994. At the inauguration date, it was the world record of the cable-stayed bridges with an 856 m (2808 ft) span and a total length of 2141 m (7024 ft).



Fig. 1-5: The Normandy bridge, near Le Havre, France [7]

The two towers, as high as the Montparnasse Tower (Paris), are the world bridge's highest ones in prestressed concrete and weigh 20,000 tonnes each. The deck is at 52 m (170.6 ft) over the highest water.

1.2 Standards

1.2.1 CSA A135-1962, Standard for Prestressed Concrete

This first standard code was published in 1962. The 25-page 3 mm thick document overcame the last objections to using prestressed concrete in Canada. [8]

The standard was included in the CPCI Design Handbook by Cazaly & Huggins. Published in 1964, this was the first precast handbook in North America.

1.2.2 ACI 318-89, American Concrete Institute's Building Code

Back in 1949-1950, when the Walnut Lane Memorial Bridge was being constructed in Philadelphia, Pennsylvania, prestressed concrete was not recognized by the ACI Building Code or by any other official jurisdiction in the United States. [9]

Based primarily on the work of Eric L. Erickson, in Louisiana, the U.S. Bureau of Public Roads (the precursor of the Federal Highway Administration) published in 1954 the *Criteria for Prestressed Concrete Bridges*. [10] This document was to have a major impact on the future of prestressed concrete, especially for bridges. One very important outcome of this document was the inclusion of precast, prestressed concrete provisions in the AASHTO Standard Specifications for Highway Bridges and the more recent LRFD Design Specifications. With the founding of the Prestressed Concrete Institute in 1954, the early precasters found it necessary to develop their own set of "code provisions" for pretensioned concrete products. This document came in the form of a three-page pamphlet titled "Specifications for Pretensioned Bonded Prestressed Concrete," published on October 7, 1954, and made effective on November 7, 1954. [11] Then, in December 1959, the PCI

announced that its Standard Building Code Committee (T.Y. Lin, chairman) had developed a "Standard Building Code for Prestressed Concrete". Prior to official adoption, this document was open to public discussion with a deadline for comments by March 1, 1960.

With the proliferation of precast/prestressed concrete in the 50s and 60s, the American Concrete Institute felt it was desirable to have prestressed concrete covered in the ACI Building Code, which until then had provisions only for reinforced concrete, so that a practitioner would have to deal with one code only. ACI approached the PCI to explore the possibility of PCI refraining from publishing its own "code" on prestressed concrete, provided it received proper representation in the ACI 318 Building Code: Chapter 16 on Precast Concrete and Chapter 18 on Prestressed Concrete. [9]

ACI 318-02, the 2002 edition of the ACI 318 standard, for the first time, includes design provisions for precast concrete structures located in regions of moderate to high seismic risk or assigned to intermediate or high seismic design categories (C, D, E, or F).

1.2.3 CEB-FIP Model Code, European Concrete Code

In 1952 The Fédération Internationale de la Précontrainte FIP (International Federation for Prestressing) was inaugurated at an international meeting held in Cambridge, England.

A common initiative by FIP and CEB created a 'Mixed CEB-FIP Committee for Drafting of Recommendations for Prestressed Concrete' took place in 1962.

And in 1970 The second edition of what later became the "CEB-FIP International Recommendations" were ratified by the two associations, covering structures in plain, reinforced and prestressed concrete, followed by elaboration of numerous accompanying manuals in the following years. [12]

1.3 Objective

The objective of this project report is to collect and synthesize information on the corrosion of the prestressing tendons as a mechanism of deterioration in prestressed concrete. It will highlight the importance of protection against corrosion and the catastrophic consequences that can result due to improper design, construction and maintenance of the prestressed concrete structures. It will also explain the particularities of corrosion in prestressed concrete with respect to the normal reinforced concrete.

1.4 Report Outline

Chapter 1 will have a introduction on prestressed concrete, a presentation of its background and a brief history of prestressed concrete, and the end of the chapter will include the standards that are being used in Canada, United States of America and Europe for designing and construction of prestressed concrete structures. Chapter 2 will have a presentation of the concepts and types of prestressed concrete, and the particularities of materials used, both the steel and the concrete.

Chapter 3 will talk about the corrosion and mechanism of corrosion of the steel in reinforced concrete in general, and then about its particularities related to prestressed concrete, followed by the Chapter 4 which will discuss the prevention and protection against corrosion in prestressed concrete, addressing both the concrete and the steel. Cathodic protection, one of the very efficient method of protection against corrosion, outside the concrete and steel itself, it is presented in Chapter 5, and the risks on the prestressed concrete that might come with this method. Chapter 6 will summarize the conclusions of the project report.

2. Prestressed Concrete

As a definition, we can say that prestressed concrete is a type of reinforced concrete in which the steel reinforcement has been tensioned against the concrete. This tensioning operation will result in a self-equilibrating system of internal stresses (tensile stresses in the steel and compressive stresses in the concrete) that improves the response of the concrete to external loads, taking care of the concrete weakness in tension. While concrete is strong and ductile in compression it is weak and brittle in tension, and hence its response to external loads is improved by applying a precompression.

2.1 Basic concepts of prestressing

Based on the prestressing operation, prestressed concrete is divided in two: pretensioning and post-tensioning. The basic concept of reinforced concrete, for both prestressed and non-prestressed elements, is that steel reinforcement is placed in those locations of a structure where tensile stresses will occur. In prestressed concrete elements, high-strength reinforcement is used, and this reinforcement is tensioned prior to the application of external loads. This initial tensioning of the reinforcement precompresses the surrounding concrete, giving it the ability to resist higher loads prior to cracking.

2.2 Pre-tensioned concrete

Pre-tensioned concrete is cast around already tensioned tendons. This method produces a good bond between the tendon and concrete, which both protects the tendon from corrosion and allows for direct transfer of tension. The cured concrete adheres and bonds to the bars and when the tension is released it is transferred to the concrete as compression by static friction. However, it requires stout anchoring points between which the tendon is to be stretched and the tendons are usually in a straight line. Thus, most pretensioned concrete elements are prefabricated in a factory and must be transported to the construction site, which limits their size.

The first step in pretensioning is the stressing of high-strength steel tendons (usually seven wire strands) between the abutments of a pretensioning bed. The concrete is then placed in formwork. After the desired concrete strength has been reached, the tendons are detensioned and the member becomes prestressed (Fig. 2-1).

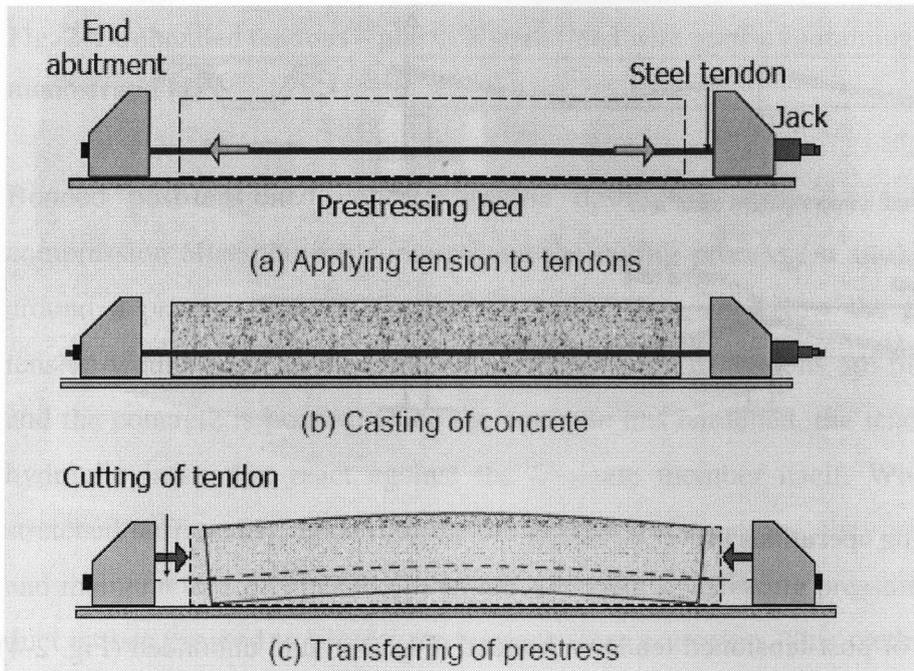


Fig. 2-1 Stages of pre-tensioning [4]

2.3 Post-tensioned concrete

The first step in producing a post-tensioned member is to place the reinforcing cage and the post-tensioning ducts in the formwork. After the casting and curing of the concrete, the tendons are tensioned and anchored using special post-tensioning jacks that react against the member. Unless unbonded tendons are being used, the duct is then grouted to complete the post-tensioning operation (Fig. 2-2).

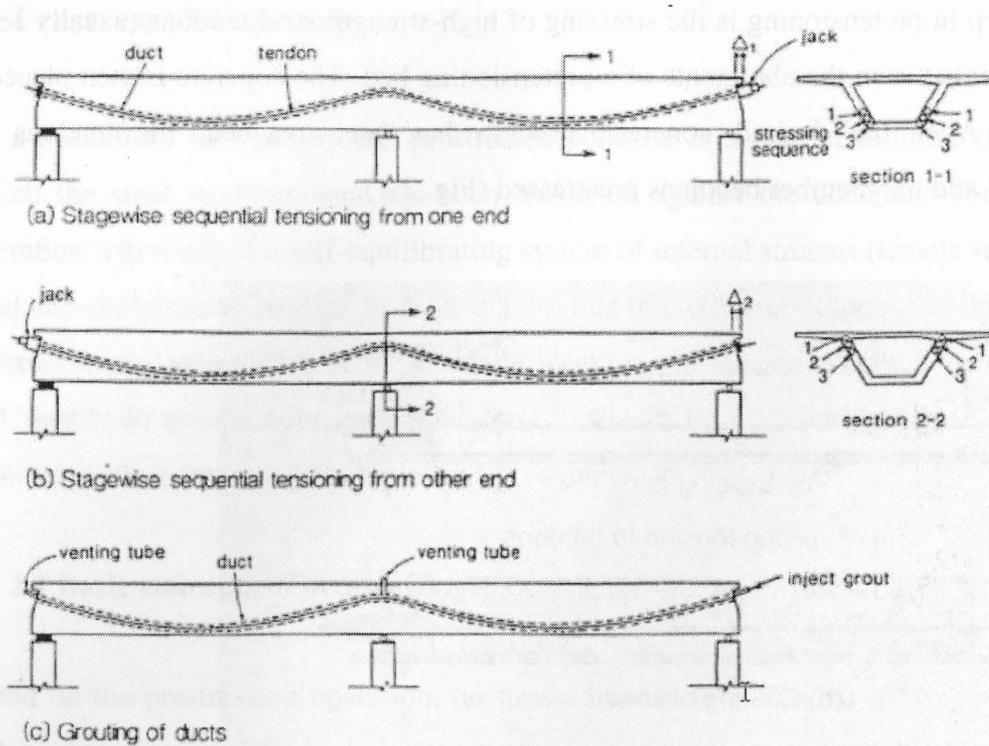


Fig. 2-2 Post-tensioning operations [4]

There are two types of post-tensioned tendons: bonded (Fig. 2-3) and unbonded (Fig. 2-4). In the bonded tendon the grout bonds the tendon to the surrounding concrete and provides corrosion protection for the tendon. The unbonded tendon is attached to the concrete only at its end anchors. Corrosion protection for the unbonded tendons is provided by grease-filled plastic tubes and by special details at the anchorages.

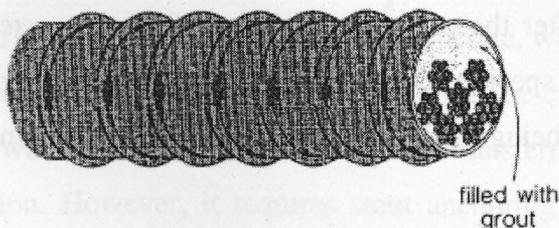


Fig. 2-3 Bonded tendons – grouted, corrugated metal sheath containing bonded multi-strand tendon [4]

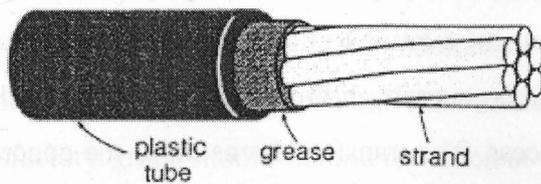


Fig. 2-4 Unbonded tendons – plastic sheath filled with grease containing unbonded monostrand [4]

Bonded post-tensioned concrete is the descriptive term for a method of applying compression after pouring concrete and the curing process (in situ). The concrete is cast around a plastic, steel or aluminium curved duct, to follow the area where otherwise tension would occur in the concrete element. A set of tendons are fished through the duct and the concrete is poured. Once the concrete has hardened, the tendons are tensioned by hydraulic jacks that react against the concrete member itself. When the tendons have stretched sufficiently, according to the design specifications, they are wedged in position and maintain tension after the jacks are removed, transferring pressure to the concrete. The duct is then grouted to protect the tendons from corrosion. This method is commonly used to create monolithic slabs for house construction in locations where expansive soils (such as adobe clay) create problems for the typical perimeter foundation. All stresses from seasonal expansion and contraction of the underlying soil are taken into the entire tensioned slab, which supports the building without significant flexure. Post-stressing is also used in the construction of various bridges, both after concrete is cured after support by false work and by the assembly of prefabricated sections, as in the segmental bridge.

The advantages of this system over unbonded post-tensioning are:

- large reduction in traditional reinforcement requirements as tendons cannot destress in accidents.
- tendons can be easily 'weaved' allowing a more efficient design approach.
- higher ultimate strength due to bond generated between the strand and concrete.
- no long term issues with maintaining the integrity of the anchor/dead end.

Unbonded post-tensioned concrete differs from bonded post-tensioning by providing each individual cable permanent freedom of movement relative to the concrete. To achieve this, each individual tendon is coated with a grease (generally lithium based) and covered by a plastic sheathing formed in an extrusion process. The transfer of tension to the concrete is achieved by the steel cable acting against steel anchors embedded in the perimeter of the slab. The main disadvantage over bonded post-tensioning is the fact that a cable can distress itself and burst out of the slab if damaged (such as during repair on the slab).

The advantages of this system over bonded post-tensioning are:

- the ability to individually adjust cables based on poor field conditions (For example: shifting a group of 4 cables around an opening by placing 2 to either side).
- the procedure of post-stress grouting is eliminated.
- the ability to de-stress the tendons before attempting repair work.

Unbonded tendons are often used in applications such as post-tensioned two-way slabs where the small duct diameter, the low friction between the strand and greased duct, and the elimination of the grouting operation all offer considerable economies. With unbonded tendons, special attention is required to ensure that the strand is protected from corrosion, where it enters the end anchorage. Further, it may be necessary to place additional reinforcing bars to provide adequate crack control.

2.4 Grouting of ducts

In the bonded post-tensioned construction the ducts are grouted as soon as possible after stressing the tendons. The objective of the grouting operation is to fill the duct completely with materials that provides an alkaline environment for corrosion protection of the prestressing steel and has sufficient strength to bond the tendons to the surrounding concrete. To minimize the time that the ungrouted tendons are exposed to corrosive conditions, it is advisable to insert the tendons into the ducts just prior to stressing the tendons.

Grout usually consists of a mixture of cement and water (water/cement ratio of about 0.50) together with a water-reducing admixture and an expansive agent. Grout for larger-diameter ducts may also contain filler materials such as sand, fly ash, or pozzolans. A satisfactory grout should maintain sufficient fluidity during the grouting operation, should exhibit minimum bleeding and segregation, should not shrink, should have adequate strength, and should not contain detrimental amounts of chlorides, nitrates, sulfides, or other compounds that contribute to corrosion.

The grout is injected at low points of the tendon or at the end of the member. Venting tubes are provided at high points of the tendon, as shown in Fig. 2-2c. If the duct is not properly vented, pockets of air may be trapped at high points of the duct (Fig. 2-5). Freezing of water that may collect in these air pockets can result in serious deterioration of the structure and may lead to corrosion of the tendon.

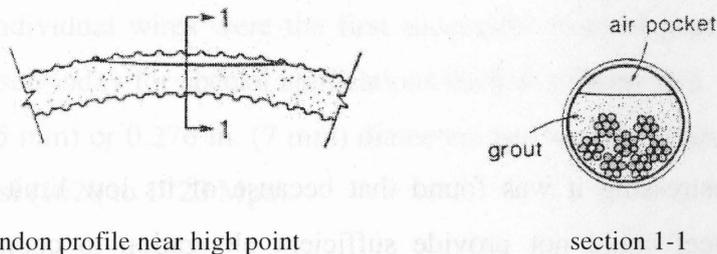


Fig. 2-5 Air pocket formed at tendon due to inadequate venting [4]

It is not usually possible to grout the ducts successfully if the temperature of the surrounding concrete is less than about 5°C (40°F). Hence it may be necessary to leave ducts ungrouted for some period of time if the temperature stays near freezing. In such cases, particular care must be taken to prevent water from entering the ducts and to ensure that all low points in the duct are properly drained.

Prior to grouting, the ducts should be blown out using oil-free compressed air to remove debris from the ducts. Sometimes the ducts are flushed out with water prior to grouting. If

blockage of a duct occurs during grouting, the ducts need to be flushed out immediately by injecting water into air vent, against the direction of grouting.

Records of the grouting pressures, volume of grout used, temperatures, and other details of the grouting operation need to be kept. High grouting pressure (e.g., over 1.5 MPa or 200psi) may indicate blockage in the duct and may cause segregation of the grout or splitting of the concrete surrounding the duct.

The long-term durability of a bonded, post-tensioned structure depends on the success of the grouting operation. Specifications for this highly specialized and critical procedure are given by the Post-Tensioning Institute, and Recommended Practice for Grouting of Post-Tensioned Prestressed Concrete by PCI. [13], [14]

2.5 Materials

2.5.1 Steel

Early in the development of prestressing it was found that because of its low limit of elasticity ordinary reinforcing steel could not provide sufficient elongation to counter concrete shortening due to creep and shrinkage. It is necessary to use the high tensile steels that were developed to produce a large elongation when tensioned. This ensures that there is sufficient elongation reserve to maintain the desired pre-compression.

Types of reinforcement

Prestressed concrete structures can be reinforced with prestressing tendons, deformed reinforcing bars, and welded wire fabric.

The term "tendon" is used to describe either an individual wire, strand, or bar or a group of wires, strands, or bars. The most widely used type of prestressed reinforcement is seven-wire strand, and a few sizes are shown in Fig. 2-6. The strands are used in both

pretensioned and post-tensioned construction. The ultimate tensile strength of these strands ranges from 250 to 270 ksi (1720 to 1860 Mpa).

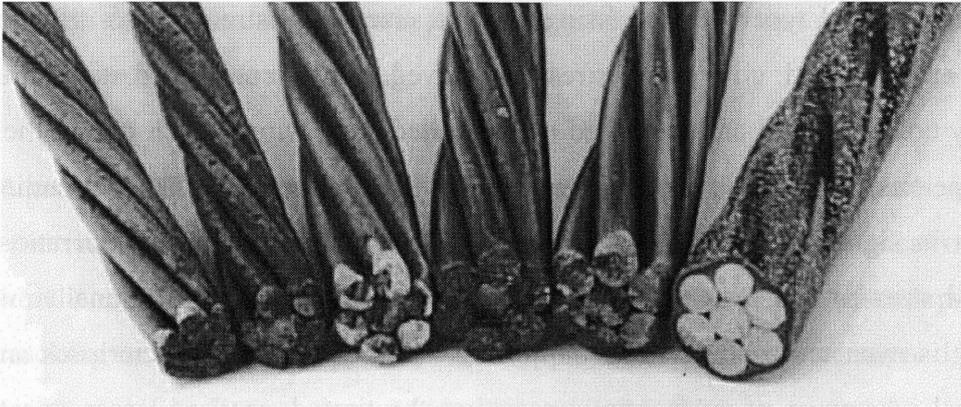


Fig. 2-6 7-Wire prestressing strands – from left to right: 3/8 in.; 3/8 in. indented; 1/2 in.; 0.6 in.; 0.6 in. epoxy-coated with embedded grit [15]

Individual wires were the first successful form of prestressed reinforcement and are still used today for special applications such as railway ties. The wires typically have 0.196 in. (5 mm) or 0.276 in. (7 mm) diameters and have ultimate strength ranging from 235 to 250 ksi (1620 to 1720 Mpa).

The properties of standard prestressing strands, wires, and bars are conforming to CSA Standard G279 or ASTM Standards A416, A421, and A722. [16], [17], [18], [19]

Welded wire fabric consists of a grid of cold-drawn wires welded together at their intersection. The wires may be either smooth or deformed and are fused together at their intersection by automatic, electrical-resistance welding. Welded wire fabric used in North America conforms to CSA Standard G30.5 and G30.15 and ASTM Standards A185 and A497. [20], [21], [22], [23]

It is important for the designer to recognize that the CSA and ASTM standards do not currently require that welded wire fabric be stress-relieved after fabrication, as is required in some European standards. Non-stress-relieved welded wire fabric, particularly for the

smaller wire sizes, may fail near the welded intersections at relatively small strains and hence such reinforcement may not be suitable where significant ductility is needed.

The most commonly used type of prestressing steel is seven-wire strand. Two different types of strands are produced: cold-drawn stressed-relieved wire or cold-rolled steel wires manufactured by cold forming, and tempered or hot-rolled steel wires which obtain their mechanical properties as a result of heat treatment. The cold-drawing and stranding operations result in significant residual stresses in the strand. Stress relieving operations removes residual stresses and results in a strand with a much higher proportional limit. Strain tempering is even more effective in improving the stress-strain characteristics and has the additional advantage of substantially reducing the time-dependent losses due to relaxation of the strand.

2.5.2 Concrete

To accommodate the degree of compression imposed by the tensioning tendons and to minimize prestress losses, a high strength concrete with low shrinkage properties is required. Units employing high strength concrete are most successfully cast under controlled factory conditions. A high strength concrete is also a high performance concrete which has a much lower permeability than the normal concrete, thus, providing a very good protection for the steel tendons.

Mehta (1994) stated that his holistic model of concrete deterioration "provides a clear justification why impermeability of concrete should be the first line of defense against any of the physico-chemical deterioration processes described earlier. If adequate attention in concrete making and processing is paid to hold the first line of defense, why would one need epoxy-coated reinforcement for protection against corrosion, ASTM Type V cement for protection against sulfate attack, and low-alkali cement or nonreactive aggregates for protection against expansion associated with the alkali-silica reaction?" [24]

Cracking and the effect of concrete cracking on corrosion

One of the most common forms of defects and deterioration that occur in the concrete elements of prestressed concrete and has a great influence on the occurrence of corrosion in prestressed concrete is cracking.

A crack is defined as an incomplete separation into one or more parts, with or without a space between them. The significance of cracks in concrete is dependent on their origin and whether the length and width increase with time. There are several possible causes of cracking in concrete structures. [26], [27] The most common causes and characteristics of cracks resulting from each cause are summarized below:

- a) Plastic shrinkage cracks result from rapid drying of the concrete in its plastic state. The cracks are usually wide but shallow and often in a well-defined pattern or spaced at regular intervals.
- b) Drying shrinkage cracks result from drying of restrained concrete after it has hardened. They are usually finer and deeper than plastic shrinkage cracks and have a random orientation.
- c) Settlement cracks may be of any orientation and width, ranging from fine cracks above the reinforcement that result from settlement of the formwork to wide cracks in supporting members caused by foundation settlement.
- d) Map cracking (a closely spaced network of cracks) usually results from chemical reaction between the mineral aggregates and the cement paste. The number and width of the cracks usually increase with time. A number of reactions are possible, although the reaction between the alkalis from the cement, or from external sources, and some aggregates (producing alkali-silica or alkali-carbonate reactions) are the most widespread. Both types of reaction result in serious damage to the concrete by causing abnormal expansion, cracking, and loss of strength. [27]

- e) Corrosion-induced cracks (resulting from the corrosion of the embedded reinforcement) are usually associated with shallow cover and are located directly above the reinforcement. The cracks often terminate at the reinforcement, and rust stains may be associated with them. The width of the cracks increases with time as the corrosion continues.
- f) Structural cracks may result from any of the following: inadequate flexural and shear capacity, incorrect consideration of the thermal stresses, insufficient attention to stresses developed by curvature of tendons, improper or inappropriate construction techniques, failure to meet tolerance requirements, and under-strength of materials. [28]

It should be noted that cracks are not totally avoidable in prestressed concrete structures because not all the portion of the structure are precompressed in three directions and shrinkage is not always controllable. Certain cracks may be structurally serious, although others are not. It is therefore important to determine the structural significance of a crack and its effect on the serviceability of the structure.

Flexural cracking is associated with tensile stresses that exceed the tensile capacity of the concrete. Consequently, in continuous girders, they are usually found at the bottom of the girder in positive moment areas and at the top of the girder in negative moment areas. Flexural cracks in some segmental bridges have resulted from underestimation of the moments from any of a number of causes, including the redistribution of creep effects, lack of consideration of thermal gradients, and stress distribution in the vicinity of anchorage or tendon terminations. In areas near the support, the effect of shear will be superimposed upon the flexural stresses.

Cracking may occur at or near anchorage positions because of inadequate design or poor details.

Cracking may be associated with either vertical or horizontal curvature of the tendons. In both cases, the cracks are oriented roughly parallel to the tendons. It may also result from the misalignment of the tendons. Depending of the geometry of the component, a laminar crack may develop between adjacent tendons, or, in an extreme situation, a spalling failure can occur.

The influence of cracking and crack widths on the corrosion of steel reinforcement in concrete members is a subject that has received much debate. In the past, considerable research and discussion has been devoted to this topic without arriving at a general consensus. [29] In general, two points of view exist:

- 1) Cracks reduce the service life of the structure by permitting a more rapid means of access for moisture, chloride ions and oxygen to reach the reinforcement, thus accelerating the onset of corrosion. (i.e., cracking has a significant effect on corrosion)
- 2) Cracks may accelerate the onset of corrosion, but such corrosion will be localized to the region of the crack. It is suggested that over time, chloride ions will eventually penetrate even uncracked concrete, initiating more widespread corrosion. Thus, after a long service life, the difference between the amount of corrosion in cracked and uncracked concrete will be minor. (i.e., cracking does not have a significant effect on corrosion)

The two points of view are illustrated in Fig. 2-7 and Fig. 2-8. Both points of view indicate that the presence of cracks will accelerate the onset of corrosion. The first point of view suggests that the accelerated onset of corrosion will lead to more corrosion damage in a shorter period, and thus reduce the service life of the structure. The second point of view suggests that the corrosion rate in uncracked concrete will reach the corrosion rate at the crack locations after some duration. This point of view implies that the length of time between corrosion initiation at a crack, t_i , and corrosion initiation in uncracked concrete, t_{icr} , is not significant. This latter point of view means that the two curves in Fig. 2-8 are close together when the entire service life is considered, and thus the early onset of

corrosion at cracks has little effect on the service life in comparison to the case where the concrete was entirely uncracked.

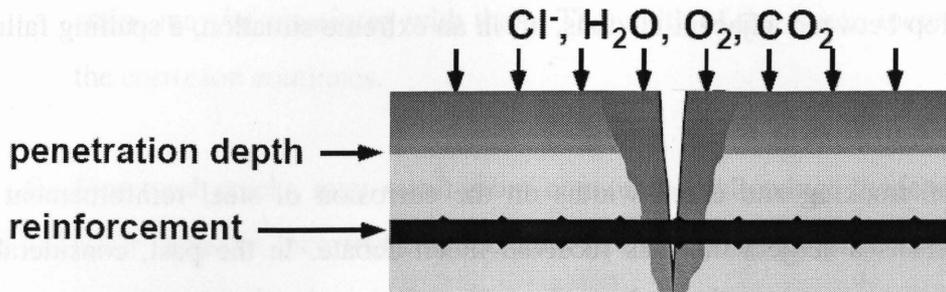


Fig. 2-7 Increased Penetration of Moisture and Chlorides at Crack Location Accelerates the Onset and Severity of Corrosion [27]

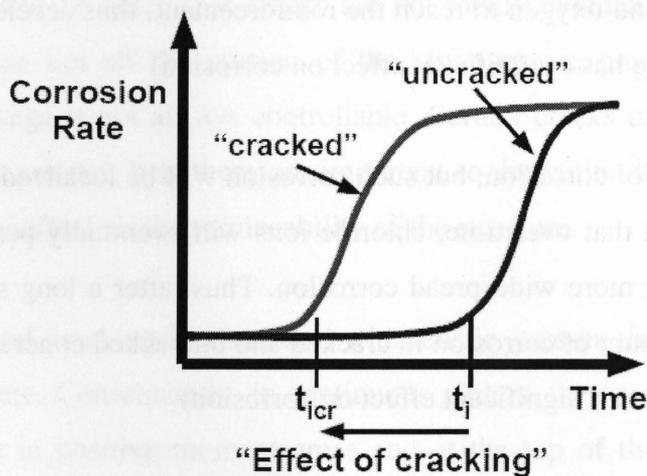


Fig. 2-8 Cracking Accelerates Onset of Corrosion, But Over Time Corrosion is Similar in Cracked and Uncracked Concrete [28]

The effects of cracking can be summarized in general terms as follows:

- cracks may significantly reduce the time to corrosion initiation in comparison to sound, good quality concrete. This effect is relatively insensitive to crack width.
- cracks may affect the corrosion rate. However, no general conclusions can be made on the rate of corrosion as a function of crack width. One research program

The suggested corrosion rate is a function of crack spacing, not width. It is generally assumed corrosion rates will be higher in cracked concrete in comparison to sound, good quality concrete.

- the level of unacceptable damage in a structure can be considered at a local scale (loss of section) or a global scale (disruption of concrete). Different situations may exist where one or the other form of corrosion damage may control.

3. Corrosion

3.1 Introduction

The deterioration of concrete structures due to chloride-induced reinforcement corrosion is recognized as one of the major challenges facing the owners of facilities and infrastructure systems made up of concrete structures. Chloride-induced corrosion of reinforcing steel in concrete bridge decks, parking garage slabs and marine structures has been identified as the primary cause of concrete deterioration. Structures exposed to marine environments or deicing salts are particularly at risk. Despite of the durability of reinforced concrete structures when compared to steel and timber structures, reinforced and prestressed concrete structures are vulnerable to the damaging effects of corrosion induced primarily by chlorides (from deicing slats and seawater) and to a lesser extent to carbonation-induced corrosion. It is estimated that one-third to one-half of the projected bridge rehabilitation costs in North America are related to bridge deck deterioration (Cady and Weyers 1983; Lounis and Mirza 2001). The corrosion of the reinforcement leads to concrete fracture through cracking, delamination and spalling of the concrete cover, reduction of concrete and reinforcement cross sections, loss of bond between the reinforcement and concrete, reduction in strength (flexural, shear, etc.), and ductility. As a result, the safety and serviceability of concrete structures are reduced, and their useful service lives shortened.

Most structures are free from corrosion trouble and many have given very satisfactory long-term service. When corrosion of the reinforcement does occur it is usually associated with faulty construction, a very aggressive environment or a change of use such as the introduction of wet conditions into a previously dry structure.

The corrosion products can occupy up to six times as much volume as steel, leading to cracking and disruption of the concrete. Corrosion of steel reinforcement in concrete may significantly affect structural integrity through reduced capacity or fracture of the steel reinforcement, due to loss of bond between the steel and concrete or through cracking and spalling of the concrete. [30]

The corrosion of prestressing steel can be more serious than that of normal reinforcing steel for mainly two reasons. First, the prestressing steel has a smaller cross section and is therefore proportionally more adversely affected by local corrosive attack than mild steel reinforcement. Second, the removal by corrosion of a certain portion of the steel in a prestressing tendon represents a greater proportion of the strength of the structure because the tendon is working at a higher tension than normal reinforcement.

On the other hand, the protection provided by the concrete has been enhanced by the use of high-strength concrete, as it is uneconomical to use low-strength concrete because the prestressing losses are too great. Also, a better control environment and better quality control programs, makes prestressed concrete less prone to faulty execution. High-strength concrete is also used that the member can achieve sufficient strength for prestressing in the shortest possible time to speed production. The requirements for a specified strength at the time of prestressing often results in higher concrete strength at later ages than would otherwise be required by the loading conditions.

However, except for external tendons, the steel is inaccessible, and nondestructive methods for detecting corrosion have serious limitations. The structural repair of prestressed concrete components is also very difficult from both the technical a practical point of view. Furthermore, a substantial number of prestressed concrete bridges in North America are reaching an age when deterioration might be anticipated, specially the ones in more corrosive areas, cold climates, where deicing is used extensively, or marine environment.

The formation of a good passive oxide film depends upon the steel being initially clean. Rust, scale, grease, drawing compounds or temporary protective coatings would be expected to impair the formation of a continuously adherent film of oxide. The passive film is a thin, tightly adherent, non-conducting layer of iron oxide that slows or stops the corrosion process by preventing ferrous ions (Fe^{2+}) from entering solution in the electrolyte. Initially, the steel must corrode to produce the passive film. The process begins soon after construction as hydration of the cement raises the pH of the concrete. Recalling the anodic, cathodic and secondary reactions listed in the preceding section, ferrous

hydroxide $\text{Fe}(\text{OH})_2$ is produced by the corrosion process. If oxygen and moisture are present, the ferrous hydroxide is converted to gamma iron oxide ($\gamma\text{-Fe}_2\text{O}_3$) and a thin passive film is formed on the surface of the steel. The passive film remains stable when the alkalinity of the concrete exceeds about pH 12, and the steel is protected from further corrosion. [31], [32] The passive film only affects the anodic reaction, and the cathode reaction may occur on surfaces where the film is intact. Areas not so oxidized would be anodic to oxidized areas and would corrode without generating protection, leading to a pitting type of attack. Pitting also occurs where free chloride ions either prevent the formation of a passive oxide film or damage such a film already formed. Such damage would not be able to heal in the presence of chloride so that the attack might be sustained over a long period. Other ions that form water-soluble calcium salts are likely to behave like chloride in this respect. The protection is also impaired if, in the course of time, the cement in contact with the steel becomes carbonated. The effective pH of the environment then falls to about 8.5 at which level the adherent oxide film is no longer stable. As the film breaks down, anodic areas (relative to those still protected) can form, leading to pitting or to more general corrosion should moisture and oxygen be available. This situation would be aggravated by the presence of traces of chloride. Macro-anodic areas could be produced in this way where concrete was locally porous.

3.2 Corrosion of Embedded Reinforcement

Corrosion of reinforcement is the electrochemical degradation of steel in concrete in which steel is affected by a particular environment, resulting in a measurable loss of metal. The deleterious effects of corrosion are illustrated in Fig. 3-1. The corrosion occurs when the passivity of the steel is destroyed by either carbonation of the concrete or by presence of more than threshold concentration of chloride ions at the steel surface and an electrochemical cell develops.

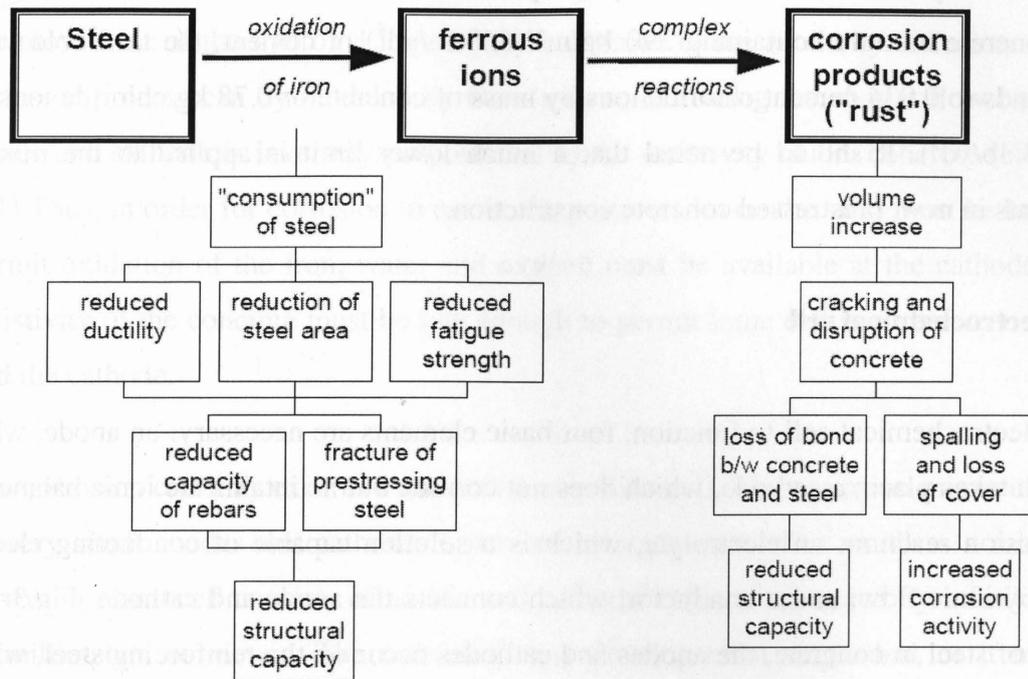


Fig. 3-1 Deterioration Mechanism for Corrosion of Steel in Concrete [31]

3.2.1 Carbonation of concrete

Carbonation of concrete is the result of the reaction of carbon dioxide and other acidic gases in the air (which forms weak acids in solution) and the alkaline constituents of cement paste. As a result, the alkalinity of the concrete is reduced and the steel is no longer protected against corrosion. The depth of carbonation increases with age but is a very slow reaction, typically no more than 1 mm (0.04 in) each year. The rate is highest in structure with a high water-cement ratio and in a dry environment. Carbonation has not been identified as a serious problem in highway structures, and corrosion is normally initiated by the ingress of chloride ions and not carbonation.

3.2.2 Chloride ion content

The quantity of the chloride necessary to destroy the passivity of steel in concrete is still a matter of some dispute, but the generally accepted value is 0.20 percent acid-soluble

chloride ions by mass of cement. This quantity is termed the threshold value for corrosion. For a concrete mixture containing 390 kg/m^3 (660 lb/yd^3) of cement, the threshold value corresponds to 0.034 percent chloride ions by mass of concrete or 0.78 kg chloride ions per m^3 (1.28 lb/yd^3). It should be noted that a much lower limit is applied to the mixture ingredients in new, prestressed concrete construction.

3.2.3 Electrochemical cell

For an electrochemical cell to function, four basic elements are necessary: an anode, where corrosion takes place; a cathode, which does not corrode but maintains the ionic balance of the corrosion reaction; an electrolyte, which is a solution capable of conducting electric current by ionic flow; and a conductor, which connects the anode and cathode. Fig.3-2 In the case of steel in concrete, the anodes and cathodes occur on the reinforcing steel, which also acts as a conductor. Moist concrete or grout acts as the electrolyte.

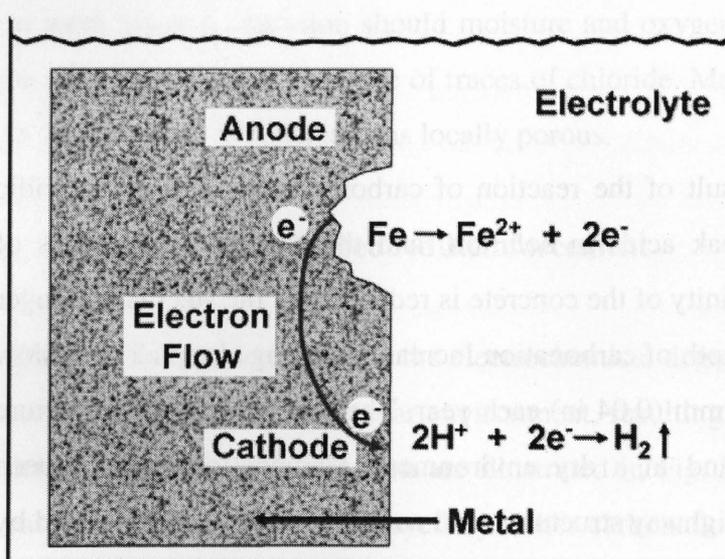


Fig. 3-2 Electrochemical cell [31]

Iron is oxidized at the anode (i.e., iron is removed from the steel, and two electrons are liberated). At the cathode, oxygen is reduced by the electrons liberated at the anode, and hydroxyl ions are produced. In order to conserve charge, the reaction rates for the anodic

and cathodic reactions must be equal. The charge flow in the corrosion cell is completed by the movement of the hydroxyl ions from the cathode to the anode by diffusion. The ionic diffusion process can be modeled by Ohm's Law, indicating the charge flow between the anode and cathode is inversely proportional to the resistivity of the concrete (electrolyte). [31] Thus, in order for corrosion to occur, the passive film on the steel must be breached to permit oxidation of the iron, water and oxygen must be available at the cathode, and the resistivity of the concrete must be low enough to permit ionic diffusion between the anode and the cathode.

The corrosion cell may exist as a macrocell or a microcell. The corrosion microcell consists of very small anodes and cathodes separated by a distance of as small as a micron. The corrosion macrocell consists of anodic and cathodic regions separated by a finite distance of millimeters or meters. The macrocell may develop on a single bar, or it may occur between different layers of steel or different areas of the structure. A common example of a corrosion macrocell is a bridge deck where the top layer of steel acts as the anode and the bottom layer as the cathode, as shown in Fig. 3-3. The top layer is closer to the source of moisture and chlorides, and therefore becomes depassivated before the bottom layer. Oxygen is readily available near the bottom layer to facilitate the cathodic reaction. Electrical continuity between the layers is provided by stirrups or other reinforcement or ties. Another form of macrocell corrosion can occur at crack locations. [33] The anodic reaction occurs in the vicinity of the crack, while the cathodic reaction takes place on the same bar within the distance between cracks, as shown in Fig. 3-4. The steel near the crack becomes depassivated due to chloride or carbon dioxide penetration at the crack. This form of corrosion cell can lead to very high corrosion rates and pitting corrosion due to the large cathode area in comparison to anode area. In most situations of corrosion in concrete structures, a combination of microcell and macrocell corrosion occurs. [31]

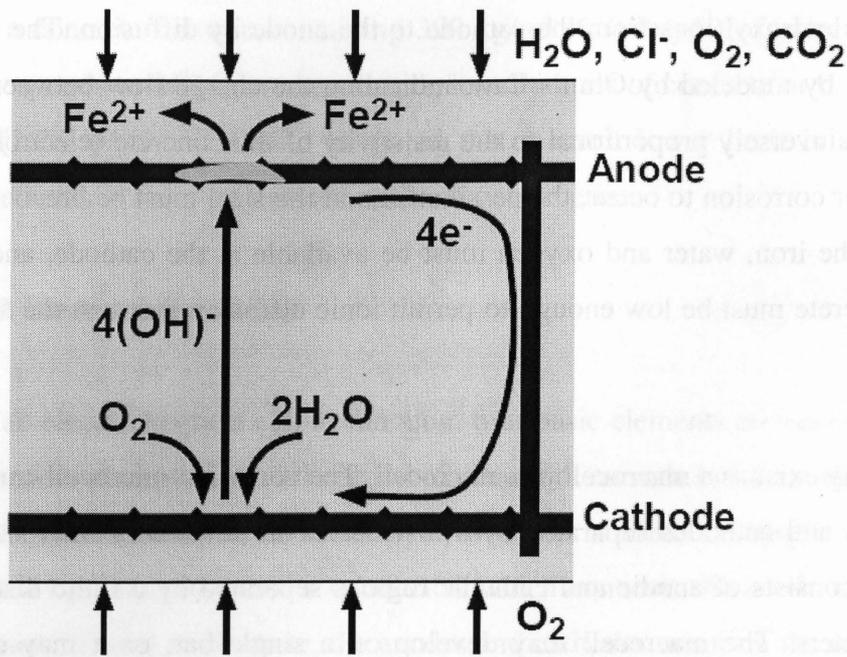


Fig. 3-3 Idealized Macrocell Corrosion [33]

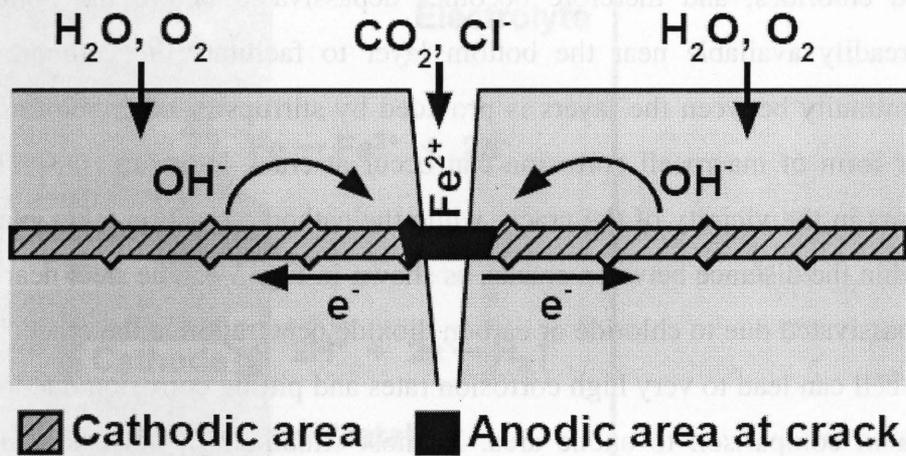


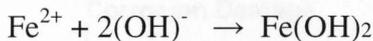
Fig. 3-4 Macrocell Corrosion at a Crack [33]

If chlorides are present in the concrete, they may act as a catalyst by introducing an additional anodic reaction: [31], [34]



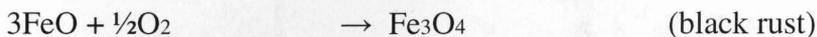
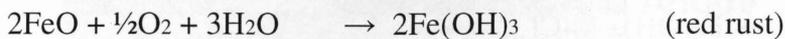
These reactions remove ferrous or ferric ions from the steel by forming complex ions with the chlorides. The ferrous or ferric ions are then deposited near the anode where they join with hydroxyl ions to form various corrosion products. The chloride ions are released to repeat the process. Since the chloride ions are not consumed, the process can become autocatalytic. The electrons released during these reactions flow through the steel to the cathodic reaction.

The anodic and cathodic reactions are followed by a variety of secondary reactions that form the expansive products of corrosion. Although the Fe^{2+} and OH^- ions both diffuse into the concrete (from the anode and cathode respectively), the corrosion products form near the anode because the OH^- ions are smaller and more diffuse through the concrete more readily. [35] If the supply of oxygen is restricted, ferrous oxides and hydroxides form:



If oxygen is available, ferric oxides and hydroxides form: [31], [36], [37]





The rate of corrosion is largely controlled by the size of the anodic and cathodic areas, the distance between them, the availability of oxygen and moisture at the cathode, the polarization of cell, and the resistivity of the electrolyte.

As the steel corrodes, it expands and often causes a delamination (a separation along a plane parallel to the surface of the concrete), usually located at or near the level of the reinforcement. As the corrosion process continues, delamination eventually becomes detached from the concrete body, resulting in a spall. Cracks are not necessary for the occurrence of corrosion because chloride ions will penetrate uncracked, high-quality concrete. However, the presence of cracks will help the ingress of chloride ions, thus helping the corrosion to take place. Depending on the amount of concrete cover and degree of corrosion, cracks may or may not be present within or at the extremities of delaminated areas.

3.3 Stages of Corrosion in Concrete Structures

The process of corrosion is often separated into the stages of initiation and propagation, as illustrated in Fig. 3-5. Depassivation of the steel defines the transition from the initiation stage to propagation.

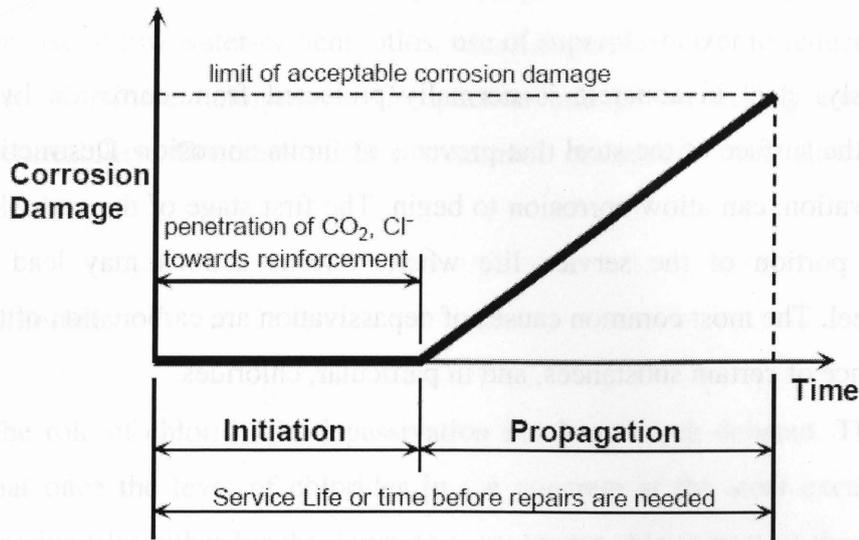


Fig. 3-5 Stages of Corrosion of Steel in Concrete [37]

This simple model can be used to illustrate the effects of time to corrosion initiation and corrosion rate on the service life of the structure. Two different situations are illustrated in Fig. 3-6. In the first situation, structure A has a lower corrosion rate, but its service life is shorter since corrosion initiation occurred earlier than structure B. In the second situation, structure B has a shorter service life although the corrosion rates are the same as the first situation. Thus, it is important to consider both the time to corrosion and corrosion rate, as a single parameter is not sufficient to determine the service life of the structure.

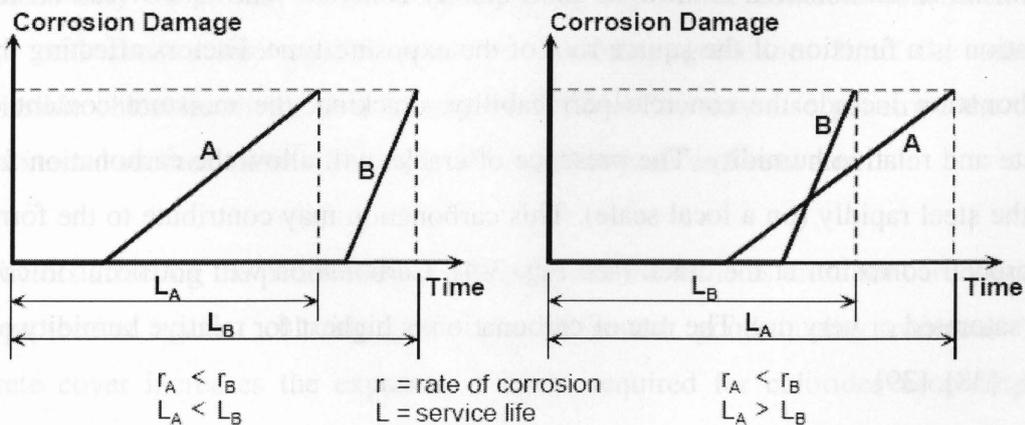


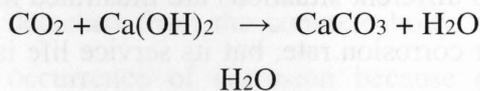
Fig. 3-6 Effect of Time to Corrosion and Corrosion Rate on Service Life [37]

Stage 1: Initiation – Factors Affecting the Time to Corrosion

As described previously, steel in concrete is normally protected from corrosion by a passive oxide film on the surface of the steel that prevents or limits corrosion. Destruction of the film, or depassivation, can allow corrosion to begin. The first stage of the corrosion process concerns the portion of the service life where various actions may lead to depassivation of the steel. The most common causes of depassivation are carbonation of the concrete and the presence of certain substances, and in particular, chlorides.

Carbonation

Carbonation of concrete occurs when atmospheric carbon dioxide penetrates the concrete. In the presence of moisture, carbon dioxide will react with calcium hydroxide in the concrete to produce calcium carbonate. The reaction is:



The formation of calcium carbonate reduces the pH of the concrete to as low as 8, where the passive film is no longer stable allowing corrosion to begin. [38]

The process of carbonation is slow in good quality concrete. The rate of carbon dioxide penetration is a function of the square root of the exposure time. Factors affecting the rate of carbonation include the concrete permeability, cracking, the moisture content of the concrete and relative humidity. The presence of cracks will allow the carbonation front to reach the steel rapidly (on a local scale). This carbonation may contribute to the formation of macrocell corrosion at the crack (see Fig. 3-4). Carbonation will not occur in concrete that is saturated or very dry. The rate of carbonation is highest for relative humidity of 50% to 70%. [38], [39]

Carbonation may be slowed by specifying concrete with low permeability. Options include the use of low water-cement ratios, use of superplasticizer to reduce water demand, and the use of mineral admixtures. Compaction and proper curing are needed to ensure low permeability. Concrete surface treatment or sealers may slow penetration of carbon dioxide.

Chlorides

The role of chlorides in depassivation has been much debated. The general consensus is that once the level of chlorides in the concrete at the steel exceeds a certain limit, the passive film either breaks down or is no longer able to protect the steel from corrosion. In general terms, the role of chlorides in depassivation may take one or a combination of the following forms:

- chloride ions may disperse the passive film.
- chloride ions may make the film permeable to Fe^{2+} ions allowing the anodic reaction to occur even when the passive film is present.
- chloride ions may penetrate the passive film and anodic reactions with Cl^- acting as a catalyst may occur.
- chloride ions may reduce the pH, making the passive film unstable.

Chlorides may be present in the concrete from any of the concrete constituents. Most design and construction specifications or durability guidelines limit the amount of permissible chlorides in concrete constituents. Chlorides may also penetrate the concrete from external sources, most commonly deicing chemicals and seawater.

The factors affecting the penetration of chlorides are the same as those for carbonation. The best protection against chloride penetration is sound, good quality concrete. Thicker concrete cover increases the exposure duration required for chlorides according to the square root time relationship. Doubling the cover will increase the exposure duration before chlorides reach the steel by approximately four times. [30]

Stage 2: Propagation – Factors Affecting the Rate of Corrosion

Once the passive film on the steel has been destroyed, the corrosion process may begin provided that the essential components of the corrosion cell are present. The rate of corrosion, or the rate at which iron is removed from the steel, is a function of many factors including the presence of moisture and oxygen, concrete permeability and resistivity, gradients in chloride ion concentration, heterogeneity's in the concrete and steel, pH of the concrete pore water, carbonation, cracking and stray currents. [29]

Corrosion Potential (Driving Force)

The electrochemical corrosion cell works in the same manner as an electrical circuit. The current flowing in the circuit is related to the electromotive force in the corrosion cell. The electromotive force is measured as a potential (voltage) difference between the anode and cathode. The electromotive force indicates the potential energy in the system, but does not directly indicate the corrosion rate for reasons that will be discussed under polarization effects. [31]

The electromotive force or potential difference in the corrosion cell can result from many sources. In general, any non-uniformity between the anode and cathode or non-uniformity within the electrolyte (concrete) will produce a potential difference. Examples include dissimilar metals and concentration cells. The latter term refers to a potential difference resulting from non-uniform concentrations of oxygen, moisture, chloride ions or metallic ions in the concrete. Temperature gradients or variations in pH may also produce concentration cells. Concentration cells commonly lead to macrocell corrosion. Examples include higher chloride levels for steel near an exposed surface, non-uniform concrete moisture contents along the length of a bar and the presence of cracks in the concrete. [31]

Polarization Effects

Several factors may limit the rate of corrosion in spite of a large electromotive force. These

conditions are referred to as polarization effects. The discussion of polarization effects is best illustrated using mixed potential corrosion theory. [40], [41]

Polarization is defined as a shift in the half-cell potential of an electrode away from its free or reversible potential due to current flow. Typical mixed electrode plots are shown in Fig. 3-7. E_{ca} and E_{an} indicate the reversible half-cell potentials of the cathode and anode, respectively. When the anode and cathode are coupled together, current begins to flow and equilibrium is reached at E_{corr} (open circuit potential of the corroding element) and i_{corr} (corrosion current). The curves (lines) in the figure indicate the polarized anodic and cathodic half-cell reactions. [29], [31]

The corrosion rate of the system may be controlled by three different kinds of polarization: Activation Polarization, Concentration Polarization and Ohmic Polarization. Activation polarization refers to situations where the corrosion rate is controlled by reactions at the metal-electrolyte interface. An example would be the passive film on steel that limits oxidation of iron, slowing the anodic reaction as is illustrated in Figure Fig. 3-7 (a). If the cathodic reaction is slower than the anodic reaction, the process is said to be cathodically controlled as shown in Fig. 3-7 (b). [29]

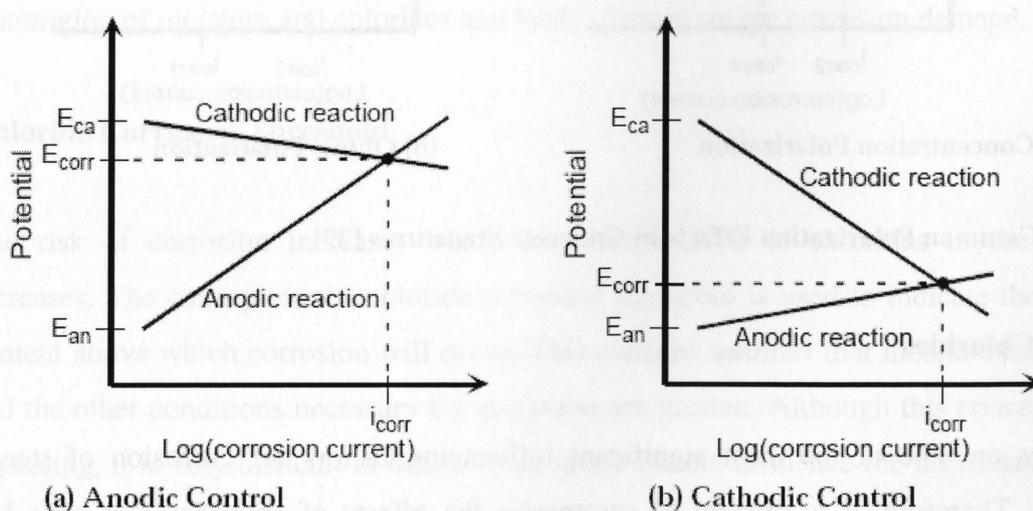


Fig. 3-7 Electrochemical Processes Under Activation Polarization [29]

The most common corrosion rate controlling mechanisms in concrete structures are concentration polarization and ohmic polarization. Concentration polarization occurs when the conditions change in the electrolyte near the anode or cathode, slowing the reaction rate. An example of concentration polarization in concrete is depletion of oxygen near the cathode, as illustrated in Fig. 3-8 (a). This condition is referred to as cathodic diffusion control. The slope of the polarized cathodic reaction curve approaches infinity, resulting in very negative half-cell potentials but low corrosion rates. The oxygen diffusion rate for case 2 is lower than case 1 in Fig. 3-8 (a). Ohmic polarization occurs when the resistivity of the concrete is high. The electromotive force (potential) available for corrosion is reduced by the resistance of the concrete, sometimes referred to as the “IR effect” or “IR drop.” As a result, the corrosion rate is limited as shown in Fig. 3-8 (b). The corrosion rate decreases as the IR drop increases, as indicated in the figure.

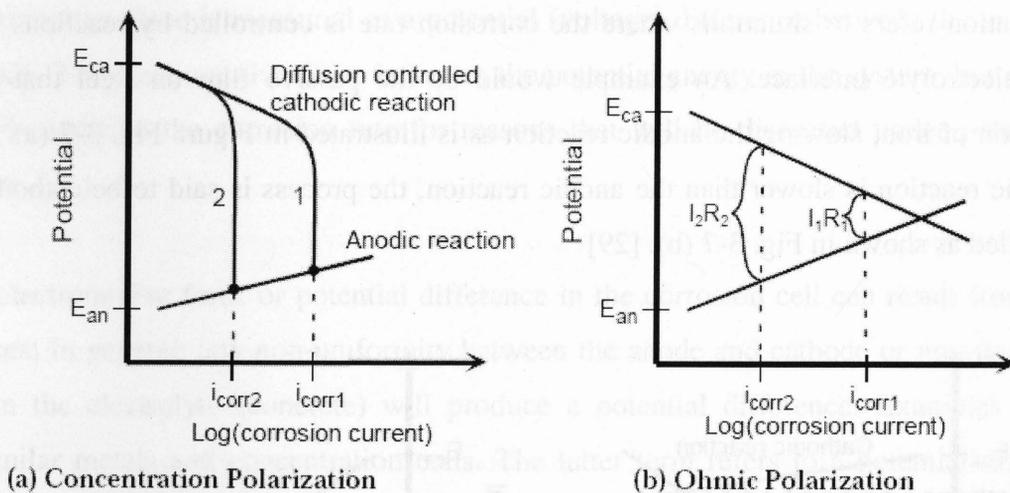


Fig. 3-8 Common Polarization Effects in Concrete Structures [29]

Role of Chlorides

Chlorides are one of the most significant influencing factors for corrosion of steel in concrete. Therefore, it is prudent to summarize the effects of chlorides that have been discussed already, and introduce some issues not addressed thus far. The role of chlorides in corrosion includes: [31], [34], [36], [29]

- 1) **Catalyst for oxidation of iron (anodic reaction):** Chlorides may accelerate corrosion by providing additional anodic reactions.
- 2) **Depassivation of the steel:** The presence of chlorides may result in destruction of the protective passive film on the steel.
- 3) **Concentration Cells:** Concentrations of chlorides in the concrete will cause a shift in the half-cell potential of the steel. The resulting potential difference provides the corrosion cell with the electromotive force for corrosion.
- 4) **Concrete Resistivity:** The presence of chloride ions in the concrete reduces the resistivity of the concrete. Also, the chlorides increase the saturation of the concrete, further decreasing resistivity.
- 5) **Reduced pH:** The presence of chlorides may reduce the pH, affecting the stability of the passive film and increasing the possibility of pitting corrosion.
- 6) **Freezing damage:** Freeze-thaw damage of concrete is generally more severe in the presence of chlorides. Cracking and spalling from freeze-thaw damage allows faster penetration of moisture and chlorides and leads to more severe corrosion damage.

Chloride Corrosion Threshold

The risk of corrosion increases when the concentration of chlorides in the concrete increases. The concept of the chloride corrosion threshold is used to indicate the chloride content above which corrosion will occur. This concept assumes that moisture and oxygen and the other conditions necessary for corrosion are present. Although this concept is very appealing, it is very difficult to define since many factors influence the necessary chloride content for corrosion. Some of the issues include:

- a) **How the chloride level is determined:** total chloride, acid-soluble chloride or water-soluble chloride. [29] Chlorides may be bound into various compounds in the concrete, making them unavailable for corrosion. [42], [43], [44], [45] This binding suggests water-soluble chlorides should be used. However, the chlorides may later become available for corrosion, suggesting that total or acid-soluble chloride content is more conservative. [42], [46], [47], [48], [49]
- b) **Whether chlorides were admixed or penetrated from external sources:** More chlorides will be bound if admixed into the concrete. [29], [42]
- c) **Cement composition:** C₃A content of the cement affects chloride binding. [43], [44], [45]
- d) **Exposure conditions:** Moisture and oxygen concentrations influence corrosion activity, and thus affect the threshold. The presence of sulfates may cause chlorides to become “unbound.” [42], [46], [47], [48], [49]
- e) **pH of concrete pore solution:** Since the pH influences the passive film on the steel, some suggest that instead of a chloride threshold, the corrosion threshold should be in terms of the ratio of chloride ions to hydroxyl ions (Cl⁻/OH⁻). [46], [50]

Many researchers have proposed chloride corrosion threshold values. Kahhalehn performed a thorough literature search on the subject and reported threshold values of 0.14% to 0.35% acid-soluble chlorides by weight of cement. A value of 0.2% acid-soluble chlorides by cement weight is often reported and recommended. The Comité Euro-International du Béton (CEB) provides the chart shown in Fig. 3-9. This figure shows the influence of environment, concrete quality and carbonation on the critical chloride content. The CEB suggests that the chloride corrosion threshold in concrete (not carbonated) is approximately 0.05% chloride by weight of concrete, or about 0.4% by weight of cement. It is not clear whether these values are in terms of total, acid-soluble or water-soluble chlorides. Due to the significance of chlorides in the corrosion process, design for corrosion protection is

primarily concerned with the preventing chlorides from entering the concrete and reaching the steel. [29], [30], [51], [52]

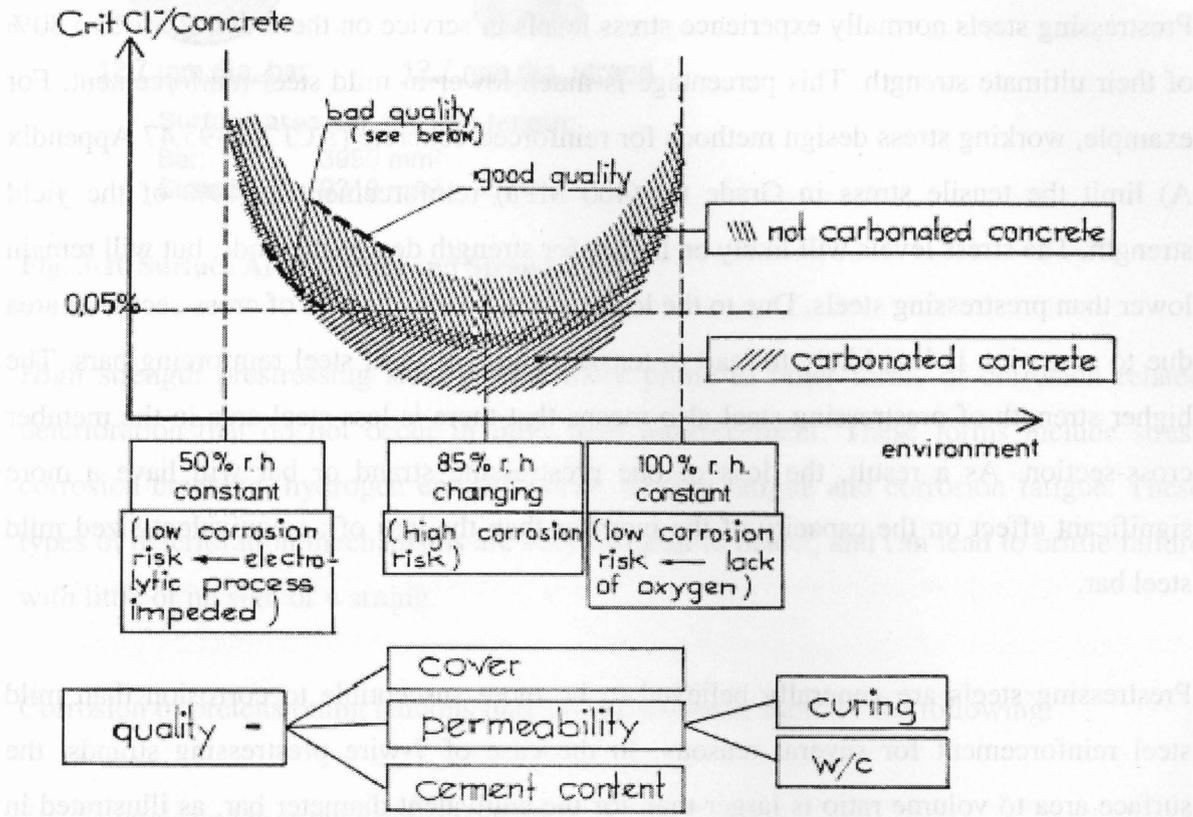


Fig. 3-9 CEB Critical Chloride Ion Content for Corrosion [30]

3.4 Corrosion of Prestressing Steel

The corrosion of prestressing steel is fundamentally the same as that of reinforcing bars. The high-strength steel used is marginally more reactive than mild steel and highly stressed steel is somewhat more readily corroded than lightly stressed steel, but these differences do not have any practical significance. [53]

Corrosion damage, when it does occur, is likely to have more serious consequences in prestressed structures, and stress corrosion – and hydrogen embrittlement – type failures are more likely in the highly stressed steel used for prestressing. It is possible to distinguish

several types of corrosion that may occur on prestressing steel, and the characteristics of each type are discussed in the following paragraphs.

Prestressing steels normally experience stress levels in service on the order of 70% to 80% of their ultimate strength. This percentage is much lower in mild steel reinforcement. For example, working stress design methods for reinforced concrete (ACI 318-95, 47 Appendix A) limit the tensile stress in Grade 60 (400 MPa) reinforcement to 40% of the yield strength. The stress levels will likely be higher for strength design methods, but will remain lower than prestressing steels. Due to the lower stress levels, the loss of cross-sectional area due to corrosion is less likely to lead to tensile failure of mild steel reinforcing bars. The higher strength of prestressing steel also means that there is less steel area in the member cross-section. As a result, the loss of one prestressing strand or bar will have a more significant effect on the capacity of the member than the loss of an equivalent sized mild steel bar.

Prestressing steels are generally believed to be more susceptible to corrosion than mild steel reinforcement for several reasons. In the case of 7-wire prestressing strands, the surface area to volume ratio is larger than for the equivalent diameter bar, as illustrated in Fig. 3-10. This larger ratio means that more surface area is available for corrosion, and the cross-sectional area of the strand may be reduced at a faster rate. The configuration of the 7-wire prestressing strand also makes the strand more susceptible to crevice corrosion. Crevice corrosion is a type of severe corrosion occurring in small spaces or crevices, such as the interstices between wires. The geometric constraints of the crevice enhance the formation of chloride ion concentration cells. Once corrosion has initiated, it progresses similar to pitting corrosion. Due to the geometry of the crevice, Fe^{2+} ions cannot disperse easily, and chloride ions are drawn into the crevice by the positive charge accumulation. The process becomes autocatalytic as the presence of chloride ions leads to formation of hydrochloric acid (HCl) and higher corrosion rates ensue.

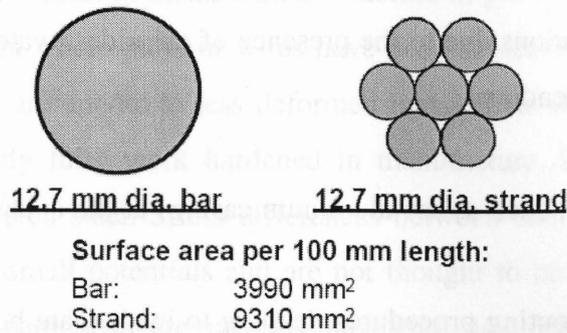


Fig. 3-10 Surface Area of Bars and Strands [30]

High strength prestressing steel is also more prone to other forms of corrosion related deterioration that do not occur in mild steel reinforcement. These forms include stress corrosion cracking, hydrogen embrittlement, fretting fatigue and corrosion fatigue. These types of deterioration mechanisms are very difficult to detect, and can lead to brittle failure with little or no sign of warning.

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

- voids under or next to the tendons.
- lack of passivation of the tendons due to a decrease in alkalinity.
- a corrosive environment at the tendons due to the presence of chlorides, water, and oxygen.
- joints that are not sealed or not watertight.
- chlorides from the mix water or aggregates.
- inadequate concrete cover due to poor construction practices.
- concrete with a high permeability due to a high water-cement ratio and/or poor consolidation.

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

- no protection between the time when the tendon is placed in the ducts and is stressed and grouted.

- poor-quality concrete and improper construction practices.
- a corrosive environment at the tendons due to the presence of chlorides, water, and oxygen (for bonded or unbonded tendons).
- chlorides in the grout mix.
- contact between dissimilar metals, such as the aluminum casings at the anchorages and the steel strands.
- voids in the ducts, due to poor grouting procedures, leading to inadequate bonding between the tendon and the grout.
- inadequate sheathing (damaged during transportation and placement within the structure) leading to possible exposure of the steel tendons to a corrosive environment.
- excessive bleed water.
- chlorides from external sources penetrating the concrete and accumulating at the tendons through perforated ducts or sheathings and at anchorages.

3.5 General Corrosion

General corrosion is the uniform diminution of a metal by chemical attack. [54] Such attack ideally requires that every atom in the exposed surface be equally sensitive to the corrosive medium and that the medium have uniform access to all surface atoms. This is more of an abstract case because probably never occurs in practice, but is approached in the action of acids on some metals – for example, when iron is immersed in hydrochloric acid.

3.6 Localized Corrosion (Pitting Corrosion)

Corrosion will not usually be uniform but will be localized because of the formation of local hydrolytic cells that arise because of differentials within the system. These differentials may be in the chemical composition of the metal or in the concrete, variations in stress or metallurgical structure, or differences in the environment surrounding the steel.

Differentials in metallurgical structure in prestressing tendons are unlikely to be important except where parts of wires have been subjected to sharp bends or kinks. Work hardened parts are anodic to less deformed metal. This would not affect cold drawn wire, which is already fully work hardened in manufacture, but could be significant in quenched and tempered steel. Stress differences between one piece of wire and another would produce only small potentials and are not thought to be significant. The potential arising between high carbon prestressing steel and mild steel reinforcement is greater but does not seem to have led to corrosion damage that has been reported. Greater potentials still arise between steel and metals like zinc and aluminium but in these cases the steel, being cathodic, would be protected from general corrosion though it might become liable to hydrogen embrittlement in certain circumstances, as is discussed later.

Perhaps the most common chemical concentration cell is that caused by differential oxygen potential. These arise in "crevice" situations; where steel emerges from concrete into air; or where the porosity of the concrete varies; where the moisture content of the concrete varies, because moisture restricts the availability of oxygen at the steel surface. Crevices occur at many locations; for example, wire against wire, wire against duct, and wire against aggregate particle. In such circumstances, the steel exposed to higher oxygen concentration may not be passivated. Because many of the situations described cannot be avoided, the danger must be minimized by limiting the supply of oxygen so that the driving potentials disappear and the reactions are stifled.

Another concentration cell of major importance that can result in a dangerous potential is that caused by differential concentrations of chloride, or any other soluble ion. Once the chloride-ion concentration exceeds the threshold value, the passivity of the steel is destroyed and corrosion occurs. Much lower potentials can be dangerous when soluble ions are present than when they are absent because the electrolytic resistance is then low, so that currents flowing are relatively high.

If the localized attack is independent of the crystal structure, the corrosion is named pitting corrosion. If the attack is concentrated at grain boundaries, than we are dealing with intercrystalline corrosion.

Unlike uniform corrosion, pitting corrosion is a localized form of galvanic corrosion. Typically galvanic corrosion occurs in metals having a passivating surface film. Corrosion starts at weak points or at a point of rupture of this film and proceeds until the cross-sectional area of the wire is reduced to such an extent that tensile stresses exceed its ultimate tensile strength, and as a result, the wire fails. This type of corrosion is more harmful than the uniform corrosion since it is very difficult to predict or detect. This makes it difficult to predict the service life of a structure, and besides, it may result in a sudden fracture of the wire. Since the rate of pitting corrosion is usually relatively high, time from construction to failure of the structure may be short.

As mentioned above, pitting corrosion serves as points of stress concentration that localize plastic strain in a small volume of metal. In conventional reinforcement, stresses are uniformly distributed around pitting corrosion. Therefore this steel is less susceptible to failure than high-strength steels of low ductility that do not redistribute the stresses readily, thus increasing considerably the magnitude of stresses in the area affected by pitting corrosion.

According to Roberts, a 10-mil deep pit on the surface of an 80-mil diameter high-strength wire decreases its tensile strength by approximately 15 percent. Further increase in depth of the pit result in a reduction of tensile strength of approximately 15 percent for each 10mil of pit depth. [55]

In addition to the mechanical effect, pitting corrosion serves a source of atomic hydrogen, thus creating conditions favorable for hydrogen embrittlement.

In pretensioned tendons pitting corrosion occurs as a result of insufficient concrete cover over tendons and is commonly observed on the underside of the tendons, where concrete

mixing water accumulates during settlements of freshly placed concrete. In post-tensioned tendons, this type of corrosion may be caused by poor grouting operations.

3.7 Stress Corrosion

Stress-corrosion cracking is another type of highly localized corrosion that can be defined as the cracking of an alloy that results from the combined action of corrosion and static tensile stress, though the amount of corrosion that occurs before cracking failure is often not measurable. [56] The stress may be either residual or externally applied. The effect on the metal may be an intercrystalline crack, the same as occurs in intercrystalline corrosion. Stress-corrosion cracking, however, does not occur in the absence of stress, whereas intercrystalline corrosion does. Unlike intercrystalline corrosion, the cracking may be trans-intercrystalline, which means that the cracks pass through individual crystals. Generally there is a threshold stress below which stress-corrosion cracking does not occur.

Although the mechanism of stress-corrosion cracking is not completely understood, the fact that this type of attack occurs only in alloys indicates that the internal metallurgical structure, which is influenced by composition, heat treatment, and chemical processing, must be a major factor. Failure is in the form of a sudden fracture without deformation.

Because prestressed steel is used in the tensioned state, all such steel is, in theory at least, susceptible to stress-corrosion cracking. In practice, only a very small number of instances of stress corrosion are known, and it is thought that this type of corrosion is caused by nitrates. [57], [58] Further more, some considerable length of exposure to nitrates solution is required at normal temperatures. [59]

Stress-corrosion cracking of steel in chloride-contaminated concrete is very unlikely. [56]

Although stress-corrosion cracking is rare, failure because of hydrogen embrittlement are much more common.

3.8 Hydrogen Embrittlement

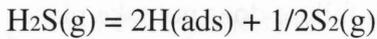
The ductility of steel is reduced as the hydrogen content is increased, and this can result in embrittlement and sudden failure. Reports of failure in the literature most of the time fail to distinguish between stress corrosion and hydrogen embrittlement. In both cases the failure is a sudden and brittle fracture, but the processes leading to failure are fundamentally different. Stress corrosion is the result of anodic process, while hydrogen embrittlement is a cathodic process. [57]

It is believed that hydrogen embrittlement occurs as follows: atomic hydrogen diffuses into the steel, where it exists interstitially. At interior cavities, such as the interfaces produced when sulfide inclusions contract on cooling from the melt, or decohere during rolling, the hydrogen atoms can recombine to form gas. Hydrogen gas atoms in the grain boundaries are at high pressure and exert a significant tensile force tending to separate the iron atoms. The equilibrium partial pressure of hydrogen in a chloride-contaminated pit in a steel surface can be as high as 1000 atm. Embrittlement occurs when the pressure created by hydrogen gas atoms in a combination with a high tensile stress in prestressing steel exceeds the yield strength of steel, some 3 to 20000 atm. In addition to creating an internal pressure, hydrogen gas atoms also lower the cohesive forces between adjacent grains. The danger of embrittlement increases at a higher external hydrogen activity. [60]

For hydrogen embrittlement to occur there must be present a source of atomic hydrogen such as steel itself, hydrogen sulfide (H_2S), galvanic cells, and some special solution used in the laboratory to evaluate susceptibility of steel to stress corrosion.

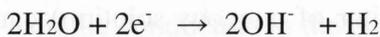
Atomic hydrogen may be present in the prestressing steel from the time when it was manufactured. Hydrogen can originate from the molten state, for example from steel refining. It also may originate during pickling of hot-rolled rods in a solution of sulfuric acid prior to cold drawing. Some impurities that may be present in sulfuric acid, such as sulfur compounds, do not allow atomic hydrogen evolving during pickling to combine into molecules.

Atomic hydrogen may also be produced by gas dissociation from hydrogen sulfide present in the atmosphere in industrial and farming regions, in high-alumina cement concrete, and in concrete made with blast furnace slag as follows:



It is believed that in hardened high-alumina cement concrete, hydrogen sulfide is formed as a result of reaction between sulfide compounds and carbon dioxide from the atmosphere. [61]

A common internal source of atomic hydrogen is a galvanic cell developed in a reinforced concrete structure. In a corrosion cell hydrogen evolves as a result of water dissociation at cathodic sites:



Studies showed this reaction can occur when moisture is present, when the pH of the electrolyte does not exceed 9, and when the metal-solution potential is in the range of -0.6 to +0.3 V. these conditions may exist in concrete under field conditions, especially in the presence of chlorides. [62] The potentials measured in their experiments were in range of -0.4 to +0.5 V. The pH in the corrosion pit was as low as 5. That means that hydrogen embrittlement may occur even when hydrogen sulfide is not present. This is in agreement with results of field observations of prestressing steel showing occurrence of hydrogen embrittlement is quite common in normal atmosphere, not polluted with H₂S, especially in the presence of chloride ions. This is also consistent with the development of electrolytic cells, which are the primary cause of failure. The reaction also occurs in zones of oxygen impoverishment, such as at the contact points between wires, or between wires and the duct. [57], [63]

Hydrogen embrittlement is enhanced by slow strain rates and elevated temperatures. Steel that has been embrittled by hydrogen will show the following characteristics: [64]

- the notch tensile strength may be less than normal and directly reflects the loss of ductility caused by hydrogen.
- delayed failure may occur over a wide range of applied stresses.
- there is only a slight dependence of the time to failure on the applied stress.
- there is a minimum critical value of stress below which failure does not take place.

There are a number of possible sources for the hydrogen, but that which results from corrosion reactions is probably of the greatest practical importance. In the “normal” case, the atomic hydrogen that is formed in every corrosion process quickly recombines to molecular hydrogen, which, at normal temperatures, is harmless to prestressing steel.

In the presence of contaminants, of which hydrogen sulfide and sulfur dioxide are of the greatest practical importance, the hydrogen is able to penetrate into the lattice of the steel, where it preferentially accumulates at the grain boundaries. Very small amounts of hydrogen sulfide (as little as a few milligrams of gas per liter of aqueous solution) can cause fracture. Although the mechanism is not perfectly understood, the hydrogen atoms combine into molecules and the steel becomes brittle. [64]

Other examples of brittle fracture have been reported in which hydrogen was absorbed into the steel during corrosion before it was stressed. Failure occurred within a few hours of stressing. [53]

In general, cold drawn wire has a much lower tendency to stress corrosion and hydrogen embrittlement than oil quenched and tempered wire of equal tensile strength. [57], [58], [65], [66]

3.9 Galvanic Corrosion

Galvanic corrosion occurs when two dissimilar metals are present in an electrolyte. Because a potential difference exists between the two metals, the anode will be attacked and consumed.

If the two metals are electrically connected outside the electrolyte, hydrogen will be evolved at the cathode. This phenomenon has been found to occur in practice when galvanized ducts were used and grouting was inadequate, and electrical contact occurred where the prestressing steel touched the duct. [57] If at such point in the incompletely grouted duct, pockets of water and air are present and a galvanic cell will exist. This means that atomic hydrogen may penetrate the steel and hydrogen embrittlement could result.

The possibility of galvanic corrosion is also important when considering the selection of metallic coatings for the prestressing strands.

3.10 Stray-Current Corrosion

Stray current are defined as electrical currents that flow along unintended paths. Stray-current corrosion occurs when a direct current flows at least partly electrolytically. Any metallic conductors in the electrolyte may then become a path for the current.

The classical conditions for stray current corrosion were found in pipes and cables lying beneath and parallel to electric tramway and railway lines carrying the return direct current from passing vehicles. The pipes behaved as shunts on lengths of rail between the points at which electrical earth returns were connected to the rails, the current passing ionically through the soil down from rail to pipe and back up again at areas in the pipe which became cathodic and anodic respectively. Severe corrosion was often observed at the anodic areas and could occur even if there was no free oxygen available. In principle, reinforcing steel or prestressing steel similarly disposed in the same kind of electric field would be vulnerable in the same way except in so far as the concrete cover would normally be less electrolytically conductive than wet soil. Reinforcement carrying direct current for any other reason could be at risk provided some of the current was leaving the steel and leaking to earth through the concrete so that anodic reaction of necessary had to occur at the metal surface.

The current entering the conductor causes cathodic effects. The point where it leaves the conductor is subject to anodic effects (corrosion), usually concentrated at the point of discharge from the structure.

Because stray-current effects tend to be concentrated at the ends of the tendons, a post-tensioned structure is more prone to damage than a comparable pretensioned structure. This situation arises because in post-tensioned structures the full length of the tendon is subjected to tension, whereas in pretensioned structures there is a reduction in tension at the ends because of the transfer length of the strand.

The cathodic reaction should not be ignored. Because hydrogen ions are produced, embrittlement of the steel is possible, although the risk is diminished by the fact that cathodic effects take place over a larger metal surface than do the anodic effects. However, in concrete containing chloride, there is a possibility of hydrogen evolving under even very small currents.

The phenomenon is well known among engineers concerned with the provision of urban supply services. The obvious precautions to be taken, where the possibility of the occurrence of this type of corrosion exists, include the use of high quality chloride-free concrete, the interposition of electrical insulation such as polythene sheeting between the source of the current and the vulnerable members and the provision of permanent electrical connections by means of which the voltages arising in the reinforcement can be monitored and through which protective cathodic currents can be supplied if necessary.

There are very few documented cases of stray-current damage to prestressed structures, but where it occurs, the consequences can be serious. Direct-current welding activities and direct current transmission in connection with transit systems are the most common sources of stray currents. No effects had been found stray alternating currents of normal frequency (50 to 60 Hz) on prestressing steel in concrete. [65]

3.11 Corrosion of reinforcement before embedding in concrete

The effects of rusting of reinforcing bars during storage before inclusion can be assessed as follows:

a) Loss of metal in relation to surface area and cross-section. The formation of rust on reinforcement may be associated with a reduction of the cross-sectional area of the bars. If the remaining cross-sectional area falls below the lower tolerance limit required by the design, the amount of reinforcement must be correspondingly increased. The removal of metal by rusting generally affects the whole of the surface uniformly, so that the shape of ribbed reinforcing bars remains in practice.

In the case of prestressing steel, in general only very slight rust-colored patches are permissible before inclusion in the concrete. Loss of metal through rusting is therefore not permitted.

b) Effect of corrosion on the properties of the bars. Rusting if corrosion pits form on the surface and produce a notching effect may reduce the fatigue strength of the bars. This effect is generally more marked in plain bars than in ribbed bars, in which as a rule the notching effect in the ribs masks that of the corrosion pits.

In prestressing steel the fatigue strength of the steel and the risk of stress corrosion (including intergranular corrosion) are adversely influenced by surface corrosion. Corrosion pits in the surface act as notches, which under fatigue stress or in a corrosive atmosphere when in a stressed condition, can lead to premature fracture. The influence of corrosion notches is relatively more important with steels of higher strength. Corrosion has a marked influence on the mechanical characteristics such as elongation at fracture and on the number of stress repetitions but less influence on strength. The deformation capacity (ductility) of surface-corroded steel is lower than that of uncorroded steel. Also the detrimental influence of corrosion occurs more frequently in smooth round bars or wires than in ribbed bars or wires. Stress corrosion is a particularly serious matter wherever high stresses, due to any cause, may be imposed.

c) Effect of corrosion on welding (tack welding). Because of the prefabrication of reinforcement assemblies, whether on site or in a bending shop, the use of tack welding has increased. The greater the amount of rust, the more difficult welding becomes and the risk of faults is increased. Harmful penetration notches can occur more easily. This is particularly the case with electric arc welding with coated electrodes because a layer of rust makes initiation of the arc more difficult. Shielded arc welding with protecting gas (e. g. CO₂) is somewhat better in this respect, although in principle the same risks are present. Considerably greater difficulty can arise in the case of electrical resistance spot welding. Where there is rusting, the contact resistances are no longer defined, and this makes satisfactory welding difficult or impossible; the bars should therefore be both dry and free from rust or, at the worst, have only a thin rust-colored film.

3.12 Permissible degree of corrosion in structures

Prestressed concrete structures that have been designed and built in accordance with codes and standards based upon research should not show corrosion damage during their lifetime. The cracks that usually and inevitably occur in the zones subjected to tension should represent no risk for the reinforcement if the above rules are observed. Only in rare cases does a "permissible degree" of corrosion occur. This is a progressive phenomenon that must be stopped by remedial treatment.

In the case of unsightly corrosion defects it is difficult to give guidance on the extent of the faults that can be permitted since it will depend on the type of structure, its use, accessibility, environment and in certain circumstances on its age and on the amount of use it has already had. Although there is still a considerable amount of disagreement on the value of placing limits on chloride content in mix ingredients, some recommendations have been made by AASHTO and the American Concrete Institute, shown in Fig. 3-11. Since chlorides added in the concrete mix tend to be more uniformly distributed than chlorides from external sources, it is not as likely to lead to the creation of concentration cells. However, when concrete members are expected to be exposed to chlorides, it is advisable to keep any chlorides added to the concrete from the mix ingredients to a minimum.

	Acid Soluble (Performed by ASTM C1152)	Water Soluble (Performed by ASTM C1218)
Prestressed Concrete	0.08	0.06
Reinforced Concrete in Wet Conditions	0.10	0.08
Reinforced Concrete in Dry Conditions	0.20	0.15

Fig. 3-11 ACI-recommended chloride limits for new construction [52]

Defects of this kind should not be permitted on the visible surfaces of concrete structures. Such defects generally consist of areas of rust staining which arise from water running over steel parts that are exposed or protrude from the concrete, depositing the rust product on the concrete.

If we are talking about corrosion defects that are not endangering the safety of the structure these defects generally occur at the onset of corrosion and can be recognized by spots of rust on the visible surfaces of the concrete. They may be purely local or be extended over an area or along a line (generally along a crack). Among the causes of these defects are:

- pockets of segregated aggregate in the concrete.
- mechanical damage (e.g. spalling) in the concrete cover.
- cracks running parallel to the axis of the nearest reinforcing bar (longitudinal cracks). They generally occur as a result of overstressing of the bond due to faulty construction or inadequate concrete cover.
- open transverse cracks due to faulty construction.
- incipient corrosion of steel parts that protrude from the concrete. These include for example handrail standards, corrosion in which leads to danger to the user.

At this stage the defects do not represent any danger to the safety of the structure. However, they should not be tolerated, but should be restored, otherwise the corrosion will spread.

Corrosion defects that will endanger the safety of the structure can occur if the defects mentioned above are not prevented or treated. If the incipient corrosion described above spreads, then by the removal of metal, the cross-section of the reinforcing rod is decreased and hence the strength of the structure and the safety of it are progressively reduced to the point of collapse. If the concrete cover over the reinforcement is burst away from the steel by the formation of rust, this will accelerate the propagation of the corrosion. These defects are a danger to the safety of the structure and must be cured.

Corrosion defects that may lead to collapse of the structure are more likely to occur in prestressed concrete than in the regular reinforced concrete. With prestressing steel, corrosion cracks can occur and, without warning, can lead to fractures of individual cables, wires or bars and to collapse of the structure. The lack of warning arises because the fractures can occur without any indication of rust formation.

3.13 Influence of environmental exposure on corrosion

Environmental exposure of the structure it is a very important factor that will affect the corrosion and the rate of corrosion. Marine exposure and cold climate (because of the deicing used in this type of climate combined use freeze and thaw cycle) are the most aggressive environment exposure for a structure, from the corrosion perspective.

3.13.1 Coastal exposure

Coastal exposures are one of the most severe environments for concrete structures. This danger is particularly true for structural components located directly in the seawater, as in the case of bridge substructures. Seawater contains dissolved salts that affect the durability of concrete. The most prevalent salts in order of quantity are sodium, magnesium and

potassium chlorides and magnesium, calcium and potassium sulfates. These salts provide sources of chlorides and sulfates, which can lead to corrosion of reinforcement and sulfate attack on concrete. To a lesser extent, these salts also provide a source of alkalis, which may lead to expansive alkali-aggregate reactions if reactive aggregates are present. There are four main exposure zones for a structure in a coastal exposure. Each zone and the associated forms of attack are described below and shown in Fig. 3-12. [30]

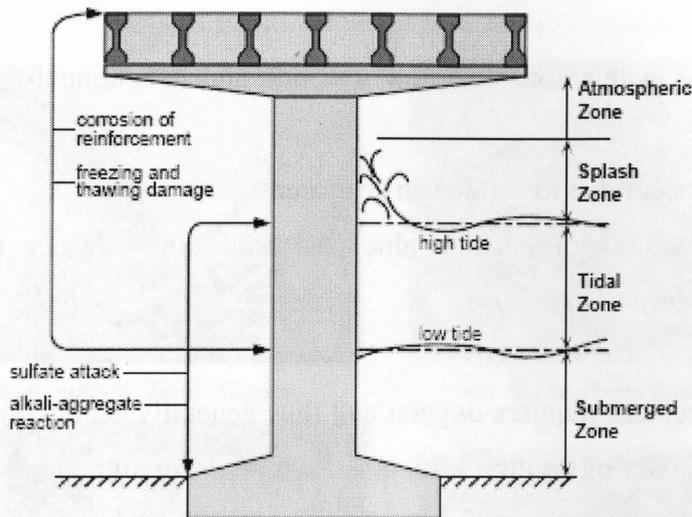


Fig. 3-12 Substructure exposure zones and forms of deterioration in coastal seawater exposure [30]

Atmospheric Zone: concrete in this zone is never in direct contact with the seawater, but is exposed to moisture and salts from spray and salt laden mist. Possible Attack:

- corrosion of reinforcement may occur due to chlorides.
- freezing and thawing damage may occur (dependent on climate).

Splash Zone: concrete in this zone is above high tide but is subjected to direct wetting due to wave action. Possible Attack:

- corrosion of reinforcement may occur due to chlorides.
- freezing and thawing damage may occur (dependent on climate).

Tidal Zone: concrete in this zone lies between low and high tide, and is subjected to continual cycles of submerging and exposure to air. Possible Attack:

- corrosion of reinforcement may occur due to chlorides.
- freezing and thawing damage may occur (dependent on climate).
- sulfate attack may occur due to sulfates in seawater.
- alkali-aggregate reaction may occur due to alkalis in seawater (if reactive aggregates are present).

Submerged Zone: concrete in this zone is below low tide and is continually submerged.

Possible Attack:

- sulfate attack may occur due to sulfates in seawater.
- alkali-aggregate reaction may occur due to alkalis in seawater (if reactive aggregates are present).

Corrosion of steel reinforcement requires oxygen and thus generally occurs only in zones, which experience some amount of drying. Also, corrosion rates are highest when humidity is in the 90-95% range. The greatest risk of corrosion occurs in the splash and atmospheric zones for these reasons. [30]

Corrosion in the tidal zone is normally limited due to the shorter drying periods and slower rate of oxygen diffusion through saturated concrete. The submerged zone of concrete has a low risk of corrosion due to lack of oxygen. Several examples of typical corrosion damage in coastal exposures are shown in Fig. 3-13. These photos are all of Texas bridges located along the Gulf of Mexico.

Frost damage is most severe in concrete that is saturated, and therefore concrete within the tidal zone or immediately above the high tide level may experience the most significant damage. Freeze-thaw damage rarely occurs below the low tide level since the seawater would also have to freeze.

The temperature range to which a structure is subjected also affects durability. Increases in temperature have an accelerating effect on many chemical reactions, including corrosion. The general rule of thumb is that a temperature increase of 10° C doubles the rate of reaction. Traditionally, seawater environments in cold climates, such as the North Sea, were viewed as the most severe exposure for structures. More recently, the accelerating effects of high temperatures have been recognized as equally or possibly more severe than the combination of freezing temperatures and corrosive environments. [30]



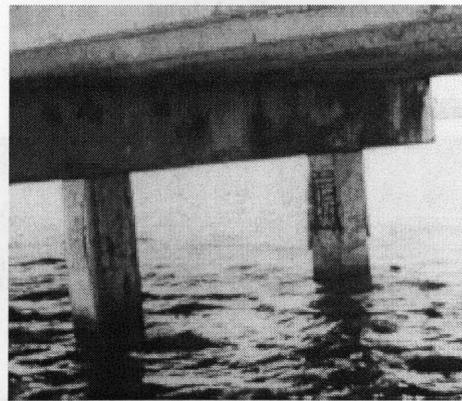
(a) Atmospheric Zone - Girder



(b) Atmospheric Zone - Bent Cap



(c) Splash Zone - Column



(d) Splash Zone - Column

Fig. 3-13 Coastal exposure corrosion damage in a bridge [67]

3.13.2 Freezing exposure

Environments where structures may be exposed to freezing temperatures may lead to freeze-thaw damage of concrete. A secondary effect is that the use of deicing chemicals in

freezing exposures can exacerbate freeze-thaw damage and may lead to corrosion of steel reinforcement if the deicing agents contain chlorides.

The severity of freeze-thaw damage of concrete is a function of the presence of moisture in the concrete and the number of times the moisture freezes and thaws. Frost damage worsens when repeated cycles of freezing and thawing occur. Thus a moderate winter climate which experiences many freeze-thaw cycles can cause more frost damage than a severe winter climate that remains below freezing for long periods.

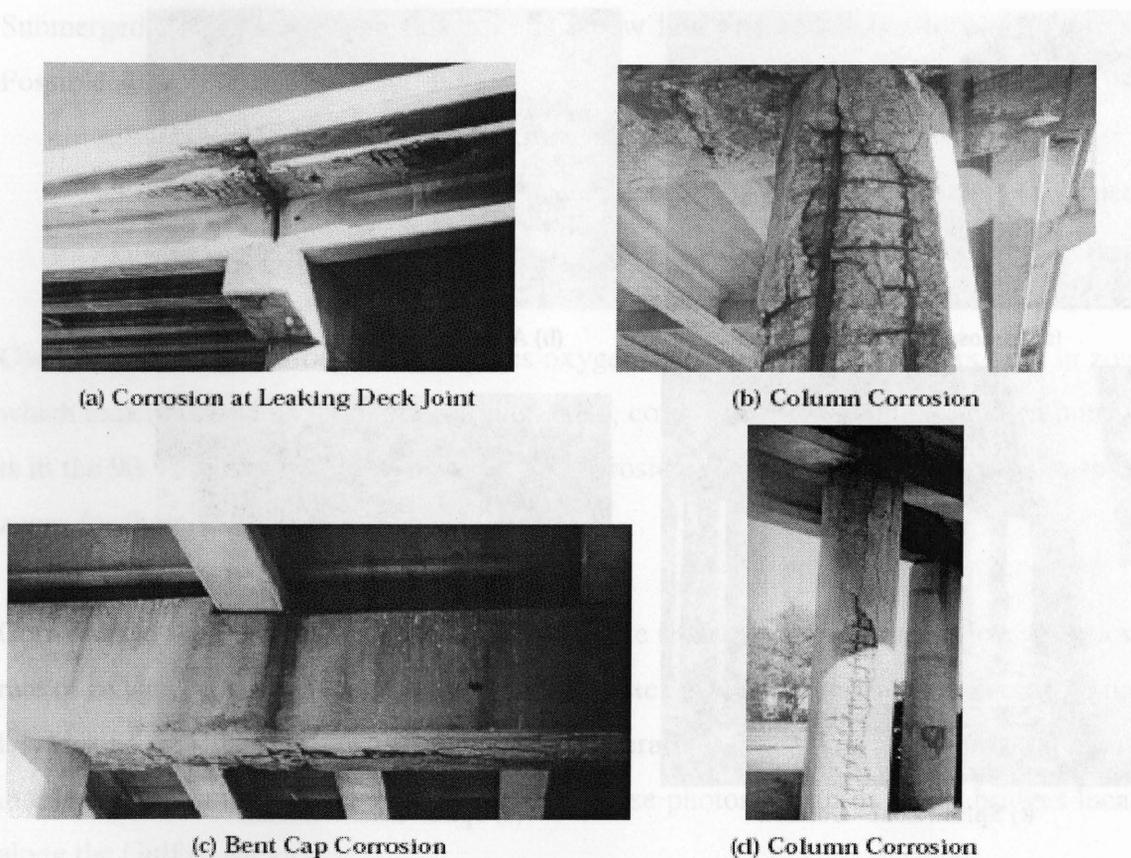


Fig. 3-14 Corrosion due to deicing chemicals in freezing exposure [67]

Even when deicing chemicals would not normally be applied directly to structure components, moisture and chlorides may come into contact with the respective element through leaking joints, inadequate drainage. Corrosion of steel reinforcement from this type of exposure can be severe due to its localized nature and often very high concentration of

chlorides. Photos of substructure corrosion resulting from drainage of deicing chemicals are shown in Fig. 3-14. Typical damage locations include columns and the underside of bent caps located near leaking deck joints. Very severe corrosion damage results due to the localized nature of the attack.

3.13.3 Aggressive soils

Chemical attack on concrete in the form of sulfate attack or alkali-aggregate reactions may occur in soils containing sulfates or alkalis. The presence of these aggressive agents must be accompanied by moisture for attack to occur (assuming the concrete is susceptible to either form of attack). Moisture provides the transportation mechanism for sulfates and alkalis to penetrate the concrete, and is also necessary for the deleterious reactions to occur.

4. Corrosion prevention and protection for prestressed concrete

Many aspects of corrosion protection for prestressed concrete structures are similar to corrosion protection for conventional reinforced concrete structures. The basics of corrosion protection for reinforced concrete structures are provided by many sources. Prestressed concrete introduces several extra variables in corrosion protection, including additional hardware and grouting of the tendon ducts. Corrosion protection requires attention during design and construction of the structure, and through the life of the structure with proper maintenance.

4.1 Protection against corrosion

Corrosion of steel reinforcement or prestressing steel in concrete is a complex phenomenon influenced by many factors. For this reason, it is best to approach corrosion protection by providing several measures to guard against different influences and breakdowns or limitations in any single protective measure. This strategy is often referred to as providing multilevel corrosion protection. Corrosion protection in posttensioned concrete can take many forms. There are four general categories of protection mechanisms, and they are explained below, together with the protection method:

1) Prevent chlorides from entering concrete:

- reduce permeability by low w/c ratio; use fly ash/silica fume/slag; eliminate voids (thorough compaction, reinforcement detailing); moist or steam curing; reduce/control cracking.
- waterproof membranes.
- surface polymer impregnation.
- structural form.
- structural detailing (drainage, anchorage locations).
- crack width limitations.

2) Prevent chlorides from reaching steel (in the event chlorides penetrate the concrete):

- increased cover.
- coated reinforcement & tendons (epoxy-coated, copper clad, etc.).
- encapsulated posttensioning systems.
- plastic post-tensioning ducts.
- cement grout (bonded posttensioning).
- greases and waxes (unbonded posttensioning).

3) Control corrosion reactions:

- galvanized reinforcement/tendons.
- electrical isolation.
- corrosion inhibitors.
- cathodic protection.

4) Remove reactive substance (steel):

- FRP reinforcement & tendons.
- stainless steel reinforcement/tendons.

The corrosion of prestressing steel should normally be prevented by so investing the steel with concrete or grout that:

- a good passive oxide film is formed on its surface initially.
- the rate at which atmospheric gases permeate the concrete should be very low so that the carbonation front will not reach the steel in the lifetime of the structure and so that oxygen will not be continuously available at such points at which passivity may accidentally be impaired.

- a high ionic resistance in the concrete ensures that electrolytic corrosion reactions can proceed only negligibly slowly.

Of these considerations the need to exclude air is the most important and designers, consultants and contractors should be united in their determination to keep the air out of the system.

In order to ensure success, rules should be formulated and adopted concerning the following:

- **Concrete quality and thickness.** Quality should be defined in terms of permeability to air or in similarly appropriate terms. Means of measuring quality should be developed preferable non-destructively. Means of measuring cover thickness should be developed. Rules should be devised prescribing minimum thickness appropriate to different qualities of concrete.

- **Chloride.** A measuring technique for determining chloride content of mixed concrete, on the site, should be developed. A strict minimum chloride content at that stage should be agreed and enforced, relative to type of concrete in use.

- **Cracks.** If cracks are to be permitted at all, strict limits must be agreed as to where they can be tolerated and how large. Depth should be considered more important than width.

Where there is special difficulty in achieving the necessary degree of protection with concrete alone consideration should be given either to coating the wires to exclude oxygen by their physical isolation or to coating the exterior of the concrete in such a way as to exclude oxygen from the whole system. It is probable that the cost of these measures is so high that they could be applied only in special circumstances.

Temporary coatings for prestressing tendons are a very effective way of preventing the corrosion before installation. A number of temporary protective coatings for use during

transit, storage or in ducts prior to grouting have been considered. These must be effective in their purpose without affecting the steel adversely in any way and without affecting the efficacy of the permanent protection by grout, or otherwise, which will later be applied. Suggested coatings include a variety of oils, waxes and greases, phosphate and zinc coatings, and, in ducts, flushing with dry air or inert gases, or inhibiting aqueous solutions including lime-water. Each method has its own possible special applications and no set of rules can be offered for general use.

4.2 Concrete as corrosion protection

Concrete is the single most important factor in ensuring structural longevity since it affects all aspects of durability, including corrosion of reinforcement. Concrete acts as a physical barrier to moisture and chlorides, and provides the alkaline environment necessary for formation of the passive film on the surface of the steel. The effectiveness of concrete as corrosion protection for steel reinforcement is dependent on the concrete material properties and on design and construction practices.

The corrosion protection provided by the concrete can be improved in many ways, including reduced permeability of concrete, increased clear cover, corrosion inhibitors and concrete surface treatments. Each is described below.

4.2.1 Concrete Permeability

The permeability of the concrete is the single most important factor for providing durable concrete, including the corrosion protection. The permeability of concrete controls the rate at which moisture, oxygen and carbon dioxide penetrates the concrete. Because the penetration of moisture provides the transport mechanism for chlorides and other aggressive substances, lowering the concrete permeability increases the length of time before aggressive agents reach the steel, and thus improves corrosion protection. The permeability of concrete is affected by four general factors: [68], [69]

4.2.1.1 Pore structure of the cement paste

The transportation of water and gases through concrete is significantly affected by the size, distribution and continuity of pores within the cement paste. The cement paste is assumed to consist of C-S-H gel, void spaces or capillary pores and various products of the hydration process. The C-S-H gel is defined as the cohesive mass of hydrated cement, and consists primarily of calcium silicate hydrates (C-S-H). In addition to the capillary pores within the cement paste, the C-S-H gel also contains voids referred to as gel pores. These pores are the interstitial spaces between the gel particles.

Due to their nature, the gel pores are much smaller than the capillary pores. The size of the gel pores is in the range of approximately 15 to 20 Angstroms in diameter, or only about one order of magnitude larger than a molecule of water. Because the gel pores constitute the space between gel particles, the size and distribution of the gel pores is characteristic of the type of cement and is relatively independent of the water-cement ratio (w/c ratio) and the extent of hydration. Thus, the porosity of the C-S-H gel is affected by the properties of the cement itself. In general terms, finer cements will produce gel with a lower porosity than coarse cements. In mature concrete, gel pores constitute approximately 28% of the volume of the C-S-H gel. However, due to the small size of the gel pores and the fact that the pores are generally well distributed and discontinuous throughout the gel, the overall effect of the gel pores on the permeability of the concrete is minimal. [68]

Capillary pores are the remnants of the water filled space that exists between the partially hydrated cement grains. The typical size range for capillary pores is 500 to 500,000 Angstroms. [70] Due to their larger size in comparison to gel pores, the permeability of the cement paste is primarily controlled by the capillary pores. The size and distribution of capillary pores is a function of the hydration of the cement. As the process of hydration proceeds, the water within the pores is replaced by solid hydration products (C-S-H gel) and the volume of capillary pores is reduced. In addition, the formation of C-S-H gel also tends to obstruct the capillary pores, causing them to become discontinuous and further

reducing the permeability of the cement paste. Thus, the permeability of the cement paste due to capillary porosity is influenced by the factors affecting the hydration of the cement.

The capillary porosity is also significantly affected by the water content of the concrete. For lower water-cement ratios, the permeability of the cement paste may be considerably reduced due to the greater extent of C-S-H gel formation that reduces the volume and continuity of the capillary pores. In general, for water-cement ratios less than 0.38, the volume of C-S-H gel formed during hydration of the cement is enough to completely fill the capillary pores. For water-cement ratios between 0.38 and 0.6, the amount of C-S-H gel formation is usually significant enough to disrupt the continuity of the capillary pores, provided that complete hydration of the cement is allowed to occur. For water-cement ratios higher than 0.7, gel formation is insufficient to block the capillary pores even with complete hydration. Tests have shown that concrete permeability decreases by up to four orders of magnitude as the water-cement ratio is reduced from 0.75 to 0.26. [71] The effect of water-cement ratio on chloride ion penetration was demonstrated by Clear, as shown in Fig. 4-1. ACI Committee 201 reports chloride ion permeability for concrete with water-cement ratios of 0.40 and 0.50 to be 400 to 600 percent higher than concrete with a water-cement ratio of 0.32. For concrete exposed to aggressive environments, Committee 201 recommends that the water-cement ratio should be as low as possible, and preferably below 0.40. [72], [73]

The use of mineral admixtures with pozzolanic characteristics influences both the gel and capillary porosity of the cement paste, and hence affects the permeability. The most commonly used mineral admixtures in North America are fly ash and silica fume. Considerable research has shown lower permeability and reduced penetration of chlorides for concrete containing fly ash and silica fume in comparison to ordinary Portland cement concrete. [74], [75]

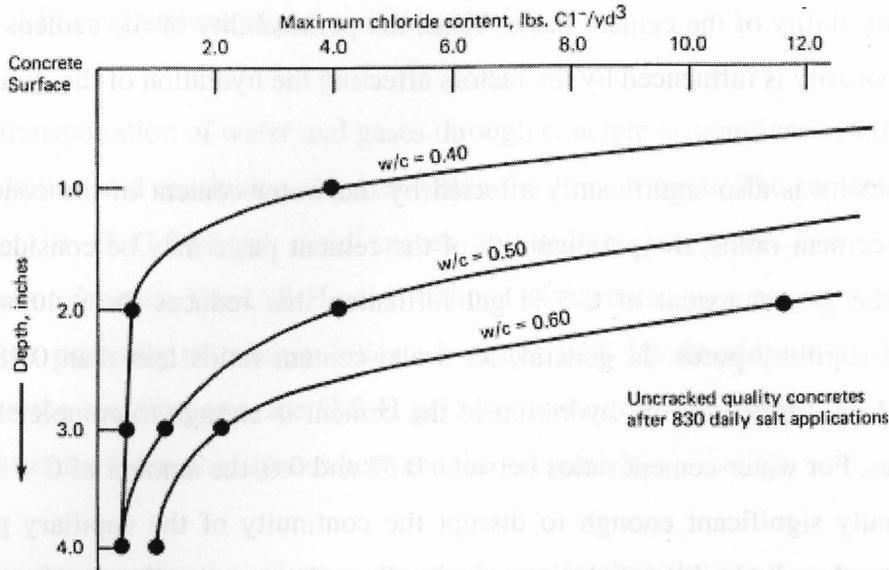


Fig. 4-1 Effect of Water-Cement Ratio on Chloride Ion Penetration [71]

Fly ash is normally used to replace a portion of the cement in concrete. Typical replacement amounts range from 20 to 35 percent by mass. Partial cement replacement with fly ash may reduce concrete permeability in three ways: [68], [75]

- the hydration of a material with pozzolanic properties (i.e., the pozzolanic reaction) consumes calcium hydroxide ($\text{Ca}(\text{OH})_2$) and produces additional or secondary calcium-silicate-hydrate (C-S-H) gel. (The primary C-S-H formed by hydration of Portland cement comprises the largest portion of the hardened cement paste, and is primarily responsible for the strength of the paste. Calcium hydroxide is a by-product of cement hydration. It is soluble, and may be leached out of the concrete, leaving a network of pores that increases permeability.) The pozzolanic reaction increases the strength and reduces the permeability of the paste by removing soluble $\text{Ca}(\text{OH})_2$ and filling capillary pores with C-S-H.
- fly ash particles have a spherical shape, and thus have lower interparticle friction than the angular cement particles. Therefore, less water is needed to attain a given slump, leading to fewer capillary pores. Typical water reduction ranges from 5 to 15

percent in comparison to concrete with Portland cement only (similar total amount of cementitious material). [68]

- the lower interparticle friction of fly ash particles also reduces permeability through better consolidation or “packing” of the fly ash and cement particles. This improved consolidation tends to reduce entrapped air and the volume of large capillary pores.

Silica fume may be used to replace a portion of the cement, or it may simply be added to the concrete to increase the total cementitious content. Silica fume also possesses pozzolanic properties, reducing permeability through production of additional C-S-H gel and removal of Ca(OH)_2 . Silica fume particles are spherical, but are ten to one thousand times smaller than fly ash and cement particles. Due to their small size and large surface area, silica fume normally increases water demand and superplasticizer is needed to maintain low water-cement ratios. The small particle size is beneficial though, contributing to lower permeability in three ways: [68], [75]

- the small particle size and increased surface area limits bleedwater. Permeability is reduced by eliminating voids resulting from bleedwater trapped under the aggregate.
- the hydration of cement is enhanced as the very small silica fume particles provide nucleation sites for calcium hydroxide. Ca(OH)_2 forms in numerous small crystals rather than large crystals.
- the very small spherical particles improve “packing” of silica fume and cement particles, filling the voids in the transition zone between cement particles and aggregate.

The importance of complete cement hydration on the permeability of the concrete is evident from the preceding paragraphs. In general, steam or moist cured concrete will have a lower permeability due to the potential for thorough hydration. Concrete that is allowed

to dry prematurely may have a significantly higher permeability due to the lesser extent of hydration. In addition, drying may produce shrinkage cracks through the C-S-H gel separating capillary pores, thus rendering them once again continuous. Proper curing is particularly important when mineral admixtures such as fly ash and silica fume are used. Guidelines for curing are provided by the ACI Committee 308 report "Standard Practice for Curing Concrete". [70], [76], [77], [78]

4.2.1.2 Aggregate

The permeability of concrete is also affected by the properties of the aggregate, although to a lesser extent than water-cement ratio. The influence of the aggregate is usually small in comparison to the cement paste since the aggregate is completely surrounded by the paste. [68] For aggregates with very low permeability, the permeability of the concrete may be reduced due to the longer flow path required for moisture to circumvent the aggregate.

4.2.1.3 Voids in the concrete

Consolidation or compaction of concrete is also necessary for low permeability. Voids or excessive entrapped air resulting from poor placing practices, lack of vibration or congested reinforcement will increase permeability. The effect of inadequate consolidation on chloride penetration is illustrated in Fig. 4-2. In some situations, slightly larger sections may be required to relieve reinforcement congestion and facilitate thorough placement of the concrete.

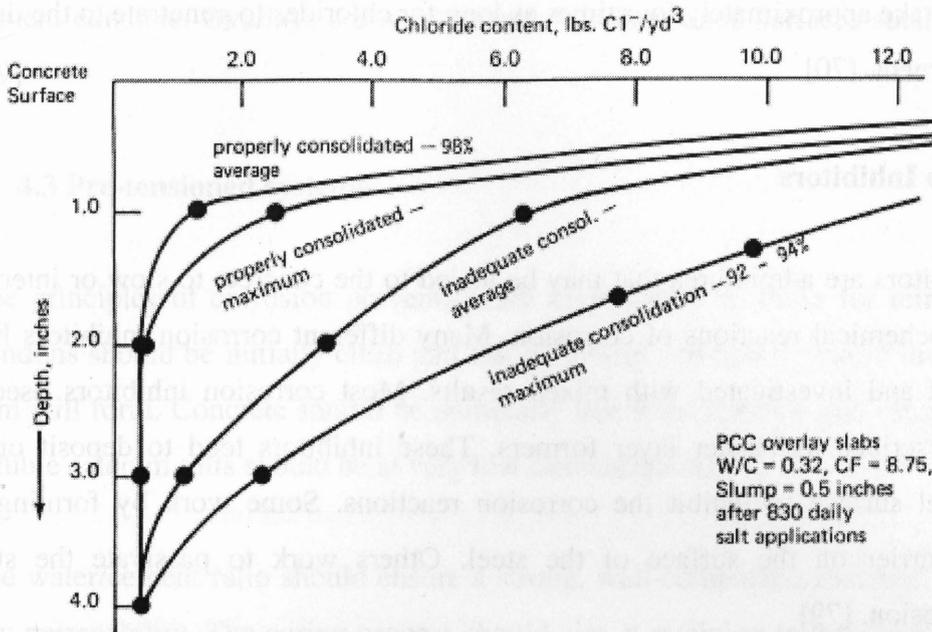


Fig. 4-2 Effect of Consolidation on Chloride Ion Penetration [71]

4.2.2 Cracking of the concrete

Cracks within concrete will have the obvious effect of increasing the permeability of concrete by providing direct routes for the movement of moisture. In order to ensure low permeability, special attention must be given to the prevention of cracks due to sources such as creep, shrinkage, temperature effects, plastic settlement during curing, load effects, either due to design loads or overloads, and imposed deformations due to support settlement. In severe exposure conditions, the use of prestressing to reduce the cracking resulting from normal reinforced concrete behavior is a possible approach for maintaining low permeability of concrete.

4.2.3 Concrete Cover Thickness

The thickness of concrete cover over the reinforcement plays a significant role in corrosion protection. Increased clear cover provides improved protection for the steel, particularly if low permeability concrete is used. The penetration of chlorides over time can be approximated by a square-root time law. For example, if the concrete cover thickness is

doubled, it will take approximately four times as long for chlorides to penetrate to the depth of the reinforcement. [70]

4.2.4 Corrosion Inhibitors

Corrosion inhibitors are admixtures that may be added to the concrete to slow or interfere with the electrochemical reactions of corrosion. Many different corrosion inhibitors have been developed and investigated with mixed results. Most corrosion inhibitors used in concrete are described as barrier layer formers. These inhibitors tend to deposit on or around the steel surface to inhibit the corrosion reactions. Some work by forming an impermeable barrier on the surface of the steel. Others work to passivate the steel, preventing corrosion. [79]

One of the more commonly used inhibitors in concrete is calcium nitrite ($\text{Ca}(\text{NO}_2)_2$). This compound reacts with ferrous ions produced during corrosion to stabilize the passive layer on the steel and prevent further corrosion. This corrosion inhibitor does not affect the time to corrosion initiation, but rather limits the corrosion rate after corrosion has started by re-passivating the steel. The calcium nitrite is consumed during the process, and therefore the corrosion protection is not indefinite. Neville states “inhibitors are no substitute for concrete of low penetrability: they are merely an additional safeguard”. [68]

4.2.5 Concrete Surface Treatments

Concrete surface treatments work to improve corrosion protection by preventing moisture and chlorides from entering the concrete. Surface treatments include waterproof membranes and surface polymer impregnation. In the latter, the exposed surface of the concrete is impregnated with polymer that fills the voids and cracks in the concrete, providing a barrier with very low permeability. Various overlays may be used to provide a low permeability barrier over existing concrete. Options include polymer concrete overlays, latex-modified concrete overlays and overlays with low permeability Portland

cement concrete. Overlays are most practical on horizontal surfaces such as bridge decks. [29]

4.3 Pre-tensioned systems

The principles of corrosion prevention are very similar to those for reinforced concrete. Tendons should be initially clean and free of grease and rust to ensure that a good passive film will form. Concrete should be nominally free from chloride and other anions yielding soluble calcium salts should be at very low concentrations.

The water/cement ratio should ensure a strong, well-compacted concrete, which will have low permeability. The curing process should aim at retaining in the concrete the water not needed for hydration. The concrete cover should be adequately thick and dense so that the effective permeation rate for air is not so high that carbonation to the depth of the prestressing wire is possible within the intended lifetime of the structure. Ends at which tendons emerge from concrete should be sealed with air-tight mastic. Designs should ensure absence of cracks in concrete at full load.

4.4 Post-tensioned systems

In these the environment is made up of grout within a lined duct. It is extremely difficult to ensure that this duct is perfectly filled so that there are, probably inevitably, pockets of "grout water" and pockets of moist air trapped in the duct and enveloping certain lengths of cable. As long as there is free oxygen present, these pockets of moist air and, more especially, the meniscus against the steel at the air/water interface are extremely dangerous corrosion sites. It is clearly impossible to eliminate excess water from grout but every effort should be made to eliminate air pockets and so keep the oxygen in the system initially as low as possible. All practicable steps must also be taken to prevent oxygen from reaching the steel from the outside concrete cover should be adequately thick and dense to maintain the transverse flow of oxygen appropriately low, all vents and anchorages should be sealed

off positively against the inward flow to oxygen and care should be taken that air from outside cannot flow along between the wires of the assembled Tendon.

Tendons and cables should be clean and free of rust and grease when assembled and placed in ducts, and should then be kept dry until grouted up.

The grout for filling the ducts should be nominally free of chloride and other dissolved salts. The concrete used for the structure should likewise be free from chloride and dissolved salt unless there is an impermeable and durable barrier, in addition to the grout, between the structural concrete and the tendons. In special situations where the use of grout is difficult, ducts may be filled with other protective media such as greases or even warmed dry air. Provided these can be maintained in good condition they may well afford more satisfactory protection than imperfect grouting but each such job requires a positive approach to be made to the basic necessity to avoid corrosion from any cause.

4.5 Wrapped systems

Similar considerations apply to wrapped systems. Guniting must be particularly low in chloride and must be of adequate thickness and uniformly dense over the wires to be protected. The adjacent core concrete should be low in chloride too. It is desirable to wrap the wire on to a bed of cement paste of similar chemical composition to the guniting to avoid trapped air pockets under the wire. The joint between the guniting and the concrete should be sealed at exposed ends against the possible ingress of either air or water.

4.6 Elimination of cause of damage

Damage by corrosion is due entirely to the following three causes, which can occur singly or in any combination:

- failure to appreciate the risk of corrosion due to harmful influences that arise from the use of the structure.

- inadequate design of the structure or building.
- faulty construction.

In every case, it is necessary to establish the causes of the damage that has occurred. Repair of the defects is only satisfactory if these are due to faults in the building of the reinforced or prestressed concrete structure. A typical example is where, as a result of faulty work on the site, the concrete cover is too thin or the permeability of the concrete forming the cover is too high.

In a case where damage is due to an incorrect assessment of the corrosive conditions arising from the use of the structure, repair of the damage alone is not generally sufficient. In these cases, it is necessary to remove the causes. Typical examples are where there has been a change in the use of a structure or a building and the underestimation of the effects of attack to such an extent that the unclad concrete no longer offers adequate protection.

4.7 Method of repair

Methods of removing the rust stain by washing the surface of the concrete have not proved successful in all cases. The rust does not always dissolve with water or other common cleaning agents, and with iron solvents, the natural color of the concrete is changed. Sandblasting often leads to a modification of the existing structure of the visible face.

Local pockets of segregated aggregate must be cleanly removed and filled with mortar sealing.

The wide cracks on the side from which the moisture can penetrate should be enlarged to a groove (by chiselling or grinding) and sealed by proven methods of jointing.

The repair of major corrosion defects requires the complete removal of all loose particles of concrete, and also of concrete covering which is defective in respect of thickness or

density. Care must be taken to ensure that no carbonated concrete remains behind in the vicinity of the steel reinforcement.

The exposed steel must be derusted and measured to determine its residual cross-section. Where necessary, missing reinforcement should be supplemented, particular attention being given to the interaction with the existing reinforcement.

For the reconditioning of the concrete cover, mortar applied by a spray gun is the most suitable. The composition of the new concrete must be similar to the existing concrete and must give adequate impermeability. Before the application of the sprayed-on mortar, the old concrete must be wetted for as long as possible. Adhesion of the fresh concrete to the old requires an almost dust-free and lightly dried surface. In some cases, steel ties between the old and the new concrete can be useful. In the case of large areas and thick concrete covering, the new concrete should be anchored to the underlay by means of additional steel inserts. Too early drying out of the patches should be prevented by suitable treatment. The same kind of cement must be used for the new concrete as for the old.

It is not sufficient to make up faulty concrete by surface coating, application of a film or rendering. Coatings and films crack under stress. The inadequate covering exposes the reinforcement in the crack to corrosion. Rendering adheres poorly to old concrete and is quite ineffective if the old concrete under it is already carbonated.

4.8 Epoxy Coatings for Prestressing Steel

Epoxy coating is a widely used organic coating for corrosion protection that isolates the steel from contact with oxygen, moisture and chlorides. Epoxy-coated seven-wire prestressing strand and threaded prestressing bars are widely available in North America and Europe.

Epoxy-Coated Strand

Epoxy-coated strand is available in two configurations: coated, and coated and filled. The two configurations are shown in Fig. 4-3. In the coated configuration, a thick epoxy coating is provided around the exterior circumference of the seven-wire strand. In the coated and filled version, the interstices between the individual wires are filled with epoxy in addition to the external coating. By filling the interstices with epoxy, migration of moisture and chlorides along the strand interstices is prevented. Both configurations of epoxy-coated strand are available either with a smooth surface or with grit particles embedded on the surface to improve bond transfer characteristics. The smooth surface epoxy-coated strand is intended for use in applications where bond is not critical, such as unbonded posttensioning systems, external post-tensioning systems and stay cables. When used in unbonded systems, the strand must still be encased in a duct, as the smooth epoxy coating is not a replacement for the sheathing used in monostrand post-tensioning systems. The grit impregnated epoxy-coated strand is intended for used in bonded post-tensioning systems and in pretensioned applications. Epoxy-coated strand is available in conventional sizes of 10 to 16 mm (3/8 in. to 0.6 in.) diameter at Grade 1860 (270 ksi). Details of installation and stressing procedures are provided in a PCI report on the use of epoxycoated strand. [80]

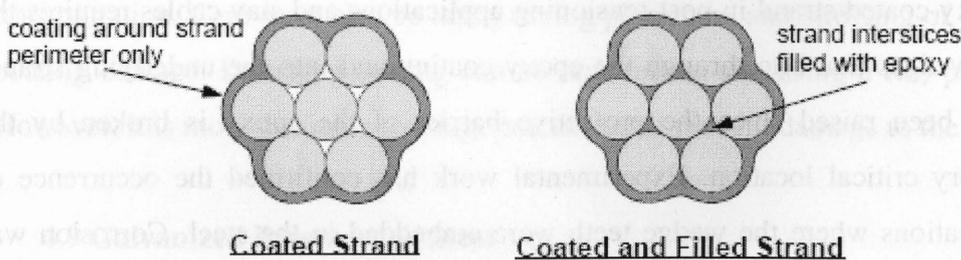


Fig. 4-3 Epoxy Coated Strand Types [81]

Epoxy-coated strand is manufactured to meet the requirements of ASTM A882-92, “Standard Specification for Epoxy-coated Seven-Wire Prestressing Steel Strand” and ASTM A416, “Standard Specification for Seven-Wire Prestressing Steel Strand.” The physical properties of the epoxy coating used for prestressing strand are significantly

different from those used to coat mild steel reinforcement. The epoxy coating developed for prestressing strand is very tough and ductile with good bond to the steel to withstand the elongation during stressing. The coating is also durable and abrasion resistant to minimize damage during handling, placement and stressing. The design final coating thickness for the strand is usually 0.76 mm (0.03 in.), although the thickness can range from 0.63 to 1.14 mm (0.025 to 0.045 in.) according to ASTM A882.79 This strand coating thickness is considerably thicker than the coating thickness for mild steel reinforcement, 0.18 to 0.30 mm (0.007 to 0.012 in.). [81], [82], [83], [84]

The coating is a thermo-setting, fusion-bonded epoxy applied in a continuous process to the bare strand. The manufacturing process starts with strand that meets ASTM A416. The strand is mechanically cleaned and then preheated to 300°C prior to application of the coating. The strand is then run continuously through a fluidized bed of electrostatically charged epoxy particles. As the electrically grounded strand passes through the bed the charged particles are attracted to the surface of the strand. To manufacture the coated and filled strand the outer six wires are separated from the inner wire just prior to entering the fluidized bed. When the wires are re-stranded with the epoxy still in a plastic state the interstitial space between the wires is completely filled with epoxy. [85]

The use of epoxy-coated strand in post-tensioning applications and stay cables requires the use of special wedges that bite through the epoxy coating and into the underlying strand. Concerns have been raised since the protective barrier of the epoxy is broken by the wedges at a very critical location. Experimental work has confirmed the occurrence of corrosion at locations where the wedge teeth were embedded in the steel. Corrosion was also found under the epoxy coating between the wedge teeth marks. The significance of corrosion at the wedge locations may vary. In bonded post-tensioned construction, corrosion at the wedge locations should not have a significant effect on the integrity of the structure, particularly if coated and filled strands are used. However, in unbonded post-tensioned applications or stay cables, anchorage failure due to corrosion at the wedges could lead to failure of the tendon or cable. In these situations, additional protection must be provided for the strand at the wedge locations. [86]

Epoxy-Coated Prestressing Bars

High strength threaded bars commonly used for post-tensioning may be specified with epoxy coating. Epoxy-coated threadbars are coated according to ASTM A775-97, "Standard Specification for Epoxy-Coated Reinforcing Steel Bars." This standard is the same one used for epoxy-coated mild steel reinforcement. Anchorage hardware, including bearing plates, nuts and couplers are also epoxy-coated. Nuts and couplers are proportioned to allow free movement over threads without damaging the epoxy coating. [83]

The fusion-bonded epoxy coating process is similar to that for prestressing strand. The bars are first cleaned and preheated, and then the epoxy powder is applied on the bars electrostatically. The final thickness of the epoxy coating ranges from 0.18 to 0.30 mm (0.007 to 0.012 in.). [83]

Epoxy-coated prestressing bars face similar issues in quality control as epoxy-coated mild steel reinforcement. The effectiveness of the corrosion protection provided by the epoxy coating is dependent on the quality of the coating and the amount of damage to the coating. Transportation and handling are common sources of coating damage. Padded bundling bands, frequent supports, and nonmetallic slings are required to prevent damage during transportation. Care must also be taken during placement and stressing of bars to minimize coating damage. Damaged coating can be repaired on-site using a two-part liquid epoxy. However, it is more desirable to adopt practices that prevent damage to the coating.

4.9 Galvanized Prestressing Steel

Zinc galvanizing has proven to be the most effective metallic coating for corrosion protection. Zinc provides protection by sacrificially corroding in place of steel when exposed to a corrosive environment. Zinc is anodic to steel in the electromotive force (EMF) series and will corrode sacrificially to steel when there is electrical contact and a sufficiently conductive electrolyte is present. The advantage of a sacrificial protection

system is that it theoretically does not have to completely cover the protected part, and nicks and abrasions in the zinc should not permit corrosion of the underlying steel.

Zinc is widely used to protect exposed steel from atmospheric corrosion. The effectiveness of zinc-coated mild steel reinforcing bars in concrete has been uncertain. Galvanized bars were found to increase time to concrete cracking in some cases, while reducing time to cracking in others. A detailed discussion is provided in ACI 222R-96. There are additional concerns when using zinc-coated steel, especially high strength steel, in contact with cement paste. In the high-alkaline environment of concrete or cement grout the corrosion rate of zinc can be very high. One product of zinc corrosion in this environment is hydrogen gas, raising concerns of hydrogen embrittlement of the high strength steel. [29]

Galvanized Prestressing Strand

The use of galvanized prestressing strand is not common in North America and is currently prohibited by the Federal Highway Administration for use in bridges. However, the use of galvanizing in prestressing applications and stay cables is very popular in Europe as well as Japan.

In addition to concerns for increased risk of hydrogen embrittlement in galvanized strand, the galvanizing process may also affect the mechanical properties of the strand. Galvanizing of cold drawn wire for prestressing strand may reduce the tensile strength of the wire and degrade relaxation properties. The ultimate elongation of the wire may increase, and the elastic modulus of the seven-wire strand is normally decreased. Questions have also been raised about the effects of zinc galvanizing on the bond of prestressing strand. Mixed results have been reported. [85]

Galvanized seven-wire strand suitable for prestressing applications is commercially available in standard sizes from 10 to 16 mm (3/8 to 0.6 in.) diameter and in standard grades. The strand is stress relieved (normal relaxation) and conforms to all the requirements of ASTM A41674 except that the wires are galvanized. During the production

process, the wires are zinc coated individually and then stranded. The minimum weight of zinc coating for the strand ranges from 275 to 305 g/m² (0.90 to 1.0 oz/ft²). The single wires before galvanizing meet the requirements of Grade 270 (1860 MPa, 270 ksi) strand (ASTM A416) when fabricated to the corresponding finished strand size. [87]

Galvanized Prestressing Bars

Threaded galvanized prestressing bars are commercially available in standard sizes and strengths of threadbar for prestressing.

Although prestressing bars are not cold drawn like prestressing wire (strand), the process of zinc galvanizing still raises concerns for hydrogen embrittlement. A specification for galvanizing prestressing bars to minimize the effects of galvanizing on the potential for hydrogen embrittlement and on mechanical properties has been developed. The highest potential for damage due to hydrogen embrittlement occurs during acid pickling of the bars prior to hot-dip galvanizing. Flash pickling of the bars should be carefully controlled in terms of pickling time and acid temperature, and hydrogen inhibitors should be used in the acid bath. The bars should be galvanized immediately after pickling. The maximum weight of zinc coating is 0.82 oz/ft² (250 g/m²). Provisions for maintaining the threadability of the bars after coating should also be considered. [88], [89]

Zinc galvanizing of prestressing bars has some effect on the mechanical properties of the bars. Galvanizing may lower the yield strength of the bar up to 5%, and may alter the stress-strain relationship. However, the ultimate strength and ductility of the bars is not adversely affected by the galvanizing process. [88]

4.10 Non-Metallic Prestressing Materials

Fiber reinforced plastic products have been used for pretensioning and post-tensioning in bridges, buildings, marine structures, pavements and rock anchors. The use of fiber reinforced plastic reinforcement in concrete structures can have many benefits. In most

applications, the main benefit of using fiber reinforced plastics is that they do not corrode and therefore eliminate structural deterioration related to corrosion of steel reinforcement.

Fiber reinforced products normally consist of continuous fibers of glass, aramid or carbon embedded in a polymer matrix. The matrix transfers stresses between the fibers and allows them to work as a single element. The matrix also provides stress transfer between the fibers and concrete and protects the fibers. Common matrix materials are polyesters and epoxies.

The material properties of fiber reinforced plastic tendons for prestressing can be significantly different from prestressing steel, and thus their use requires special design considerations. An excellent source of information on this subject is ACI 440R-96, State-of-the-Art Report on Fiber Reinforced Plastic (FRP) Reinforcement for Concrete Structures. The reader is referred to this reference for further information. [90]

4.11 Ducts for Post-Tensioning

Ducts have several functions in post-tensioned concrete. The duct provides the void to allow placement and stressing of the tendons after concrete has been cast, and transfers stresses between the grouted tendon and the concrete. As corrosion protection, the duct works as a barrier to moisture and chlorides. In order for the duct to work effectively as a barrier, it must be impervious to moisture and must itself be corrosion resistant. Duct splices and connections to anchorage hardware must also be watertight. Requirements for ducts are provided in Clause 10.8 of the AASHTO LRFD Construction Specifications. [91]

Galvanized Steel Duct

The most widely used duct material is corrugated galvanized steel. The steel is sufficiently strong to prevent crushing and damage during concrete placement, and can withstand the frictional forces associated with post-tensioning. Galvanizing provides some resistance to duct corrosion. However, research studies have found that the corrosion protection

provided by galvanizing is limited and severe corrosion damage, including corrosion through the duct, can occur in marine or deicing salt exposures. [92]

Some galvanized steel ducts are manufactured with a longitudinal crimped seam. The crimped seam may not be watertight, allowing moisture ingress even if the steel duct is undamaged. Grout bleed water observed leaking from ducts with longitudinal seams has confirmed the potential moisture pathway in ducts manufactured in this manner.

Splices for galvanized steel ducts are performed in many different ways. Common practice is to wrap the joint between ducts with ordinary duct tape. Sometimes a short length of oversized duct is used to span the joint between duct segments to maintain alignment. Heat shrink tubing developed for sealing electrical connections has also been used for duct splicing. Laboratory tests found that duct tape splices were not waterproof, while heat shrink tubing "...produced essentially water and chloride-tight joints..." [92]

In view of the limitations listed above, galvanized steel ducts should not be used in situations where exposure to deicing salts or seawater may occur.

Epoxy-Coated Duct

Epoxy-coated steel duct eliminates several of the problems associated with galvanized steel duct. The epoxy coating protects the duct from corrosion and seals the longitudinal duct seams. A laboratory study showed excellent performance of epoxy-coated ducts in comparison to galvanized steel ducts. Performance was evaluated in terms of grout chloride levels and extent of duct and strand corrosion damage. [92]

Epoxy-coated ducts are not widely used and may be faced with some shortcomings. Similar to epoxycoated reinforcement, the quality of the epoxy coating and level of coating damage will influence the effectiveness of the coating as corrosion protection. Questions have also been raised regarding the ability of the epoxy coating to withstand the deformations associated with fitting the duct to the desired profile.

Plastic Duct

The use of plastic duct systems can provide the highest level of corrosion protection for post-tensioning tendons since they are non-corroding and provide an impermeable barrier to aggressive agents. Plastic ducts have been developed with sufficient strength, rigidity, abrasion resistance and bond properties to satisfy structural requirements. Testing has also shown lower friction losses and reduced fretting fatigue for plastic duct systems in comparison to steel ducts. Commercially available plastic ducts for post-tensioning are normally provided with fitted watertight couplers for duct splices and connection to anchorage hardware. [92], [93], [94]

Plastic ducts made from polypropylene are available in a two-strand system for slabs, and in multistrand systems for tendon configurations of up to fifty-five 12.7 mm (0.5 in.) diameter strands or up to thirty-seven 16 mm (0.6 in.) diameter strands. [95]

4.12 Temporary Corrosion Protection

The time between stressing and grouting of internal tendons should be as short as possible to minimize the opportunity for corrosion while the tendons are unprotected. Many specifications limit the length of time between stressing and grouting. The AASHTO LRFD Construction Specifications provides time limits for grouting ranging between seven days and twenty days, dependent on the ambient humidity (Clause 10.4.2.2.1). The PTI Guide Specification for Grouting has a similar requirement, with time limits for grouting ranging between seven days and forty days. If the permissible time limits between stressing and grouting are exceeded, temporary corrosion protection measures are required by both specifications. [91], [96]

A range of temporary protection measures are available. The most common form of temporary corrosion protection is to coat the prestressing steel with water-soluble oils or vapor phase inhibitors. Other materials, including sodium silicate and biodegradable soap (normally used as coolant for cutting metal), have been used for temporary corrosion

protection. The ducts must be thoroughly flushed with water immediately prior to grouting to remove all traces of the temporary corrosion protection materials that may inhibit bond between the steel and grout. With the exception of the vapor phase inhibitor, these materials can have the added benefit of reducing friction losses during post-tensioning if they are applied on the strands before stressing. Other options for temporary corrosion protection include sealing the ducts to prevent moisture entry, continuous pumping of dry air through the ducts and purging with compressed air or dry gas. [96], [97]

A comprehensive study of materials for temporary corrosion protection and lubrication of post-tensioning tendons found that water soluble oils could not be flushed from the strands completely and adversely affected bond between the strands and grout. Adhesion tests found that bond was reduced by 90% if the ducts were not flushed. When the ducts were thoroughly flushed with water, bond was still reduced by 75% in comparison to untreated strands. The effect of sodium silicate on bond was not as significant as the water-soluble oils, reducing bond by 50% before flushing and 10% after flushing. Stearate soap did not affect bond. These findings illustrate the potential negative side effect of many agents used for temporary corrosion protection. The use of water-soluble oils for temporary corrosion protection should be avoided if the tendons are to be bonded. [97]

4.13 Cement Grout for Post-Tensioning

Cement grout bonds the post-tensioning tendon to the surrounding concrete and provides corrosion protection for the tendon. Corrosion protection is in the form of a barrier to moisture and chloride penetration and in the presence of an alkaline environment for the tendon.

An optimum grout for post-tensioning combines desirable fresh properties with good corrosion protection. The fresh properties of the grout influence how well the grout fills the duct. The corrosion protection provided by the grout is rendered ineffective if the duct is only partially or intermittently filled with grout. These situations can lead to severe conditions for corrosion. The presence of voids or discontinuous grouting may also permit

movement of moisture and chlorides along the length of the tendon. Important grout fresh properties are listed below.

- **Fluidity:** is a measure of how well the grout flows or pumps. Insufficient fluidity may lead to difficulties in placement, blockages and incomplete grouting. Excessive fluidity may lead to void formation near crests in draped tendon profiles and incomplete grouting. Grout fluidity also influences the ability of the grout to fill the space between strands in a multistrand tendon.
- **Bleed Resistance:** this is very important in grouts for post-tensioning. Unlike concrete where bleed water can evaporate, bleed water in grouted ducts tends to migrate to high points in the duct, forming bleed lenses. Eventually, the bleed water will be re-absorbed into the grout, leaving a void. Bleed lenses are particularly a problem where large vertical differences are encountered along the tendon profile.
- **Volume Change:** Reduction in volume or shrinkage in the plastic state can lead to voids and must be avoided. In some cases, it may be desirable for the grout to possess expansive properties while in the plastic state to offset shrinkage and possibly fill voids resulting from entrapped air or bleed water collection.
- **Set Time:** Rapid setting grouts lead to insufficient fluidity, hindering placement and leading to incomplete grouting.

The fresh properties of grout can be controlled through water-cement ratio, by the use of chemical and mineral admixtures, and by the type of cement. Without the use of admixtures, fluidity is primarily a function of the water-cement ratio. In most cases, it will be desirable to reduce the water content to lower permeability and minimize bleed water. In this situation, sufficient fluidity can be provided through the use of superplasticizer. Partial cement replacement with fly ash will tend to increase fluidity for the same ratio of water to total cementitious material. The addition of silica fume or partial cement replacement with silica fume tends to decrease fluidity due to its small particle size. Bleed can be minimized

by reducing the water-cement ratio and by using fly ash or silica fume. Antibleed admixtures may also be used, particularly in situations where the tendon profile has large variations in vertical distance and bleed water accumulation may be severe. Antibleed admixtures are sometimes referred to as thixotropic admixtures. This class of admixture gives the grout gel-like properties to minimize bleeding, while permitting the grout to become fluid when agitated (mixed, pumped, etc.). Expansive properties may be provided through the use of chemical admixtures. Expanding or non-shrink admixtures are generally categorized as gas-liberating, metal oxidizing, gypsum forming or expansive cement based. Set time is normally controlled through the use of set retarding admixtures. Some control may also be available through the selection of cement type. [86]

The corrosion protection provided by the grout is primarily related to its permeability. Low permeability will reduce or slow the ingress of moisture and chlorides. Similar to concrete, the permeability of grouts may be lowered by reducing the water-cement ratio and by the use of mineral admixtures such as fly ash and silica fume. Reduced water-cement ratios may require the use of superplasticizers to provide sufficient fluidity. Admixtures such as corrosion inhibitors may also be used to improve the corrosion protection provided by the grout.

The selection of suitable grout proportions and admixtures requires careful consideration of the grout fresh properties and corrosion protection. The effects of various admixtures and grout proportions on fresh properties and corrosion protection have been studied by several researchers. [86], [98] Schokker performed an extensive research study to develop two optimized grouts for post-tensioning. [99] The study was part of this overall research project. The first grout was developed for use in applications where the tendon profile was primarily horizontal. The grout contained 30% fly ash (by weight), superplasticizer and had a water-cement ratio of 0.35. This grout had excellent fluidity, good bleed resistance and provided excellent corrosion protection. The second grout was recommended for vertical tendons where resistance to bleed is critical. This grout had a water-cement ratio of 0.33 and contained a combined superplasticizer and antibleed admixture. Additional information on mix proportioning and guide specifications for grouts for post-tensioning is provided by

the PTI “Guide Specification for Grouting of Post-Tensioned Structures” and by the U.K. Concrete Society report “Durable Bonded Post-Tensioned Bridges.” The requirements for grout in the AASHTO LRFD Construction Specifications are minimal. [91], [96], [100]

The corrosion protection provided by the grout is also heavily dependent on the construction practices. Many corrosion problems have resulted from poor construction practices and inexperienced contractors. An optimized grout design is of no use if it is not placed properly and the ducts are not completely filled with grout. Attention must be given to batching and grouting/injection equipment, locations of vents along the duct and grouting procedures. Requirements for grouting procedures are given in Clause 10.11 of the AASHTO LRFD Construction Specifications. [91]

4.14 Anchorage Protection

The post-tensioning anchorages and end stubs of the strands must be carefully protected. Although anchorage corrosion may lead to failure of the anchorage, bond between the tendon and concrete will prevent a complete loss of prestress. However, corrosion of the anchorage hardware may lead to cracking and spalling of the concrete in the vicinity of the anchorage and increased corrosion action. Corrosion of the anchorage and strand stubs may also allow moisture entry into the duct and subsequent tendon corrosion. Multistrand anchorage systems may be fitted with a sealed end cap to protect strand ends. The cap is grouted or filled with corrosion inhibiting grease. Not all multistrand post-tensioning systems include an end cap. Anchorages are commonly recessed in a pocket at the end or edge of the concrete element. Corrosion protection for the anchorage normally consists of filling the anchorage recess or pocket with mortar or concrete. Common practice is to coat the anchorage and pocket surfaces with an epoxy-bonding agent prior to filling the anchorage pocket with a non-shrink mortar.

The location of the anchorage within the structure can also play a role in corrosion protection and corrosion damage. In many structures, the anchorages are located at the ends of structural elements below expansion joints, or at exterior member ends or slab edges.

These locations are prone to concentrated exposure with moisture and chlorides, and often lead to severe anchorage corrosion damage. The location of post-tensioning anchorages is often dictated by the method of construction. In instances where the anchorage cannot be located away from a possible source of aggressive agents, the anchorage must be detailed to provide multiple layers of corrosion protection. The Concrete Society (U.K.) Technical Report No. 47 on bonded post-tensioned bridges provides suggestions for anchorage protection details. [100] The report discusses two approaches for anchorage protection. The first is to provide an anchorage that is not encased in mortar or concrete after stressing. Exposed anchorage hardware is protected by end caps and waterproof membrane, and has the advantage that the anchorage can be readily inspected for corrosion damage. The second approach provides a higher level of corrosion protection at the expense of inspectability by recessing the anchorage in a filled pocket. Details of multilevel corrosion protection for this form of buried anchorage are shown in Figure 4-4. The details of the member end can also be designed to minimize contact with moisture and chlorides draining through expansion joints, as shown in Fig. 4-5. The member end is detailed to prevent water from dripping onto the anchorage region. An abutment gallery is provided to allow inspectors to gain access to the anchorage.

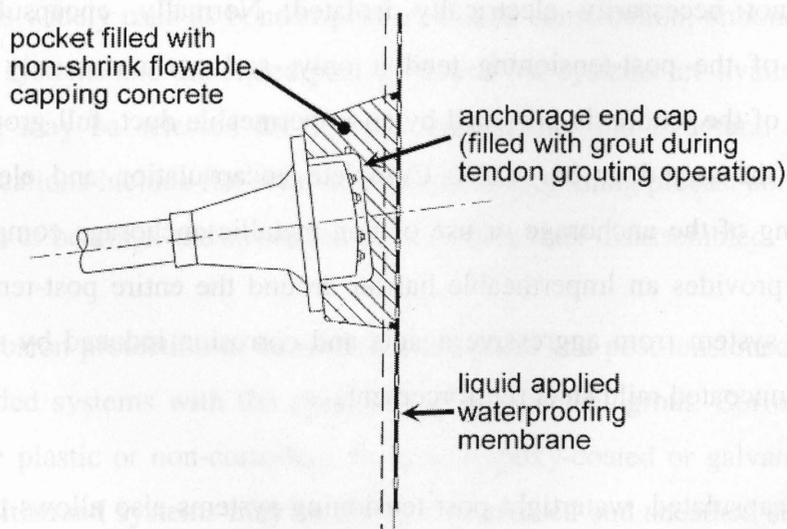


Fig. 4-4 Multi-Layer Corrosion Protection for Buried Post-Tensioning Anchorages [97]

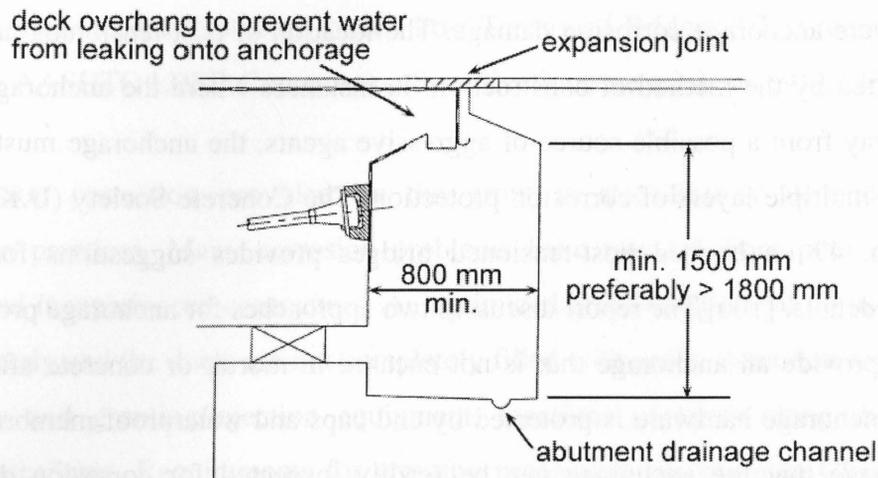


Fig. 4-5 Member End Details for Anchorage Corrosion Protection [97]

4.15 Encapsulated and Electrically Isolated Systems

Encapsulated and electrically isolated multistrand post-tensioning systems provide the highest level of corrosion protection. Encapsulation and electrical isolation are terms that are sometimes used interchangeably. However, it is important to note that encapsulated systems are not necessarily electrically isolated. Normally, encapsulation refers to encapsulation of the post-tensioning tendon only, and not necessarily the anchorage. Encapsulation of the tendon is provided by an impermeable duct, full grouting and sealed end cap over the strand or bar stubs. Complete encapsulation and electrical isolation includes coating of the anchorage or use of non-metallic anchorage components. Ideally, this approach provides an impermeable barrier around the entire post-tensioning system, protecting the system from aggressive agents and corrosion induced by stray currents or coupling with uncoated mild steel reinforcement.

The use of encapsulated, watertight post-tensioning systems also allows the system to be tested for leaks. Pressure testing is performed immediately prior to grouting with all vents closed and end caps sealed. The duct is pressurized with air and the leakage rate is monitored using a flowmeter. Sources of leaks can be identified using soapy water or other

means. Significant sources of leaks should be repaired as applicable, including re-sealing of end caps, grouting or patching and crack sealing. The use of a watertight and airtight duct system also permits vacuum grouting. Vacuum grouting applies a negative pressure at one end of the duct while the grout is pumped under pressure from the opposite end. Vacuum grouting is particularly useful for long tendons or where large vertical distances are involved.

An encapsulated multistrand post-tensioning system is available in three configurations, each providing an increased level of corrosion protection. All ducts, connections and trumpets are plastic. The bearing plate is a composite of metal and high performance mortar, and a sealed end cap is provided. The highest level of protection is provided by a configuration that provides electrical isolation for the tendon, and the ability to electrically monitor the tendon throughout the life of the structure. [95], [101]

4.16 Unbonded Post-Tensioning System Details

Embedded Post-Tensioning

Although not as widely used as bonded post-tensioned construction, unbonded multistrand posttensioning systems and unbonded post-tensioned bar systems are available. Unbonded post-tensioning may be selected for various applications and structural design criteria. Common applications include flat slabs and foundations, joining precast concrete elements, precompression of bearings and structures that are to be later disassembled.

Multilevel corrosion protection in unbonded multistrand and post-tensioned bar systems is similar to bonded systems with the exception of the cement grout. Corrosion protection options include plastic or non-corroding ducts and epoxy-coated or galvanized strands or bars. Some multistrand systems may be fitted with greased and sheathed strands normally used in monostrand systems. Anchorage protection for unbonded systems is the same as for bonded post-tensioning.

External Post-Tensioning

External post-tensioning has various applications, including precast segmental bridge construction and strengthening of structures. Stay cables may also be considered in this category. External post-tensioning tendons are not embedded in the concrete, but rather are bonded to the structure at discrete locations including anchorages and deviators.

Several options are available to provide multilevel corrosion protection for external tendons. Most external multistrand tendons are encased in a sheath, as shown in Fig. 4-6. The steel or plastic sheath provides an exterior protective barrier around the tendon. Similar to considerations for post-tensioning ducts, the use of a plastic or other non-corroding material for sheathing provides the highest level of corrosion protection. Strands or bars used for external tendons may be epoxy-coated or galvanized. Greased and sheathed strands (as used in monostrand systems) are also commonly used for external multistrand tendons. The space between the strands or bars inside the outer sheathing can be filled with cement grout, grease or wax to provide additional corrosion protection. Grout properties should meet similar requirements as grouts used in bonded post-tensioned construction. Greases or waxes should be similar to those used for greased and sheathed monostrand tendons. [102], [103]

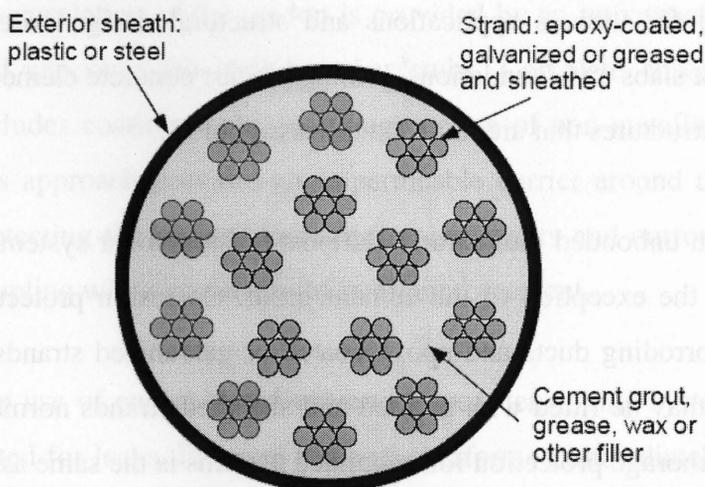


Fig. 4-6 External Post-Tensioning Tendon Corrosion Protection [102]

5. Cathodic protection and cathodic prevention

There are lots of countermeasures to prevent the steel corrosion, like conventional patch repair, coating etc., and they may be applied to all concrete structures. However, if the concrete has already been contaminated by chloride ions it can be difficult to stop corrosion by using these techniques. One of the very effective countermeasures is cathodic protection, which supplies a direct current from anode materials placed at the concrete surface to the steel in concrete. This technique has been used to lots of reinforced concrete and prestressed structures all over the world and they have had success applying it so far.

Cathodic protection of steel reinforcement in atmospherically exposed concrete structures is achieved by means of an anode system usually laid on the concrete surface and it is schematically shown in Fig. 5-1. This has proved to be an effective method of stopping or preventing localized pitting corrosion of reinforcement caused by the presence of chloride [104], [105].

The first mention of cathodic protection applied to structures heavily contaminated with chlorides dates back to the late 1950s. The application of the technique to protect bridge decks contaminated by deicing salts began in 1973 in North America. Soon, suitable hardware (anodes, overlays, reference electrodes etc.) had been set up, as well as specific protection and design criteria. In the 1980s, the development of new meshed anodes based, at first, on conductive polymeric materials and then on much more reliable mixed metal oxide activated titanium and the development of carbon containing paints, led to further applications such as bridge decks, slabs, piles, marine constructions, industrial plants and garages.

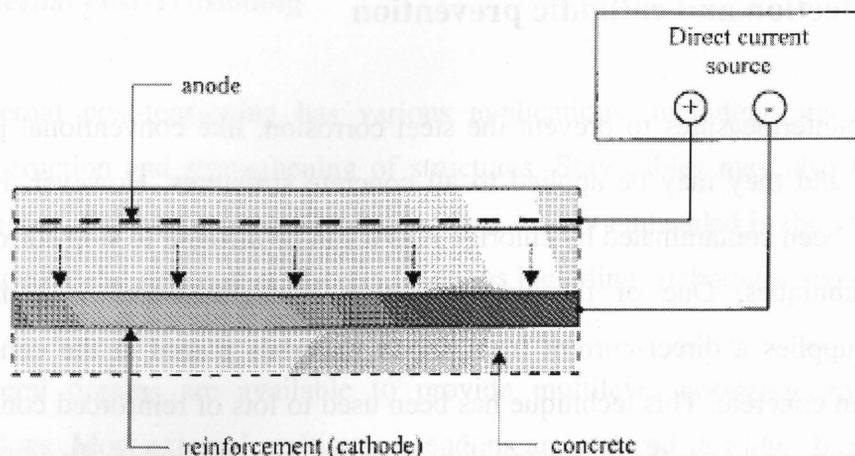


Fig. 5-1 Schematic representation of the cathodic protection system for concrete structures [105]

The technique has also been applied to new structures exposed to chlorides to prevent corrosion. This type of cathodic protection was named cathodic prevention. It can be applied to prestressed structures without risk of embrittlement of the high strength steel of tendons. Recently this technique has been proposed in conjunction with conventional patch repair of chloride contaminated structures in order to avoid the initiation of incipient pitting around the repaired zones, by utilizing sacrificial anodes embedded near the periphery of the repair patches. Cathodic protection also proved to be effective in repassivating steel in carbonated concrete. This type of cathodic protection was called continuous realkalization, since the continuous application of low currents leads to protection effects similar to those achieved by the technique of electrochemical realkalization. [105], [106], [107], [108]

Cathodic protection has been applied to more than 600,000 m² of corroding reinforced concrete structures in North America, where cathodic protection has been installed on more than 500 bridges, in Europe (mainly in UK, Norway and Italy), in Asia (Middle East, Korea and Japan) and Australia. About 150,000 m² of new and almost all prestressed concrete structures have been protected in Italy in the early 1990s.

5.1 Behaviour of steel in chloride containing concrete

As described in chapter 3, steel reinforcements embedded in sound concrete (that is alkaline, with pH usually higher than 12, and chloride free) are in passive conditions. Passivity breaks down when the chloride content on the steel surface exceeds a critical threshold and pitting attack can initiate. Chloride content also affects the range of potentials in which the reinforcing steel is passive. The upper potential of this range, called the pitting potential (E_{pit}), diminishes typically from +500 to -400mV passing from sound to very heavily chloride contaminated concrete as shown in Fig. 5-2. The pitting potential is characterized by a great variability, since, beyond the chloride content, it depends on many other parameters such as pH near the steel surface, temperature, cement and type and content and concrete porosity. The highest chloride content compatible with passive conditions for a given potential is the critical chloride content, which decreases with the potential. In correspondence to the usual corrosion potentials observed on a structure exposed to the atmosphere the critical content is in the range 0.4-1% of cement weight.

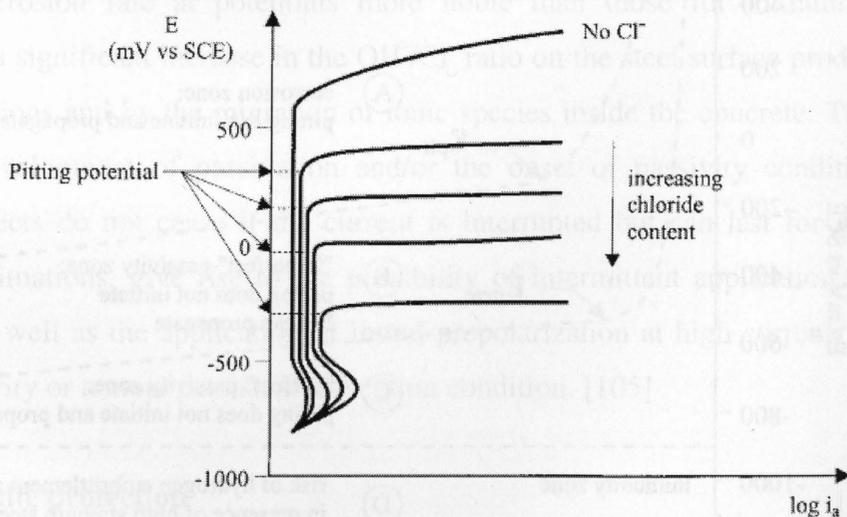


Fig. 5-2 Schematic illustration of the anodic behavior of steel in the presence of chlorides [109]

The conditions for pitting initiation and propagation were pointed out by Pourbaix who, during the 1970s, introduced the concepts of imperfect passivity and perfect passivity intervals. [109] Besides the pitting potential, E_{pit} , he defined the repassivation potential or protection potential, E_{pro} , below which the pitting corrosion stops. At potentials in the range from E_{pit} to E_{pro} corrosion will not initiate but can propagate.

The different domains of potentials and chloride contents are plotted in Fig. 5-3. The extension of these domains depends on pH, temperature and other parameters mentioned above. The domain A (corrosion zone) indicates the conditions which cause initiation and stable propagation of pits; the domain B (imperfect passivity zone) the conditions which do not allow the initiation of new pits but the propagation of preexisting ones; the domain C (perfect passivity zone) the conditions which do not allow either the initiation or the propagation of pits; the domain D (immunity and also hydrogen evolution zone) where corrosion cannot take place for thermodynamic reasons and where hydrogen evolution and consequently hydrogen embrittlement of high strength steel can take place. [108]

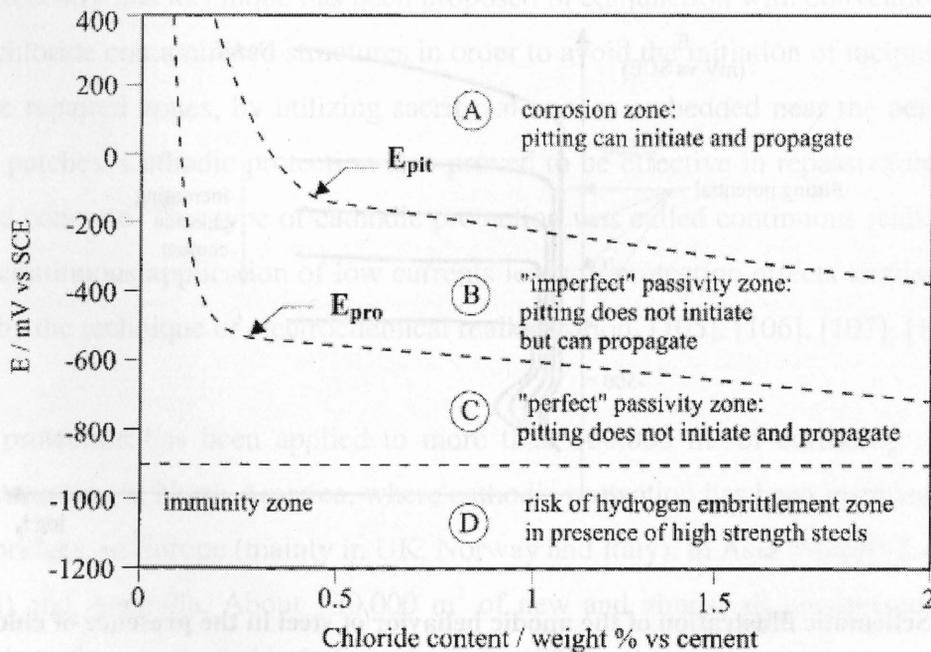


Fig. 5-3 Schematic illustration of steel behavior in concrete as a function of the chloride content [108]

5.2 Beneficial effects of cathodic protection

To achieve the cathodic protection of steel reinforcement embedded in chloride contaminated concrete it is not necessary, as some people claim, to establish immunity conditions, that is lowering the potential below the equilibrium potential E_{eq} given by Nernst's law. These immunity conditions are normally required on steel in active condition such as on structures in soil or immersed in sea water, where potentials more negative than -850mV in the presence of Cu/CuSO_4 or than -950mV in the presence of sulphate reducing bacteria are imposed. Conversely, the target of cathodic protection in concrete structures is to reduce the corrosion rate by taking the steel into the passivity range or by reducing the macrocouple activity on its surface, and this can be done with a small reduction in potential and a smaller current.

If the potential is taken into the range from E_{pit} to E_{pro} the initiation of new pits is prevented and the corrosion rate of the existing ones is reduced, the driving voltage of the functioning of active-passive macrocells being decreased. The possibility of achieving protection or negligible corrosion rate at potentials more noble than those for immunity are also connected to a significant increase in the OH^-/Cl^- ratio on the steel surface produced by the cathodic reactions and by the migration of ionic species inside the concrete. This increase favors the development of passivation and/or the onset of passivity conditions. These beneficial effects do not cease if the current is interrupted but can last for months and, under some situations, give rise to the possibility of intermittent application of cathodic protection, as well as the application of initial prepolarization at high currents in order to achieve passivity or a more persistent protection condition. [105]

5.3 Cathodic protection

To follow the variation in the potential of steel reinforcement of a concrete structure damaged by chlorides and then protected by a cathodic protection system it is helpful to refer to the map of Fig. 5-4. The initial condition is represented by the dot #1 where the chloride content is nil and the steel is passive. By increasing the chloride content, the

working point shifts to dot #4 within the corrosion region. Corrosion of the steel occurs rapidly by macrocell mechanism. The cathodic protection leads to #5 so that the passivity is restored or to #6 without restoring passivity. In all cases the corrosion rate is reduced.

Initial current densities in the range $5\text{-}15 \text{ mA m}^{-2}$ are generally needed for protecting constructions exposed to the atmosphere. Much lower current densities are required under conditions that will reduce the access of oxygen towards the surface of the steel such as in water-saturated concrete. For components operating underwater, the oxygen diffusion limiting current is very low, typically in the range $0.2 \text{ to } 2 \text{ mA m}^{-2}$ of reinforcing steel surface area.

The experience on bridge decks shows that the current required to maintain protection conditions decreases even after months or years from start up. This happens because the cathodic current can bring about repassivation of steel in active zones, by improving the ratio OH^-/Cl^- that increases E_{pro} and/or also because the reduction of the current exchanged by the macrocell leads to the inhibition of the macrocell itself.

In the cases in which the cathodic protection path runs according to $4 \rightarrow 5 \rightarrow$ of Fig. 5-4, and thus passivity is established on the entire surface of the steel, the current required to maintain passivity is reduced to a few mA m^{-2} (e.g. $2\text{-}5 \text{ mA m}^{-2}$). If the cathodic protection path runs according to $4 \rightarrow 6 \rightarrow$, the current density to fulfill the protection criterion remains high and does not decrease with time, since passivity is not obtained. For the interpretation of the behavior in the case of the restoration of passivity, results obtained from a laboratory test carried out on concrete slabs exposed to a ponding with sodium chloride solutions can be considered. From the same test, it should be noted that the polarization of the steel takes place mainly in the first year of cathodic protection. A further decrease in potential (of about 100 mV) is recorded in the long run at the highest current densities applied. One hypothesis is that the cathodic functioning increases the water content around the steel, thus reducing the oxygen availability on the rebar. [110], [111]

Another result is that higher cathodic polarization occurred during wintertime than in summertime. Feeding voltage also changed during the year. The anode potential is rather constant and the ohmic drop is negligible; hence the cathodic polarization is the main contribution of the increase in the feeding voltage in the cold season.

An important effect is the chloride migration, which opposes the penetration of chloride ions by diffusion. Increasing the current density (for example from 0.5 to 20 $\text{mA}\cdot\text{m}^{-2}$) of concrete surface, the chloride content tends to be lower. This effect is much more evident at 28 months in some tests, when the chloride content was higher and for current densities greater than 10 $\text{mA}\cdot\text{m}^{-2}$.

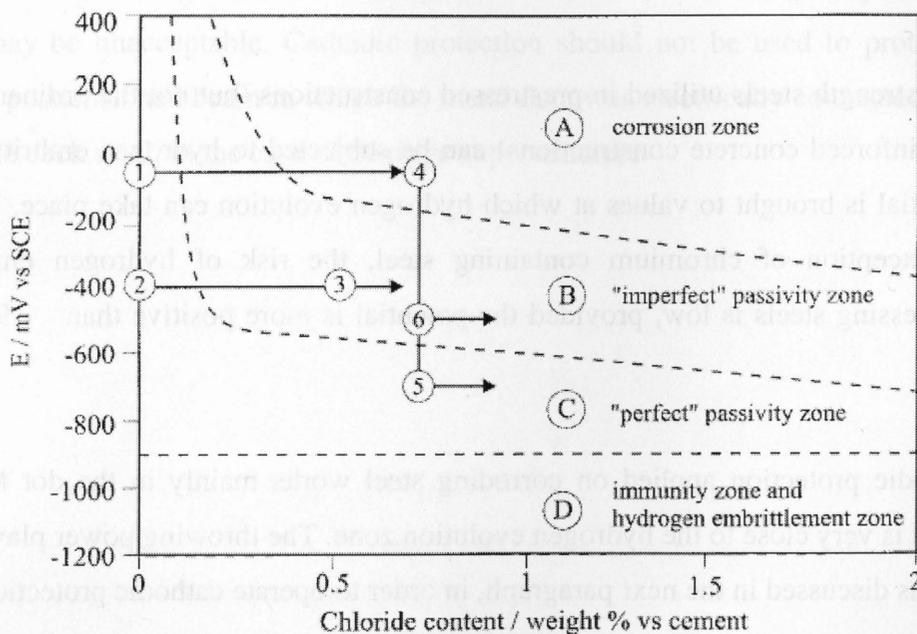


Fig. 5-4 Schematic illustration of steel behavior in concrete for different potentials and chloride contents. Evolution paths of potential and chloride content on the rebar surface of an aerial construction during its service life for: cathodic prevention (1→ 2→ 3→); cathodic protection restoring passivity (4→ 5→); cathodic protection reducing corrosion rate (4→ 6→). Cathodic prevention is applied from the beginning, cathodic protection only after corrosion has initiated. [108]

5.4 Cathodic prevention

Cathodic protection is also applied to non-corroding structures in order to prevent corrosion otherwise expected during the service life. This technique, called cathodic prevention, is based on the strong influence of the potential on the critical chloride content. In Fig. 5-4 a typical evolution path (1→ 2→ 4→) (in terms of potential and chloride content) of cathodic prevention is shown. At usual current densities in the range 1 to 2 mA m^{-2} , a decrease in potential of at least 100-200 mV is produced, leading to an increase in the critical chloride content higher than one order of magnitude.

5.5 Hydrogen embrittlement

High strength steels utilized in prestressed constructions (but not the ordinary steel utilized for reinforced concrete constructions) can be subjected to hydrogen embrittlement if their potential is brought to values at which hydrogen evolution can take place. However, with the exception of chromium containing steel, the risk of hydrogen embrittlement of prestressing steels is low, provided the potential is more positive than -900 mV. [112], [113]

Cathodic protection applied on corroding steel works mainly in the dot #5 of Fig. 5-4, which is very close to the hydrogen evolution zone. The throwing power plays an important role, as discussed in the next paragraph, in order to operate cathodic protection properly.

5.6 Conclusions

Cathodic protection has proved to be an effective method to control chloride induced corrosion of atmospherically exposed reinforced concrete structures even in the presence of high chloride levels. The risk of hydrogen embrittlement makes its application to prestressed concrete structures possible only in cases of simple geometry and under strict monitoring control.

Cathodic prevention has proved to be a viable and safe technique to increase the corrosion resistance of reinforcement in reinforced or prestressed structures, and in carbonated concrete can take the pH on the rebar surface from values lower than 9 to values higher than 12 and thus transform the state of the reinforcement from corroding to passive conditions.

Caution should be exercised if cathodic protection is to be used to protect pretensioning strands in pretensioned bridges. Uniform distribution of cathodic protection current to the strands is difficult to obtain. Satisfactory performance may be possible only when an anode is placed on the top surface of the deck slab and on all exposed surfaces of the girders and even with this anode arrangement the variation in current density to different portions of the strands may be unacceptable. Cathodic protection should not be used to protect the strands in the posttensioned beam in which the metal duct was electrically connected to the strands and the deck steel, in the case of a bridge superstructure.

6. Conclusions

One of the major causes of deterioration of prestressed concrete structures is chloride-induced corrosion of the pre-stressing steel. The magnitude of the damage is especially large in structures exposed to marine environment and de-icing salts. If we consider three parameters, the concrete cover thickness, chloride diffusion coefficient and surface chloride concentration as analytical variables, based on the reliability theory, the structural durability life in chloride environment when the limit reliability index equals to zero can be predicted. The thickness of concrete cover has a biggest influence on durability of prestressed concrete structure in marine environment, and other two parameters (chloride diffusion coefficient and the surface chloride concentration) are second. And it is important to choose and establish suitable values of w/c in order to obtain better durability for prestressed concrete structures in chloride environment.

Although prestressed concrete members are generally manufactured with a higher strength concrete under better quality control, they are subject to the same effects of corrosion as is conventionally reinforced concrete. However, because of the high stresses in the prestressing strands, the corrosion process is accelerated. Even small corrosion pits could cause a strand to fracture, compared to conventional reinforcing that may rust to virtually nothing before breaking. There have been documented cases of prestressing strands breaking due to corrosion. This is a serious problem as prestressed concrete members rely on the tensile strength of the strands to resist applied loads.

One of the more prominent early designs that experienced costly corrosion problems, including structural collapse, is the West Berlin Congress Hall. On May 21, 1980, one-third of the 3400 m² (37000 ft²) concrete shell roof, including one of the two support arches, collapse, as it can be seen in Fig. 6-1. Dynamic stress and corrosion in the prestressing tendons were the main causes. The condition of stress corrosion in the tendons was accelerated by inadequate protection against moisture (insufficient concrete cover) and by the presence of corrosive agents. One person died and four other were injured in the collapse.

The West Berlin Congress Hall roof was a totally cast-in-place doubled curved surface structure 76 mm (3 in) thick. The shell roof was supported by an edge on either side and a tension ring in the center. Steel tendons were anchored under tension in the arches and the central tension ring and embedded in thin reinforced concrete slabs for protection. Vertical movements of the edge arches due to concrete creep, temperature stresses, snow, wind, settlements, and other factors caused high fluctuating stresses in the tensioning steel. These stresses over the 23 years life of the structure cracked the roof. Humidity and carbon dioxide were able to penetrate through the cracks to the tensioning steel, causing severe corrosion and a gradual break of the wires. [114]

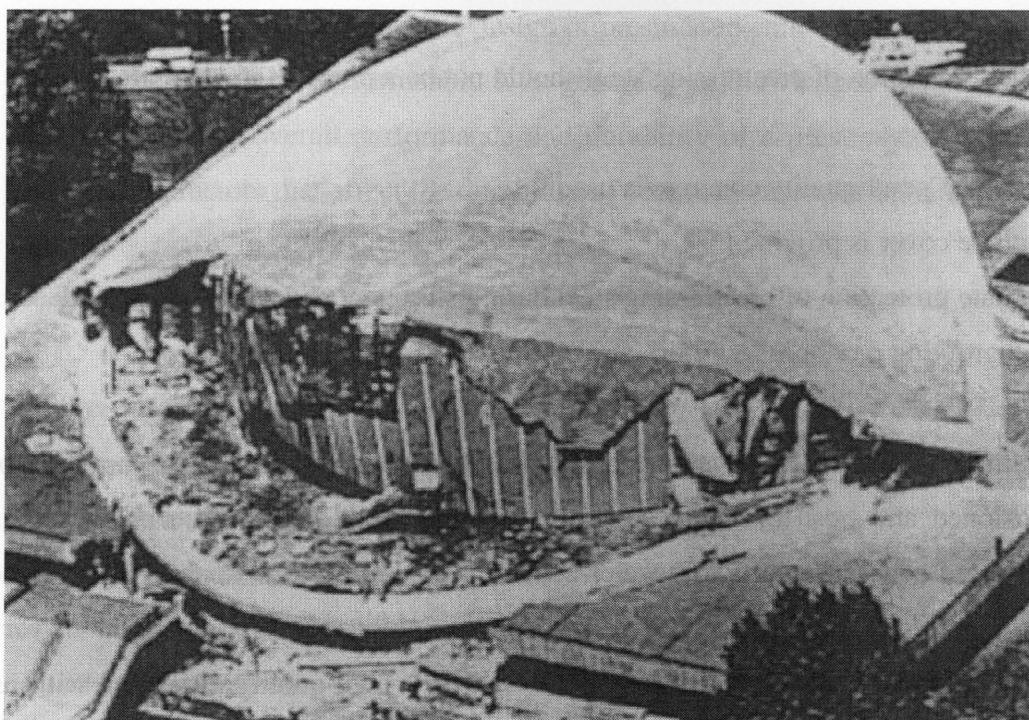


Fig. 6-1 Partial collapse of West Berlin Congress Hall [114]

The collapse was the result of details that permitted corrosion of the steel tensioning elements supporting the thin shell roof, according to an investigation report prepared by Jorg Schlaich of the Stuttgart Institute for Massive Construction (ENR 1980). The prestressing steel, embrittled by corrosion and hydrogen induction, finally broke. While the design was blamed for the collapse, there were also some construction deficiencies noted in

the report. The roof joint concrete was of “insufficient, uneven quality, very porous and had, by today’s standards, a too high chloride content”.

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

To summaries, corrosion of prestressing steel should not be a problem if:

- uncracked good-quality concrete is used.
- adequate cover is provided.
- adequate protection of prestressing steel during shipment and storage is provided.
- good grouting practices to minimize or eliminate voids in ducts are used.

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

There are some corrosion-protection measures that can be applied to the tendons themselves. Epoxycoated strands are coated with an extra-thick coating of epoxy to allow for elongation, and often have coarse grit embedded in the epoxy to aid in bonding. The epoxy coating provides a mechanical moisture barrier. The coating thickness is

considerably more than that on epoxy-coated steel reinforcing bars. The performance of epoxy-coated strands in both pretensioned and post-tensioned specimens under severe exposure conditions has been excellent. However, there are a number of issues besides cost-effectiveness that need to be resolved before epoxy-coated strands are commonly used in either pretensioned or post-tensioned construction. An "intelligent strand" is a strand where a fiber-optic sensor is placed through the center wire. It can be used to monitor strains and for any breaks in the strands.

6.1 General design provisions

Precautions can be taken during the design of prestressed concrete beam structures that will enhance the corrosion performance. It is generally acknowledged that the design details will influence the overall performance and durability of a prestressed concrete structure. Some design factors that affect the durability of concrete structures include:

- construction type.
- expansion joints.
- construction joints.
- tendency of concrete to crack.
- duct and anchorage layout in post-tensioned concrete.
- drainage details.
- access for inspection and maintenance.
- proximity to seawater.
- exposure to deicing chemicals.

Prestressed concrete designs have improved greatly in the last few decades. Bette joints details (or even the elimination of some of the them in the case of integral abutment bridges), denser and better quality concrete, corrosion inhibiting additives and improved surface coatings are just a few examples that the industry is active in supporting research in topics that need attention. Also, there are more published application guidelines for

designers and constructors that provide better corrosion protection for prestressing strands, anchorage and other hardware.

6.2 General constructions provisions

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

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

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

Corrosion is probably less frequent in prestressed concrete than in ordinary reinforced concrete, but when it does occur, the results can be more dangerous and more costly.

The good performance of the prestressed concrete indicates that prestressing steel, completely protected by dense Portland cement concrete or grout of adequate thickness and free of cracks and aggressive chemicals, does not suffer corrosion problems.

6.3 Recommendations

Based on the cited literature, long-term performance of prestressed concrete structures in corrosive environments can best be assured by proper design in new construction. Corrosive agents, such as deicing salt solutions, should be directed away from prestressed concrete structure elements by proper design and maintenance of drain systems and joints. The use of properly designed integral (joint-less) bridges can also address the problem of leaky expansion joints. Consideration should be given to encapsulation of anchorages in corrosion resistant materials, use of corrosion inhibitors in surrounding concrete, and use of very low permeability, high performance concrete mixtures. In structures where these precautions have not been taken, corrosion may occur and significant concrete distress, in the form of cracking and spalling, may develop. Conventional concrete repairs, as observed in field and laboratory investigations, are a temporary measure, and do not appear to ensure long-term protection of the prestressing steel and associated conventional reinforcement; periodic repair and reapplication of protective systems may be necessary to maintain such structures. Where such periodic repairs are difficult to carry out, complete replacement of distressed members may be a long-term cost-effective alternative.

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