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Corrosion of Prestressing Steels

Reported by ACI Committee 222

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This report reflects the current understanding of corrosion of prestressing steels in concrete. The report includes chapters that cover the various types of prestressing steel, including some discussion on metallurgical differences. Deterioration mechanisms are discussed, including hydrogen embrittlement and stress-corrosion cracking. Methods to protect prestressing steel against corrosion in new construction are presented, along with a discussion of field performance of prestressed concrete structures. Finally, field evaluation and remediation techniques are presented.

Keywords: anchorage; corrosion; duct; durability; grout; hydrogen embrittlement; post-tensioned; prestressed; tendon; strand; stress-corrosion cracking; unbonded.

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CHAPTER 1—INTRODUCTION

1.1—Background

While several attempts were made during the 1800s to prestress concrete, the modern development of prestressed concrete is credited to E. Freyssinet of France in 1928 (Lin and Burns 1981). Freyssinet understood the importance of using high-strength steel for prestressing to avoid prestressing losses that can significantly reduce the applied prestressing force. Use of prestressed concrete began in the United States with circular-wrapped prestressed tanks in 1941 (Schupack 1964). The first prestressed bridge in the United States was located in Tennessee and opened to the public on Oct. 28, 1950. The Walnut Lane Bridge, located in Pennsylvania, was completed in the fall of 1950 and opened to traffic in Feb. 1951 (Manning 1988). Since that time, applications of prestressing in bridge and building construction have spread rapidly. In the United States prestressed concrete has proven to be a very successful method of construction. Its benefits include increased load-bearing capacity, improved crack control, and slenderness of elements.

Corrosion is not as well documented in prestressed concrete structures as in non prestressed concrete structures. Corrosion of these structures appears to be restricted to specific circumstances, including construction details, improper design, and construction practices. The potential for widespread problems still exists, however. Indeed, in environments contaminated with known corrosion promoters, such as chloride ions and hydrogen sulfide, it is imperative that prestressing steel be protected.

A number of surveys provide information concerning the potential for corrosion of prestressed structures. Burdekin and Rothwell summarized current practice, specifications, and corrosion mechanisms (1981). Failure to follow proper construction details and practices, as well as poor-quality materials, account for the vast majority of poor performance. Schupack reached similar conclusions in two additional surveys (1978a; Schupack and Suarez 1982).

Because corrosion in prestressed concrete members potentially has more serious consequences than in nonprestressed concrete, more information needs to be developed and disseminated. The magnitude of the corrosion problem, its projected extent, and measures that can be taken to resolve it are of vital concern to designers, contractors, and owners, and form the basis of this report.

Prestressed concrete is used in several forms. It is important to the discussion of corrosion of prestressed concrete that the basic differences in these forms of construction be described.

Pretensioned concrete refers to systems in which high-strength wire or strand is stressed before placement of the concrete. Concrete is cast around the prestressing steel (bonding the steel to the concrete) and allowed to cure to a specified strength. Tension in the steel is then released, placing the member in compression. Pretensioning is common practice for both standardized bridge beams and on building elements, such as solid joists, solid and hollow-core planks, and single-and double-tee joists.

Post-tensioned concrete is prestressed after the concrete is placed and allowed to cure to a specified strength. This term is used for both bonded and unbonded tendons. Bonded post-tensioning requires that deformed tubes or ducts be placed in the forms before concrete placement. After the concrete is placed and cured to a specified strength, bundles of strand, wire, or bar (the bundle is referred to as a tendon) are placed in the duct and are stressed against and anchored to the concrete.

After the tendons have been installed and stressed, ducts are usually injected with grout (a fluid mixture of portland cement, water, and perhaps admixtures) with a grout pump. Ducts can be metallic or nonmetallic and include uncoated or coated steel, high-density polyethylene (HDPE), and polypropylene. The injected grout fills the spaces among the individual elements of the tendon and between the tendon and duct. Grout injection provides two benefits: corrosion protection for the tendon with the highly alkaline environment provided by the grout, and bond between the concrete and the tendon. This form of prestressing is popular on structures such as bridges, buildings, dams, tanks, and tie-backs.

The other post-tensioning technique, unbonded post-tensioning, mainly uses single-strand tendons. Each strand is placed inside an individual sheath, and the annular space is filled with a corrosion-inhibiting grease or wax. The tendon is installed in the formwork before placement of concrete. The sheath provides a barrier between the greased strand and the concrete, allowing the strand to be stressed after concrete placement. In general, the anchorages are cast into the concrete along with the tendon. Unbonded multistrand tendons have also been used extensively in nuclear pressure vessels. Multiple wire or strands are placed in a cast-in-place duct, which is then usually filled with heated, corrosion-inhibiting, wax-like hydrophobic grease.

Prestressing bar is widely used in bonded post-tensioning of liquid-containing tanks, in geotechnical applications such as foundation tie-downs and tie-backs, and in segmental bridge construction.

Prestressing wire is used mainly in the construction of prestressed concrete tanks and the manufacture of concrete pipe. In tank construction, the wire is wrapped (under tension) around the tank circumference. This provides a circumferential compressive prestressing force that resists the tensile stresses developed when filling the tank. Prestressed concrete pipe is manufactured in a similar manner where the pipe is wound with high-strength prestressing wire.

1.2—Scope

This state-of-the-art report is intended to cover known practice and research relating to corrosion in prestressed concrete systems. ACI 222R covers factors that govern corrosion of steel in concrete, measures for protecting embedded metals in new construction, techniques for detecting corrosion in structures in service, and remedial procedures. In general, ACI 222R focuses on the mechanisms that relate to the corrosion of nonprestressed steel. Many of the mechanisms discussed in ACI 222R are valid for and apply to prestressing steels in concrete. This document supplements ACI 222R by presenting information concerning mechanisms that affect prestressing steels in concrete. It includes basic coverage of the metallurgy of the various commonly used prestressing steels as necessary background information to understand the mechanisms of deterioration of prestressing steels. Techniques, both current and proposed, for evaluation of prestressed structures with respect to corrosion of strands and tendons are also reviewed. A history of field performance, covering documented cases of corrosion-induced failures in prestressed concrete members, is included in this report. Finally, this document describes methods of protection as well as remedial techniques that can be applied to existing structures with corrosion.

CHAPTER 2—PRESTRESSING STEELS

2.1—Wire

Prestressing wire is produced for use in prestressing applications, such as prestressed pipe, wire-wound concrete tanks, luminaire and signal poles, and rail ties. Prestressing wire is also manufactured for seven-wire prestressing strand. Typical chemical compositions and manufacturing techniques for the production of wire are described in this section. Details relating to the use of wire in the production of prestressing strand are discussed in Section 2.2.

The high tensile strength obtained in cold-drawn, high-carbon steel wire is the result of three strengthening characteristics:

- Chemical content;
- Thermal treatment; and
- Deformation strengthening (cold working).

2.1.1 *Chemical composition*—Current ASTM standards applicable to the several types of high-strength steel prestressing wire specify steel compositions reflective of ingot cast steel. Before the late 1980s, open-hearth and electric-furnace ingot-cast steel had been the primary source material for high-strength wire rod. The composition of high-carbon, ingot-cast steels for those products has been based on high carbon and manganese with limitations on deleterious elements, such as sulfur and phosphorus. Ingot-cast steel has been virtually replaced in the United States by continuous-cast steel for high-strength steel wire and strand products. The traditional high carbon and manganese composition of ingot-cast steel rod has been supplemented with microalloying additions of grain refining and strengthening elements to produce a steel composition more suitable to the requirements of the continuous casting process.

In addition to high carbon and manganese, continuous cast steel contains relatively small amounts, generally less than 0.20%, of alloying elements, such as chromium, vanadium, or both, to achieve the minimum required mechanical properties in the finished rod. Current ASTM standards do not specify composition ranges or limits for either the microalloying elements or minor and tramp elements, such as nitrogen. These elements, however, are generally included in contemporary rod purchase specifications and are analyzed for control by the steel producers.

2.1.2 *Thermal treatment*—As-rolled rod does not have the proper microstructure or mechanical properties required for drawing into high-tensile-strength wire. Thermal treatment of the rod is required to produce a very fine lamellar pearlitic microstructure with the proper tensile strength and ductility for wire drawing. Two different thermal treatments have been used.

Before the 1980s, the most widely used thermal treatment for rod was lead patenting. Lead patenting, generally performed by the wire manufacturer at the wire mill, requires uncoiling multiple coils of rod and simultaneously pulling the parallel strands through a furnace to heat the rods above the transformation temperature. Immediately upon exiting the furnace, the heated rods are quenched in a molten lead bath to isothermally transform the microstructure into fine lamellar pearlite. The rods are subsequently cooled and recoiled before cleaning and wire drawing.

With the emergence of continuous casting of high-carbon steels and the high relative cost and environmental concerns associated with molten-lead baths for patenting, thermal treatment of rod has become incorporated in the finishing process at the rod mill. This thermal treatment practice, known as the Stelmore Process, has all but replaced lead patenting in the manufacturing of high-carbon, high-strength prestressing wire. In the Stelmore Process, hot-rolled rod emerging from the last stand on the mill is rapidly cooled with water to approximately 1500 F (815 C) and laid in a continuous spiral on a moving chain or a roller bed. The spiral of rod immediately passes over large fans that blow air against the exposed hot strands. The rapid cooling of the rod by the air transforms the steel into the fine pearlite microstructure required for drawing high-tensile-strength wire. The tensile strength of the finished, thermally treated rod, typically 160 ksi (1100 MPa) or higher, provides the foundation required to achieve the high tensile strengths required for prestressing-wire products.

2.1.3 *Deformation strengthening: cold work*—The first step in the wire-drawing process is acid cleaning of the rod to produce a clean, uniform surface free of mill scale and debris that could result in surface defects in the finished wire. Acid cleaning is performed in either hydrochloric or sulfuric acid for a specific period of time, followed by rinsing and immersion in a hot aqueous solution of zinc phosphate, borate, or lime. The coating is applied to protect the surface and enhance bonding of wire drawing lubricants during the drawing process.

Cold work by wire drawing increases the tensile strength of the product. This process may add over 100,000 psi (700 MPa) of tensile strength. The amount of cold work will generally result in a 60 to 85% total reduction in cross-sectional area and is carried out by drawing the wire through a number of consecutive wire drawing dies of decreasing diameter in a continuous operation. The dies are made of polished tungsten carbide and are precisely shaped and sized for each step in the reduction of rod to wire. Typically, 18 to 25% and generally no more than 30% reduction in area is accomplished by any one die. To produce wire with optimum mechanical properties, the wire-drawing variables, such as speed, lubrication, die configuration, interpass temperatures, and finish, should be closely controlled.

The reduction in diameter at each die generates large amounts of heat that should be removed from the wire. Lubrication, in the form of soap, is applied immediately ahead of each die to provide a barrier between the die and the wire. Water and air cooling of both the capstans, or drawing blocks, and the dies is incorporated in the wire-drawing machines to remove the heat generated in the wire. Drawing speed affects both rate of wire production and heat generation and removal, and should be adjusted to optimize both. Maintaining the wire temperature at 360 F (182 C) or less during drawing is critical to prevent the potentially damaging effects of dynamic strain aging in the wire. This subject is discussed in more detail in Section 3.3.

2.1.4 *Stress-relieving*—A number of types of single wire are stress-relieved after drawing. Stress relieving is performed by heating the wire to a temperature of 600 to 800 F (315 to 1380 C). This can be accomplished by a hot-air furnace, a fluid bath of lead or fluidized particles, or by resistance or induction heating. Stress relieving increases the material's modulus of elasticity, yield strength, and ductility as measured by percent elongation. Stress relieving also gives the product a constant relationship of stress to strain below the elastic limit so that the length of the extension in tensioning can be used as a measurement of stress.

2.2—Strand

Strand is produced from cold-drawn, high-strength wire as described in Section 2.1. The finished wire (before stress-relieving) is wound into a seven-wire bundle in which six wires are helically wrapped around a single straight wire. The strand is then stress-relieved, as discussed in Section 2.14, or stabilized, as discussed in Section 2.2.1.

ASTM A 416, A 882, A 886, and A 910 do not give a required chemical composition of the finished strand. The strand manufacturer is free to adjust the chemical content as necessary to meet the mechanical requirements of the standard. The same is true for ASTM A 779, except for the limit on phosphorus (less than 0.04%) and sulfur (less than 0.050%).

2.2.1 *Low-relaxation*—Prestressing strand is also manufactured in a stabilized, stress-relieved condition. Stabilization is performed to reduce relaxation characteristics of the strand by thermal-mechanical treatment—simultaneously stretching and heating the strand. The mechanical action of stretching compacts the individual wires in the strand, which reduces the relaxation of the strand to below the specified level. For example, low-relaxation strand produced to ASTM A 416 is required to have a maximum relaxation loss of 3.5% in 1000 h when loaded to 80% of its specified minimum ultimate tensile strength. The thermal treatment stress relieves the strand.

2.3—Bar

2.3.1 *Chemical composition*—As with prestressing wire, high-strength bar steel is manufactured with controlled additions of carbon and manganese to iron. Residual elements and harmful impurities are kept below specified limits. ASTM A 722 does not give a required chemical composition of the finished bar, except for limiting the impurities phosphorus (less than 0.04%) and sulfur (less than 0.05%), as in ASTM A 779. The manufacturer is free to adjust the chemical content as necessary for the bar to meet the mechanical requirements of the standard. ASTM A 911 specifies a specific chemical composition in which impurities are limited.

2.3.2 *Fabrication*—Threaded prestressing bars are manufactured from billet steel produced in an electric furnace in which the steel is melted, alloyed, and cast into billets for use in the bar hot-rolling process. The billets are reheated and hot-rolled into the bar configuration. The rolling process involves a continuous feed of bar through several stands that successively reduce the diameter of the bar to the proper size. The last rolling strand places the thread impressions in the bar. Unlike the strand-drawing operation, the bar is rolled under tension-free conditions so the threads on the bar are not distorted. Following the rolling operation, the bars are cut to length and cooled.

2.3.3 *Cold stretching and stress relieving*—After cooling, the bars' yield point is raised by cold stretching. The residual stresses from cooling and stretching are then removed with thermal stress relieving. The bars are heated to 700 F (370 C) in a furnace or with electrical induction heating.

CHAPTER 3—DETERIORATION OF PRESTRESSING STEEL

3.1—Introduction

As noted in Chapter 1, prestressing steel is used in both pretensioned and post-tensioned applications. Pretensioned applications involve direct contact between concrete and prestressing steel. The alkaline environment, provided by quality chloride-free concrete, protects the prestressing steel from corrosion just as for reinforcing bars. The same is true of post-tensioned concrete structures with grouted tendons. The degree to which concrete provides satisfactory protection is, in most instances, a function of the concrete (grout) quality, depth of cover, and the degree to which good practices are followed throughout the entire design and construction operation.

Unbonded tendons are generally protected by a plastic sheath and anticorrosive grease placed in the annular space around the strand. Tendon technology and field experience with greased and sheathed tendons has improved over the past 15 years. ACI 423.4R covers the history of unbonded tendons, including problems encountered in the field, and provides guidance on evaluating corrosion damage and repair methods. It does not, however, cover the metallurgy and deterioration mechanisms in as much detail as this document.

It is generally agreed that if pretensioned and post-tensioned systems are properly detailed and constructed, the protection provided will be adequate for the life of the structure. In the case of poorly detailed or constructed systems, or in systems in which the environment is more severe than expected, however, corrosion of the prestressing can occur. Experience has shown that a reduction in the cross-sectional area in nonprestressed steel reinforcement due to corrosion is generally not a primary concern. The corrosion of nonprestressed steel causes other problems with the structure, including unsightly staining, spalling of the concrete, and other serviceability-related issues, long before the loss of steel cross section becomes an issue. On the contrary, loss of cross-sectional area of prestressing steel due to general or pitting corrosion is a major problem for two reasons. One reason is that the prestressing steel experiences a continuous applied stress level of about 55 to 65% of its ultimate tensile strength throughout its life, which can temporarily reach even higher values during stressing. Loss of cross section increases the net stress in the prestressing, possibly leading to local yielding and fracture. The second reason is that the strength of the prestressing steel is normally four to five times higher than that of nonprestressed reinforcing steel. Consequently, if a prestressed concrete structure and a nonprestressed steel-reinforced structure have the same corrosion rate, the prestressed concrete structure sustains four to five times the damage of the nonprestressed reinforced structure. This difference will be even higher because prestressing tendons are generally made up of smaller-diameter wires that will lose relative cross-sectional area faster than larger bars undergoing the same corrosion rate. Therefore, remedial efforts for corrosion-damaged, prestressed concrete structures should consider this difference.

In addition to the traditional general or pitting corrosion damage of reinforcing bars, prestressing strand and wire have increased susceptibility to other kinds of damage that are not usually of concern for lower-grade steel. Damage from hydrogen embrittlement (HE), stress-corrosion cracking (SCC), fretting fatigue, and corrosion fatigue all can significantly affect the performance of the prestressing steel. One disturbing fact about these mechanisms is that there can be relatively little visible warning, such as corrosion product, immediately before failure. In addition, the failures are usually brittle, involving little elongation before fracture. While relatively few failures have been attributed to one or more of these mechanisms, it is important that designers be aware of the potential problems and the environments that may cause them.

3.2—Deterioration of prestressing steels

Corrosion is defined as the destructive attack of a material through a reaction with its environment (Fontana 1986; Uhlig and Revie 1985). The fundamental mechanisms for corrosion of prestressing steel in concrete are essentially the same as those for lower-grade reinforcing bars (Manning 1988; Perenchio, Fraczek, and Pfeifer 1989; Whiting, Stejskal, and Nagi 1993). ACI 222R provides complete coverage of the fundamental mechanisms of general and pitting corrosion of metals in contact with concrete. In subsequent discussions it is assumed that the reader is familiar with these fundamentals.

In bonded, post-tensioned construction, the tendon is usually in contact with a portland-cement grout. This grout is injected into a polyethylene or galvanized steel duct embedded in the concrete. This system should give superior corrosion protection over pretensioned construction because of the additional barriers (Whiting, Stejskal, and Nagi 1993). This can be true when the duct is properly and solidly filled with grout. A number of problems have been attributed to the lack of or improper injection of grout (Novokshchenov 1989a, 1991; Ohta et al. 1992; The Concrete Society 1996; Woolley and Clark 1993). In addition, the performance of prestressing steel embedded in concrete may not necessarily be indicative of the behavior of bonded post-tensioned systems (Perenchio, Fraczek, and Pfeifer 1989). Most problems associated with bonded post-tensioned construction occur as a result of inadequate grout injection. Further discussion of this issue is presented in Chapter 7. The remainder of this chapter focuses on brittle deterioration mechanisms, HE, and SCC.

3.3—Metallurgy of prestressing steels

3.3.1 *Microstructure*—Iron is a crystalline solid in which the atoms are arranged in a regular, tightly packed array, or lattice. This tightly packed lattice is due to the strong atomic forces that exist between the iron atoms. The two lattice patterns in which iron can exist are the face-centered cubic (FCC) and body-centered cubic (BCC). Most crystalline materials, including iron, are seldom found as single crystals but rather as polycrystalline aggregates in which the whole body is made up of large numbers of small interlocking crystals or grains. Each grain in the aggregate is connected to its neighbor by a grain boundary that is generally of irregular shape and bears no relation to the pattern of the grain. The individual crystalline axis of each grain is usually randomly oriented. Grain boundaries, theoretically, are only several atoms thick in a pure metal. In commercial metals, however, the grain boundaries are wider because impurities usually collect there.

3.3.2 *Crystal imperfections*—Imperfections within the crystalline structure greatly affect the mechanical properties of the metal. While there are a number of different types and causes of flaws, the most significant are dislocations. These can be point, line, or screw dislocations. Screw dislocations are the most significant in explaining plastic deformation.

3.3.3 *Plastic deformation*—Atoms in single-grain crystals are ordered into regular geometrical patterns, such as BCC and FCC. The cohesion in the solid is a result of the attraction between the atoms and gives the material its strength. Applied stress on the steel causes elastic deformations that result from temporarily displaced atoms. The atoms return to their original position when the stress is removed. Plastic deformation occurs when the atoms undergo permanent deformation. This is due to the relative displacement of atoms along slip planes. In general, the slip planes occur along planes of greatest interplanar spacing where the atomic forces are the weakest. This slip is caused by shear stresses within the crystalline structure. The stress necessary to pull the crystalline structure apart in tension is much higher than the shear stress necessary to cause slip.

Multicrystalline structures, such as steel, have a more complex plastic deformation mechanism. The crystalline structure in a multicrystalline material is randomly oriented between crystals. This impedes the slip mechanism of a single grain because of the restraint imposed by the surrounding crystals to slip deformation. The change in shape of any particular crystal should conform to the change in shape of the surrounding crystals. Dislocations inside the crystal allow slip to propagate through a single crystal. The slip causes the dislocations to travel though the crystal until they encounter an imperfection, such as an impurity atom, a precipitated particle, or another dislocation. After encountering an imperfection, an increase in applied stress is required for a dislocation to continue its forward motion. This increased stress is called strain hardening. Fracture occurs when the dislocations accumulate and can no longer travel through the grain. The more dislocations there are within the grain, the more plastic deformation will occur before fracture.

3.3.4 *High-strength steel microstructure*—Pearlite is a lamellar aggregate made up of alternate plates of ferrite (BCC iron with carbon in solution) and cementite (Fe_3C). Pearlite is formed when eutectic austenite (0.80% carbon) is cooled below the critical temperature. The transformation occurs as a cementite nucleus forms. The carbon from the surrounding austenite is depleted during the formation of the cementite plate. This depletion causes the transformation of the surrounding layer of austenite to ferrite. This process continues as new layers of alternating ferrite and cementite form during cooling. Nucleation and growth of the layers occurs at several locations along the austenite grain boundary. At each location, approximately hemispherical pearlite nodules are formed. These nodules grow until the entire austenite grain is consumed. This transformation occurs with relatively slow cooling and results in a pearlite structure. If the austenite is rapidly cooled, then it forms into a phase known as martensite. Martensite is the product of a different mechanism of transformation with no precipitation of carbon. It is a single-phase, supersaturated solution of carbon in ferrite, with carbon located interstitially in a body-centered tetragonal lattice. This is a distorted version of the normal BCC to BCC tetragonal, which results in high strength and hardness and low ductility.

Cherry and Price (1980) indicate that the microstructure of typical prestressing steel consists of fine pearlite oriented so that the lamellae lie parallel to the axis of the wire. Lead patenting the wire (rapidly cooling the eutectoid steel from the austenite region in a lead bath) provides the finest pearlitic structure possible. During drawing, the cementite is plastically deformed and the resulting microstructure consists of alternate lamellae of ferrite and cementite that are aligned with the axis of the wire.

3.3.5 *Brittle and ductile behavior*—A ductile material allows significant plastic deformation before fracture. A brittle material allows very little plastic deformation before fracture. Standard test methods generally call for a minimum percent elongation before rupture as a measure of the material's inelastic deformation capacity (ductility). Elongation usually varies along the length of the specimen and is the greatest where necking occurs. Another measure of ductility is the reduction of cross-sectional area, usually measured at the minimum diameter of the neck. Microscopic inspection of the fracture surface can also give an indication of whether the fracture is brittle or ductile.

3.4—Stress-corrosion cracking and hydrogen embrittlement

The susceptibility of steel to SCC and HE generally increases with increasing strength (Fontana 1986; McGuinn and Griffiths 1977; Uhlig and Revie 1985). This characteristic, along with the trend in the late 1960s toward the use of higher-strength tendons in bridges and parking decks, contributed to the concern regarding possible damage due to SCC or HE (Klodt 1969). HE is defined as the reduction in ductility due to the absorption of atomic hydrogen into the metal lattice (Fontana 1986; Uhlig and Revie 1985). Before discussing the factors that affect HE and SCC, it is necessary to define these behaviors. Unfortunately, there is some debate concerning the differences (or similarities) between SCC and HE in prestressing steel (Manning 1988).

For a brittle failure to qualify as SCC, the metal should be under tensile stress and simultaneously exposed to a corrosive environment (Fontana 1986; Manning 1988; Uhlig and Revie 1985). The tensile stress may be either applied or residual, and the corrosive environment should be specifically damaging to the metal or alloy. Furthermore, the damaging environments must not be usually corrosive in the normal sense; that is, corrosion products and weight loss at or near the failure may be negligible (Uhlig and Revie 1985). The cracking takes the form of transgranular or intergranular cracking. Intergranular cracking occurs at the grain boundary, while transgranular cracking propagates through the grains. The type of cracking that occurs depends on the environment and metallurgy (Novokshchenov 1994).

HE occurs only with the absorption of hydrogen atoms, because the hydrogen molecule is too large to penetrate the crystalline structure of the steel. Unlike SCC, the material need not be stressed for HE to occur. Hydrogen can be introduced before installation during the manufacturing process or storage. For instance, HE can occur while a tendon is stored in a duct waiting stressing and grout injection. The HE

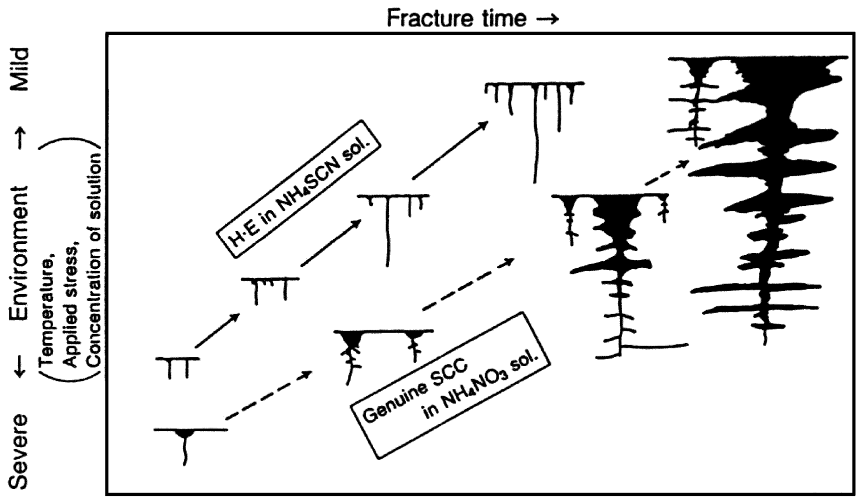

Fig. 3.1—Change in crack pattern of HE and SCC fractures (Yamaoka and Tanaka 1993).

becomes apparent when the tendons are prestressed and the tendon fails. HE can occur when the steel is stressed and, if sufficient atomic hydrogen is available for absorption, it will eventually fracture. This behavior falls under the definition of SCC, and some indicate that HE is a form of SCC. The reverse, however, is not true—SCC is not a form of HE (Fontana 1986). This form of HE is sometimes referred to as hydrogen-induced stress cracking, hydrogen-induced cracking, hydrogen cracking, HE stress cracking, or hydrogen-induced SCC (HISCC) (Gramberg 1985; Isecke 1982; Uhlig and Revie 1985). This is an indication that there is some combined effect of applied stress and hydrogen absorption.

One of the difficulties in distinguishing between the two types of failures is that they both occur by brittle fracture and may both have the same appearance (little necking). In both, pitting or general corrosion may or may not be present, and little associated elongation and reduction of cross-sectional area occurs before fracture.

The two basic theories that have been proposed to explain SCC are electrochemical dissolution and stress sorption (Uhlig and Revie 1985). The electrochemical dissolution theory proposes that galvanic cells are set up along grain boundaries in the metal. The localized electrochemical dissolution opens a crack. The stress disrupts the brittle oxide film over new anodic material, which is corroded. This process continues as the crack works through the material. Intergranular stress corrosion of carbon steel in nitrate solutions is an example of dissolution SCC caused by the anodic reaction (Whiting, Stejskal, and Nagi 1993).

The stress-sorption theory proposes that the cohesive bonds between the metal atoms are weakened by the adsorption of damaging components of the environment. The applied stress causes crack growth along the weakened boundary. Therefore, the boundary weakening is caused by elements that weaken the atomic bond, rather than by anodic dissolution.

One HE theory proposes that the atomic hydrogen diffuses into the lattice of the metal and accumulates near slip dislocation sites or microvoids. The dissolved hydrogen then interferes with the slip mechanism, reducing the ductility of the metal.

Regardless of the mechanism, the presence of atomic hydrogen in significant quantities can promote nonductile behavior in high-strength steels.

Yamaoka and Tanaka suggest that genuine SCC and HE are different phenomena and can be distinguished as such by the pattern of cracking (Yamaoka and Tanaka 1993). Experimental testing with cold-drawn, stress-relieved wire using NH_4SCN and NH_4NO_3 solutions produced HE and SCC, respectively. The results are schematically represented in Fig. 3.1, which indicates that SCC is a dissolution-based phenomenon and takes longer to occur than HE. Furthermore, nitrate solution causes dissolution of the ferrite structure, but the cementite structure is the cathode in the reaction and does not allow the corrosion process to proceed until the cementite platelet is fractured by the stress and exposes fresh anodic ferrite.

This document does not provide exhaustive coverage of the various mechanisms of HE or SCC but rather a summary of research and field experience. Its focus is on the environments and field conditions under which HE or SCC is expected to occur, as well as methods to test prestressing steels for susceptibility to SCC or HE.

3.5—Case histories

Relatively few failures in the literature are attributed to brittle mechanisms such as SCC and HE. One possible reason is that the prestressing steels normally used in prestressed concrete construction resist this type of failure quite well. A number of problems have occurred on prestressed concrete structures in recent years that are not reported in the literature. This is probably because the failures have generated litigation with closure of trial proceedings or nondisclosure agreements between litigants. Another possible reason is that failures may have occurred in conjunction with pitting corrosion. In this case, the investigators may not realize that the failures are due to brittle HE because of the heavy pitting damage that may be present. The committee does not believe that this is a widespread issue, and that the occurrences of HE and SCC failures

Table 3.1—Media that promote stress corrosion cracking of structural steel (Flis 1979)

Type of steel	Medium	Conditions for SCC	Cracking paths
Ferritic steels	Aqueous nitrates	NH_4NO3, $Ca(NO_3)_2$, $NaNO_3$, pH 3 to 8, potentials approximately –200 to +1600 mV_{NHE}; high corrositivity of conc. Solutions (20 to 60%) and hot solutions (at boiling point)	Intergranular; ferrite grain boundaries, and in quenched steels, the former boundaries of austenite grains
	Aqueous NaOH	Concentrations above 5% at 373 K, up to 50% at 323 K, O_2 in minor amounts, potentials approximately –850 to –550 mV_{NHE}, and +500 to 700 mV_{NHE}	As above, and transgranular at high temperatures
	Liquid NH_3	Anhydrous NH_3 with trace O_2, potentials above –450 mV_{NHE}, 0.2% addition of H_2O inhibits SCC	Inter- or transgranular
	Aqueous $CO^{2/3}$ HCO_2-solutions	1 *M* $(NH_4)_2$ CO_3, 0.5 *N* Na_2CO_2 + 0.5*N* $NaHCO_3$, pH 8 to 10, 343 to 368 K, potentials approximately –500 to –350 mV_{NHE}	Inter- or transgranular
	H_2O-CO-CO_2	Temperature 293 to 373 K, pH 6	Transgranular
	Aqueous phosphates	1 *M* (NaH_2PO_4, pH 4.8; 295 or 373 K, potentials at 373 K –400 to –250 and –130 to –80 mV_{NHE}, tensile stress applied at a rate of 10^{-6} s^{-1}	Transgranular
	Aqueous acetates	Saturated steam on the low-pressure side of a steam turbine, 0.1 to 1.0 *M* CH_3COONH_4, pH 8, 363 K, potentials from –360 to –200 mV_{NHE}	Intergranular
	Aqueous HCN	Pure solutions for containing NH_3, H_2S, CO, CO_2 (gas liquor)	Transgranular
	Aqueous $FeCl_3$	10^{-4} to 10^{-3} *M* $FeCl_3$, temp. 583 K	Inter- and transgranular
	Aqueous ethanolamine solutions	15%, ethanolamine is water containing H_2S and CO_2, temp. up to 423 K	Unreported
	Aqueous Na_3AlO_3	Na_3AlO_3 in water with bauxite and lime, at 418 K	Unreported
	AlC_3 + SbC_3 in hydrocarbons	Catalyst—10% $AlCl_3$ + 90% $SbCl_3$ in hydrocarbons, 363 K	Intergranular
Martensitic steels	Water, humid air, acids, salts, alkalis	3% NaCl, 3% Na_2SO_4, 20% H_2SO_4, room temperature or above, cathodic or anodic polarization; hot NH_4NO_3 or $Ca(NO_3)_2$ solutions	Previous boundaries of austenite grains or intergranular

remain small when compared to failures attributed to general or pitting corrosion of prestressed concrete structures.

Canonico, Griess, and Robinson reported failure of prestressing tendons in a prestressed concrete reactor vessel (1976). The tendons were bundles of ASTM A 416 prestressing strand. During detensioning of the tendons, it was found that nearly the entire inner row of tendons had failed. The tendons were unbonded and protected by wax containing chlorides and nitrates. Ammonium salts, such as nitrate and carbonate, are known to cause SCC. SCC in an environment of free ammonia and atmospheric CO_2 was postulated. An epoxy sealer was used to coat the seal plates and concrete surfaces around the tendon. The investigators indicated that an improper formulation of the epoxy resin was the cause of the high levels of nitrogen.

Probably the most prominent failure caused by HE is the 1980 collapse of the Berlin Congress Hall, in which the failure of the quenched and tempered prestressing rods was due to HISCC (Isecke 1982). The report, however, indicates that when fracture surfaces were inspected there were signs of "heavy grain-boundary corrosion and dissolution of former austenite grain-boundary regions." This would indicate that SCC due to dissolution could be the cause of the fractures. Isecke indicates that the appearance of the fracture surface is typical of HISCC in the type of steel used in the construction. It was also noted that the fracture surfaces contained micro- and macroscopic cracking, and that the surfaces of the rods were deeply pitted.

Schupack and Suarez conducted a survey that indicated, in general, good performance of prestressing strand/wire in the United States (1982). They reported receiving information on 50 corrosion incidents of which there were 10 cases of probable brittle failure caused by either SCC, HE, or a combination (40% of them were parking structures that were treated with deicing salts). This low number suggests that prestressing strand and wire have been performing well in service when properly protected.

Yamaoka and Tanaka reported two examples of field failures attributed to SCC (1993). One was a prestressing strand that was left stressed and ungrouted in a post-tensioning duct for 7 months at a construction site. Another was a prestressing wire wrap for a pipe that failed after 6 years of service. Microscopic examination of the failed wires indicated SCC.

3.6—Stress-corrosion cracking

A list of environments that may cause SCC is shown in Table 3.1 (Flis 1991). It is believed that nitrates have the strongest effects on steel and these environments are often used to rapidly evaluate steels for susceptibility to SCC (Flis 1991; Uhlig and Revie 1985). In prestressed concrete systems, the metallurgy of the steel and the environment to which it is exposed are unique. Most of the environments and conditions shown in Table 3.1 are not likely to be encountered by

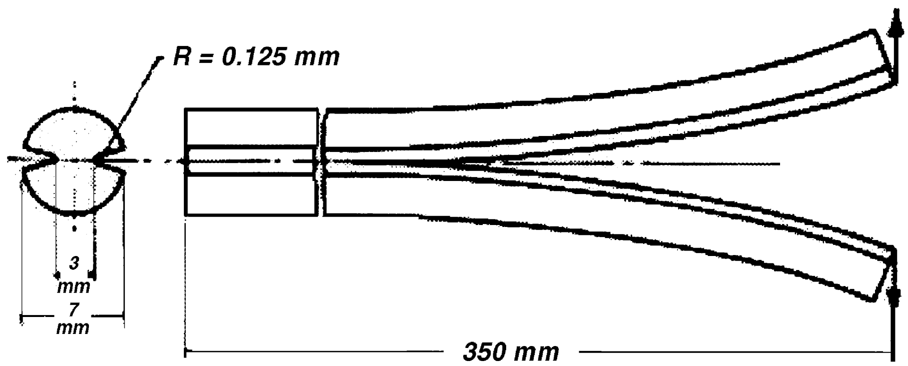

Fig. 3.2—Fracture test specimen (McGuinn and Griffiths 1977).

prestressing steel in normal applications. Klodt indicates that of the many possible combinations of environments that cause SCC in iron-based alloys, the only environment that prestressing steel might be exposed to in service is H_2S (Klodt 1969).

It is necessary to test a particular metal in the expected operating environment to effectively determine its susceptibility to SCC. Considerable work was conducted from the late 1960s to the early 1980s on the susceptibility of cold-drawn prestressing wire to SCC and HE. Various environments and conditions were investigated to determine specific circumstances in which SCC or HE could be anticipated. The results are mixed. Much of the research focuses on an accelerated test condition and applied potentials. While this approach gives interesting findings, it does not give definitive answers concerning the performance of prestressing steel under actual conditions. The literature reviewed in this section covers testing conducted using prestressing steels exposed to environments intended to represent concrete or contaminated concrete.

Monfore and Verbeck performed several tests of prestressing wire to determine the effect of calcium-chloride additions on the strength of the wire under long-term exposure (1960). Both as-drawn and stress-relieved wires were tested. Each specimen was composed of a single wire encased in mortar. The calcium-chloride content varied from 0 to 4%. The specimens were stressed and placed in dry and wet storage for 2 years. Following the test period, the wires were removed from the mortar encasement and tensile tested. As expected, the results indicated that the tensile strength of the wire was reduced in the specimens with a higher calcium-chloride content and wet storage. In addition, the stress-relieved wires tended to have a higher reduction in tensile strength than the as-drawn wire. There was no indication whether the tensile tests resulted in brittle or ductile failure modes.

Klodt performed experimental studies in which smooth stress-relieved wire; as-drawn, cold-drawn prestressing wire; and quenched and tempered wire were tested for SCC at stress levels of 175, 200, and 225 ksi (1210, 1380, and 1550 MPa) in various environments (1969):

- 3.5% NaCl and $CaCl_2$ solutions at room temperature;
- Saturated $Na(OH)_2$ solution with 3.5% NaCl and $CaCl_2$ at room temperature; and
- 3.5% NaCl and $CaCl_2$ solutions at 200 F (93 C).

In all cases, the specimens underwent general or pitting corrosion without brittle failure. It was concluded that SCC was not a problem in a chloride environment. Other research cited by Klodt indicated that SCC of cold-drawn steel in concrete contaminated with chlorides is not a problem.

McGuinn and Griffiths' report that the application of a fracture mechanics approach to the SCC problem in prestressing steel was useful (1977). The approach involved the use of precracked, prestressing wire specimens as opposed to smooth specimens that had been used previously. They found that this approach avoided lengthy initiation times associated with smooth specimens (necessary to form a pit), allowed the use of a less severe and more realistic environment, and resulted in more reproducible data. The stress-intensity factor K_I gives an indication of the intensity of the stress at a crack tip and is a function of the crack and specimen geometry and applied stress. At the critical stress-intensity factor K_{Ic}, fracture occurs. If the same specimen is tested in an environment that causes SCC and the failure occurs at a lower stress, then the critical stress-intensity factor for environmental cracking is $K_{ISCC} < K_I$. The investigators claim that this approach gives two distinct advantages:

1. If no SCC is observed up to K_{Ic}, then the material is presumed immune to SCC for the duration of the test; and
2. If SCC is observed in testing, then K_{ISCC} allows determination of maximum working stresses for a particular defect size so that safe operation will occur ($K_I < K_{ISCC}$).

The test program included the testing of 3.6 in. (7 mm) diameter cold-drawn wire with axial precracking (Fig. 3.2).

The specimens were tested in a saturated $Ca(OH)_2$ solution in which the pH was varied to simulate carbonation of the concrete and also in $Ca(OH)_2$ solutions with varying pH and NaCl content. The effect of stress relieving was also investigated. The specimens were not artificially polarized. In the tests to determine the effect of pH on SCC, some susceptibility to SCC was found in solutions with a pH up to 12.3, above which no SCC was noted. The addition of chlorides to the solution increased the critical pH for stress cracking, indicating that the amount of chloride necessary to produce SCC is strongly dependent on pH (Fig. 3.3). The authors postulate that at pH levels from 2 to 10, the likely actual mechanism of the SCC is HE. In addition, the SCC observed in the continuous pH range from 7 to 12.7 likely involves hydrogen, although no specific mechanism is proposed.

Cherry and Price conducted tests with two different strain rates on smooth, cold-drawn, stress-relieved prestressing wire

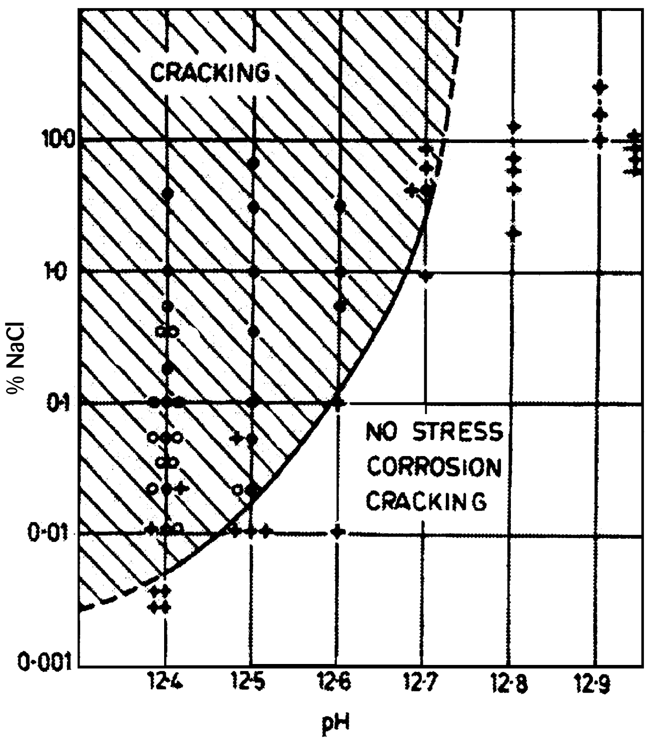

Fig. 3.3—Stress corrosion crack resistance as a function of pH and chloride content (Klodt 1969).

(260 ksi [1800 MPa] ultimate tensile strength) to determine if sodium-chloride solutions of varying pH (10, 12, 14) and anodic polarization would cause SCC (Cherry and Price 1980). The first test was a long-term constant strain test that lasted over a year. These were conducted at 218 ksi (1500 MPa). The second test was an ultra-slow SCC test at a strain rate of 2×10^{-6} s^{-1}. Wires fractured in both tests. The failures, however, were attributed to loss of section due to corrosion and not SCC.

McGuinn and Elices indicate that there are few reports of SCC of prestressing steel at the time of their article in 1981 (1981). SCC with smooth specimens of cold-drawn prestressing wire is difficult to reproduce in the laboratory in realistic environments. They also indicate that failure can be induced quite easily when using precracked specimens.

Tests were conducted on cold-drawn, stress-relieved wire and a control sample of nonstress-relieved wire. The wires were notched transversely and cracks were initiated at the notches under fatigue loading. The wires were then stressed to a constant strain and exposed to solutions of $Ca(OH)_2$ with Na_2S, $Ca(OH)_2$ solution with 1% NaCl, distilled water, and ammonium-thiocyanate solution. Cathodic polarization was imposed on some specimens by connection to zinc and magnesium anodes. The alkaline-sulfide solution exhibited no indication of SCC at any stress level. SCC occurred to some degree, however, in all other environments tested. In addition, stress-relieved wires showed a slight decrease in resistance to SCC when compared to nonstress-relieved wires. Specimens connected to zinc and magnesium anodes had significant decreases in resistance to SCC when tested in neutral artificial seawater. In an alkaline environment, however, there was no increase in SCC. Even when the alkaline solution was contaminated with NaCl there was no increase in SCC. The investigators indicated that the SCC of high-strength ferrous alloys may well be due to build-up of HE.

Stoll and Kaesche conducted SCC tests on smooth cold-drawn and quenched and tempered steels (1981). The tests were conducted in $Ca(OH)_2$ solution with the pH varied between 7 and 12.6. Both static and slow strain-rate tests were conducted. No SCC was evident in the solutions when unpolarized. If the specimens were cathodically polarized to potentials more negative than $-850mV_{SHE}$ (standard hydrogen electrode), however, the specimens became severely embrittled, as evidenced by the decrease in the necking of the fracture area. The investigators attribute this behavior to the onset of cathodic hydrogen evolution.

Parkins et al. conducted a similar set of tests for SCC and HE on cold-drawn, stress-relieved wire (1982). Three types of specimen preparation were used: smooth, Charpy v-notch, and fatigue precracked. Test environments included $Ca(OH)_2$ solution in which NaCl or HCl were added in varying amounts. Specimens were also tested with applied potentials at a slow strain rate. The most obvious conclusion to be drawn is from the tests in alkaline solutions containing HCl for pH adjustment. Below $-900mV_{SCE}$ (saturated calomel electrode), failure is related to hydrogen, but at potentials above $-600\ mV_{SCE}$, pitting at nonmetallic inclusions allows acidification within the pit that leads to cracking that is dissolution related. Parkins et al. found that at applied potentials more positive than $-600\ mV_{SCE}$, SCC is present in the form of dissolution at the tip of the crack rather than being caused by hydrogen evolution (1982). The major trend is that enhanced cracking occurs as the applied potential is reduced below $-900mV_{SCE}$. Results of tests at intermediate pH are most readily interpreted in terms of hydrogen-assisted cracking at low potentials and dissolution-related failures at higher potentials. Evidence of selective pearlite dissolution within pits and on stress-corrosion fracture surfaces suggests that acidification occurs within pits and probably within precracks or growing cracks (Fig. 3.4). Parkins et al. suggests that acidification occurs within cracks in the pH range of 3 to 4 and that crack-tip potential is below that required for hydrogen discharge (1982). They go on to say that such observations are not an unequivocal demonstration of failure by a hydrogen-related mechanism. This is supported by the fact that there is an appreciable range of potential below that necessary for hydrogen discharge but in which dissolution of iron is still possible in acidic solutions (suggesting a possible combination of dissolution and embrittlement). The main difference is that in the Parkins et al. study the wires were notched, while they were not in the tests conducted by Cherry and Price.

Hampejs et al. indicates that there are two types of stress corrosion (1991). The first involves the anodic dissolution of the steel in the crack tip, and the second is hydrogen-induced stress corrosion with cathodic hydrogen absorption. The absorption reduces the cohesion between the steel molecules.

Yamaoka and Tanaka describe three different fracture mechanisms: delayed fracture; HE; and SCC (1993).

3.7—Hydrogen embrittlement

Investigations have revealed that steel embrittled by hydrogen displays the following characteristics (Troiano 1960):

- The notch tensile strength may be less than normal, directly reflecting a loss of ductility due to hydrogen absorption;
- Delayed failure can occur over a wide range of applied stress;
- Time-to-failure depends slightly on the applied stress; and
- Below a critical stress, failure does not occur.

As discussed previously, HE can cause a reduction in ductility and strength of prestressing wire. HE requires the presence of atomic hydrogen in the steel lattice. HE can be induced by presence of poisons that prevent the combination of atomic hydrogen into gaseous form. Sulfides and arsenides are the most common poisons and have been used in much of the research on HE. Other poisons include phosphorus (P), antimony (Sb), selenium (Se), tellurium (Te), and cyanide (CN). The most likely to be encountered in practice are sulfur dioxide (SO_2) and hydrogen sulfide (H_2S). Another method that can be used to generate atomic hydrogen is cathodic polarization. This method cathodically polarizes the steel to a level such that the oxygen reduction reaction is reversed, producing molecular oxygen and hydrogen ions from water.

While most steels are susceptible to HE to some extent, high-strength steels are particularly susceptible (Jones 1992). The type of steel also affects the tendency toward HE. There are four main types of prestressing steels:

1. Hot-rolled, stretched, and stress-relieved bars;
2. Quenched and tempered martensitic wires/bars;
3. Cold-drawn, stress-relieved wire/strand; and
4. Cold-drawn wire.

Of these four, the cold-drawn, stress-relieved wire/strand (prestressing wire/strand) is the most resistant to HE cracking, while the quenched and tempered steel is the least resistant (Hampejs et al. 1991).

The previous review of SCC research indicates some debate as to the exact failure mechanism that can cause environmental cracking of prestressing steels. The following presents the available literature on HE.

Klodt indicates that HE can be characterized by the sensitivity of HE to low strain rates and elevated temperatures, which suggests that embrittlement is controlled by the diffusion of hydrogen (1969). Klodt also indicates a possible problem with HE due to contact between galvanizing and cement paste, because hydrogen is generated (1969). The presence of poisons can prevent the combination of atomic hydrogen into molecules and allow the atomic hydrogen to enter the steel. Tests of prestressing wire indicated that brittle failure occurred in wires exposed to H_2S after a short exposure period. The stress-relieved wires had longer time-to-failure than those that were not stress relieved.

Rehm indicates that fractures of prestressing steel are always due to anodic stress corrosion or hydrogen-induced cracking (1982). Anodic stress corrosion only appears in the passive state, in which there has to exist a specific medium adapted to the special sensitivity of the steel and, in addition, sufficiently

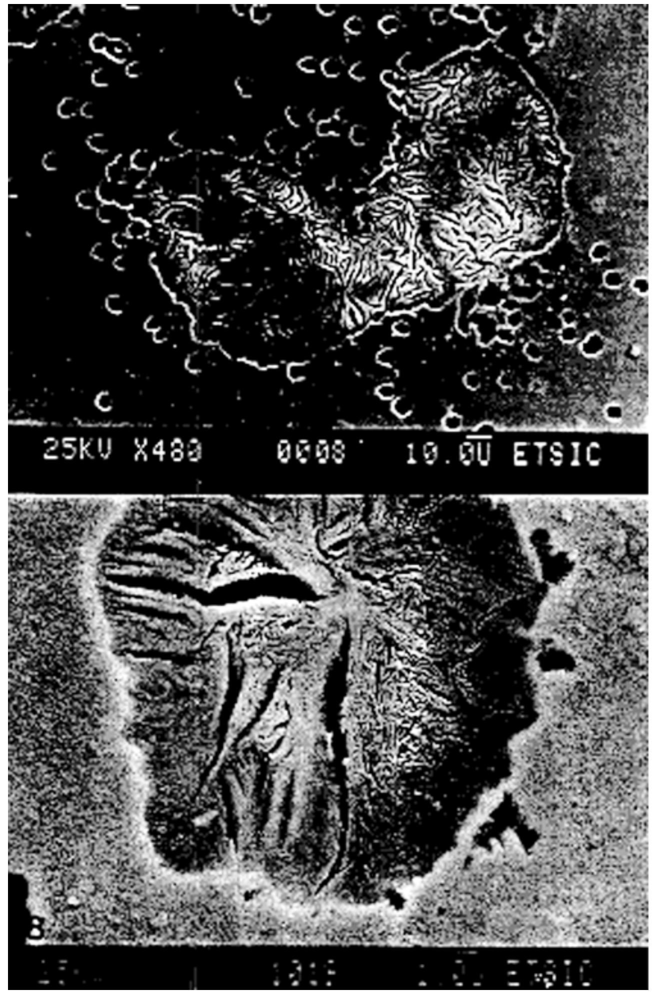

Fig. 3.4—Corrosion pits with cracking at base of pits (Parkins et al. 1982).

high tensile stresses and a sensitive material. Conditions for HE are less specific. Rehm also indicates that the testing for cracking susceptibility is geared toward determining the sensitivity toward HE in which stressed steels are exposed to highly concentrated chemical solutions and tested until failure. Rehm describes a mechanism in which the steel is embrittled by the cathodic reaction. The cathodic reaction can be either the reduction of oxygen or the reduction of water molecules, with the resulting production of hydrogen atoms. Only a small part of the hydrogen remains in the atomic state and enters the steel lattice, while the remainder of the hydrogen combines as molecular hydrogen, which is too large to enter the lattice. The hydrogen-reduction cathodic reaction can control in acidic environments such as industrial areas, at the base of a pit, or in splits in the steel where there is a lack of oxygen.

Price et al. investigated the role of notches in the hydrogen-assisted cracking of prestressing steels (1992). The investigators indicate that hydrogen may enter the tendons as a result of cathodic protection or the acidification of the solution within corroding pits. Smooth, unnotched specimens are considered to be more representative of service conditions than notched or precracked specimens. Notches present at the prestressing wedges were not considered. The investigators

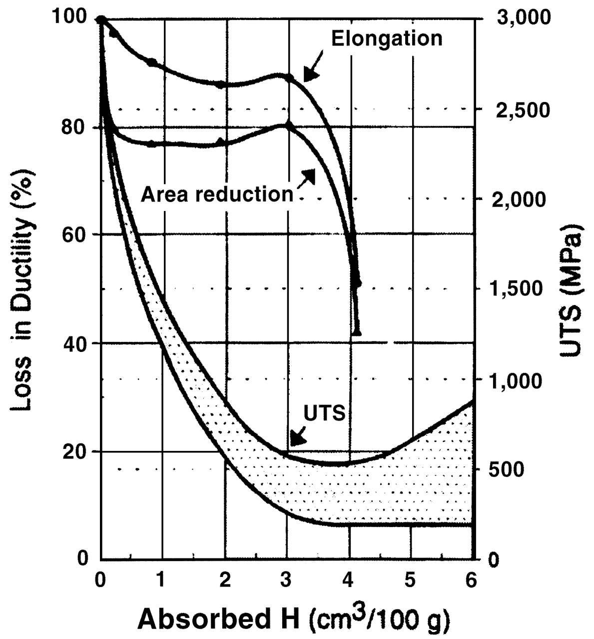

Fig. 3.5—*Ductility and UTS of high-strength steel versus total H content (Klodt 1969).*

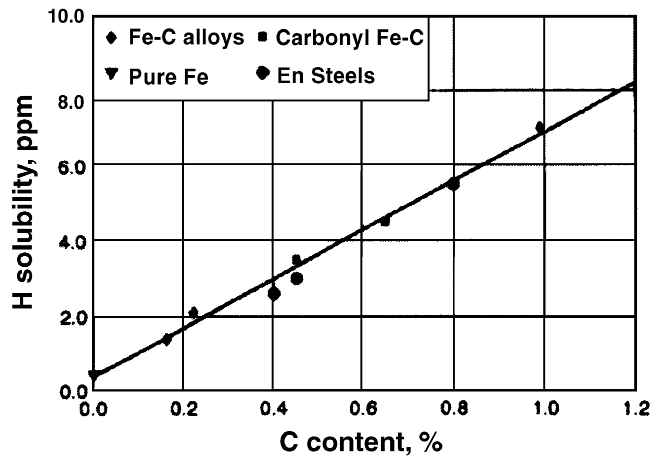

Fig. 3.6—*Effect of C content on H absorption of a steel (Klodt 1969).*

indicate that when pitting corrosion does occur, it spreads along the wire and results in flat-bottomed corrosion damage as was found in previous research. This particular investigation included the effect of two notch configurations on hydrogen cracking. The first was a flat-bottomed notch 1/2 in. (10 mm) long and 0.04 in. (1 mm) deep with rounded corners. The second was Charpy V-notch. The specimens were charged with hydrogen and slow strain rate tests were conducted, after which fractographic analyses were conducted to determine the nature of the fracture. The flat-bottomed specimens fractured by ductile shear. The percentage reduction in cross-sectional area, however, indicated a brittle failure, while the elongation was reduced significantly over that of the uncharged specimens. It was concluded that the reduction in ductility in the flat-bottomed notches is due to HE.

Novokshchenov indicates that there are controlling factors that can affect the rate of hydrogen diffusion into the lattice (1994). The most important factor is the concentration of hydrogen ions in the steel lattice. Figure 3.5 shows the variation of ductility with the hydrogen content of the steel. This is controlled by a variety of manufacturing and environmental factors. Susceptibility to HE increases with:

- Increased carbon content, because of the increase in cementite (Fe_3C) particles and enlargement of the ferrite-cementite interface area (Fig. 3.6);
- Increased cold working;
- Increased stress relieving, because of the change in the ferrite-cementite interfaces;
- Increased chloride concentration because of the increase in concentration of chlorides; and
- Increased temperature. Susceptibility to cracking increased with a temperature up to a maximum of 77 F (25 C), then decreased.

3.7.1 *Effect of metallurgy*—Carbon steels are more susceptible to HE when heat-treated to form martensite but are less susceptible if the structure is pearlitic (Uhlig and Revie 1985). Quenched and tempered steel generally has a tempered martensitic microstructure produced by quenching (Hampejs et al. 1991). This type of structure has high residual stresses due to quenching, which are somewhat reduced by the tempering process. The martensitic microstructure has relatively few dislocations into which hydrogen may diffuse. Consequently, a relatively small amount of hydrogen can interrupt the yielding, causing premature brittle fractures. Hot-rolled bars have a microstructure similar to that of quenched and tempered steel, a relatively low dislocation density, and are relatively free from residual stresses. Cold-worked steels with elongated pearlitic microstructure are the least sensitive to HE. The cold-drawing process introduces a large dislocation density and a large number of sites that are available to which hydrogen can diffuse. Therefore, higher pressures are necessary to embrittle cold-worked steel than quenched and tempered or hot-rolled steels.

Wire-drawing temperatures exceeding about 400 F (204 C) compromise ductility and produce dynamic strain aging (DSA), making the wire susceptible to HE (Bradish, Quinn, and Lewis 1995). Longitudinal splitting of the wire indicates failure by DSA. Hundreds of splitting fractures in prestressing wires were the cause of prestressed pipeline failure (Flis 1991). It was found that no coolants were used on the die during manufacturing to keep the wire cool during drawing. Corrosion can occur in these longitudinal splits because the protective concrete or grout is not able to penetrate the split. Moisture can collect in the crack and initiate crevice corrosion in which the anodic reaction produces hydrogen.

The carbon content of the steel affects the solubility of hydrogen into the lattice. With increasing carbon, there is an increase in cementite particles and enlargement of the ferrite-cementite interface, allowing more hydrogen atoms to be trapped.

3.7.2 *Corrosion-generated hydrogen*—The concentration of chlorides at the base of a crack in concrete or grout can lead to pitting corrosion. While use of the electrochemical theory to explain pitting corrosion is still the subject of much research and discussion, a few basic principles are agreed upon (Schiessl 1988). Figure 3.7 shows the formation and propagation of a pit. Pitting generally describes a unique type of anodic reaction that is autocatalytic or self-propagating in nature. The process is initiated by the arrival of sufficient quantities of chloride ions at the surface of the steel to disturb the passive layer and initiate active corrosion. When the locally available oxygen is depleted, the reduction reaction ceases in the pit and begins at sites adjacent to the pit where oxygen is available for the reduction reaction. The dissolution of the iron ions continues, producing an excess of positive charge in the pit region. The negatively charged chloride ions are drawn to the iron ions in the pit, resulting in the increased concentration of ferric chloride that hydrolyses, producing insoluble rust and free acid

$$FeCl_2 + H_2O \rightarrow Fe(OH)_2 + H^+Cl^-$$

With the production of free acid, the potential in the region becomes more negative. The adjacent sites produce hydroxyl ions that increase the pH at the cathodic sites. Novokshchenov indicates that one possible source of hydrogen is at the base of this pit (Fig. 3.7) (1994). The pH in the pit can be as low as 1.5 to 5.0, resulting in a shift in the corrosion potential below the reversible hydrogen potential. Inside the pit, the H_2O dissociates at the anode site to form H^+ protons that migrate to the cathode site where they are discharged to atomic hydrogen (with electrons freed during iron dissolution) and adsorbed into the steel. As the pH decreases, the combining of electrons and hydrogen protons tends to replace the cathodic reaction occurring outside the pit, and the process continues with atomic hydrogen collecting in the imperfections in the steel. It has also been suggested that promoters such as sulfur, arsenic, or thiocyanate are necessary at the base of the pit for the atomic hydrogen to form (Isecke and Miletz 1993).

3.7.3 *Cathodic protection*—If the concrete or grout surrounding prestressing strand or wire is contaminated with chlorides, cathodic protection can be applied to prevent corrosion of the steel. Cathodic protection essentially consists of cathodically polarizing the prestressing steel with respect to a sacrificial anode. Polarization can be provided by the natural difference in polarization levels between two materials or by impressed current. This forces the anode to corrode sacrificially to protect the cathode. Depending on the level and extent of corrosion, an impressed current may need to be applied to the systems to sufficiently polarize (and protect) the steel. If the applied protection potential is low, then the predominant cathodic reaction is (Isecke and Miletz 1993)

$$1/2O_2 + H_2O + 2e \rightarrow 2OH^-$$

If the cathodic protection exceeds the equilibrium hydrogen potential value, then water is reduced as follows:

$$H_2O + e \rightarrow H_{ADS} + OH^-$$

where H_{ADS} denotes adsorbed atomic hydrogen. The equilibrium potential of the hydrogen cell depends on the pH. At the

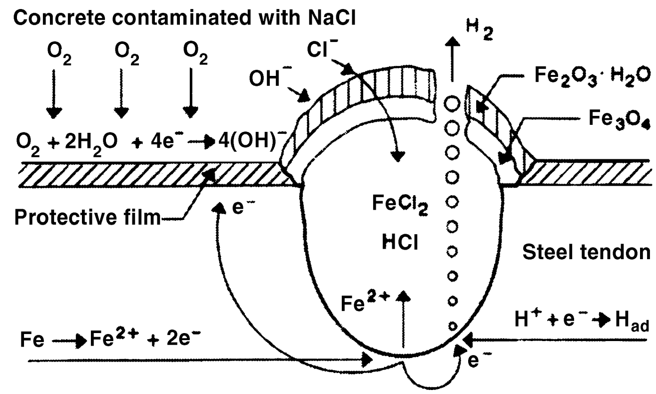

Fig. 3.7—Evolution of H inside a corrosion pit (Novokshchenov 1994).

usual grout pore-water pH of 12.6 to 14.6, hydrogen evolution values can range from –730 to –840 mV_{SHE}. Isecke and Miletz suggest that the actual potential required to support the significant evolution of hydrogen is lower than this (200 to 300 mV_{SHE} more negative than the equilibrium potential) (1993). The significance is that reinforced concrete cathodic protection is typically no more negative than –400 mV_{SHE}. Isecke and Miletz also indicate that the hydrogen recombination reaction in which gas is formed competes with uptake of dissolved hydrogen, further inhibiting the reaction.

Several researchers have investigated HE in association with cathodic protection (polarization) and brittle fracture of prestressing steels. Parkins et al. reports that enhanced cracking occurred at potentials less negative than –900 mV_{SCE}, but that enhanced cracking was seen at all potential levels if the environment was acidic (1982).

Hartt, Kumria, and Kessler indicates that in tests using notched and smooth wire, and with different values of pH, Cl^-, and precharging time, HE was observed in specimens polarized more positive than –900 mV_{SCE}. Some of the notched specimens, however, showed susceptibility to HE even at potentials less negative than –900 mV_{SCE} (1993).

Funahashi, Wagner, and Young found that hydrogen was generated on steel embedded in mortar at potentials more negative than –970 mV_{SCE} (1994). They also found that hydrogen could be generated near –750 mV_{SCE} when the pH was near 9.0.

Smeltzer, Hodgson, and Russell tested cathodic protection on beams 1.5 m in length using graphite electrodes along the tendons to determine the polarized voltage at the tendon/concrete interface (1992). Cathodic-protection levels of –0.60 to –1.30V_{CSE} (copper-copper-sulfate electrode) were applied for up to 3 months to determine if HE was a problem. Neither loss of capacity nor evidence of brittle fracture was detected. The concrete beams containing the tendons were constructed in a wedge shape to induce varying levels of resistance between the anode and the tendon.

Hope and Ip performed tests to determine if stray currents from cathodic protection of the reinforcing of a bridge deck would cause HE of prestressing steel in a supporting girder (1987). The investigation considered both prestressed and post-tensioned configurations. A single girder 0.9 x 0.6 x 2.0 m containing a metal duct was constructed to allow post-tensioning of the tendon. Instant-off potentials from the test specimen were compared to a small-scale test in which varying amounts of potential were compared with measured hydrogen evolution. This experiment indicated that the generation of hydrogen was initiated at about –850 mV_{CSE}. Comparison of this value with the instant-off potentials in the test specimens indicated that cathodic protection of the deck should not adversely effect the prestressing steel. Cathodic protection was not able to protect the tendon within the metal duct. If cathodic protection is used to protect steel, a uniform distribution of current to the steel is difficult, and it is necessary to apply the anode to the top and sides of the beam.

Although the test results differed somewhat among the various researchers, the recommendations from these investigations were in general agreement. Parkins et al. concluded that HE risk could be kept low by using polarization values less negative than –500 mV_{SCE}, while use of potentials more negative than –900 mV_{SCE} can result in HE (1992). Hartt, Kumria, and Kessler suggested that excessive CP can cause HE of the prestressing steel (more negative than –900 mV_{SCE})(1993). The severity and extent of corrosion present on the steel can have an effect on the risk of HE when applying cathodic protection. In the potential range –500 to –900 mV_{SCE}, the presence of sharp pits or defects can increase the tendency for HE. Funahashi, Wagner, and Young suggested a lower limit on potential of –720 mV_{SCE} (1994). These potentials should be adjusted to remove the IR drop from the potential reading. In addition, because most cathodic-protection systems are current controlled, it was suggested that the equipment be provided with a current-off potential limitation to prevent negative polarization values.

3.7.4 *Galvanizing and grout*—Zinc is used in galvanizing prestressing strand and wire. In contact with fresh portland cement grout, it reacts with the alkali ingredients and generates hydrogen (Hope and Ip 1987)

$$Ca(OH)_2 + Zn \rightarrow Ca(Zn(OH)_4) + H_2$$

Although not conclusively confirmed experimentally, there is concern that the hydrogen evolved in this reaction may enter the steel lattice and embrittle the wire. There are few data, however, concerning the relationship between the volume of adsorbed hydrogen and the brittleness of the steel.

Yamaoka and Tanaka conducted tests on 0.20 in. (5 mm) diameter drawn prestressing wires (1993). The behavior of bare wires was compared with that of galvanized wires and galvanized wires that had been redrawn after galvanizing. Examination of the microstructure indicated that the galvanizing on the surface of the wire that had not been redrawn was crack free. The redrawn wire, however, had micro-cracks in the zinc layer. These cracks extended down to the steel surface. The specimens were dipped in a 35% aqueous solution of NH_4SCN for 65 h. Tensile and torsion tests were conducted along with measurement of absorbed hydrogen. The results indicated that the bare wire suffered a reduction in ductility, while the galvanized wire did not. The galvanized and redrawn wire suffered some loss in ductility, but not as much as the bare wire. The zinc layer absorbed the hydrogen and prevented it from penetrating into the steel. Wire that had been redrawn after galvanizing, however, had cracks that allowed the hydrogen localized access to the surface of the wire. Measurement of hydrogen absorption confirmed these findings. The investigators indicated that the microstructure of the zinc was very similar to titanium, a known hydrogen adsorber. The zinc had a large volume of interstitial space that adsorbed a large quantity of hydrogen. Based on the results of their work, Yamaoka and Tanaka concluded that the hydrogen formed during the contact of zinc with fresh grout should not cause HE (1993). This confirms findings by Klodt in which zinc-coated specimens had longer times-to-failure than the specimens that had the zinc removed, indicating that the zinc provides some level of protection by absorbing the hydrogen (1969). Other research has confirmed that hydrogen released from the reaction between the zinc and wet grout is effectively prevented from entering the steel by the zinc barrier (Cook 1981).

Klodt conducted HE testing on hot-dipped galvanized stress-relieved wire in cement paste to determine if the cathodic reaction at the steel surface produced sufficient hydrogen to embrittle the steel (1969). The tests indicated no problems with brittle failure. These specimens prevented failure for test times up to 10 h. In contrast, specimens in which a narrow strip of galvanizing had been removed from the surface gave times to failure similar to those that had not been galvanized (1 to 2 h). This indicates that the layer of zinc prevents (or delays) the adsorption of the hydrogen by the steel.

3.8—Corrosion effect on fatigue performance

3.8.1 *Fretting fatigue*—Fretting fatigue is a corrosion-related phenomenon that can affect prestressing strand/wire used in post-tensioned applications. Fretting fatigue is an extension of fretting corrosion that occurs at the contact area between two materials. For damage to be considered fretting corrosion (as opposed to wear), the following conditions should be satisfied (Fontana 1986):

- The interface should be under load;
- Repeated small relative motion must occur between the two surfaces; and
- The load and relative motion on the interface should produce relative slip and deformation on the surface.

The relative motion necessary to produce fretting corrosion can be as little as 4×10^{-8} in. (10^{-7} mm). The relative motion of the surfaces in the presence of oxygen causes wear and corrosion at the interfaces.

The process that causes fretting corrosion can also cause fatigue cracking in prestressing strand used in post-tensioned girders (Ryals, Breen, and Kreeger 1992). Inside post-tensioning ducts, strands are in close contact with one another. This contact, along with cyclic loading, can lead to premature fatigue failures due to fretting. In curved tendons, strands that are in contact with the duct material can experience fretting. Fatigue cracks can be initiated prematurely from the combination of surface damage (from the fretting corrosion) and the very high local contact stresses.

3.8.2 *Corrosion effect on fatigue performance*—A metal that progressively cracks under cyclic stress is said to fail by fatigue (Uhlig and Revie 1985). The number of cycles required to cause fracture is known as the fatigue life. The fatigue life can be reduced when the specimen is subjected to a selected environment during cycling. This phenomenon is known as corrosion fatigue. While corrosion fatigue of steel is well documented and studied, little study has been devoted to the corrosion fatigue of prestressing strand or wire.

Martin and Sanchez-Galvez investigated the corrosion fatigue of high-strength, cold-drawn, 0.25 in. (7 mm) diameter stress-relieved wire (1988). Notched wires were precracked in air before fatigue testing. The environment selected was seawater with a pH of 8.2 to 8.0. The tests were conducted at three frequencies: 10; 1; and 0.1 Hz. One set of specimens was coupled with zinc to polarize the steel, while another set of specimens was tested at the free corrosion potential. The prenotched specimens coupled with zinc experienced a higher crack growth rate than the uncoupled ones. The smooth specimens coupled with zinc, however, experienced slower crack growth than the uncoupled ones. The investigators concluded that cathodic protection of prestressing steels by coupling with zinc is effective in retarding crack initiation but could be dangerous if flaws exist in the steel.

In an investigation of the fatigue strength of high-strength rope, the fatigue strength was reduced by 1/2 when tested in seawater compared with tests in air (Takeuchi and Waterhouse 1987). High-strength wire rope was tested in fatigue with and without fretting. Application of cathodic protection or –850 mV_{SCE} increased the fatigue strength to nearly the same strength of the rope tested in air.

3.9—Testing for SCC and HE

In general, testing in aqueous environments involves monitoring time-to-failure of stressed, high-strength prestressing wires of various compositions exposed to solutions of various pH and temperature. Investigations into the susceptibility of materials to SCC and HE should also incorporate some type of fracture-surface analysis and determination of metallurgical characteristics.

A report published in 1980 by the Federation Internationale de la Precontrainte (FIP) recommends a test for SCC of prestressing steels (FIP 1980). The report includes a short discussion of the HE and SCC mechanisms and the difference between HE and SCC in terms of dissolution versus the development of cathodic hydrogen. FIP also had many testing laboratories to evaluate various methods of SCC testing. The FIP recommended test method involves the use of ammonium thiocyanate (NH_4SCN) solution to determine the susceptibility of prestressing wire to SCC. The FIP report indicates that the NH_4SCN test is really a test of HE susceptibility and is not representative of the media to which prestressing steels are exposed. Other researchers have agreed that the failure induced by this test method is actually HE rather than SCC (Elices et al. 1980). In addition, the test is very sensitive to the surface condition of the steel and testing of smooth samples leads to scattering in the time to fracture.

Several reported investigations have assessed the SCC/HE character of prestressing tendon materials (FIP 1980). Almost all of these, however, have used aqueous electrolytes instead of concrete. Consequently, little information from these studies is useful in the design and maintenance of prestressed concrete structures. Rehm (1982) indicates that short-duration tests, such as the FIP test, cannot be used to compare the HE and SCC susceptibility of different steels, such as cold-rolled, quenched and tempered, and hot-rolled bars. Rehm also suggests that more appropriate methods of testing should be developed to test for HE and SCC.

Yamaoka and Tanaka claim that there is no relationship between the susceptibility to genuine SCC and the static mechanical properties, such as torsion value, elongation, and reduction of area, and that SCC cannot be avoided even if such values are high (1993). Plating and coating cannot enhance the resistance of wire to SCC. The addition of alloying elements to wire makes the wire vulnerable to pitting and consequently to SCC.

Often, tests incorporate poisons, such as sulfide and arsenate, which decreases the rate of formation of molecular H_2 in the following reaction

$$H^+ + H_{ads} + e^- \Rightarrow H_2$$

thereby increases the quantity of adsorbed hydrogen atoms on the steel surface. The entrance of hydrogen atoms into the metal lattice is enhanced as the concentration of adsorbed hydrogen atoms increases.

CHAPTER 4—PROTECTION AGAINST CORROSION IN NEW CONSTRUCTION

4.1—Introduction

Corrosion-protection measures for prestressed concrete structures are essentially the same as for conventional reinforced concrete structures. For unbonded tendons, however, additional measures are required because protection is more dependent on the sheath and the protective coating. Much of the information provided in ACI 222R is applicable to prestressed concrete structures and is referenced appropriately here to avoid duplication. This chapter addresses issues unique to corrosion protection of new prestressed concrete structures. Persons unfamiliar with corrosion of metals in concrete are referred to ACI 222R.

Corrosion of steel reinforcement or prestressing steel in concrete is complex and influenced by many factors. For this reason, the best corrosion protection provides several measures to guard against different influences and against breakdowns or limitations in any one protective measure. This is often referred to as providing multilevel protection. Corrosion-protection measures in prestressed concrete structures can take many forms and should be considered at all stages of design. Corrosion protection for prestressed concrete falls into three categories:

1. Measures that maximize the protection provided by the concrete;

2. Measures that prevent chlorides from entering the concrete; and

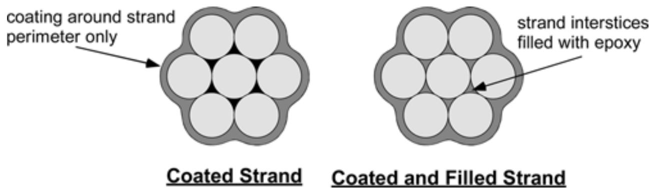

Fig. 4.1—Epoxy-coated strand types.

3. Measures that directly prevent corrosion of the prestressing system.

The first two categories are the same as those for conventional reinforced concrete structures.

For those prestressing elements encased in concrete, concrete quality is the single most important factor in ensuring structural longevity, affecting all aspects of durability, including corrosion of reinforcement. The effectiveness of concrete as corrosion protection for steel reinforcement is dependent on two equally important factors: concrete material properties, and design and construction practices. Many excellent sources of information are available on the effect of concrete quality for corrosion protection (ACI 222R, ACI 201.2R, Perenchio, Fraczek, and Pfeifer1989). These sources address requirements for material properties, design, and construction. ACI 301 and ACI 318 provide additional guidance in the form of standard specifications. The second category of corrosion-protection measures involves techniques for preventing chlorides from entering the concrete. These include treatments such as waterproof membranes, overlays, and polymer impregnation. ACI 222R provides an excellent treatment of this subject. The final category of corrosion protection measures involves aspects unique to prestressed concrete and is the focus of this chapter. Items to be considered are selection of prestressing tendon materials, protection systems for pretensioned and post-tensioned concrete, and cathodic protection.

4.2—Prestressing tendon materials selection

The selection of the type of material for prestressing tendons is dictated by both structural and durability requirements. This chapter describes some of the choices for prestressing tendon materials when corrosion is a concern.

The selection of prestressing wire, seven-wire strand, or bar for a particular application is dependent primarily on structural requirements, construction considerations, and economics, rather than on durability.

The permissible materials and manufacturing processes for prestressing steels used in structures in North America are governed by standard specifications and building code requirements, such as ACI 301 and ACI 318. In some countries, quenched and tempered steels have been used for prestressing. This method of manufacturing, without special alloying and manufacturing processes, can leave the steel more susceptible than cold-drawn prestressing steels to SCC (ACI-ASCE 423.3R; Schupack 1976). The material requirements of ACI 301 and ACI 318 preclude the use of quenched and tempered steel for prestressing; therefore, only cold-drawn prestressing wire or hot-rolled bar are used in North America.

Metallic and nonmetallic coatings have been investigated as protection methods for mild steel reinforcement (ACI 222R; McDonald, Pfeifer, and Blake 1996). The most common and widely used coatings are epoxy coating and zinc galvanizing. Good results have been obtained in laboratory testing of stainless steel-clad bars, copper-clad bars, and zinc alloy-clad bars (McDonald, Pfeifer, and Blake 1996). Some coatings have been considered for prestressing steels (Moore Klodt, and Hensen 1970; Perenchio, Fraczek, and Pfeifer 1989). Prestressing steel coatings should have certain important properties that often preclude the use of some coatings normally applied to mild steel reinforcement (Moore, Klodt, and Hensen 1970). For example, the coating should not have an adverse effect on the strength or ductility of the steel and should have sufficient flexibility and ductility to withstand stranding during manufacture and elongation during stressing without cracking or peeling. Coatings should not have a detrimental effect on bond between the steel and concrete and should be able to withstand handling, placement, and stressing without damage. Finally, the improvement in corrosion protection provided by the coating should not be at a prohibitive price. Presently, only epoxy coatings and zinc galvanizing have been successfully applied to prestressing steels. Each is discussed in this chapter.

The ongoing development of advanced composite materials or fiber-reinforced polymers has produced an additional choice in the selection of prestressing tendons. These materials are noncorroding, but long-term durability information is still limited. Some special design considerations are discussed in the final section of this chapter.

4.2.1 *Epoxy coatings for prestressing steel*—Epoxy coating is a widely used organic coating for corrosion protection that can isolate the steel from contact with oxygen, moisture, and aggressive chemicals. Epoxy-coated, seven-wire prestressing strand and threaded prestressing bars are available in the United States and require special considerations in design and construction.

4.2.1.1 *Epoxy-coated strand*—Epoxy-coated strand is available in two configurations: coated, and coated and filled (Fig. 4.1). In the coated configuration, a thick epoxy coating is provided around the exterior circumference of the seven-wire strand. In the coated and filled configuration, the interstices between the individual wires are also filled with epoxy, preventing migration of moisture and chlorides along the strand interstices. Both configurations are available either with a smooth surface or with grit particles embedded on the surface to improve bond-transfer characteristics. Epoxy-coated strand with smooth surface is intended for use in applications where bond is not critical, such as unbonded post-tensioning systems, external post-tensioning systems, and stay cables. When used in unbonded systems, such strand should still be encased in a duct, because the smooth epoxy coating is not a replacement for the sheathing used in monostrand post-tensioning systems. Epoxy-coated strand with a grit-impregnated coating is intended for use in bonded post-tensioning systems and in pretensioned applications.

Epoxy-coated strand is manufactured to meet the requirements of ASTM A 882 and ASTM A 416. The physical properties of the epoxy coating used for prestressing strand are significantly different from those used for 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 finished coating thickness for the strand is usually 0.03 in. (0.76 mm) (Moore 1994); although, the thickness can range from 0.025 to 0.045 in. (0.63 to 1.14 mm), according to ASTM A 882. This is considerably thicker than the coating thickness for mild steel reinforcement [0.007 to 0.012 in. (0.18 to 0.30 mm)] (ASTM A 775).

Thermo-setting, fusion-bonded epoxy coating is applied in a continuous process to the bare strand (Moore 1994). The manufacturing process starts with the strand that meets ASTM A 416. The strand is mechanically cleaned and then preheated to 572 F (300 C) before application of the coating. The strand is then continuously run 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 before they enter the fluidized bed. When the wires are restranded with the epoxy still in a plastic state, the interstitial space between the wires is completely filled with epoxy.

The use of epoxy-coated strand in post-tensioning applications and stay cables also requires special wedges that bite through the epoxy coating and into the underlying strand. Concerns have been raised because 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 (Hamilton 1995). Corrosion was also found under the epoxy coating between the marks created by the wedge teeth. 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. In unbonded post-tensioning tendons or stay cables, however, anchorage failure due to corrosion at the wedges could lead to failure of the tendon or cable. In these situations, additional protection should be provided for the strand at the wedge locations (Hamilton 1995).

Details of installation and stressing procedures are provided in a PCI report on the use of epoxy-coated strand (Prestressed Concrete Institute 1993).

4.2.1.2 *Epoxy-coated prestressing bars*—High-strength threaded bars commonly used for post-tensioning may be specified with epoxy coating. Epoxy-coated threaded bars are coated according to ASTM A 775, the same standard used for epoxy coating mild steel reinforcement. Anchorage hardware, including bearing plates, nuts, and couplers, is also epoxy coated. Nuts and couplers are proportioned to allow free movement over threads without damaging the epoxy coating.

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 electrostatically to the bars. The required final thickness of the epoxy coating ranges from 0.007 to 0.012 in. (0.18 to 0.30 mm) (ASTM A 775).

Epoxy-coated prestressing bars face issues in quality control similar to those with epoxy-coated mild steel reinforcement. The effectiveness of the corrosion protection provided by the epoxy coating depends 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, closely spaced supports, and nonmetallic slings are required to prevent damage during transportation. Care should also be taken to minimize coating damage during placement and stressing of bars. Damaged coating can be repaired on-site using a two-part liquid epoxy; it is more desirable, however, to adopt practices that prevent damage to the coating.

4.2.2 *Galvanized prestressing steel*—Zinc galvanizing has proven to be the most effective metallic coating for corrosion protection. Zinc provides protection by corroding sacrificially to the 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 between them and a sufficiently conductive electrolyte is present. The advantage of sacrificial protection is that it theoretically does not have to completely cover the protected part. Localized 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 corrosion resistance of zinc-coated mild steel reinforcing bars in concrete varies. Galvanizing was found to increase time-to-concrete-cracking in some cases, while reducing time-to-cracking in others (ACI 222R). There are additional concerns when using zinc-coated steel, especially high-strength steel, in contact with cement paste. In the highly 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 HE of the high-strength steel. Some research (cited in Chapter 3 of the report) indicates that HE due to the hydrogen gas is unlikely.

4.2.2.1 *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. The use of galvanizing in prestressing applications and stay cables, however, is very popular in Europe as well as in Japan.

The galvanizing process also affects 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 normally decreases. Questions have also been raised about the effects of zinc galvanizing on the bond of prestressing strand. Mixed results have been reported (Moore, Klodt, and Hensen 1970).

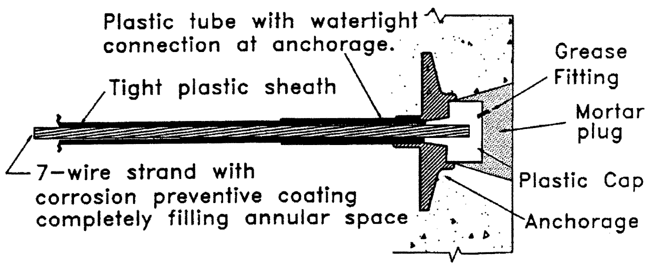

Fig. 4.2—PTI recommended monostrand system details (Schupack 1994c).

Galvanized seven-wire strand suitable for prestressing applications is commercially available in standard sizes from 3/8 to 0.6 in. (9.5 to 15.2 mm) diameter and in standard grades. The strand is stress-relieved (normal relaxation) and conforms to all the requirements of ASTM A 416 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 0.90 to 1.0 oz/ft^2 (275 to 305 g/m^2). The single wires before galvanizing meet the requirements of Grade 270 strand (ASTM A 416) when fabricated to the corresponding finished strand size (Florida Wire and Cable).

4.2.2.2 *Galvanized prestressing bars*—Threaded galvanized prestressing bars are commercially available in standard sizes and strengths of threaded bar for prestressing.

Although prestressing bars are not cold-drawn like prestressing wire (strand), the process of zinc galvanizing still raises concerns for HE. A specification has been developed for galvanizing prestressing bars to minimize the effects of galvanizing on the potential for HE and on mechanical properties (Dywidag Specifications). The highest potential for damage due to HE occurs during acid pickling of the bars before 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 (Dywidag Specifications, ASTM A 123). The maximum permissible weight of zinc coating is 0.82 oz/ft^2 (250 g/m^2) (Dywidag Specifications). Measures for maintaining the threadability of the bars after coating should also be considered (ASTM A 123).

Zinc galvanizing has some effect on the prestressing bar's mechanical properties. Galvanizing may lower the yield strength of the bar up to 5% and may alter the stress-strain relationship. Ultimate strength and ductility, however, are not adversely affected by the galvanizing process (Dywidag Specifications).

4.2.3 *Nonmetallic prestressing materials*—Fiber-reinforced polymer (FRP) products have had limited use for pretensioning and post-tensioning in bridges, buildings, marine structures, pavements, and rock anchors. Their use in concrete structures can have many benefits. Principal among these is that they do not corrode and therefore eliminate structural deterioration related to corrosion of steel reinforcement.

FRP composites normally consist of continuous fibers of glass, aramid, or carbon embedded in an epoxy or polyester matrix. The matrix transfers stresses among the fibers and allows them to work as a single element. The matrix also transfers stresses between the fibers and the concrete and protects the fibers.

FRP prestressing tendon material properties can be significantly different from prestressing steel; therefore, their use requires special design considerations. The reader is referred to ACI 440R for further information.

4.3—Corrosion protection for prestressing systems

4.3.1 *Pretensioned construction*—Basic corrosion protection for pretensioned concrete is provided by the quality and the cover of the concrete over the prestressing steel. Because pretensioned concrete is usually fabricated on a repeated basis in a plant, the quality control is much higher than is generally found in the field. Higher-strength concrete is generally used to permit detensioning of the prestressing strands at early concrete ages for production purposes. Modern high-performance concrete can possess the required early strength, low permeability, and excellent durability. Construction in a plant environment also provides controlled curing conditions, contributing to lower permeability.

In aggressive environments, corrosion protection is provided by using high-quality, low-permeability concrete with large covers, corrosion-inhibiting admixtures, or epoxy-coated strands, either single or in combinations. Adequate cover and protection should also be provided for nonprestressed, mild steel reinforcement in pretensioned elements to ensure that corrosion of this steel does not lead to cracking and spalling of the concrete and loss of protection for the prestressing tendons.

The trimmed ends of the prestressing strands should also be protected with suitable end cover that will be resistant to the particular corrosion environment and climatic conditions. Otherwise, moisture and chlorides may reach the ends of the strands, causing corrosion. In addition, cover tends to spall because of poor bond or freezing-and-thawing damage. Over time, moisture and chlorides can progress along the center wire of the strand, causing corrosion of the entire strand with consequent splitting of concrete.

4.3.2 *Unbonded monostrand post-tensioned construction*—Unbonded single-strand tendons (monostrand tendons) represent the largest volume of post-tensioning steel used in North America (Schupack 1994c). Monostrand tendons are used primarily in buildings, parking structures, and slabs-on-grade. As described in Section 1.1, the monostrand system uses a greased (or waxed) and sheathed seven-wire prestressing strand and single-strand anchorages. Details of a typical monostrand system are shown in Fig. 4.2 and 4.3. The Post-Tensioning Manual (1990) provides additional information on monostrand systems.

The manufacturing process for modern monostrand tendons begins with standard seven-wire prestressing strands. The strands are destranded, greased (or waxed), and restranded. The grease or wax normally contains corrosion-inhibiting admixtures. Excess grease is removed and a tightly fitting, high-density polyethylene sheathing is extruded over the strand. The sheathing protects the strand

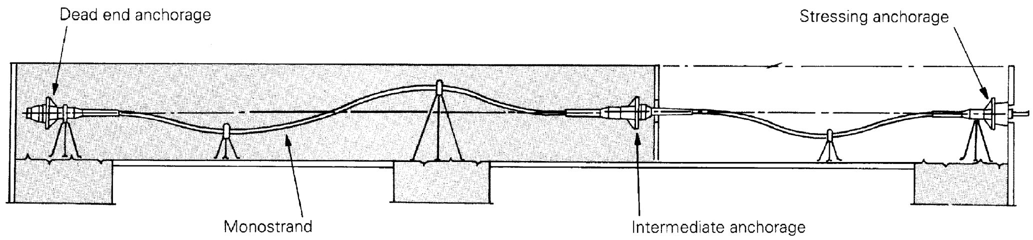

Fig. 4.3—Typical monostrand post-tensioning system components (VSL post-tensioning).

and acts as a barrier to moisture and chlorides. The grease provides the necessary lubrication to allow post-tensioning and also acts as secondary corrosion protection in the event that moisture and chlorides penetrate the sheathing. The general preference for tightly fitting extruded sheaths was largely in response to corrosion problems with loose push-through sheaths and heat-sealed sheaths (Schupack 1994c). Tightly fitting sheaths minimize voids in the grease and provide less opportunity for moisture movement into and along the length of the monostrand tendon.

Guide specifications for monostrand systems, published by the Post-Tensioning Institute (PTI) (1990, 1993), include requirements for sheathing materials and corrosion-preventive grease. Additional information is provided in ACI 423.3R.

Modern corrosion protection for monostrand tendons has multiple levels described in the following sections.

4.3.2.1 *Corrosion-preventive coating*—The grease (or wax) used to coat the strand should be continuous over the entire length of the tendon and should meet the requirements of the PTI Specifications (1993). In general, the grease or wax should contain corrosion-inhibiting admixtures and should not be water soluble, dry out, or become brittle.

4.3.2.2 *Tendon sheath*—The sheathing should be continuous over the entire length of the tendon and meet the requirements of the PTI Specifications (1993). Damage to the sheath during transportation, handling, and construction should be avoided, and all damage should be repaired before concrete placement. Damage to the sheath during post-tensioning cannot be detected or repaired because the concrete is in place. Common locations of sheath damage during stressing include crimped sheathing caused by tie wire, tendon supports, and mild steel reinforcement. Sharp curvatures in the tendon profile may also cause sheath damage during stressing due to high frictional forces.

4.3.2.3 *Transition between sheathing and anchorage*—Usually, the sheathing is cut so it falls short of the anchorage (see Fig. 4.2). In the past, only the concrete enclosing it generally protected the unsheathed portion.

Since 1985, the PTI specification has required that in an aggressive environment the sheathing be "connected to all stressing, intermediate, and fixed anchorage in a watertight fashion." There are considerable differences of opinion regarding what constitutes an aggressive environment. Particularly in a loosely fitted sheath, water can enter through the unsheathed portion of the tendon (Post-Tensioning Institute 1993).

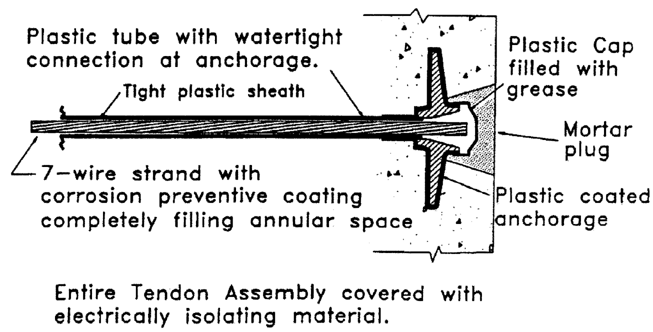

Fig. 4.4—Fully encapsulated, electrically isolated monostrand system details (Schupack 1994c).

It is preferable that the strand be completely enclosed and that any sleeves joining the sheath to the anchorage be filled with grease.

4.3.2.4 *Anchorage protection*—The anchorage and end stub of the strand should be carefully protected. The anchorage can provide a pathway for moisture and chlorides to reach the strand and cause corrosion. Corrosion of the anchorage itself may damage the concrete near the anchorage and subsequently lead to increased penetration of moisture and chlorides. According to recent recommendations, minimum anchorage protection should consist of a plastic, grease-filled cap over the strand stub and a mortar or concrete plug filling the anchorage recess, as shown in Fig. 4.2. The mortar or concrete plug should have low permeability and good bond with the surrounding concrete for effective corrosion protection. The effectiveness of the anchorage protection provided by the mortar or concrete plug can be determined using a vacuum tester (Schupack 1991).

4.3.2.5 *Encapsulation and electrical isolation*—Encapsulated and electrically isolated monostrand systems provide the highest level of corrosion protection of any prestressing system. Encapsulation and electrical isolation are terms that are sometimes used interchangeably. It is important to note, however, that encapsulated systems are not necessarily electrically isolated. Normally, encapsulation refers to encapsulation of the strand only, provided by the sheathing, transition sleeve, and greased-filled plastic cap over the strand stub. While the strand is completely encapsulated, the anchorage may still be exposed. Complete encapsulation and electrical isolation includes plastic coating of the anchorage, as shown in Fig. 4.4. This approach provides positive corrosion protection for the anchorage and protects the entire system from corrosion induced by stray currents or coupling with uncoated mild steel reinforcement. The electrically

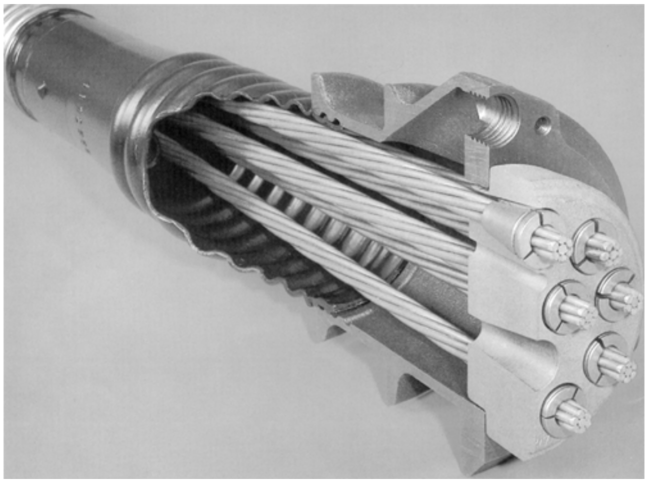

Fig. 4.5—Multistrand tendon anchorage details, cut away view (Dywidag International).

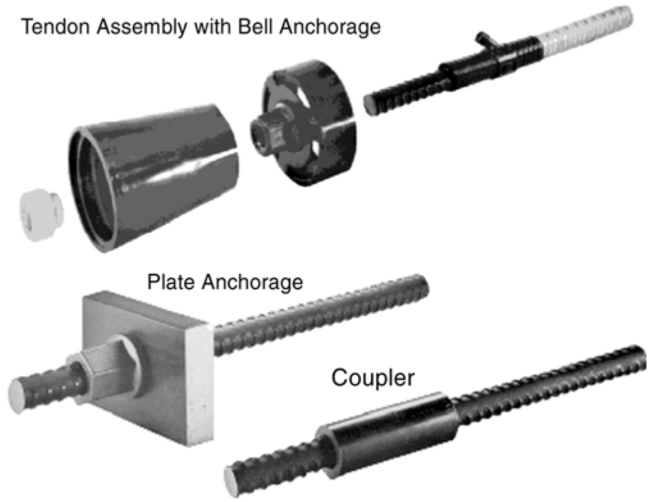

Fig. 4.6—Threadbar post-tensioning system anchorage details (Dywidag International).

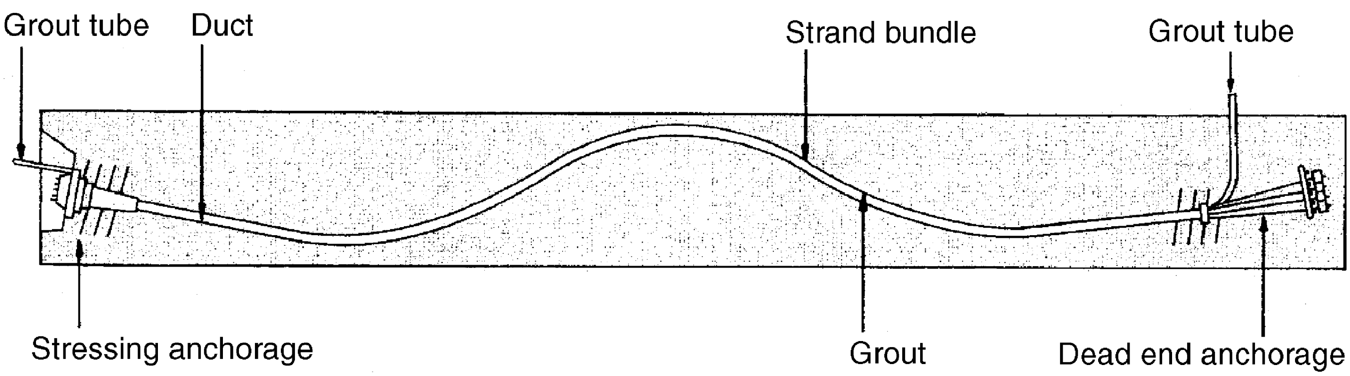

Fig. 4.7—Typical multistrand post-tensioning system components (VSL post-tensioning).

isolated monostrand system was first developed in 1980 (Schupack and Suarez 1980).

Encapsulated monostrand systems can provide complete encapsulation and electrical isolation, including coating the anchorage with high-density polyethylene (Dywidag Monostrand). Other systems provide encapsulation for the strand, while the anchorage remains uncoated (VSL Post-tensioning).

4.3.3 *Bonded post-tensioned construction*—The most common forms of bonded post-tensioned construction are multiple-strand tendons and bonded prestressing bars. Typical hardware for each is shown in Fig. 4.5 and 4.6. Typical components of a multistrand system are shown in Fig. 4.7. Additional information on bonded post-tensioning systems is given in the PTI Post-Tensioning Manual (1990) and other references (Dywidag Multistrand, Dywidag Threadbar, VSL Post-tensioning).

Corrosion protection for the bonded post-tensioning system consists of multiple levels. Corrosion protection specific to bonded post-tensioning includes duct, grout, and anchorage protection. Coatings for the prestressing steel also provide an additional layer of corrosion protection. Epoxy-coated and galvanized prestressing steels are both options in bonded post-tensioned systems, as described in Sections 4.2.2 and 4.2.3. The remaining levels of corrosion protection are described below.

4.3.3.1 *Ducts for post-tensioning*—Ducts have several functions in post-tensioned concrete. They provide a void to allow placement and stressing of tendons after concrete has been cast and transfer stresses between the grouted tendon and the concrete. Duct also works as a barrier to moisture and chlorides. For a duct to work effectively as a barrier, it should be impervious to moisture and be corrosion resistant. Duct splices and connections to anchorage hardware should also be watertight.

Galvanized steel ducts—The most widely used duct material is corrugated galvanized steel. The steel is sufficiently strong to prevent crushing or other damage during concrete placement and can withstand the frictional forces associated with post-tensioning. While galvanizing provides some resistance to duct corrosion, research studies (Perenchio, Fraczek, and Pfeifer 1989; West 1999) and field performance (see Section 7.5.2.3) have shown that this protection is limited, and that severe corrosion damage, including corrosion through the duct, can occur under exposure to marine environments or deicing salts.

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. Observed leakage of grout bleedwater from such ducts has confirmed the potential moisture pathway in these ducts.

Galvanized steel ducts are spliced in many different ways. One 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..." (Perenchio, Fraczek, and Pfeifer 1989).

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. One laboratory study showed excellent performance of epoxy-coated ducts in comparison to galvanized steel ducts (Perenchio, Fraczek, and Pfeifer 1989). Performance was evaluated in terms of grout chloride levels and extent of duct and strand corrosion damage.

Epoxy-coated ducts are not widely used and can have some shortcomings. As with epoxy-coated 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 coiling for shipping and the deformations associated with fitting the duct to the desired profile (Perenchio, Fraczek, and Pfeifer 1989).

Plastic duct—Plastic ducts can provide the highest level of corrosion protection for post-tensioning tendons because they are noncorroding and impermeable 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 (Ganz 1992; Perenchio, Fraczek, and Pfeifer 1989) and reduced fretting fatigue (Ganz 1992; Wollman, Yates, and Breen 1988) for plastic ducts compared with steel ducts. Commercially available plastic ducts for post-tensioning are normally provided with fitted watertight couplers for duct splices and connection to anchorage hardware.

Plastic ducts made from polypropylene are available in a two-strand system for slabs and in multistrand systems for tendon configurations of up to 55 0.5 in. (12.7 mm) diameter strands or up to 37 0.6 in. (15.2 mm) diameter strands (VSL Post-Tensioning).

4.3.3.2 *Temporary corrosion protection*—To minimize the risk of corrosion while the tendons are unprotected, the time between stressing and grouting of internal tendons should be as short as possible (see Section 7.5.1). Many specifications limit the length of time between stressing and grouting. The PTI Guide Specification for Grouting (Post-Tensioning Institute 1997) gives maximum time limits for grouting ranging between 7 and 40 days, dependent on the ambient humidity. If these limits are exceeded, temporary corrosion protection is required.

A range of temporary protection measures is available. The most common 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 also been used. 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 (Kittleman et al. 1993). Immediately before grouting, the ducts should be thoroughly flushed with water to remove all traces of the temporary corrosion-protection materials that may inhibit the bond between the steel and grout. 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 (PTI 1997).

A comprehensive study of materials for temporary corrosion protection and lubrication of post-tensioning tendons found that water-soluble oils could not be completely flushed from the strands, and adversely affected bond between the strands and grout (Kittleman et al. 1993). When compared with untreated strands, bond was reduced by 90% if the ducts were not flushed and by 75% if they were thoroughly flushed with water. The effect of sodium silicate on the 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 the bond. These findings illustrate the potential negative effect of many agents used for temporary corrosion protection. Water-soluble oils should not be used for temporary corrosion protection if the tendons are to be bonded.

4.3.3.3 *Cement grout for post-tensioning*—Cement grout bonds post-tensioning steel to the surrounding concrete and provides corrosion protection for the steel. Cement grout is a barrier to moisture and chloride penetration and produces an alkaline environment for the tendon.

An optimum grout for post-tensioning combines appropriate fresh grout properties with 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 corrosion (see Section 7.5.2.1). The presence of voids or discontinuities may also permit movement of moisture and chlorides along the length of the tendon. Important fresh grout properties are listed below.

Fluidity—Fluidity is a measure of how well the grout flows or pumps. Insufficient fluidity can lead to difficulties in placement, blockages, and incomplete grouting. Excessive fluidity can lead to void formation near crests in draped tendon profiles and to incomplete grouting. Grout fluidity also influences the ability of the grout to fill the space between strands in a multistrand tendon (Schupack 1974).

Bleed resistance—Resistance to bleeding is very important in grouts for post-tensioning. Unlike concrete where bleedwater can evaporate, bleedwater in grouted ducts tends to migrate to high points in the duct, forming bleed lenses and voids. Eventually, the bleedwater will be reabsorbed into the grout, leaving a void. Bleed lenses and voids are a particular problem in ducts with significant changes in elevation.

Volume change—Reduction in volume or shrinkage in the plastic state can lead to voids and should be minimized. In some cases, it may be desirable for the plastic grout to be slightly expansive to offset shrinkage and possibly fill voids resulting from entrapped air or bleedwater.

Setting time—Rapid-setting grouts have insufficient fluidity, hindering placement and leading to incomplete grouting.

The properties of fresh grout can be controlled through the water-cement ratio (*w/c*), the use of chemical and mineral admixtures, and cement type. Without the use of admixtures, fluidity is primarily a function of *w/c*. In most cases, it is desirable to lower the water content to lower permeability and reduce bleedwater. In this situation, sufficient fluidity can be provided through the use of a high-range water-reducing admixture. Partial cement replacement with fly ash increases fluidity for the same water-cementitious material ratio (*w/cm*). The addition of silica fume or its use as a partial cement replacement tends to decrease fluidity due to its small particle size. Bleeding can be reduced by reducing the *w/c* and by using fly ash or silica fume. Antibleed admixtures can also be used, particularly in situations where the tendon profile has large variations in elevation and bleedwater accumulation can be severe. Antibleed admixtures (sometimes referred to as thixotropic admixtures) give the grout gel-like properties to minimize bleeding while permitting the grout to become fluid when agitated (by mixing or pumping). Chemical admixtures can provide expansive properties. Expanding or nonshrink admixtures are generally categorized as gas-liberating, metal-oxidizing, gypsum-forming, or expansive-cement based (Hamilton 1995). Set-retarding admixtures are normally used to control setting time; although, some control is also available with selection of cement type.

The corrosion protection provided by the grout is primarily related to its permeability. Low permeability will slow the ingress of moisture and chlorides. The permeability of grout, like concrete, can be lowered by reducing the *w/c* and by the use of mineral admixtures, such as fly ash and silica fume. Reduced *w/c* may require the use of high-range water-reducing admixtures to provide sufficient fluidity. Corrosion-inhibiting admixtures can 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 fresh grout properties and corrosion protection. Several researchers have studied the effects of various admixtures and grout proportions on fresh grout properties and corrosion protection (Hamilton 1995; Schokker et al. 1997; Schupack 1974; Thompson, Lankard, and Sprinkel 1992; Ghorbanpoor and Madathanapalli 1993). Information on mixture proportioning and guide specifications for grouts for post-tensioning are provided by the PTI *Guide Specification for Grouting of Post-Tensioned Structures* (1997) and by the Concrete Society report "Durable Bonded Post-Tensioned Bridges" (1996).

The corrosion protection provided by the grout is also heavily dependent on construction practices (Schupack 1994b). Many corrosion problems have resulted from poor construction practices and inexperienced contractors (see Section 7.5.2.1). An optimized grout design is of no use if it is not placed properly and the ducts are not completely filled with grout. Attention should be given to batching and grouting/injection equipment, tendon trajectory, bleeding caused by strands, locations of vents along the duct, and grouting procedures. Guidance for construction practices is provided in *Guide Specification for Grouting of Post-Tensioned Structures* (Post-Tensioning Institute 1997) and *Durable Bonded Post-Tensioned Bridges* (The Concrete Society 1996).

4.3.3.4 *Anchorage protection*—As with monostrand post-tensioning systems, the anchorages and end stubs of the strands should be carefully protected. Although anchorage corrosion can lead to failure of the anchorage, the bond between the tendon and concrete will prevent a complete loss of prestress. Corrosion of the anchorage hardware, however, can lead to cracking and spalling of the concrete near the anchorage and to continued corrosion. Corrosion of the anchorage and strand stubs can also allow moisture to enter the duct, causing subsequent tendon corrosion. To protect strand ends, multistrand anchorage systems can be fitted with a sealed end cap, which is then 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 ends or edges 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 before filling the anchorage pocket with a nonshrink mortar. Additional information on long-term behavior and durability of different types of end anchorage protection methods, as well as grouted tendons, is available (Schupack 1992, 1994a; Schupack and O'Neill 1997).

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 often have severe exposure to moisture and chlorides that often lead to severe anchorage corrosion. The location of post-tensioning anchorages is often dictated by the method of construction. When the anchorage cannot be located away from a possible source of aggressive agents, the anchorage should be detailed with multiple levels of corrosion protection. A Concrete Society (U.K.) report discusses two approaches for anchorage protection (The Concrete Society 1996). The first approach is to provide an anchorage that is not encased in mortar or concrete after stressing. Exposed anchorage hardware is protected by end caps and a 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 Fig. 4.8. The details

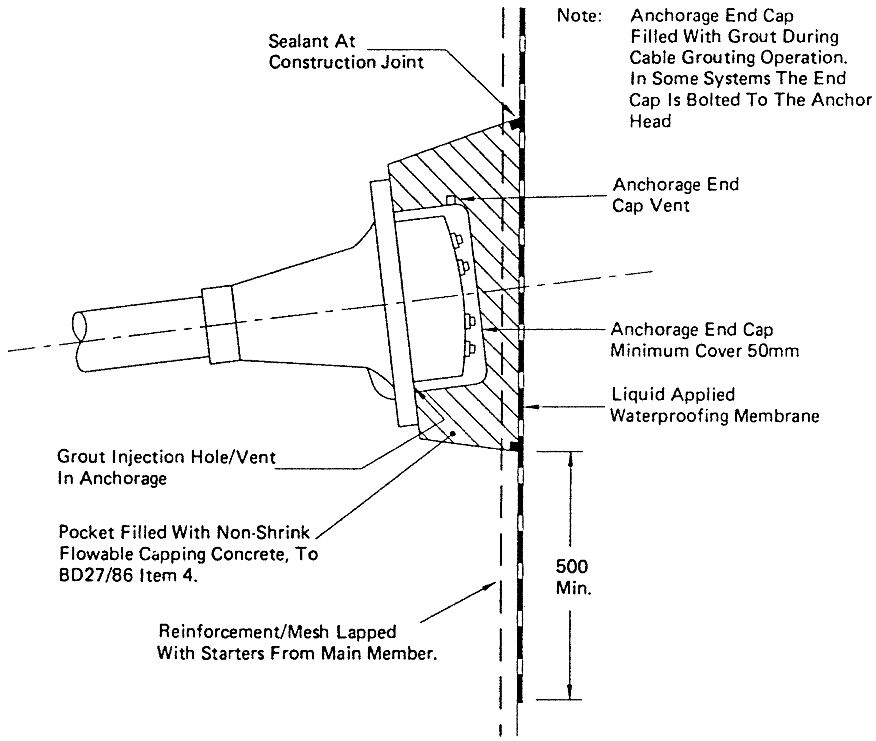

Fig. 4.8—Multilayer corrosion production for buried post-tensioning anchorages (dimensions in mm; 1 in. = 25.4 mm) (The Concrete Society 1996).

of the member end can also be designed to minimize contact with moisture and chlorides draining through expansion joints, as shown in Fig. 4.9. 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 area.

4.3.3.5 *Encapsulated and electrically isolated systems*—As with monostrand post-tensioning systems, full encapsulation and electrical isolation of the multistrand post-tensioning system will provide the highest level of corrosion protection. As mentioned previously in Section 4.3.2.5, encapsulation normally refers to the post-tensioning tendon only and not necessarily the anchorage. Encapsulation of the tendon is provided by an impermeable duct, full grouting, and a sealed end cap over the strand or bar stubs. Complete encapsulation and electrical isolation includes coating of the anchorage or use of nonmetallic 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 before 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 resealing of end caps, grouting or patching, and crack sealing. The use of a watertight and airtight duct system also permits vacuum grouting, in which a negative pressure is applied 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 those with large differences in elevation. For vertical tendons, vacuum grouting can facilitate removal of bleed-water and laitence and topping off if regrouting is required when antibleed grout is not used.

An encapsulated multistrand post-tensioning system is available in three configurations, each providing an increased level of corrosion protection (VSL Composite, VSL Post-Tensioning). 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 noninvasive methods to electrically monitor the tendon throughout the life of the structure.

4.3.4 *Unbonded multistrand post-tensioned construction*

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

Multilevel corrosion protection in unbonded multistrand and post-tensioned bar systems is similar to that of bonded systems with the exception of the cement grout. Corrosion-protection options include plastic or noncorroding ducts

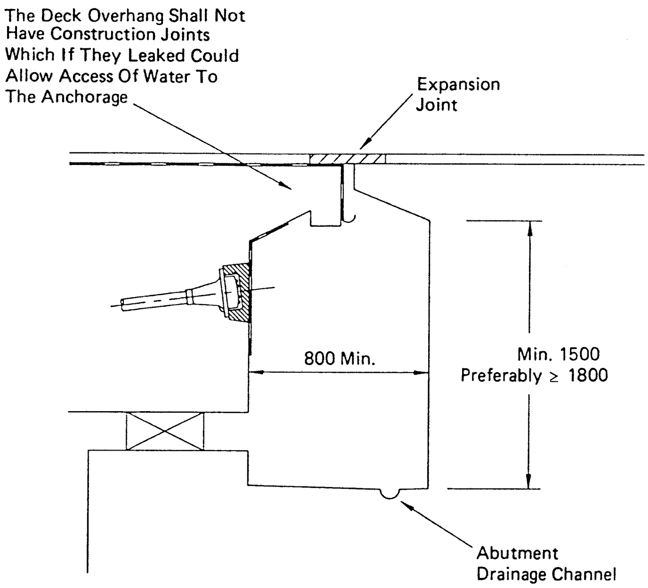

Fig. 4.9—Member end details for anchorage corrosion protection (dimensions in mm; 1 in. = 25.4 mm) (The Concrete Society 1996).

and epoxy-coated or galvanized strands or bars. Some multi-strand systems can be fitted with greased and sheathed strands normally used in monostrand systems (see Section 4.3.2). Anchorage protection for unbonded systems is the same as for bonded post-tensioning (see Section 4.3.3.4).

4.3.4.2 *External post-tensioning*—External post-tensioning has various applications, including precast segmental bridge construction, strengthening of structures, and stay cables. External post-tensioning tendons are not embedded in the concrete but 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 steel or plastic sheath (Fig. 4.10), which provides an exterior protective barrier around the tendon. As with post-tensioning ducts, a plastic or other noncorroding sheath provides the highest level of corrosion protection. Strands or bars used for external tendons can 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 (see Section 4.3.3.3). Greases or waxes should be similar to those used for greased and sheathed monostrand tendons (see Section 4.3.2.1), or for a higher level of protection, the nuclear industry requirements should be used (ASME).

4.4—Cathodic protection

Several factors render cathodic protection of prestressing steel in concrete more complex than for reinforcing steel. HE is a particular concern when applying cathodic protection to prestressing steel in concrete. If a cathodic-protection system

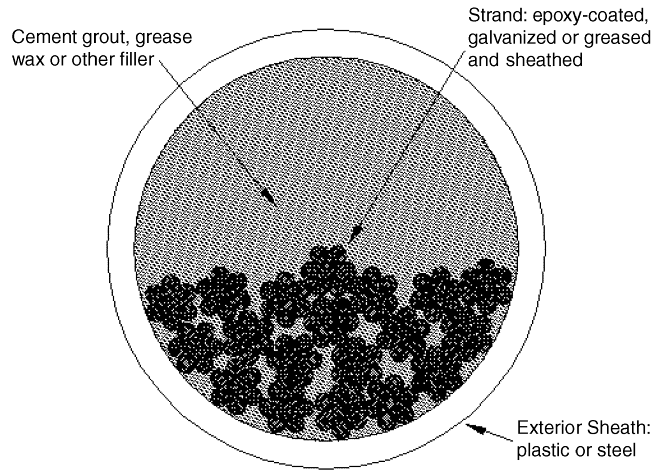

Fig. 4.10—External post-tensioning tendon corrosion protection cross section shown at tendon high point.

is installed and operated so that the magnitude of polarization is excessive, then atomic hydrogen can be generated at the surface of the steel and embrittlement may occur. Most authors agree that significant HE is unlikely at steel potentials less negative than –900 mV_{SCE} (Federal Highway Administration 1998). This is more of a concern with impressed current cathodic protection than with galvanic-anode systems.

Another concern regarding the use of cathodic protection in post-tensioned concrete structures is the shielding effect. If a metallic duct is used in conjunction with a bonded post-tensioning tendon, it will typically be electrically continuous with the steel tendon. If cathodic protection is applied to such a structure, however, the metallic duct receives most of the current and shields the tendon within. If a plastic duct is used, it also acts as an electrical insulator and shields the current. While this shielding effect reduces the possibility of excessive polarization and potential HE, it also significantly reduces the effectiveness of cathodic protection of tendons in steel or plastic ducts. Cathodic protection can be used to provide corrosion control to the metal ducts which, in turn, stops contamination to the grout and tendon inside. Also, cathodic protection could be effective in controlling corrosion of anchors and conventional reinforcing steel, which are also present in post-tensioned systems.

To date, the majority of cathodic protection for prestressing steel in new construction has been for prestressed concrete cylinder pipe. Both impressed-current and galvanic-anode systems have been installed in the soil for corrosion control of the prestressing wires in the pipe. Cathodic protection, however, has been applied to over 1,075,000 ft^2 (100,000 m^2) of new post-tensioned segmental concrete viaducts in Italy. In the viaducts, a mixed metal-oxide, titanium-anode mesh (impressed current method) was used in conjunction with a concrete overlay on the new bridge decks (Bazzoni et al. 1996).

CHAPTER 5—FIELD EVALUATION

5.1—Introduction

Field evaluation of prestressing steel corrosion in concrete structures is much like the evaluation of embedded mild reinforcing steel in concrete. Information on the field evaluation of embedded metals in concrete is covered in ACI 222R and is not repeated in this document. Although there are similarities between the evaluation of prestressed and mild reinforcing systems, there are also important differences and additions. Moreover, there are additional distinctions between pretensioned-system and post-tensioned-system evaluations. Because of the inherent differences between pretensioned and post-tensioned systems, field evaluation of these systems is discussed separately.

5.2—Evaluation goals

Before beginning an evaluation of a structure or structural system, it is important to establish the goals or purpose of the evaluation. During a corrosion evaluation of a prestressed system, it is typical to attempt to determine the presence, extent, and rate of any active corrosion. Low corrosion rates can permit remediation to be delayed to a future date; high corrosion rates may indicate the need for immediate repair or rehabilitation. To assess the effect of corrosion on a structure's capacity or serviceability, it is important to determine the extent and magnitude of any corrosion-induced damage to the structure, such as wire section loss, fracture, or concrete damage (spalling or delaminations). Additionally, to effectively mitigate future deterioration of the structure, it is important to determine the cause of any corrosion or the presence of conditions that will promote future corrosion.

5.3—Pretensioned systems

5.3.1 *Introduction*—Pretensioned systems consist of high-strength wire or strand that is completely surrounded by the parent concrete of the member. Therefore, like mild reinforcing in a typical reinforced concrete member, the prestressed steel is in intimate contact (bonded) with the concrete. Therefore, the field evaluation of a prestressed system is very similar to a typical corrosion evaluation of mild reinforcing for a reinforced concrete member. ACI 222R provides extensive guidance on the field evaluation of embedded steel in concrete. There are, however, important distinctions between pretensioned systems and mild reinforcing systems.

5.3.2 *Electrical evaluation methods*—Pretensioned systems are not electrically isolated from the parent concrete of the structure by a duct, as can be the case with post-tensioned systems. Electrical evaluation methods originally developed for the evaluation of mild reinforcement can be used.

Half-cell corrosion potentials—Active corrosion can be detected by the use of half-cell corrosion potentials (ASTM C 876). Elsener and Bohni (1990) and ACI Committee 228 provide background theory and field experience on half-cell potential mapping for mild reinforcement. The basic concept behind the use of half-cell potentials is illustrated in Fig. 5.1. During active corrosion of a prestressing wire or strand, an electric potential field is set up around the corrosion site. The magnitude of the equipotential line intersecting the concrete surface can be measured relative to a reference half-cell electrode by electrically connecting a reference electrode to the strand through a high-impedance voltmeter. The reference

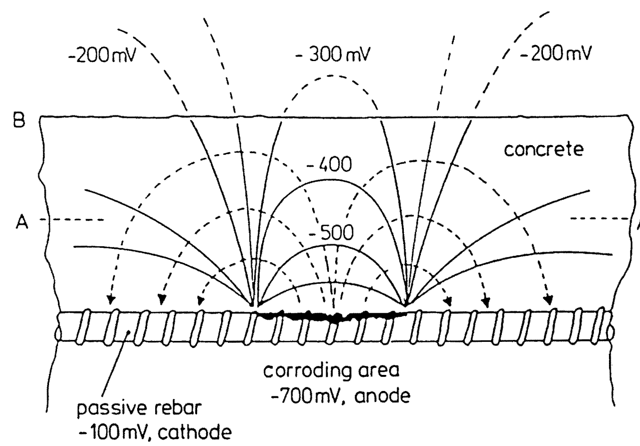

Fig. 5.1—Schematic of electric fields and current flow for corroding steel in concrete (Elsener and Bohni 1990).

electrode called for in ASTM C 876 is a CSE; although, other types of electrodes may be used, such as a saturated calomel electrode (SCE).

According to ASTM C 876, a measured half-cell potential more negative than –350 mV indicates a probability greater than 90% of active corrosion at the test location. A potential less negative than –200 mV indicates a probability of active corrosion less than 10%; while potentials between –200 mV and –350 mV indicate that corrosion activity is uncertain at the test location. These values are for use with a CSE. The use of other reference electrodes require the measured values to be offset based on the electrode type. Moreover, these threshold values provided in the standard should be used with caution, especially when evaluating prestressed systems. Examination of Fig. 5.1 indicates that because the electrode cannot be placed directly on the corroding steel, a mixed potential is measured on the nearest concrete surface. This measured potential will be affected by a significant difference in concrete cover; note that the measured potential directly over the corroding area would be approximately –460 mV if the concrete surface was at level A. If the cover depth was doubled to level B, however, the potential reading would be approximately –340 mV. Because the cover depth to prestressing steel is often greater than typical cover to mild reinforcement, the potential thresholds described in ASTM C 876 may not be applicable.

Polarization resistance—Corrosion rates can be measured using polarization resistance. Clear (1989), Flis et al. (1993), and Fontana (1986) provide theoretical background and field experience on polarization resistance for measuring and interpreting corrosion rates for embedded steel in concrete structures. Basically, the use of polarization resistance to measure the rate of active corrosion is based on the fact that the amount of applied current required to alter the natural half-cell potential of corroding steel a few millivolts is proportional to the corrosion rate of the steel. The relationship between the half-cell potential (voltage) and the applied current is linear up to approximately 15 mV (Fontana 1986). The application of current above this threshold during field testing can lead to significant inaccuracies.

Table 5.1 (Feliu, Gonzalez, and Andrade 1996)

Corrosion current density, μA/cm^2	Corrosion level
< 0.1	Negligible
0.1 to 0.5	Low
0.5 to 1.0	Moderate
> 1.0	High

For a typical test setup, three electrodes are needed. The first electrode (working electrode) is the steel at which the corrosion rate is to be measured. The second electrode (counter electrode) is a metallic object used to apply the polarizing current to the steel. The third electrode (standard half-cell) measures the change in potential in response to the applied current from the counter electrode to the steel. During testing, current is applied to the steel to change the free half-cell potential over any reasonable interval up to a total change of 12 to 15 mV. Applied current is plotted against voltage (relative to the initial potential). The slope of this line is used in the Stern-Geary equation (Fontana 1986) to determine the corrosion current in milliamps. The area of steel polarized during the test is determined and the corrosion current density is calculated. The rate of corrosion can be calculated in terms of actual metal loss per unit time using Faraday's Law (Fontana 1986). The polarization resistance and the use of the Stern-Geary equation are only applicable to microcell corrosion and cannot give any indication of macrocell nor galvanic corrosion currents.

Research studies and field experience (Feliu et al. 1996; Flis et al. 1993) have shown that measured current densities give a reasonable estimation of the corrosion rate at the test site. Typical values are shown in Table 5.1. Although corrosion rates and subsequent metal loss per unit time can be calculated by linear polarization, this tool is probably better suited for qualitative rather than quantitative evaluation. There is substantial controversy about the actual area of bar or strand polarized during testing. This is especially the case where there is a high concentration of interconnected strands and mild reinforcement within the test region. It is typically assumed that current flows perpendicular from the counter electrode to the working electrode (strand or bar). There are several studies, however, that suggest this assumption is not true (Feliu et al. 1990a; Feliu et al. 1990b; Feliu et al. 1996). In an attempt to confine the polarized area of the strand or bar, some testing devices use a guard electrode around the counter electrode that is maintained at the same potential as the reference electrode. The actual ability of the guard electrode to completely confine the polarizing current applied during the testing has been questioned by some researchers (Flis et al. 1993). In addition, the top portion of the steel can be polarized more than the remainder of the steel; in some cases, the polarization can occur exclusively on the top portion of a bar (Flis et al. 1993). Other variables that can affect test results include type of corrosion (carbonation- or chloride-induced), concrete temperature, resistivity, moisture content, and oxygen availability. Despite the potential for measured corrosion-rate inaccuracies, linear polarization can be used to identify the presence and extent of active corrosion and, when used in conjunction with half-cell potential measurements, can better define the location and extent of corroding areas.

Unfortunately, research to date has been exclusively on corrosion-rate measurement of solid reinforcing bars. There is no published work on the use of polarization resistance to measure corrosion rates of prestressed systems. Although the basic theory is the same, it is not known how multiple wires in close proximity, such as a seven-wire strand, affect the test results or the area of polarized steel used when calculating the current density. Published data on studies using mild reinforcement should be applied with care to prestressed systems.

Polarization resistance or any other corrosion-rate measurement technique can give an indication of the corrosion rate only at the time of testing, not in the past and future. Thus, service-life estimations taken from single tests or multiple tests over brief time periods should be used with caution.

5.3.3 *Material analysis*

Chloride-ion content—Significant concentrations of chloride ions have been shown to promote the corrosion of embedded steel in concrete in the presence of adequate moisture and oxygen (Houseman 1967; Nakamura and Kanako 1975). High chloride concentrations at the level of embedded prestressing, while not a direct indication of active corrosion, is an indication that corrosion will occur under the proper environmental conditions. An additional benefit of chloride content testing of the concrete is that the cause of any existing corrosion may be better understood, which allows for more effective mitigation of future corrosion.

Chloride contents can be determined from concrete powder samples taken from rotary-hammer drilling or from extracted drilled cores. Traditionally, acid-soluble chloride contents have been used to evaluate the chloride level of existing concrete but water-soluble methods have also been used (ASTM C 1218). The presumed advantage of water-soluble methods such as ASTM C 1218 is that chemically bound chloride that will not promote steel corrosion will not be included in the measurement. Because of the fine grinding of samples required by the method, some chemically bound chloride is actually released during the hot-water extraction. How much chloride is released depends upon the particular chemistry of the cements, the amount of chloride present, and other factors. Powdering the concrete sample (including aggregates) before extraction can lead to incorrect conclusions regarding the corrosivity of a concrete where only chemically bound chloride aggregates have been used.

To overcome this problem, a second water-soluble chloride test based on chloride extraction using a soxhlet extractor has been developed (ACI 222.1). The ASTM water-soluble chloride test, however, is still the water-soluble test that can be used in most instances. The soxhlet-based test should be used only when bound water-soluble chlorides in the aggregate are encountered or suspected. Test methods for acid-soluble chlorides consist of titration methods (ASTM C 1152) and a rapid method using a specific ion probe (AASHTO TP55).

Tests are typically conducted on samples at incrementally increasing depths to obtain a chloride-ion profile for the concrete. This profile is useful because the chloride concentration at the level of the embedded steel can be determined. The profile also gives an indication of the rate of chloride contamination. Additionally, a uniform concentration of chloride over a significant depth may indicate that chlorides were introduced into the concrete during batching.

Carbonation—Typically, the high pH of concrete protects embedded steel by creating a thin passive oxide surface layer on the steel. Long-term exposure to carbon dioxide can significantly reduce the pH of the concrete because free hydroxide ions react with the carbon dioxide to form calcium carbonate. If the pH of the concrete falls below a critical value, the existing passivating layer will be destroyed, and corrosion of the steel will commence under the proper environmental conditions. The pH required for passivation is dependent on the electrochemical potential of the environment [see the Pourbaix diagram for the Fe-H_2O system (Fontana 1986)]. It is generally accepted, however, that passivation will occur above a pH of 9 to 9.5 under typical conditions.

The pH and carbonation of the concrete can be evaluated by applying phenolphalien dye to freshly fractured concrete. Single dyes that denote a single pH change or dye mixtures, which can indicate a pH profile may be used. If a concrete core is split longitudinally and tested, the depth of carbonation can be determined.

Chloride permeability—Although it does not indicate corrosion, chloride permeability can offer information about the susceptibility of the concrete to chloride penetration. Currently, testing is conducted only in the laboratory on drilled cores or sawn samples from the field, although in-place methods are being developed. Traditionally, a 90-day ponding test (AASHTO T259) has been used to evaluate the chloride penetration of concrete. This test directly measures the penetration of chloride through analysis of concrete powder samples at the completion of the test. Due to its extreme length, this method has largely been replaced with a much more rapid test procedure (AASHTO T277; ASTM C 1202). This rapid method does not measure chloride penetration directly but rather correlates the total charge passed through a water-saturated concrete sample during a 6-h period with chloride permeability. Pfeifer, McDonald, and Krauss reviewed the correlation of this rapid electrical test with the 90-day ponding test and found poor correlation in many instances, especially when pozzolans, such as silica fume and blast-furnace slag, were used in the concrete (1994). When the rapid method is to be used extensively on a particular project, it may be necessary to conduct 90-day ponding tests to correlate the two methods.

5.4—Post-tensioned systems

5.4.1 *Introduction*—Post-tensioned systems consist of high-strength wire or strand contained within a metal, plastic, or paper duct. The duct is encased by the parent concrete of the member. The duct is typically filled with cement grout, grease, or petroleum wax to provide corrosion protection for the steel. Grout also bonds the strands or wires to the rest of the member. Tendons that are not grouted or that are external to the member are considered to be unbonded. Evaluation of these systems is very different from the evaluation of pretensioned systems or of mild reinforcement in reinforced concrete structures.

5.4.2 *Evaluation of anchorages*—Anchorages are a critical component of unbonded post-tensioned systems; loss of the anchor would result in an effective loss of the entire tendon. Evaluation of anchorages is essential during a corrosion evaluation of an unbonded post-tensioned system; although, it should not be ignored during evaluation of a bonded system.

Visual inspection—The anchorage of many systems is cast into the concrete with a blockout, providing access to the anchor for future stressing. Corrosion protection is often provided by dry-packed mortar or by formed and placed or pumped grout, mortar, or concrete, or by a tight-fitting, grease-filled plastic cap. Bituminous materials may be applied for protection as well. The first step during a visual inspection is to look for efflorescence or staining from corrosion products around the blockout. Significant efflorescence can indicate moisture migration through the anchorage, which can promote corrosion if levels of moisture, chloride, and oxygen are elevated around the anchor. Invasive probing and selective removal of material within the blockout can provide further information about possible corrosion of the anchor.

Material analysis—As with pretensioned systems, the cementitious material surrounding the anchor in the blockout can be evaluated for chloride content, carbonation, and chloride permeability. These methods are covered briefly elsewhere in this document and more fully in ACI 222R.

5.4.3 *Evaluation of unbonded tendons*—Unbonded systems are commonly used in slabs of buildings and parking structures and typically consist of wire or strand within plastic or paper ducts. The ducts are filled with a protective material, typically grease. A significant exception, external tendons within box girders, typically consists of multiple strands encased in a polyethylene duct filled with grout. This section focuses on typical internal unbonded systems. Additional information on the evaluation of unbonded single-strand tendons is also given in ACI 423.4R.

Remote electrical methods, such as half-cell potentials and polarization resistance, that are common for prestressed systems typically cannot be used on an unbonded system. A notable exception to this is near anchorages where the sheathing that serves as the duct is sometimes cut back to facilitate construction. At these locations, the steel is in contact with the concrete and electrical methods can be employed. These methods should be used with care in this instance. For example, Poston, Carrasquillo, and Breen (1987) found that uncharacteristically large negative half-cell potentials can be recorded near anchorages without corresponding active corrosion. This was attributed to the large metallic mass of the nearby anchor.

Invasive probing—The most common evaluation method is visual inspection through invasive probing of the member. The location and cover to unbonded tendons can be determined nondestructively using a cover meter or impulse radar. Concrete can be removed to gain access to the tendons by careful chipping or coring of the concrete. Demolition should be conducted so that the tendons and ducts are not damaged. Before examining the tendon itself, the duct should be care-

Fig. 5.2—Existing damage to tendon duct.

Fig. 5.3—In-place examination of an unbonded tendon.

fully examined for damage or other signs of distress. An example of existing damage to a duct is shown in Fig. 5.2. In this case, the polyethylene sheath around a monostrand tendon was damaged by a vibrator during concrete placement. Damage to the duct can be important because it gives moisture, oxygen, and chlorides easier access to the tendon. After examining the ducts, the tendons themselves should be examined by removing a portion of the duct. This should be done carefully so important information is not missed during the examination. For example, in Fig. 5.3, careful breaching of the polyethylene sheathing demonstrated the presence of significant moisture in the sheath (at the tip of the knife blade) and an absence of significant grease.

It would be useful to determine if exposed strand had failed or lost significant tension or if individual wires had broken within the strand. A simple method to evaluate a strand for failure or loss of tension is by attempting to displace the exposed strand with a pry bar or other device. If prying can significantly displace the strand, it is likely to have little or no tension. This is certainly the case if there is any residual displacement after prying. This test is highly subjective; the length of strand exposed, the presence or absence of grease and corrosion products, and the amount of annular space between the sheathing will affect the amount of displacement obtained from a given effort. A common method employed to evaluate individual wire breaks is known as the screwdriver test. To conduct this test, the operator attempts to insert a flat-bladed screwdriver between the wires in the strand. Penetration of the screwdriver into the strand between wires indicates one or more wire breaks. This test is also highly subjective; the size of the screwdriver and the presence or absence of grease and corrosion products will affect the screwdriver's ability to penetrate the strand between wires. Additionally, wire breaks may not be located with this test, especially if they have occurred some distance away from the test location.

A few advantages of invasive probing are that there is minimal damage to the structure and minor disruption to the use of the structure. The primary drawback, however, is that only small areas of the tendon are evaluated while no information is obtained about the remaining portions of the tendon. This is significant because the strength of an unbonded tendon is controlled by its weakest location. Therefore, limited probe openings can miss critical deterioration to the tendons. Field experience has indicated a poor correlation between visual observations of a strand at inspection recesses and anchorages and the condition of the same strand upon removal (Schupack 1998).

Tendon removal and examination—An entire tendon can be visually examined if it is removed from the structure. This overcomes the limitation of examining small areas through probe openings. An obvious drawback to this type of examination is that the removed or new tendon should be reinstalled in the structure. Tendon removal and replacement is typically more destructive and disruptive than examination through probe openings. Selective removal and examination of a small number of tendons, however, can provide insight on the condition of the post-tensioning in a larger area. Selective removal and examination has allowed existing structures with post-tensioning corrosion to be evaluated and kept in service with repairs and rehabilitation that were a fraction of the cost of complete tendon replacement.

A typical procedure involves the detensioning of the tendon and its subsequent removal from the structure. Grease or other protective coatings that would otherwise impair visual examination can be removed with a solvent and a wire brush. The strand can then be cut into convenient lengths for examination. If possible, the strand should not be cut until after the grease is removed so that damaged areas may remain within the same strand segment; this allows for later tensile testing of representative damaged areas.

Strand segments should be examined visually under low magnification (2× to 5×) for corrosion damage, especially pitting. This may be more readily accomplished under laboratory conditions. Pitting depths and densities should be recorded. Precise measurement of pit depths is difficult and usually involves the destruction of the strand in a laboratory (ASTM G 46). A successful approximate procedure uses a caliper with a wire probe attached to one of the jaws (Fig. 5.4). To evaluate the effect of different levels of corrosion damage, tensile testing can be conducted on representative samples.

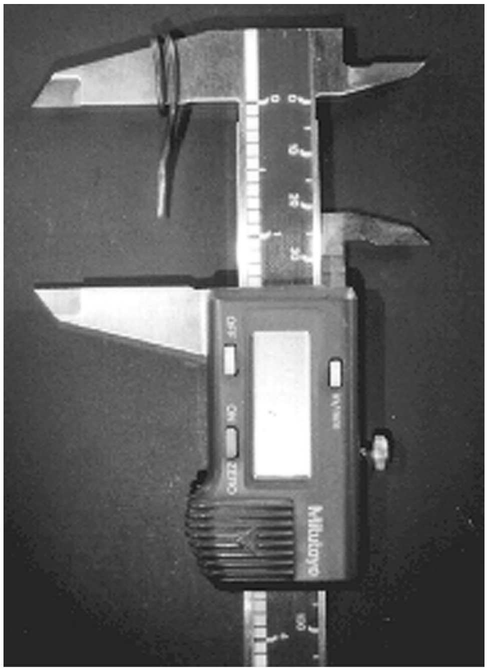

Fig. 5.4—Probe used to estimate pitting depths.

Any failed wires should be examined carefully. Brittle fractures (Fig. 5.5) may indicate HE or SCC. If this is suspected, metallurgical analysis by a metallurgist experienced with the corrosion of prestressed wire may be appropriate.

Nondestructive evaluation of tendon damage—Continuous acoustic monitoring of post-tensioned structures has been used successfully in Canada to monitor fractures in unbonded tendons in post-tensioned structures since 1994 (Halsall, Welch, and Trepanier 1996). This technique has had limited use in the United States. Acoustic monitoring relies on the fact that a wire or strand failure results in the release of strain energy that in turn sets up transient stress (acoustic) waves in the structure. Accelerometers are mounted throughout the structure to detect the acoustic energy from wire or strand breaks due to ongoing corrosion.

For acoustic monitoring to be effective, it should differentiate signals generated by wire fractures from other ambient noise in the structure. Previous work in Canada has shown the method to be successful with unbonded systems. Although acoustic monitoring shows great potential as a management tool for continuing corrosion damage, it cannot provide information on existing damage to the post-tensioning of a structure.

5.4.4 *Evaluation of bonded tendons*—Corrosion protection for bonded tendons consists of the duct and cementitious grout. The most important component of this system is the grout. If the tendon ducts are not completely filled with grout, if the grout is absent, or if the grout is of poor quality, the tendon is more susceptible to corrosion (Poston and Wouters 1998). This has been documented in numerous research studies and field investigations (Clark 1992; Freyermuth 1991; Matt 1996; Novokshchenov 1989a,b, 1991; Schupack 1994a,b). An important part of an evaluation of a bonded tendon is the determination of the extent and quality

Fig. 5.5—Brittle wire fracture.

of the grouting. Although the presence and level of any existing corrosion (wire fractures and section loss) should be evaluated, if possible, numerous studies have shown that significant corrosion of grouted tendons is unlikely if the ducts are completely filled with a high-quality grout.

Location of the tendon duct—The first step in an evaluation is to locate the tendon ducts. Although as-built plans can be useful, it is not uncommon, especially in cast-in-place structures, for tendons to be misplaced or to move during concrete placement. If the tendon is reasonably close to the concrete surface and a metal duct was used, a standard cover meter can be used to locate the duct; otherwise, impulse radar can be used. Although metallic ducts are by far the most common, plastic ducts are sometimes used. Plastic ducts cannot be located with a cover meter and may be difficult to locate with impulse radar, although impulse radar may be used to locate the tendon itself. Impact-echo can be used to locate a plastic duct embedded within concrete (Carino 1992; Malhotra and Carino 1991).

Nondestructive evaluation of grouting—One nondestructive method for locating voids in grouted bonded tendon ducts is impact-echo, which uses transient stress waves (Jaeger, Sansalone, and Poston 1996, 1997; Malhotra and Carino 1991; Woodward, Hill, and Cullington 1996). Both laboratory research and field testing have located voids in bonded metal tendon ducts (Jaeger, Sansalone, and Poston 1996, 1997; Woodward, Hill, and Cullington 1996). Impact-echo cannot locate air voids within grouted plastic ducts (Poston and Wouters 1998). An advantage of this method is that access to both sides of the member is not required. A disadvantage is that skilled operators are required to interpret the test signals.

Though impulse radar has been used with limited success to locate voids in tendon ducts, its best application may be in the location of the ducts or tendons for further testing or invasive inspection (Poston and Wouters 1998; Woodward, Hill, and Cullington 1996).

Nondestructive evaluation of tendon damage—It is useful to assess the existence and level of corrosion damage (wire section loss or fracture) to the tendons. Currently, radiography has been the most successful method (Darley 1996; Woodward, Hill, and Cullington 1996). Although radiographs can be interpreted by an engineer with an average familiarity with post-tensioning, highly skilled operators are required to conduct the testing. An additional drawback is that evacuation of a large area near the testing

Fig. 5.6—Field examination of grouted tendon duct with invasive probing.

site is required because of the high radiation output required. Also, access to both sides of the tendon is required. A new radiography system called the Scorpion, developed in France, has been used successfully (Darley 1996, Woodward, Hill, and Cullington 1996). The Scorpion is vehicle-mounted and contains a telescopic arm that positions the radiation source and detector on opposite sides of the beam being tested. The same safety procedures are required for the Scorpion as for conventional radiography.

Italian and Swiss engineers have developed an electrical reflectometer test known as Reflectometric Impulse Measurement (RIMT). A short-duration electrical pulse is applied at the anchorage, and the return signals are interpreted. Trials of the method in England were not successful, and the method is not recommended at this time (Woodward, Hill, and Cullington 1996).

Continuous acoustic monitoring has been successful with unbonded systems. Trials on bonded systems in England had encouraging results; although, more work needs to be conducted on evaluating fully grouted tendons (Paulson and Cullington 1998). Although acoustic monitoring shows great potential as a management tool for continuing corrosion damage, it cannot provide information on existing damage to the post-tensioning of a structure.

Invasive probing—Invasive probing can be used to evaluate the grout and any existing damage to the tendons. Probes can be made into the member through hammer drilling or coring. A rigid borescope or a flexible fiberscope will cause the least damage to the structure (Fig. 5.6). Invasive probing is best used in conjunction with other nondestructive testing to verify and supplement information from nondestructive testing.

Telltale signs of sheath or prestressing steel corrosion are similar to reinforcing steel corrosion. Cracks along the tendon trajectories can indicate corrosion or water freezing in a duct. Local invasive probing can identify substantial voids in grouted post-tensioned ducts but provides only a very small sampling of the corrosion condition. Autopsies of post-tensioned members confirm that the findings from localized invasive probing are not representative of the rest of the tendon (Schupack 1994a).

CHAPTER 6—REMEDIATION TECHNIQUES

6.1—Introduction

Corrosion-induced damage to prestressing systems can be successfully controlled or mitigated by remediation measures that are virtually identical to those for embedded steel in concrete, covered in ACI 222R. Additional information on remediation techniques for unbonded single-strand tendon systems is given in ACI 423.4R. Remedial techniques for post-tensioned systems are discussed in subsequent sections of this chapter.

6.2—General

The extent and type of remediation required for post-tensioned systems is dependent upon a number of factors, including the level of confidence required, the type of structure, the type and extent of damage, and the cause and nature of the corrosion. Many types of remedial options have been performed on post-tensioned structures; the extremes have ranged from ignoring the problem to complete replacement of the post-tensioning systems (Kesner and Poston 1996). Fortunately, there are options within these extremes.

6.3—Grouted post-tensioned systems

Grouted post-tensioned systems consist of wire or tendons within a grout-filled duct. The grout consists of portland cement and water with the possible addition of admixtures. Numerous studies have suggested that corrosion problems are associated, at least in part, with incomplete grouting of the tendon ducts (Freyermuth 1991; Matt 1996; Novokshchenov 1989a,b, 1991; Schupack 1994a,b; Woodward 1989). It is likely that significant voids can be present within grouted tendon ducts experiencing corrosion problems. Several of the available remedial options for grouted post-tensioned systems are listed below:

- Delay remediation;
- Regrout voids within the tendon ducts;
- Strengthen the member/structure; and
- Replace the member/structure.

Delay remediation—In terms of corrosion and corrosion-induced damage to grouted post-tensioned members, this option can be appropriate under limited circumstances. If the level of existing damage is considered acceptable and there is reasonable confidence that the current rate of corrosion is minimal or low, remedial action can be delayed until a future date, but not indefinitely. Inaction should be viewed with caution, because corrosion rates can be highly variable over the life of a structure (Fontana 1986). Frequent and careful inspection of structural elements is recommended if inaction is proposed for a structure experiencing corrosion, corrosion-induced damage, or both, to its post-tensioning. Additionally, future repairs will probably be significantly more difficult and costly if remedial action is significantly delayed.

Regrout voided areas within the tendon ducts—If significant grouting voids are found within a tendon duct, regrouting the duct is a common remedial action to mitigate future corrosion. Care should be taken during the regrouting to ensure effective corrosion protection. The grouting should completely encapsulate prestressing steel in high-quality grout, with a low *w/c*, low bleeding and separation characteristics, high flowability, and an adequate working time. PTI and The Concrete Society provide guidance for the specification of grouts for both new construction and remedial grouting (1996). If voids lie within existing ducts, high-range water-reducing admixtures should be considered to promote the flow of grout around partial obstructions. Corrosion-inhibiting admixtures should be considered if the member is in an aggressive environment or if existing corrosion is evident.

Remedial grouting procedures are critical to ensure complete encapsulation of the tendons. The ducts are typically flushed with water and compressed air before grouting to remove foreign matter that can later obstruct the flow of grout. Vents and injection ports should be located to allow grout to flow to all voided areas and to allow air and water within the ducts to escape. PTI, The Concrete Society (1996), and others (FIP 1989; Lapsley 1996) provide guidance on appropriate grouting practices and quality assurance.

Strengthen the member or structure—If structural analysis, load testing, or both, of a member indicates damage such that the member is structurally inadequate, strengthening the member is an alternative to replacement. Strengthening is also appropriate if corrosion-mitigation techniques are judged ineffective in arresting the existing corrosion or reducing it to acceptable levels. Several options are available for strengthening an existing post-tensioned member, including installation of external, or additional internal, post-tensioning; addition of bonded reinforcement; increasing the member size, including the use of additional post-tensioning, bonded reinforcement, or both; and installation of additional members. The most appropriate option or combination of options depend on the existing details and conditions.

Replace the member or structure—Replacement is the final option if the structure or member is so badly damaged that it cannot be readily or economically strengthened. Although this option can be necessary in rare cases, such as corrosion-induced collapse or failure, it is likely that strengthening of the structure or some other mitigation strategy is not only possible, but economical. Where aesthetic considerations preclude viable strengthening strategies, however, replacement may be the only option.

6.4—Ungrouted post-tensioned systems

Ungrouted post-tensioned systems consist of high-strength wire or strand that is contained within a metal, plastic, or paper, which is typically filled with grease to provide corrosion protection for the steel. Similar to grouted post-tensioned systems, poor corrosion of unbonded systems has been associated with incomplete coverage of the strands with grease or poor performance of the grease. Several of the available remedial options for ungrouted post-tensioned systems are listed below:

- Delay remediation;
- Dry-gas purge and regrease the tendon ducts;
- Inject urethane into the tendon ducts;
- Strengthen the member or structure; and
- Replace the tendon.

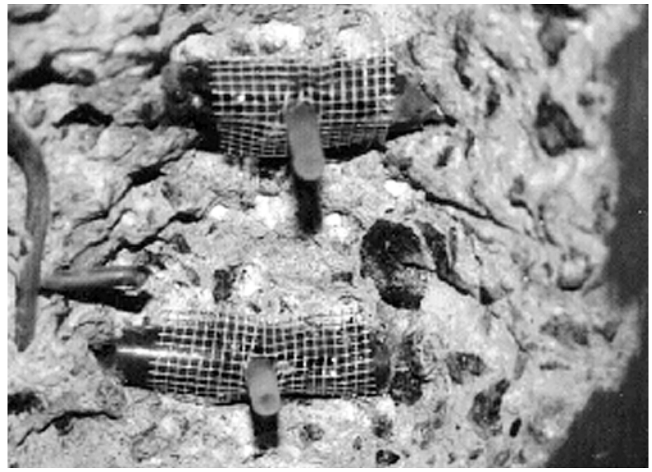

Fig. 6.1—Installation of injection ports for urethane grouting.

Delay remediation—For unbonded post-tensioned members experiencing corrosion and corrosion-induced damage, this option may be appropriate under limited circumstances, especially if accompanied by extensive monitoring of the post-tensioning by acoustic or other means (Section 6.3).

Dry-gas purge and regrease the tendon ducts—Gas-purging of post-tensioned strand sheathing was developed and tested in Canada in the early 1990s. Dry gas is injected into the sheathing system and is circulated until free water within the system is removed, and the relative humidity within the ducts is sufficiently lowered. Lowering the relative humidity to 50% is sufficient to stop corrosion in most cases (Uhlig and Revie 1985). The dry-gas purging is used once to remove water before repairs and continuously with a dry-gas manufacturing plant and automated control system. Single-use purging is sometimes followed by grease injection of the strand sheathing to provide additional corrosion resistance. This technique has been used primarily in Canada and has not been readily available in the United States because the system is patented. This remediation technique is suitable only where sufficient annular space exists between the duct, strand, and grease to permit the passage of gas, grease, or both, through the sheathing. Generally, this technique is not suitable for extruded-sheath tendons.

Inject urethane into the tendon ducts—A technique for injecting low-viscosity urethane grout material into existing strand ducts was developed in Canada and patented in the United States. First, the strands are located and injection ports are installed at appropriate locations (Fig. 6.1). The grout fills the duct and displaces water as it travels through the duct. When uncontaminated grout is captured at the end opposite the injection point, the grouting is stopped and the injection system sealed.

The urethane grout cures to a solid closed-cell foam (Fig. 6.2), which encapsulates the strand inside the sheathing. This system has been shown to mitigate future corrosion of strands by preventing moisture and oxygen from reaching the steel surface. Besides corrosion protection, an additional benefit of the urethane grouting is that the foam material is reported to act as an energy-absorbing material, which greatly

Fig. 6.2—Cured urethane grout completely encapsulating a monostrand tendon.

reduces the risk of strand eruption through concrete surfaces should failures occur. As a result, potential property damage and personal injury are minimized.

Like dry-gas purging, urethane grouting is patented by a Canadian firm. This has limited its use and availability in the United States; although, the system was implemented on a trial basis on a project in 1997. This remediation technique is suitable only where sufficient annular space exists between the duct, strand, and grease to permit the passage of the urethane grout through the sheathing. Generally, this system is not suitable for extruded strand tendons.

Strengthen the member or structure—Strengthening an unbonded post-tensioned member or structure is similar to strengthening a grouted post-tensioned system (Section 6.3). In contrast to bonded systems, however, failure of an unbonded tendon anywhere along its length renders the tendon completely ineffective.

Replace the tendon—Tendon replacement is the most drastic remedial action that can be taken with an unbonded post-tensioned system. Complete replacement of entire post-tensioning systems has been implemented, often unnecessarily (Kesner and Poston 1996). Tendon replacement may be necessary in the case of high levels of corrosion-induced damage. It is rarely necessary to completely replace an entire unbonded system; it is likely that strengthening the structure or some other mitigation strategy is not only possible, but economical.

CHAPTER 7—FIELD PERFORMANCE OF PRESTRESSED CONCRETE STRUCTURES

7.1—Introduction

The objective of this chapter is to describe typical corrosion problems in prestressed concrete structures, according to type of prestressing and time of occurrence. Where possible, specific case studies are provided for illustration. The field performance of prestressed concrete structures can provide a useful perspective on the corrosion of prestressing steels. Observed corrosion problems can be used to identify deficiencies in the processes of design, construction, and maintenance of the structure. In learning from the past, similar problems can be avoided in the future, and the service life of structures can be extended.

The overall performance of prestressed concrete structures worldwide has been very good and the number of serious cases of corrosion has been limited. In 1970, the FIP Commission on Durability surveyed 200,000 prestressed structures and reported that "an extremely low proportion of cases causing concern," and that "occurrences of corrosion where the consequences have been serious are rare" (FIP 1970). More recent reviews of research and experience have also concluded that post-tensioned structures are very durable (Freyermuth 1991).

It is not possible to obtain precise numbers for the incidence of corrosion in prestressed concrete structures, because many cases are not reported and some occurrences of corrosion have not yet been detected. Attempts to estimate the occurrence of corrosion have been based on surveys of reported problems. A survey of almost 57,000 prestressed structures had 0.4% incidents of damage and 0.02% incidents of collapse due to all causes, including corrosion (Szilard 1969a). Another survey of 12,000 prestressed bridges had visual evidence of corrosion in less than 0.007% of the surveyed bridges (Moore, Klodt, and Hensen 1970). Schupack reported corrosion in about 200 post-tensioning tendons, representing only 0.0007% of the estimated 30 million stress-relieved tendons in use in the western world up to 1977 (1978a). A condition survey of all bridge types in the United States found that 23.5% of all bridge types were structurally deficient due to all causes, including corrosion (Dunker and Rabbat 1990). The survey considered steel, timber, reinforced concrete, and prestressed concrete bridges at comparable ages and time spans. As a group, prestressed concrete bridges had the best performance with only 4% deficient. Because these deficiencies include all causes, the actual incidence of corrosion in prestressed bridges will be much less than 4%. A 1994 survey of post-tensioned segmental bridges found that 98% of segmental bridges in the United States and Canada had condition ratings of satisfactory or higher, with no reported corrosion problems (Miller 1995). These figures indicate the overall performance of prestressed concrete structures.

The opinion and experience of persons familiar with the subject of corrosion in prestressed concrete are frequently reported in publications. This often reveals differences in the perceived field performance of prestressed concrete. Some conclude that "… the instances of serious corrosion in prestressed concrete structures are rare" (Podolny 1992), while others counter with personal experience where the incidence of corrosion is as high as 10% in some types of prestressed structures (Kaminker 1993). This broad range of experience and opinion on the performance of prestressed structures can be explained by several factors (Kesner and Poston 1996). The publicity associated with several large, high-profile projects with corrosion problems can make the problem appear very prominent, particularly to those outside the engineering community. This is compounded in some geographic regions where there have been a larger number of corrosion problems. An example of this is corrosion of unbonded post-tensioned parking garages and office buildings in some areas of the northern United States and Canada. The legal cases and publicity in these areas have

Table 7.1—Causes and effects of corrosion of prestressing steels

Influencing factor	Potential problems
Environment:	
Use of deicing salts	Source of moisture and chlorides
Marine environment	Source of moisture and chlorides
Soils with high salt content	Source of chlorides
Chemical exposure (acids, materials with high sulfur content)	May lead to HE or hydrogen-induces stress corrosion
Water access into ducts	Sources of moisture
Materials selection:	
Most heat-treated prestressing steels	Prone to stress corrosion and HE
Low-quality concrete	Insufficient protection for steel
Low-quality post-tensioning grouts	Excessive bleed lens or air-void formation, insufficient or excessive fluidity, chlorides in grout
Nonpermanent void formers (ducts)	No corrosion protection
Corrosion-susceptible duct materials	Limited corrosion protection
Dissimilar metals used for anchorage components	Prone to galvanic corrosion
Design deficiencies:	
Low concrete cover	Insufficient protection for steel
Congested reinforcement	Poor concrete consolidation or honeycombing
Poor drainage	Saltwater ponds collect on structural elements
Joint locations and details	Saltwater drips onto supporting structural elements and anchorage not designed for severe exposure
Anchorage protection	Insufficient protection provided
Location of post-tensioning anchorages	Saltwater comes in contact with anchorage
Post-tensioning ducts	Discontinuous ducts or poor splice details lead to leakage of grout or grease and ingress of moisture and chlorides
Vents for post-tensioning ducts	Improper vents or lack of vents lead to incomplete grouting
Inadequate bleeding control	Grout voids in more vulnerable areas
Construction deficiencies:	
Design concrete cover not provided	Insufficient protection for steel
Damaged or torn sheathing	Prestressing steel protection is compromised
Blocked or damaged post-tensioning duct	Incomplete grouting—insufficient protection for prestressing steel
Poor grouting procedures or inexperienced contractors	Incomplete or nonexisting grouting—insufficient protection for prestressing steel
Sustained period between stressing and grouting/construction	Opportunity for corrosion while tendon is unprotected
Leaking cold joints	Corrosion of tendon
Maintenance deficiencies:	
Expansion joints	Saltwater drips onto supporting structural elements not designed for severe exposure
Blocked or damaged drains	Saltwater collects on structural elements or drips onto supporting structural elements not designed for severe exposure

contributed to the erroneous perception that all unbonded post-tensioned construction suffers from corrosion problems. Another important factor is the length of time between construction and the occurrence of corrosion. Many corrosion problems encountered recently result from poor design details and construction practices in the past. In most cases, new developments and improvements in prestressing systems and specifications will prevent a repetition of problems due to such previous deficiencies.

Quantifying the incidence of corrosion in prestressed concrete structures is further complicated by limitations in techniques for detecting corrosion, particularly in post-tensioned structures. Condition surveys of prestressed concrete structures are often limited to visual inspections for signs of cracking, spalling, and rust staining. This limited inspection can overlook corrosion activity, particularly for post-tensioned structures. Corrosion damage in post-tensioned elements has been found in situations where no outward indications of distress were apparent. The collapse of a precast segmental, post-tensioned bridge in Wales was attributed to corrosion of the internal prestressing tendons at mortar joints between segments (Woodward and Williams 1988). The bridge had been inspected 6 months before the collapse, and no signs of deterioration were apparent. The collapse of this bridge occurred due to highly localized corrosion of the prestressing tendons. Because the corrosion was localized, distress indicators such as spalling, rust staining, or increased deflections of the structure were not present. Examples such as this lead some to fear that the incidence of corrosion in prestressed structures based on limited or visual inspections can be underreported, leading to a false sense of security.

In comparison to reinforced concrete, prestressed concrete can have more aspects affecting corrosion and the subsequent durability. This is particularly true for post-tensioned concrete structures, due to the larger number of elements in the prestressing system and the additional steps in the construction process. These factors are reflected in the incidence of corrosion in prestressed structures in general and post-tensioned structures in particular.

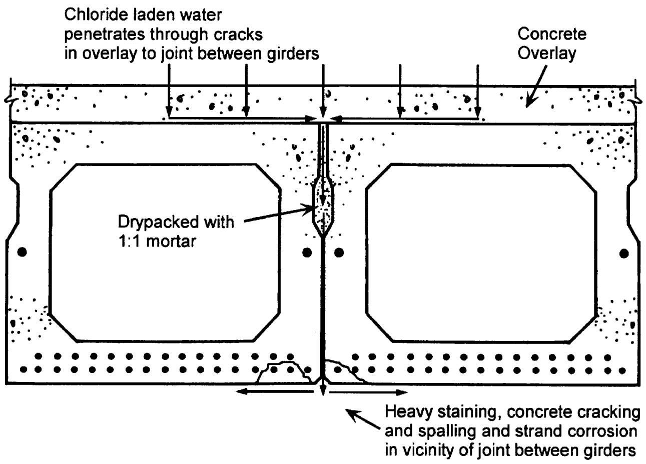

Fig. 7.1—Mechanism for moisture and chloride penetration through concrete overlay in precast, pretensioned box girders (adapted from Novokschenov 1989b).

Where corrosion problems have occurred in prestressed concrete structures, the source is normally traced to specific examples of poor design, construction, or maintenance. The common denominator for all corrosion in concrete structures is an aggressive environment. Some generalizations of factors influencing corrosion and the associated potential problems are listed in Table 7.1.

The field performance of prestressed concrete structures is not as well documented as that of nonprestressed or conventionally reinforced concrete structures. This may be attributed to a number of factors, including the larger proportion of reinforced concrete structures in service, and the shorter length of experience with prestressed concrete structures, limiting data on long-term behavior. In spite of these factors, a number of good sources of information exist on the field performance of prestressed concrete structures.

7.2—Corrosion of prestressing strand before construction

Corrosion of prestressing strand occurring before construction can lead to failures before, during, and after stressing. Corrosion before construction can result from improper storage and handling during shipping. Failures before stressing normally occur in cases where the prestressing strand is stored in tightly wound coils. This type of failure is generally attributed to SCC and is most common in quenched and tempered steel, which is more susceptible to stress corrosion. Quenched and tempered steel is not permitted by AASHTO or ACI (see Section 4.2) and is generally not available for use in North America. Reports of this type of failure come from Germany, Japan, and the former Soviet Union. Chemical contamination of the strand during storage, transport, and handling can lead to embrittlement or pitting corrosion of the strand. Common sources of contamination are splashing with fertilizers, water containing lime and gypsum, chlorides, animal wastes, or raw oils (Szilard 1969). Pitting corrosion can also occur as a result of exposure to moisture, saltwater or sea mist during storage, or transportation. Embrittlement and pitting corrosion can lead to failure before stressing. Corrosion occurring before stressing can also cause failure during and after stressing in both pretensioned and post-tensioned structures (Szilard 1969). Guidelines for assessing the degree of corrosion on prestressing strand before it is placed in the structure are provided by Sason (1992) and PCI (1985).

7.3—Pretensioned structures

Corrosion problems in pretensioned structures are not significantly different from those in reinforced concrete structures. The absence of the post-tensioning duct and anchorages in pretensioned concrete makes it more similar to reinforced concrete in terms of the protection provided for the prestressing steel. The main influencing factors for corrosion in pretensioned structures are the prestressing steel, concrete, and severity of the environment. The effect of the concrete and environment is the same for pretensioned- and reinforced concrete structures. Although prestressing steel is more susceptible to corrosion, and the consequences of corrosion can be more severe than for mild steel reinforcement, corrosion of prestressing tendons in pretensioned structures is rare for two reasons. First, pretensioned elements are always precast, generally resulting in improved overall quality control and good-quality concrete. Second, pretensioned elements normally fit the classic definition of full prestressing, that is, concrete tensile stresses are limited to prevent flexural cracking of the concrete. Where corrosion has been discovered in pretensioned structures, the cause is normally related to the structural form and details. Because pretensioned elements are precast, the structure can contain a large number of joints or discontinuities. Poor design or maintenance of these joints, or both, can direct moisture and chlorides onto the pretensioned elements of the structure in localized areas.

Novokshchenov performed an extensive condition survey of several pretensioned and post-tensioned bridges in both marine and deicing-salt environments (1989b). One bridge located in the Gulf of Mexico had approach spans with pretensioned girders and main spans with post-tensioned, segmental box-girders. The bridge was 16 years old at the time of inspection. Concrete cracking caused by corrosion of the prestressing steel was found on the ends of the girders adjacent to the expansion joint at the transition between the approach and main spans. Corrosion was attributed to chloride-laden moisture from the deck leaking through the expansion joint onto the ends of the girders, producing localized severe exposure. Novokshchenov (1989b) also examined a precast pretensioned box-girder viaduct that had been exposed to deicing salts throughout its service life, and the bridge was 29 years old at the time of inspection. Examination revealed that almost all longitudinal joints between box girders had leaks, ranging from very minor to extensive. The leaks appeared to have resulted from moisture and chlorides penetrating through cracks in the cast-in-place concrete-deck overlay and progressing through the longitudinal joints between the girders, as shown in Fig. 7.1. In the areas of heaviest leakage, extensive staining and white deposits were visible, accompanied by corrosion of the prestressing strands and deterioration of the concrete cover. In some areas, spalling exposed the prestressing strands, leading to severe deterioration and failure of up to six of the seven wires in several strands. The specified concrete cover for this bridge was 1.75 in. (45 mm), and measured cover was up to 0.25 in. (6 mm) less than this value. Novokshchenov mentioned that this type of damage was common in other similar bridges and concluded that this was an inherent problem of this particular bridge design. A third bridge examined in this report (Novokshchenov 1989b) consisted of precast, pretensioned I-girders. Corrosion-related concrete cracking and spalling were found in girders adjacent to longitudinal expansion joints and at the ends of most girders at transverse joints both expansion and fixed. Figure 7.2 shows the path of chloride-laden water at a longitudinal expansion joint and the resulting deterioration. Wire fractures were common where the strand was exposed due to spalling. Corrosion damage at the ends of the girders was less severe at fixed joints than at expansion joints and was attributed to less leakage of chloride-laden moisture from the bridge deck. The specified cover was 2 in. (50 mm), and measured covers were up to 0.25 in. (6 mm) less than this value. The reader is referred to Novokshchenov for more detail on these condition surveys (1989b).

Others have performed similar condition studies of bridges with pretensioned elements (Whiting, Stejskal, and Nagi 1993). In general, these surveys found that corrosion-related deterioration in pretensioned members was restricted to specific areas of the structure, primarily at transverse joints along the bridge. Condition surveys indicate that corrosion problems in pretensioned bridges are primarily a function of the design of the structure rather than the pretensioned elements. Improved joint design and maintenance or minimization of joints would appear to eliminate most corrosion problems.

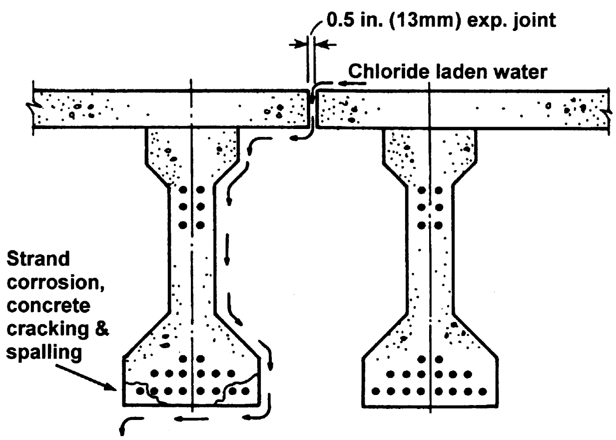

Fig. 7.2—Mechanism for moisture and chloride penetration at longitudinal expansion joints in precast I-girder bridges (adapted from Novokschenov 1989b).

7.4—Unbonded post-tensioned structures

7.4.1 *Unbonded single-strand tendons*—Unbonded single-strand or monostrand tendons refer to greased and sheathed single-strand tendons commonly used in slabs. Monostrand unbonded tendons represented approximately 80% of the post-tensioning used in the United States from 1965 to 1991 (Schupack 1994c). Most of this steel was used in buildings, including parking structures, and slabs-on-grade. A very comprehensive discussion of corrosion of monostrand tendons is provided by ACI 423.4R. Corrosion problems in monostrand systems generally result from three sources (ACI 423.4R):

- Sheathing problems;
- Detailing practices; and
- Storage, handling, and construction problems.

Figure 7.3 shows typical moisture and chloride access to the monostrand system (Demitt 1994). A detailed description of deterioration mechanisms and the field performance of monostrand systems is provided in ACI 423.4R.

7.4.2 *Unbonded internal multistrand tendons*—This section deals with unbonded internal tendons other than monostrand tendons. Included in this category are unbonded multistrand tendons, unbonded multiple-wire tendons, and unbonded post-tensioning bars. Internal unbonded tendons are not commonly used for several reasons. First, the lack of grouting to provide bond between the tendon and concrete limits the capacity of the structure. Second, unbonded internal tendons have few corrosion-protection options, except grout. Third, failure of an unbonded tendon, whether due to tendon corrosion or anchorage corrosion, leads to a complete loss of prestress.

Internal unbonded tendons consisting of seven-wire strands are uncommon. Unbonded multiple-wire tendons, however, have been used in the form of buttonheaded wire systems, which were common in the 1960s, particularly in parking garages. These are no longer used in new construction, in part due to poor durability (Nehill 1991; ACI 423.4R).

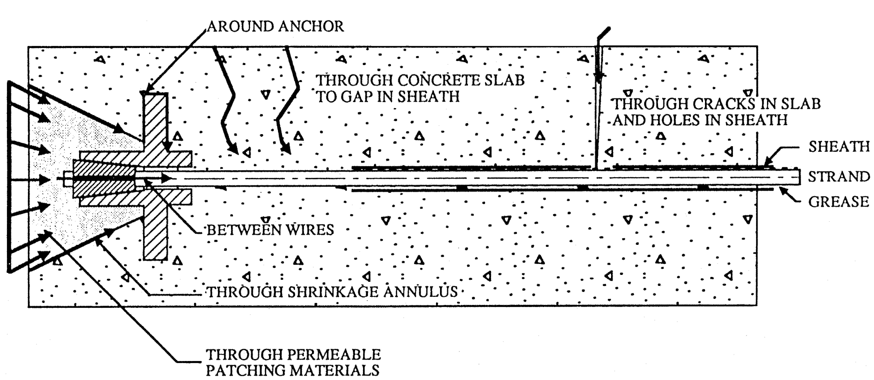

Fig. 7.3—Possible moisture and chloride access to monostrand systems (Demitt 1994).

Novokshchenov reports a condition survey of a bridge with pretensioned and post-tensioned girders in Salt Lake City, Utah (1989b). Post-tensioned girders were prestressed with unbonded post-tensioning bars. Two 1 in. (25 mm) diameter and two 1.5 in. (38 mm) diameter prestressing bars were used in each girder. Each post-tensioning bar was placed inside a galvanized steel duct, and no additional corrosion protection was provided. Bar anchorage was provided using end nuts and a steel bearing plate. Anchorages were located in pockets that were filled with mortar after stressing. The bridge was located in an environment where deicing salts were used. After 13 years of service, the post-tensioned bars began to fail. Failures were first indicated by loud noises heard by persons in the area and by bars projecting from the ends of the girders. Additional failures were discovered by removing the mortar anchorage protection and checking for loose bar ends and nuts. No cracks or rust stains were found on the exterior of the girders. Twenty-one bar failures were found. Pitting corrosion was found on the fractured bars, and the absence of necking suggested the failure was brittle in nature. The corrosion was attributed to moisture and chlorides entering the ducts at the anchorage zones and moving along the tendons. Corrosion of the steel anchorage plates, rated from moderate to very severe, caused cracking and spalling of the mortar cover. Chloride measurements in the mortar were very high. A malfunctioning drainage system and leaking expansion joints allowed chloride-laden moisture to drip onto the ends of the girders and the anchorage areas. Examination of the duct exterior at locations away from girder ends found no sign of corrosion. It was concluded that penetration of moisture and chlorides through the concrete cover and galvanized steel duct was unlikely, and that the sole cause of corrosion was penetration at end anchorages.

7.4.3 *External multistrand tendons*—External multistrand tendons most commonly occur in bridges. Cable stays can also be considered in this category. Corrosion protection for multistrand external tendons typically consists of a plastic or metal sheath filled with grout or corrosion-inhibiting grease. Observed corrosion-related failures or problems have resulted from a breakdown in the sheathing system or insufficient protection of the anchorages. These situations are worsened by poor or incomplete filling of the void space around the tendon with grout or grease that allows movement of moisture along the tendon length after penetration.

Robson and Brooman report corrosion-related distress in a precast segmental box-girder bridge with external tendons (1997). The external prestress was provided by 240 tendons, each consisting of 19 wires 19 mm (3/4 in.) diameter inside a plastic, grease-filled sheath. Severe signs of distress were observed after approximately 20 years of service. Two of the 240 tendons had failed completely, and evidence of individual wire fracture was observed in 121 of the remaining tendons. The fractures were attributed to corrosion of the wires in the anchorage zones. It was assumed that corrosion began during a 10 month construction delay, during which the tendon ends were left unprotected. Because the tendons were external, individual wire failures were detected by visual inspection. Existing tendons were removed, and the bridge was prestressed with new tendons after modifications to the anchorage areas.

7.5—Bonded post-tensioned structures

7.5.1 *After stressing, before grouting*—After stressing and before grouting, post-tensioning tendons are vulnerable to corrosion as a result of water penetrating the ducts through either the end anchorages or grouting ports and vents. HE failures have been attributed to corrosion occurring during this period (Clark 1992). Some construction specifications limit the length of time between stressing and grouting of post-tensioned tendons to 48 h to minimize the potential for such corrosion. Corrosion occurring after stressing and before grouting could also lead to failures after the structure has been in service.

7.5.2 *In service*—Corrosion in post-tensioned structures during service has been attributed to a variety of sources. A post-tensioning tendon in service has multiple layers of corrosion protection. While tendon corrosion can occur from breakdown of a single layer, it usually occurs when multiple layers fail. In most cases, corrosion-related deterioration is related to an inadequacy or breakdown in more than one component of the protection system.

7.5.2.1 *Grouting*—The most common grout-related corrosion problems are attributed to incomplete filling of the duct with grout. Common causes of incomplete grouting are construction difficulties, improper construction practices, blocked or damaged ducts, and improper placement or usage of vents. The fresh properties of the grout can also affect the grouting process through insufficient or excessive fluidity and excessive bleedwater, leading to entrapped air or the formation of bleed lenses. The severity of tendon corrosion is related to the extent of grout voids and the availability of moisture, oxygen, and chlorides. In general, the most severe corrosion occurs when the tendon is intermittently exposed and embedded in the grout. In this situation, a concentration cell can occur due to variations in the chemical and physical environment along the length of the tendon.

Tendon corrosion can also occur where the entire length of the tendon is well grouted. The most common cause of corrosion in these situations is chlorides in the grout itself (Schupack 1994b). Examples include seawater used as the mixing water or chloride-containing admixtures. In severe exposure with low cover or cracks, corrosion of the duct can lead to penetration of moisture and chlorides from an external source.

Isecke (1990) describes a detailed examination of a bonded post-tensioned bridge in Germany, which was demolished after less than 20 years of service due to corrosion-related deterioration. Isecke reports varying levels of grouting: full grouting or embedment of steel; partial grouting or embedment of steel; partial or total coating of the steel surface with a thin film of grout; and complete absence of grout. No corrosion was found where grouting was complete and the steel was fully embedded in grout. Varying amounts of corrosion damage were found under all other grouting conditions. The most severe corrosion was reported in partially grouted ducts at the boundaries between exposed and embedded steel. In ducts that were completely ungrouted, the prestressing steel was covered with a thin film of rust, but the reduction of area due to corrosion was deemed very small.

Schupack performed an extensive forensic examination on a 35-year-old post-tensioned bridge (1994a,b). The extent of corrosion damage in this bridge was not extensive enough to affect structural behavior (Schupack 1994a). Corrosion deterioration was attributed to two sources: poor and incomplete grouting throughout the bridge and the use of grout containing high levels of chloride in some girders. Like Isecke, Schupack found a range of grouting, from fully grouted, to partial grouting, to a complete lack of grout. The extent of corrosion depended on the completeness of grouting, the type of grout, and the availability of moisture. No corrosion was found in tendons where the ducts were completely filled with grout that did not contain chlorides. In partially grouted tendons with no chlorides in the grout and in ungrouted tendons, most exposed wires had surface corrosion. Severe corrosion was found at tendon low points where water had collected in the duct. Several tendons that were completely ungrouted but free of moisture showed no signs of corrosion. Several girders in the bridge were grouted using an expansive grout that contained high levels of chloride. This grout was not recommended for post-tensioning applications by its manufacturer, because expansive properties were achieved by adding iron filings and chlorides to provide expansion though corrosion of the iron. Grout samples from the bridge showed chloride levels as high as 8000 ppm by weight of grout. Where the grout was used, very severe tendon and duct corrosion, deep pitting corrosion, and random wire breaks were found. Schupack also reported significant longitudinal cracks in the webs of the girders following the tendon profile. In cases where nonchloride grout was used, the cracks were attributed to freezing of water in partially grouted or ungrouted tendons, rather than to tendon corrosion, illustrating additional deterioration that may result from poor grouting.

7.5.2.2 *Inadequate concrete cover*—Concrete cover provides additional protection for the tendon. Where the protection provided by the duct is inadequate, low concrete cover has contributed to tendon corrosion.

Novokshchenov reports a condition survey of the Gandy Bridge in Florida, which consisted of precast post-tensioned girders with a reinforced concrete deck slab (1989b). Post-tensioning was provided using 1-1/8 in. (28.6 mm) diameter prestressing bars. Each bar was located inside a 1-1/2 in. (38.1 mm) grouted metal duct. The bridge was less than 35 years old at the time of inspection and had experienced significant cracking and spalling resulting from corrosion of the post-tensioning ducts and tendons. Measured values of concrete cover for the bottom tendons were less than the specified value of 2.75 in. (70 mm), ranging from 1.25 in. (32 mm) to 2.5 in. (64 mm), with an average of 2.1 in (53 mm). Concrete in the girders was air-entrained with a low *w/c*. Rapid chloride-permeability measurements on concrete samples from the bridge indicated moderate to low permeability. Because the concrete was of good quality, Novokshchenov concluded that insufficient concrete cover was the major cause of corrosion of the post-tensioning tendons.

7.5.2.3 *Duct problems*—The post-tensioning duct is an important component of corrosion protection in post-tensioned structures. Many forms of ducts exist, ranging from nonpermanent duct formers, to galvanized steel ducts, to plastic ducts, each providing an increasing level of protection. Corrosion has resulted from damaged ducts, improper splices between ducts, corroded ducts, and nonpermanent duct formers. Holes in the duct can allow concrete to enter during casting. This can hamper placement and tensioning of the tendons and also grouting. Damage or misalignment of ducts during construction or concrete placing can also lead to post-tensioning and grouting difficulties.

As mentioned in the preceding section, Novokshchenov reports duct and tendon corrosion in a post-tensioned bridge in Florida and concludes that insufficient concrete cover led to severe corrosion of the metal ducts and post-tensioning tendons (1989b). If noncorroding plastic ducts had been used, it is possible that corrosion-related deterioration of the post-tensioning system could have been eliminated in spite of low cover. Isecke also reports total deterioration of metallic ducts due to corrosion in many areas of a post-tensioned bridge (1990). Corrosion of the duct led to moisture and chloride penetration into the grout. In most cases, deterioration of the duct corresponded to severe corrosion and occasionally fracture of the prestressing steel.

7.5.2.4 *Anchorage protection*—In contrast to unbonded tendons, anchorage corrosion in bonded tendons is generally not deemed critical failure. Bond between the tendons and concrete will prevent a complete loss of prestressing. Anchorage corrosion and inadequate anchorage protection, however, can lead to the ingress of moisture and chlorides into the tendon. This condition is particularly severe with poorly grouted ducts that allow moisture to readily move along the length of the tendon. Corrosion of anchorage components can also cause cracking and spalling of concrete near the anchorage.

Most anchorage corrosion problems result from two factors: inadequate protection and location. Inadequate protection can include insufficient cover, permeable materials used to fill the anchorage recess, and lack of bond between fill material and anchorage recess. The location of the anchorage plays a significant role. Normally, anchorages are located at the end of the member. In many structure types, expansion joints are located over the member ends. Poor detailing and maintenance of the joints has permitted chloride-laden moisture to come in direct contact with the anchorage zones of the member, creating particularly severe exposure conditions.

Dickson, Tabatabai, and Whiting reports a detailed evaluation of a 34-year-old, precast, post-tensioned girder, which was removed from a bridge that had been subjected to deicing salts throughout its service life (1993). The overall condition of the girder was excellent, and the observed corrosion was not deemed to affect structural behavior. The most severe corrosion was found on the anchorages of the girder. Anchorage protection was provided by a cast-in-place concrete end diaphragm. Surface corrosion was found on all anchorage and bearing-plate surfaces. The post-tensioning wires within the anchorage were corroded more severely than the wires within the length of the duct. In general, the ducts were very well grouted with only one void found during dissection of the girder. Corrosion of the wires within the length of the tendon was minor. Chloride analysis indicated that chlorides had infiltrated the duct through one of the anchorages in spite of the cast-in-place concrete anchorage protection.

Isecke also reports infiltration of moisture and chlorides through end anchorages during the examination of a post-tensioned bridge whose anchorages were unprotected (1990). Anchorages located near expansion joints were exposed to chloride-laden moisture runoff from the bridge deck and were heavily damaged by corrosion. Moisture and chlorides penetrated through the anchorages, leading to heavy corrosion on the post-tensioned bars used in the structure. Where the unprotected anchorages were not exposed to deck runoff, no corrosion was found on the anchorages.

CHAPTER 8—REFERENCES

8.1—Referenced standards and reports

The standards and reports listed below were the latest editions at the time this document was prepared. Because these documents are revised frequently, the reader is advised to contact the proper sponsoring group if it is desired to refer to the latest version.

American Concrete Institute

201.2R	Guide to Durable Concrete
222R	Corrosion of Metals in Concrete
222.1	Provisional Standard Test Method for Water-Soluble Chloride Available for Corrosion of Embedded Steel in Mortar and Concrete Using Soxhlet Extractor
228.2R	Nondestructive Test Methods for Evaluation of Concrete in Structures
301	Specifications for Structural Concrete
318	Building Code Requirements for Structural Concrete
423.3R	Recommendations for Concrete Members Prestressed with Unbonded Tendons
423.4R	Corrosion and Repair of Unbonded Single Strand Tendons
440R	Report on Fiber Reinforced Plastic (FRP) Reinforcement for Concrete Structures

ASTM International

A 123	Standard Specification for Zinc (Hot-Dip Galvanized) Coatings on Iron and Steel Products
A 416/A 416M	Standard Specification for Seven-Wire Prestressing Steel Strand
A 421	Uncoated Stress-Relieved Steel Wire for Prestressed Concrete
A 648	Steel Wire, Hard Drawn for Prestressing Concrete Pipe
A 722/A 722M	Uncoated High-Strength Steel Bar for Prestressing Concrete
A 775	Standard Specification for Epoxy-Coated Reinforcing Steel Bars
A 779	Steel Strand, Seven-Wire, Uncoated, Compacted, Stress-Relieved for Prestressed Concrete
A 821	Steel Wire, Hard Drawn for Prestressing Concrete Tanks
A 864/A 864M	Steel Wire, Deformed, for prestressed Concrete Railroad Ties
A 881/A 881M	Steel Wire, Deformed, Stress-Relieved or Low-Relaxation for Prestressed Concrete Railroad Ties
A 882/A 882M	Standard Specification for Epoxy-Coated Seven-Wire Prestressing Steel Strand
A 886/A 886M	Steel Strand, Indented, Seven-Wire Stress-Relieved for Prestressed Concrete
A 910	Uncoated, Weldless, 2- and 3-Wire Steel Strand for Prestressed Concrete
A 911/A 911M	Uncoated, Stress-Relieved Steel Bars for Prestressed Concrete Ties
C 876	Standard Test Method of Half-Cell Potentials of Uncoated Reinforcing Steel in Concrete
C 1152	Standard Test Method for Acid-Soluble Chloride in Mortar and Concrete
C 1202	Standard Test Method for Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration
C 1218	Standard Test Method for Water-Soluble Chloride in Mortar and Concrete
G 46	Standard Recommended Practice for Examination and Evaluation of Pitting Corrosion

American Association of State Highway and Transportation Officials (AASHTO)

TP55	Standard Test Method for Determining Chloride Ions in Concrete and Concrete Materials by Specific Ion Probe
T259	Standard Test Method for Resistance of Concrete to Chloride Ion Penetration
T277	Standard Test Method for Rapid determination of the Chloride Permeability of Concrete

These publications may be obtained from these organizations:

American Concrete Institute
P.O. Box 9094
Farmington Hills, Mich. 48333-9094

ASTM International
100 Barr Harbor Drive
West Conshocken, Pa. 19428

AASHTO
444 North Capital Street NW, Suite 225
Washington, D.C. 20001

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