Corrosion and Repair of Unbonded Single Strand Tendons

Reported by ACI/ASCE Committee 423

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CHAPTER 1—INTRODUCTION

1.1—General

This report is intended to provide historical and general information on the evaluation of known or suspected corrosion problems in unbonded single strand tendons, and to describe typical repair methods currently in use. It has been prepared to describe the state of knowledge as perceived by the committee. It is not a standard or recommended practice. Expertise in design, construction, evaluation, and repair of structures utilizing single strand unbonded tendons is strongly recommended for a team undertaking evaluation and repair of corrosion problems.

There have been corrosion problems with other types of pre- and post-tensioning systems. However, certain aspects of corrosion of unbonded single strand tendons are unique. The causes and effects of corrosion of unbonded single strand tendons are, in several respects, different from those of bonded conventional reinforcing or other post-tensioning systems. Thus, the methods for evaluating and repairing corrosion of single strand tendons are also different in some respects. For example, since the tendons are largely isolated from the surrounding concrete, they may not be affected by deleterious materials such as chlorides and moisture in the concrete. However, they also are not passivated by the surrounding concrete, and can corrode if water gains access to the inside of the sheathing or anchorage and the grease protection is inadequate. Measures taken to repair and protect the surrounding concrete may not repair or reduce deterioration of the prestressing steel where corrosion has been initiated. The tendons usually require separate evaluation and repair.

1.2—Background

Commercially viable unbonded post-tensioning systems were introduced to North America in the 1950s. At that time there were no accepted standards for design nor material specifications for prestressing steels, and guidance came in the form of tentative recommendations from a joint committee of the American Concrete Institute (ACI) and the American Society of Civil Engineers (ASCE), from the Prestressed Concrete Institute (PCI), or from the Bureau of Public Roads, United States Department of Commerce. Unbonded tendons in the early systems used bundles of wires or strands, sometimes inaccurately called “cables,” of various diameters, and protected by grease and paper sheathing that were sometimes applied by hand.

The use of unbonded tendons became more common during the late 1950s and early 1960s as progress was made in establishing design and materials standards. Acceptance of the concept was regional at first, and was largely the result of sales efforts and design tutoring by tendon suppliers. The use of post-tensioning increased rapidly during the late 1960s and 1970s as the advantages of the system were demonstrated. For many types of structures, these advantages included shorter construction time, reduced structural depth, increased stiffness, and savings in overall cost. In addition to their use in enclosed buildings, unbonded post-tensioning systems were used in parking structures and slabs on grade, and bonded post-tensioning was used on water tanks, bridges, dams, and soil tie-back systems. Unbonded multiwire and multistrand tendons have been used extensively in nuclear power structures.

Incidents of corrosion of unbonded single strand tendons began to surface during the 1970s. It had been believed by some that corrosion protection would be provided by the grease during shipping, handling and installation, and by the concrete thereafter. However, the early greases often did not provide the corrosion-inhibiting characteristics that are required in the current Post-Tensioning Institute (PTI) “Specifications for Unbonded Single Strand Tendons.” In the early 1980s, PTI recognized the structural implications of corrosion and began to implement measures to increase the durability of unbonded post-tensioning systems. In 1985, PTI published the first performance standard for single strand tendons. Relying on experience and practice in the nuclear industry using corrosion-inhibiting hydrophobic grease, similar performance standards for grease were incorporated. In the 1989 edition of ACI 318, “Building Code Requirements for Reinforced Concrete,” changes were made to incorporate measures that related the required protection of the tendons and the quality of the concrete to the environmental conditions that could promote corrosion of the post-tensioning. Structures built prior to the adoption of these new standards, especially those in aggressive environments, are more likely to experience corrosion of the post-tensioning system than those designed and built since.
Tendons that are broken, or are known to be damaged by corrosion, can be repaired or supplemented by any of several methods. The more difficult task is to determine the extent of corrosion damage and the degree to which tendon repairs are needed. This report is intended to provide guidance in the evaluation of suspected or known corrosion problems and to describe repair methods currently in use.

1.3—Scope
This report includes a review of the following:
• Codes and code changes affecting unbonded post-tensioning tendons;
• Past and present corrosion protection systems and how those systems have changed to enhance corrosion protection;
• Types of corrosion damage found in prestressing steel;
• Methods for evaluating structures which are suspected of, or known to have, corrosion damage in the post-tensioning system; and
• Basic repair options currently in use.

1.4—Limitations
This report presents a summary of typical problems experienced with unbonded post-tensioning systems and includes general guidelines for evaluating and repairing single strand tendons. While the methods presented herein are general in nature, they are not universally applicable. Standard specifications and details are not included since each structure is unique and must be analyzed accordingly.

This report is not intended to be included as part of specifications for investigations and repairs. Presently, there is no practical method to ascertain the total extent of damage to a post-tensioning system. The unpredictable nature of tendon failures exhibited by inadequately protected, corroding strand makes estimating tendon life expectancy uncertain.

A wide variation exists in the durability and rate of deterioration of older post-tensioning systems. This is due in part to the composition of the parts of the tendon (strand, anchors, grease, and sheathing), and in part to the quality of the surrounding concrete, the environmental exposure and the type of maintenance performed on the structure. The investigator must rely on a knowledge of historical performance of similar structures and must be experienced in interpreting external evidence which may give an indication of latent internal problems.

CHAPTER 2—REVIEW OF CODE REQUIREMENTS AND CHANGES
2.1—General
When evaluating corrosion damage in post-tensioned structures with unbonded tendons, the investigator must consider the age of the structure and the standards of practices available to the designer and contractor at the time of construction. Although ACI published building regulations for reinforced concrete as early as 1920, ACI 318-47 was the first to acknowledge the significance of environmental exposure. The early Codes (ACI 318-47, ACI 318-51, and ACI 318-56) also recognized the importance of clear cover and concrete quality in providing adequate corrosion protection to the non-prestressed, bonded reinforcement.

In 1958, ACI-ASCE Joint Committee 323 published “Tentative Recommendations for Prestressed Concrete” and addressed the protection of prestressing steel in three areas of recommended practice: concrete cover, allowable tensile stresses, and, for unbonded systems, protection of the strand or wire with grease and a sheathing material. Since 1958, provisions for prestressed concrete have included requirements for corrosion protection. The grease and sheathing were viewed, by most, primarily as a lubricant and bond breaker, and secondarily as a corrosion deterrent during shipping, handling, and placing. Long-term corrosion protection was viewed by some as being provided by the uncracked concrete cover.

In 1963, prestressed concrete was first included in ACI 318 with provisions for concrete cover, allowable tensile stresses, and strand protection. These items have been modified from time to time, but the substantive change came in 1989 when durability was emphasized in ACI 318.

2.2—Cover requirements for unbonded tendons
ACI 318-63 required the following under prestressed concrete-concrete cover:

a) The following minimum thickness of concrete cover shall be provided for prestressing steel, ducts and non-prestressed steel.

<table>
<thead>
<tr>
<th>Cover, in. (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete surface in contact with ground 2 (50)</td>
</tr>
<tr>
<td>Beams and girders: Prestressing steel and main reinforcing bars 1½ (40)</td>
</tr>
<tr>
<td>Stirrups and ties 1 (25)</td>
</tr>
<tr>
<td>Slabs and joists exposed to weather 1 (25)</td>
</tr>
<tr>
<td>Slabs and joists not exposed to weather ¾ (20)</td>
</tr>
</tbody>
</table>

b) In extremely corrosive atmosphere or other severe exposures, the amount of protection shall be suitably increased.

Eight years later, in ACI 318-71, the cover requirements were increased from 2 in. (50 mm) to 3 in. (75 mm) for prestressed members cast against and permanently exposed to earth. In addition, a new requirement for concrete protection for reinforcement was introduced and required that cover requirements be increased 50 percent in members with allowable concrete tensile stresses above 6,000 psi (0.5 MPa). The provisions for concrete cover in the 1963 Code were also revised to require that density and non-porosity (in addition to cover) of the concrete be considered when increasing concrete protection. The Code provisions for cover of unbonded prestressing strand did not change in the 1977, 1983 and 1989 Code revisions. In the 1983 Commentary, a discussion was added as follows:

R7.7.5—Corrosive environments
When concrete will be exposed to external sources of chlorides in service, such as deicing salts, brackish water, seawater, or spray from these sources, concrete must be proportioned to satisfy the special exposure requirements of Code Section 4.5. These include minimum air
content, maximum water-cement ratio (or minimum strength of lightweight concrete), maximum chloride ion in concrete, and cement type. Additionally, for corrosion protection, a minimum concrete cover for reinforcement of 2 in. (50 mm) for walls and slabs and 2 1/2 in. (60 mm) for other members is recommended. For precast concrete manufactured under plant control conditions, a minimum cover of 1½ and 2 in. (40 and 50 mm), respectively, is recommended.

This discussion is important as it calls the designer’s attention to the importance of air content, maximum water-cement ratio, maximum chloride-ion content, and cement type when designing for corrosion protection.

Finally, a distinction between plant-cast members and other prestressed concrete members was made in the 1971 Code with the addition of distinct cover requirements for plant-cast members.

2.3—Allowable tensile stresses in concrete

In aggressive environments, designing to minimize cracking was used to improve durability by reducing ingress of corrosive elements. Though a properly greased tendon in an intact sheathing may not be affected at first by a crack in the surrounding concrete, corrosion of nearby conventional reinforcing can cause spalling which may expose the tendon to physical damage which may then lead to corrosion of the strand.

For consideration of long-term durability and corrosion protection, the maximum allowable tensile stresses in the concrete at service loads, after allowance for all prestress losses, are of most interest. The initial recommendations presented in 1958 by ACI-ASCE Committee 323 limited allowable tensile stresses in pretensioned members to 6 $\frac{f_{pc}^U}{500}$ psi (0.5 $f_{pc}^U$ MPa) where not exposed to weather or corrosive environments. In post-tensioned members not exposed to weather or corrosive environments, tensile stresses were limited to 3 $\frac{f_{pc}^U}{500}$ psi (0.25 $f_{pc}^U$ MPa). No mention was made of allowable stresses in unbonded systems.

In ACI 318-63, allowable tensile stresses in concrete were limited to 6 $\frac{f_{pc}^U}{500}$ psi (0.5 $f_{pc}^U$ MPa) for members not exposed to freezing temperature or to a corrosive environment, provided the members contained bonded reinforcement (prestressed or non-prestressed) to control cracking. For all other members, no tensile stresses at service load levels were allowed.

The 1971 Code, “Prestressed Concrete—Permissible Stresses,” required that tensile stresses in concrete be limited to 6 $\frac{f_{pc}^U}{500}$ psi (0.5 $f_{pc}^U$ MPa) for most members, but allowed up to 12 $\frac{f_{pc}^U}{500}$ psi (1.0 $f_{pc}^U$ MPa) in tension in the concrete provided computations were made using the cracked transformed section and a bi-linear moment-deflection relationship to confirm that long-term deflection of the member satisfied “Strength and Serviceability Requirements—Control of Deflections.” In addition, the allowable tension limits could be exceeded provided that experimental and analytical work could show that performance would not be impaired.

In the 1977 Code, for an allowable tensile stress up to 12 $\frac{f_{pc}^U}{500}$ psi (1.0 $f_{pc}^U$ MPa), a provision was added requiring that the concrete cover for prestressed and non-prestressed steel be increased for prestressed members exposed to earth, weather or corrosive environments. “Concrete protection for reinforcement” required a 50 percent increase in cover for members exposed to weather, earth, or corrosive environments and with a tensile stress greater than 6 $\frac{f_{pc}^U}{500}$ psi (0.5 $f_{pc}^U$ MPa). The 1995 Code retains the allowable tensile stresses as outlined in the 1977 Code.

2.4—Protection of unbonded tendons

The protection of unbonded prestressing strands was initially described by ACI-ASCE Committee 323 to consist of a grease- or asphalt-impregnated material enclosed in a sheath. Specification and approval of the method of protection was left to the engineer. In ACI 318-63, Section 2620 indicated that “unbonded steel shall be permanently protected from corrosion.” However, no specific information was provided relative to the sheath material and the required corrosion protection. As noted previously, concrete cover was sometimes interpreted as providing the permanent protection from corrosion.

The 1971 Code included a section entitled “Corrosion Protection for Unbonded Tendons.” It stipulated that “unbonded tendons shall be completely coated with suitable material to ensure corrosion protection.” This section also required that wrapping should be continuous over the entire unbonded zone of tendons in order to prevent bonding with surrounding concrete and loss of the coating material during concrete placement.


Significant new provisions of the 1985 PTI specification are discussed below.

- Definition of exposure conditions: The 1985 specifications addressed tendons in “normal (non-corrosive) environments,” and tendons in “aggressive (corrosive) environments.” Normal environments were defined as those present in nearly all enclosed buildings with dry interiors, and exposed structures in areas with very little or no snow. Aggressive environments were defined as those that would expose the structure to direct or indirect applications of deicer chemicals, seawater, brackish water, or spray from these sources.
- Anchorages and couplings: Anchorages were required to have watertight connection to the sheathing and a watertight closing of the wedge cavity. Couplings were required to be protectively coated the same as the strand, since they become part of the strand.
- Sheathing: Sheathing material thickness was specified according to exposure, and was given as 0.025 in. (0.6
for normal environment and 0.030 in. (0.8 mm) for aggressive environment. The inside diameter of the finished sheathing was required to be at least 0.010 in. (0.3 mm) greater than the diameter of the strand.

- Corrosion-preventive coating: (Hereafter, the term “grease” will be used to describe the corrosion-protective coating, defined in the PTI specification as “…an organic coating with appropriate polar, moisture-displacing, and corrosion-preventive additives.”) The minimum weight of grease coverage of the strand was given, as was a list of test requirements for the grease itself. The corrosion protection performance was based on ASTM B-117 for the time until Rust Grade 7 developed and was set as 720 hr minimum for normal environment and 1000 hr for aggressive environment.

PTI reissued this specification in 1993 after substantial revisions were made to all chapters. Major changes are summarized below:

- Definition of exposure conditions: The terms used in previous specifications “normal (non-corrosive)” and “aggressive (corrosive),” were changed to “normal” and “aggressive.” The definitions of normal and aggressive remained similar to the 1985 specification except that exposed structures in areas with very little or no snow were no longer mentioned as being in a normal environment. The designer was advised to “evaluate the conditions carefully to determine if the environment in which a structure is located is considered aggressive in any way.”

- Definition of exposure conditions: A stipulation was added that stressing pockets not maintained in a dry condition after construction should be considered exposed to an aggressive environment.

- Prestressing steel: Protection requirements were added for packaging and identification. A criterion was added that limited surface rust to pits no more than 0.002 in. (0.05 mm) diameter or length. (This type of rust can be removed with fine steel wool and might not be felt with the fingernail.)

- Anchorages and couplers (formerly “Couplings”): Static test criteria were clarified and linked to ACI 318, and dynamic test requirements were added. Design criteria were added for bearing stresses on concrete. A stipulation was added that required anchorages intended for use in aggressive environments to be fully protected against corrosion. Encapsulation of the anchorage, the connection of the sheathing to the anchorage encapsulation, and the seal of the wedge cavity were required to sustain a hydrostatic water pressure of 1.25 psi (0.0086 MPa) for 24 hr.

- Sheathing: The thinner sheathing previously allowed for tendons to be exposed to normal environments was removed; the minimum thickness of sheathing was specified as 0.040 in. (1.0 mm) for both environments. The size of the annular space between the outside of the strand and the inside of the sheathing was increased from 0.010 in. (0.3 mm) to 0.030 in. (0.8 mm). Complete encapsulation of tendons to be used in aggressive environments was specified with the same watertightness requirements given for anchorages. A statement was added that calls for the designer to specify the amount of unsheathed strand permitted at the anchorages for tendons exposed to normal environments.

- Corrosion-inhibiting coating (formerly “Corrosion-preventive coating”) (commonly referred to as grease): All tendons are to use grease that meets the ASTM B117 Rust Grade 7 criteria after 1000 hr. (The shorter test time previously permitted for tendons to be used in normal environments was dropped.)

- Installation requirements: The minimum cover requirement for anchorages was added and was specified to be at least 1.50 in. (40 mm) for normal environments and 2 in. (50 mm) for aggressive environments.

- Tendon finishing: Permissible length of strand projection from the face of the wedges was reduced. The previous projection allowed was 0.75 in. (20 mm) minimum and 1.25 in. (30 mm) maximum; the revised projection limits are 0.50 in. (12 mm) minimum and 0.75 in. (20 mm) maximum.

**CHAPTER 3—UNBONDED TENDONS**

3.1—Evolution of unbonded tendons

The first building structures to use unbonded tendons in North America were lift slabs built during the mid-1950s. The early, unbonded tendons were greased and helically wrapped with paper. They used high-strength wires, generally 0.25 in. (6 mm) diameter, with an ultimate strength of 240 ksi (1650 MPa), with button-head type anchorages, as shown by Fig. 3.1. This system had large cumbersome anchorages, large trumpet transitions and a fixed distance between buttons at each end of the tendon. Systems also evolved during this period that used single and multiple strands.

In the early 1960s the convenience of placing unbonded single strand tendons was realized and the number of suppliers increased. The marketplace found the competitiveness of this system favorable, and the use of unbonded single strand tendons increased significantly. Anchorage hardware varied considerably and included high-strength spirals, barrels, or castings with wedges, and fittings that were swedged (mechanically attached) to the prestressing steel. The anchor that prevailed was a casting that contained a recess to house a two-piece wedge for use with a single strand. This is currently the predominant system and represents about 60 percent of all post-tensioning tonnage.

By the late 1960s, plastic began replacing paper sheathing. Three different processes have been used (Fig. 3.2 and 3.3): 1. The strand is covered by preformed push-through plastic tubes; 2. The strand is wrapped longitudinally with a heat-sealed strip; and (most recently) 3. The greased strand is covered by molten plastic that is continuously extruded around it. Thickness and composition of the sheathing material were left to the supplier and were not uniform in the industry. The first effort to regulate these items was by the PTI in their 1985 specification.
Initially, there were no corrosion protection standards for the grease (except in the nuclear industry, refer to ACI 359-74, “Code for Concrete Reactor Vessels and Containments”), so the tendon manufacturers used the grease of their choice. Corrosion-resisting properties of the grease were not specified, nor were there standards for the uniformity and amount of grease to be applied to the prestressing steel. Deterioration of the grease was not expected, but as problems became known they were addressed by the PTI in their recommended specifications.

3.2—Sheathing problems

The push-through system required that the sheathing be sufficiently oversized to allow the greased strand to be inserted without too much difficulty. This resulted in a tendon with many air voids inside the sheathing and allowed infiltration of water during storage, shipping, and installation, and in service.

With the heat-sealed system, the sheathing was provided in rolls of flat plastic tape that was usually 20 to 40 mils (0.5 to 1.0 mm) thick. During tendon fabrication, the strand would be taken from the pack and passed through a grease extrusion head. The tape was then folded over the greased strand and the lapped seam welded shut with a flame. This method also formed a slightly oversized sheathing that could have air voids. The seam weld was interrupted every time there was a pause in the process, and sometimes the sealed seam pulled apart during handling or installation. This system is frequently found to have gaps from one cause or another that expose the greased strand to contact with the concrete.

Seamless extruded sheathing first appeared in the early 1970s. The extruded sheathing minimized the problems experienced with the push-through and heat-sealed systems by providing a snug, seamless sheathing around the greased strand.

3.3—Detailing practices

Certain details that were initially considered acceptable are insufficient to provide the degree of durability required...
by recent versions of ACI 318. It is now recognized that concrete cannot provide reliable protection to the strand, even if the strand is greased.

It was common practice in both detailing and installation to allow the sheathing to stop short of the anchorages, as shown by Fig. 3.4 (the grease can get wiped away from these sections of strand during the installation of anchorages at stressing and nonstressing ends). During stressing operations, the strand at the stressing end moves about 8 in. per 100 ft (200 mm per 30 m) of length. Direct contact between the unsheathed strand and the hardened concrete is disturbed at the stressing end during this elongation, thus providing a path for water to find its way to the strand and into the sheathing.\(^2\)

Since the dead end anchor did not have to move during stressing, it was considered acceptable for the strand near the dead end to be exposed to the concrete. However, this also allowed the end of the tendon to be exposed to dirt and water during storage, handling, and placement, prior to placing the protective concrete. The end of the sheathing was, for practical purposes, open, and the voids inside the tendon sheathing provided a means for any available water to gain access to the tendon. Even if the storage time at the site was relatively brief (which was not always the case), there was ample opportunity for water to get into the tendons due to any snow and rain that might occur while the tendons were stored on the ground or on the formwork. For those portions of the strand not adequately protected by grease, water and oxygen could cause corrosion and then could be further exacerbated by the presence of chloride-ions. Some typical defects contributing to corrosion in the end anchor region are shown in Fig. 3.4.

The stressing side of the live-end anchorage was protected by filling the stressing pocket in which the anchorage was recessed with a protective cementitious grout (Fig. 3.4). The casting, wedges, and strand tail were not coated. Often, shrinkage of the grout plug in the stressing pocket caused a space between the side of the stressing pocket and the grout that allowed water to gain access to the tendon anchorage. Corrosion of the bearing plate casting is often found but has not been known to cause failures. On tendons with barrel and wedges sitting on bearing plates, heat-treated barrels

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**Fig 3.3—Plastic sheathing types (reprinted from Reference 2)**

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**Fig 3.4—Potential defects in corrosion protection at unbonded single strand tendon live end anchorage.**
have been found to suffer from brittle failure. The primary problem has been either corrosion of the wedges or the migration of water down the tendon in the void between the wires or around the outside edge of the strand. Corrosion failures of the strand have occurred in the unsheathed length in front of the anchor, at low points in the profile where water collects, and at points where sheathing was damaged and permitted direct access of corrosive materials.

At intermediate anchorages, features of both dead- and live-end anchorages are found. The sheathing may not be adequately sealed to the anchorage on the “first pour” side. Strand and anchorages may be in direct contact with concrete on the “second pour” side of the anchorage. Water could gain access to the tendon during shipping, storage, and installation, or after construction has been completed. Problems have developed at the intermediate anchorages when the construction joint over the anchorages was not sealed to prevent leaking of water (sometimes containing chlorides) through the construction joint in elements exposed to the weather, such as in parking structures and balconies. Significant chloride contamination of concrete has occurred as a result of leakage through unsealed construction joints. Corrosion of the upper surface of the anchorage castings, and of backer bars, can cause corrosion-induced spalling at an early age. Such spalls then act as small reservoirs for water, further accelerating the deterioration process. Corrosion of the backer bars, bearing plates or anchorages, and on occasion the strand, is promoted by this frequent supply of water and oxygen adjacent to the joint. With the earlier bearing plate and intermediate barrel anchorage hardware, the bearing plate was occasionally located outside the bulkhead of the slab, meaning that the construction joint would be located directly over the plate and allow water to come into contact with strand end anchorages.

The designer’s choice of locations for anchorages has sometimes led to corrosion problems. Locating anchorages in gutter lines or expansion joints of parking structures or at exposed edges of slabs in commercial and residential buildings (i.e., balconies) has resulted in water infiltration into anchorage areas and caused corrosion failures of the strand. Without special waterproofing, construction joints at beam-column connections have, on occasion, allowed water to enter tendon sheathing through the anchorages in joints in columns. These joints are seldom sealed, even though they are frequently in the drainage flow path, and can be exposed to rain and melting snow. Water running down the face of a column can gain access to the tendon anchorage if consolidation of the concrete at the construction joint is poor. Honeycombed, permeable concrete, ponding areas, leaking construction joints, misplaced conduits, and other construction defects can all contribute to water access to tendons.

3.4—Storage, handling, and construction problems

Though the tendon sheathing is fairly tough, the wear and tear to which it is sometimes subjected during shipping, storage, handling, and concrete placement can be severe. These can damage the corrosion protection provided by the grease and sheathing.

Typical problem areas have included:

- Tearing and/or cutting of the sheathing by wire or metal bands that hold the tendons in coils for shipping;
- Tearing and/or cutting of the sheathing by unpadded slings used to lift the bundles of tendons from the truck to the storage area, and from the storage area to the formwork where they are to be placed;
- Unprotected storage, such as leaving the tendons in direct contact with the ground, leaving them exposed to snow or rain, or placing them where they will be damaged by construction traffic;
- Rough handling during placement of tendons in formwork, causing splits and tears in the sheathing;
- Incomplete repair of damage to tendon sheathing;
- Leaving the unsealed tendon ends exposed to the weather, either before concrete placement or after stressing;
- Inadequate cover over the cut-off strand tail at live-end stressing pockets;
- Inadequate concrete cover to protect the tendon at high and low points of drape, and at the anchorages;
- Voids inside sleeves or trumpets, where water may collect;
- Improper grouting of the stressing pockets; and
- Using grout material that contains chlorides or other chemicals that will accelerate corrosion of the strand.

Some current encapsulation systems used in aggressive environments incorporate oversize sleeves or trumpets to assist in sealing the tendon at the transition from the sheathing to the anchorage. Since these systems rely on friction rather than on a mechanical connection between the anchor and the sleeve, these sleeves have been seen to work loose and pull away from the anchorage prior to or during concrete placement. Final inspection with reattachment as necessary has been required to achieve the intent of the PTI 1985 and 1993 “Specification for Unbonded Single Strand Tendons” for protection of unbonded tendons.

Damage to the protective grease and sheathing or exposure to moisture during these periods of the construction could adversely affect the future performance of the post-tensioning, but neither responsibility for protection nor specific measures for achieving protection were defined in an industry-wide specification or procedure. These issues are now partially addressed in “Field Procedures Manual,” 2nd edition. As of 1997, the PTI “Specification for Unbonded Single Strand Tendon” is being reviewed, rewritten, and incorporated through a standardization process by ACI/ASCE Committee 423, “Prestressed Concrete.” Concerns regarding shipping and handling are being considered.

3.5—Deterioration mechanisms

In most cases, the corrosion mechanism requires that water and oxygen be present. If a corrosion-inhibiting grease completely covers the strand, and the grease is not affected by water, corrosion generally will not occur. There is always
the possibility that grease will be discontinuous, so corrosion can begin if water and oxygen are available. Corrosion can be accelerated if chlorides are present in the water. As the wire(s) corrodes and the cross-sectional area decreases, the stress in the remaining section rises past its ultimate tensile strength, and the wire(s) fails. Failure by embrittlement of the strand wires can also occur if other aggressive materials, such as nitrates and sulfides, are present. Embrittlement failures can occur without significant loss of cross-section in the strand. It is not necessary, or likely, that all of the wires at a given cross section will be similarly affected at the same time (Fig. 3.5, 3.6, and 3.7).

A strand can burst from the concrete when it fails if the cover is small, usually less than about 3/4 in. (20 mm) (Fig. 4.1 and 4.2). Whether this happens depends on the drape of the tendon, the type of sheathing, and the presence of perpendicular reinforcement between the broken tendon and the surface of the concrete. Occasionally the strand will be released by the anchor and project past the edge of the structure.

Tendons can be subjected to vehicular damage if the concrete cover spalls off or is abraded away. This most commonly happens where the concrete cover is less than 3/4 in. (20 mm) and can be the result of misplacement of the tendon or poor screening and finishing of the concrete. The sheathing is not intended to resist direct contact of in-service traffic and is easily breached under these circumstances. Thereafter, the grease quickly disappears and contaminated water is free to enter the sheathing and cause corrosion.

3.6—Performance record
In general, tendons with extruded type plastic sheathing provide superior corrosion protection when compared with tendons from other unbonded systems and with other types of sheathing. The improvement in performance, however, may also be due to improvements in the quality and application methods of the grease that occurred at about the same time that most fabricators changed to the extruded plastic sheathing system. The extruded type sheath now predominates in the industry, probably as a result of the 1985 PTI “Specification for Unbonded Single Strand Tendons.” From random corrosion incidents, it is clear that care must be taken to ensure that no aggressive materials, including water, enter through the sheath or anchorage.
There are no recorded incidents of sudden collapse of structures using unbonded tendons while the structure is in service. Demolition of these structures has shown that they possess greater reserve of strength than is shown by structures that are not post-tensioned. One characteristic of structures that use unbonded tendons is their propensity to develop significant catenary action even while the main parts of the slabs and beams are being pulverized or broken away. While there is no guarantee against a sudden partial collapse, it is likely that a post-tensioned structure will continue to perform in a ductile manner even when a significant number of the tendons have failed.

CHAPTER 4—EVALUATING CORROSION DAMAGE

4.1—General

Almost any structure can have its useful life extended by the use of appropriate maintenance procedures and suitable repairs, whether the structure is of post-tensioned concrete or of some other material. The cost of repairs can usually be estimated to a reasonable degree of accuracy and a judgment made as to the feasibility of that investment. For concrete structures that do not use unbonded tendons, damage caused by corrosion is estimated by examination of rust-stained or spalled areas. It is assumed, based on spot checking and on past performance of similar conditions, that the reinforcement is serviceable in areas where the concrete is not stained, spalled, or delaminated. That same logic applies to evaluation of deterioration of bonded reinforcing bars in structures using unbonded tendons, but it is of less help in evaluating the tendons themselves.

An appropriate evaluation of the condition of the structure should be performed to minimize the risk of overlooking something significant. Ultimately, decisions will have to be made about the types of repairs to be made and the need to augment the reinforcing system. These decisions are generally based on an engineering evaluation of the data collected, and should be made by an experienced investigator who understands that all latent deficiencies have probably not been identified. As in any repair, the objective is to fix the obvious defects, eliminate the causes of deterioration where practical, slow continued deterioration, and determine requirements for monitoring and future maintenance.

4.2—Condition surveys of concrete

Many references in Chapter 7 provide specific information about the generally accepted methods for evaluation of concrete structures. Typical procedures are reviewed here.

The evaluation should include a careful and well-documented inspection to identify deterioration and distress, and to identify their causes. If available, the original design drawings and shop drawings should be reviewed to identify problems that might be attributed to the design, detailing, or material selection. The drawings should be compared with the findings of the condition survey to assess the in-service performance and determine whether suspected problems are local or might be widespread. It is normal for the as-built conditions to differ from the drawings to some degree. Refer to ACI 364.1R, “Guide for Evaluation of Concrete Structures Prior to Rehabilitation,” and ACI 437, “Strength Evaluation of Existing Concrete Structures,” for additional guidelines.

A crack survey is useful since cracks can be caused by structural distress, insufficient cover to reinforcing bars or tendons, incipient delaminations due to corrosion of reinforcement or embedded conduit, or restrained shrinkage. Notes should be made to document the width and length of the cracks, as well as the presence of leakage, efflorescence, and rust stains.

A survey should be performed to locate delaminations in slabs and other structural members, using methods that are appropriate to the conditions. The delamination survey can be used to estimate the extent and distribution of corrosion damage in the reinforcing system.

Typical materials testing to be performed may include:
- Chloride-ion content testing to determine the depth and intensity of chloride penetration, and to estimate the chloride content of the original concrete. Chloride contamination can promote corrosion of portions of the post-tensioning system in contact with concrete as well
as conventional reinforcing and hardware whose corrosion may damage the concrete protection of the post-tensioning.

- Evaluation of depth of carbonation into the concrete. This reaction decreases the pH of the concrete, thus making corrosion more likely in the presence of moisture and oxygen.
- Obtaining measurements of concrete cover over tendons and reinforcing steel and, if appropriate, correlating these measurements with the chloride contamination profiles or depth of carbonation.
- Copper-copper sulfate half-cell testing to determine whether corrosion activity is presently occurring in the reinforcing system. (These tests may not provide meaningful information on tendons because of the lack of electrical continuity but may indicate likelihood of corrosion activity on anchorage hardware and reinforcing where electrical continuity of steel and contact with concrete are present.)
- Extracting concrete cores for compressive strength testing and for petrographic examination to include air void system analysis. (Tendons should be located before coring begins.)

4.3—Condition surveys of tendons

It is possible to obtain some information about the likely condition of the post-tensioning system by means of the visual inspection. The visual inspection can help to identify specific external evidence of possible internal distress. The visual inspection can also help identify whether there is a pattern to the distress, and may point to the possible extent of a problem.

Cracking of concrete in post-tensioned structures can be the result of problems with the post-tensioning system, but all cracks should be evaluated for their impact on future durability. Following are some examples of cracking patterns that may provide information about the post-tensioning system:

- Cracks that are vertical in beams, and cracks that are perpendicular to the direction of span in slabs, may indicate a loss of post-tensioning force. A structural analysis taking into account possible losses of post-tensioning precompression due to built-in restraint can provide an indication of whether such cracking could be anticipated. If a large discrepancy is found between the calculated service load stresses and the actual observed extent of cracking, then a loss of post-tensioning may be indicated.
- Longitudinal cracks in beams, either on the sides or across the bottom in line with the tendons, may be an indication of water infiltration into the tendons. As the sheathing fills with water and freezes during winter months in colder climates, expansion due to formation of ice can cause splitting of the concrete cover over the tendons.
- Leaking or leaching of moisture through the cracks is an indication of water infiltration. Cracking due to water infiltration may follow the tendons a considerable distance from midspan toward the supports.
- An unintended reversal in the curvature of the tendons at any point in the beam can cause splitting of the beam due to local internal tension in the concrete around the reversal, produced when the tendons are tensioned. These cracks can form without affecting the condition of the post-tensioning tendons but can be a warning of a potential blowout.
- Cracks in the area of beam-column connections may be due to structural action, to a deficiency of reinforcement intended to control bursting stresses in the tendon anchorage zone, to restrained shrinkage, or to a combination of these effects. Water that enters these cracks has easy access to the ends of the tendons and can cause further deterioration.13

Stains on the surface of the concrete can also provide external evidence of possible internal distress due to corrosion of the post-tensioning system. Grease stains on the soffits of slabs, especially at low points of tendon profiles, can be an indication of unrepaired damage to the tendon sheath as well as shallow concrete cover over the tendons. Such grease staining may be accompanied by water stains or evidence of leaching, indicating water infiltration into the slab tendons. Rust staining in the vicinity may be the result of corrosion of the post-tensioning steel or perhaps only the supporting bolsters. A compromise in the corrosion protection of the post-tensioning is indicated and would warrant further testing and exploratory concrete removal, as discussed in Section 4.5.

Visual inspection of exposed end anchorage grout pockets should be performed, especially where exposure to moisture is evident, and correlated with any signs of distress such as described above. Evidence of shrinkage, cracking, debonding, freeze-thaw damage or rust staining coming from the grout pocket may indicate a potential breach in corrosion protection of the anchorage and post-tensioning tendon.

The most obvious external evidence of corrosion damage is the presence of loops of strand sticking out of the structure (Fig. 4.1 and 4.2). Such loops result when the strand breaks and the elastic energy is released suddenly. The strands typically will erupt from the slab at high points or low points in the tendon profile where concrete cover may be shallow, but occasionally only a single wire will burst through the surface of the concrete. Loops formed by this phenomenon can be anywhere from 1 in. (25 mm) to 2 ft (600 mm) high. Rather than bursting from the structure at some point midway between anchorages, the tendon may also protrude out of the structure a distance of several inches or several feet.

Strand breakage can occur without visible disturbance to the concrete, so the absence of strand loops or projections is not to be taken as an absence of broken tendons. Most post-tensioned structures use higher strength concrete (with higher cracking strength) and/or may incorporate (perhaps unintentionally) a significant degree of restraint or redundancy (i.e., below grade construction or two-way slab construction), so it is possible to have as many as 50 percent of the tendons broken in a beam or in an area of slab without obvious distress.
The location at which the tendon or wire has erupted out of the structure is usually some distance away from the location of actual failure. Exploratory concrete removals combined with the removal of the broken tendon will be required to identify the location, nature, and the possible causes of the failure.

The visual review of the structure can serve to determine the distribution of such eruptions in the structure, which can then be correlated to detailing problems and inadequate corrosion protection to help determine the possible appropriate repair and protection measures. It may be useful to attempt to determine the distribution of such eruptions over time as well to see if any trends are apparent in the frequency of tendon failures with the passage of time. Unfortunately, delaying repairs to monitor tendon failures may permit acceleration in the rate of deterioration. Usually it is better to begin repairs as soon as a reasonable condition study has been completed rather than to wait for months or years to gather more information. In a few cases, the rate of corrosion activity has been observed to increase over time, indicating that the rate of tendon failures could increase with time.

4.4—Nondestructive testing

Currently there are no dependable nondestructive testing methods for evaluating the existing condition of tendons inside a structure. The main shortcomings of present test methods include inability to provide information about severity or location of pitting, gross section loss, location of pre-existing cracks or breaks, or breaks and section loss of individual wires in a strand. Several common procedures are reviewed below with comments about their limitations for application to unbonded tendons.

• X-ray techniques are often used to locate embedded reinforcing in concrete, usually for a very limited area due to their expense. X-ray cannot be used to obtain information about pitting, breaks, or loss of cross section of individual wires. The images obtained are fuzzy and do not lend themselves to fine interpretation. Also, other reinforcement can shield the tendon being checked. It is usually impractical to use this method along the length of a tendon to find strand breaks.

• Radar (ASTM D 4748) can also be used to locate embedded reinforcing in concrete. Radar has the same limitations as X-ray but is more adaptable to scanning large areas to locate reinforcement, including tendons.

• Acoustic emission (not to be confused with Continuous Acoustic Monitoring discussed in Section 5.6) is a technique for locating corrosion based on detection of high-frequency sounds emitted at sites of active corrosion, and has been used successfully on steel structures such as storage tanks. For detection of corrosion damage in post-tensioning, acoustic emission is an experimental procedure from which has come no correlation between test results and the condition of tendons. With acoustic emission testing, it is not possible to determine the nature of the corrosion damage (surface attack versus pitting), the location of the damage (strand versus anchorage, or where on the strand), or whether wires or the entire strand are broken.

• Ultrasonic pulse velocity (ASTM C 597) is useful in determining certain properties of concrete, but it has not proven to be adaptable to determining the integrity of tendons.

• Half-cell potential (ASTM C 876) can be used to determine the presence of corrosion activity in bonded reinforcing bars and in anchors and strands that are in contact with concrete. Half-cell testing does not reliably locate corrosion along the length of a tendon since the grease and sheathing will electrically insulate the strand. This method is redundant where visible evidence of corrosion is present.

• Pachometer devices will locate greased and sheathed tendons as well as bonded reinforcement. They are useful in estimating the thickness of concrete cover over anchorages and along strands, and the thickness of grout protection of strand stubs at stressing pockets. Pachometer readings for anchorages and stubs as well as along strands may require calibration by comparing against actual physical measurements of concrete cover thickness.

• Vacuum testing of grouted stressing pockets can assess the quality of the grout and its bond to the sides of the block-out. Such testing can help quantify the effectiveness of the grout plug in preventing water from infiltrating the post-tensioning tendon.

• Impact-echo testing can be used to supplement a delamination survey to more accurately determine the delaminated areas. It can also be used to determine crack depths, voids, and honeycombing. Impact-echo provides no information regarding the strand condition.

4.5—Exploratory concrete removal

Usually it is necessary to chip into the concrete to inspect the strand, grease, sheathing and anchorages, and to assess the extent of corrosion activity of the bonded reinforcement.

This procedure is slow and is limited to parts of the structure that are accessible. For example, anchorages for beam tendons are frequently embedded in columns and are not readily exposed. Occupied structures present special problems since concrete removal is noisy and dirty. The number of inspection opportunities may be small compared to the total number and lineal footage of tendons in a structure, so the information obtained may not be statistically meaningful. Exploratory concrete removal can be used to calibrate or confirm results from nondestructive tests.

Repair of concrete removals should restore grease, sheathing and concrete protection. Refer to ACI 546, “Guide to Repair of Concrete.”

4.6—Exposing tendons

Exposing parts of tendons is most convenient where concrete cover is shallowest, such as at low points or high points in the tendon profile. In most structures, it is preferable to expose the strand at low points in the profile for several reasons:
Exploratory concrete removals undertaken in the area of intermediate anchorages are useful, particularly where leakage has occurred through construction joints. The stressing side of the joint is the side that can be exposed without compromising the integrity of the tendon; the concrete on the other side is under compression and should not be disturbed.

Exploratory concrete removals should not be started without a detailed plan of action. A specification should be prepared that outlines safety precautions and acceptable procedures for demolition, encapsulation repairs, and concrete patching. Individuals performing the chipping must be educated about tendons so that they understand what they are looking for and the consequences of breaking a strand. They should use slow, careful means of concrete removal, and use tools that are appropriate to the task. Refer to ACI 546.1R, “Guide to Repair.” As previously discussed, broken strands can erupt vertically through the top or bottom surfaces of the concrete, or horizontally through the anchors.

4.7—Strand removal

Strands can be removed for inspection, but it is unusual to cut a live tendon for this purpose. It is more common to extract part of a strand that is known to be broken, usually one that has erupted from the concrete. It is usually possible to remove the strand between the eruption and the break, but considerable effort is sometimes required. Laying the strand piece on the deck is a simple way to find the location of the break. Once removed, the strand can be examined to determine the condition of the grease, the frequency and severity of corrosion, and the type of failure of the wires.

Signs of corrosion are sometimes subtle and look more like stains than rust, so it is important that the strand be examined closely by someone experienced in this type of evaluation. The failed end of a strand can be submitted to a metallurgical testing laboratory for an analysis of the broken wires. Microscopic examination of the external and internal wires of the strand can be performed to look for cracking, pitting, and corrosion.

Chemical residue present on the wires at the failure location can sometimes be identified and related to the failure. Testing of physical and mechanical properties of the strand can be performed on strand taken some distance from the break to determine tensile strength, elongation, and bend testing.

4.8—Other testing and investigative procedures

Lift-off testing (pulling the strand loose from an anchor to measure its effective force) can be used to check tension in the strands, but it is difficult to do with single strand tendons in a completed structure. Since the strand has been cut close to the wedges, special tools are required to grip the strand stub sufficiently to pull on the strand and loosen the wedges. Corrosion increases the locking force of the wedges and weakens the stub, making the process difficult to execute. Welding an extension onto the strand is not feasible because of the likelihood of permanently damaging the anchorage and the strand. When barrel anchorages have been used, it
may be possible to clamp onto the barrel and perform a lift-off of the tendon in this manner. However, care should be taken since barrels are generally heat-treated, making them hard, difficult to grip, and non-weldable.

Lift-off tests will not provide any information about the amount or distribution of corrosion on the strand, nor will they give an indication of the loss of ultimate strength. It will only provide knowledge that the strand had the ability to sustain the force held by the jack after the wedges came loose. Typically, lift-off tests do not exceed the tension originally used to set the wedges. Sometime the strand will break just after the wedges come loose, indicating that strength was lost before the test. The logical next step is to pull the strand out of the sheathing for inspection.

An alternative to lift-off testing is to check for tension in the strand using equipment that analyzes the vibration characteristics of the strand under tension. This equipment is adapted from that used to check tendons in cable-stayed bridges and in similar applications.

A portion of the strand is exposed, typically about 2 ft (600 mm), and the ends of the exposed length are choked to provide node points. An accelerometer is attached to the exposed strand and the strand excited by means of tapping. The vibration is transmitted from the accelerometer to computing equipment to determine the tension in the strand as a function of the frequency of vibration. As with lift-off testing, checking tension in this manner does not provide detailed information about corrosion on the strand.

Load testing of slabs and beams in accordance with ACI 318 under “Strength Evaluation of Existing Structures” provides no detailed information about the condition of the individual tendons. A significant number of tendons could have failed without being detected by a load test. The period of time subsequent to the testing for which the test results are valid is subject to a great deal of uncertainty, and ACI 318 provides no guidance for estimating the remaining service life. Also, load testing is expensive and disruptive. Therefore, although a load test can confirm that the tested part of the structure has adequate strength at the time of the test, this method has limitations when applied to structures with suspected or known corrosion damage to unbonded tendons.

CHAPTER 5—REPAIR SCHEMES AND METHODS

5.1—General

Prior to launching a repair effort of any kind, it is prudent to determine the causes of corrosion damage and possible preventative measures which may be taken. Incorporating measures that address causes of deterioration can make a repair program more effective for extending the useful life of the structure. Whether the need for tendon repair is critical should be assessed by analyzing the structure to determine if sufficient reserve capacity exists to adequately support the design loads after a few tendons are lost. In some cases it might not be necessary to replace every broken or damaged tendon, or it might be reasonable to use smaller diameter strand to replace the strands that were removed. The smaller diameter tendon may be of a higher strength if required to maintain adequate precompression or ultimate strength. Where the type and condition of the strand and sheathing permit, it is possible to replace the old strand with one of the same diameter. If analysis indicates that a reduction in load carrying capacity below code limits, and especially below service load levels, has occurred, then evacuating or shoring the affected portions of the structure may be required in advance of repairs.

When tendon corrosion has progressed to the point where the strength of any part of a structure has been reduced to an unacceptable level, repairs should be considered to correct the deficiency. The type of repair will depend on the extent (or assumed extent) of the corrosion damage and the resulting loss of strength. If the damage is known to be minor, and its location is easily identified, the remedy may consist of simply repairing and protecting the existing tendons. Most probably a variety of conditions will be encountered, some of which might require the removal and replacement of entire tendons. Engineering assessment of the proposed repair methods and sequence is needed to determine how the repair process may further affect the integrity of the structure.

When the actual or assumed extent of damage makes localized repairs impractical, the structure may need to be strengthened with an externally applied structural system consisting of post-tensioned tendons, structural steel sections, or concrete members. External post-tensioning systems have been designed to completely replace the original tendons in some structures. These various approaches are discussed in the following sections. When external strengthening is not feasible, partial or complete demolition and replacement may be necessary.

5.2—Existing tendon repair

Visible evidence of tendon corrosion, such as that which may be seen on exposed tendon anchorages where the stressing pocket has not been grouted, should be repaired as quickly as possible. The exposed part of the anchorage should be cleaned by abrasive blasting or equivalent, coated with a rust-preventive paint or other corrosion protection, and the stressing pocket properly filled in accordance with PTI “Specification for Unbonded Single Strand Tendons.”

In a structure where tendon corrosion has been diagnosed, appropriate means of stopping or slowing the rate of corrosion in the existing tendons should be applied. Eliminating water intrusion is of primary importance, so concrete repairs should be made and cracks should be sealed. Random cracks can be routed and sealed, but consideration should be given to the application of a waterproofing membrane, possibly incorporating a wearing surface as appropriate, if extensive cracking is present or if there is widespread deficiency of protective concrete cover throughout the structure or a portion of the structure. Refer to ACI 224.1R, “Causes, Evaluation and Repair of Cracks in Concrete Structures.”

5.3—Strand replacement

When a strand has been inadvertently cut or damaged, or when corrosion damage is known or believed to be localized,
repairs are often made by replacement of part of the strand between anchorages. The old anchors are reused, and the old wedges are never unlocked. The damaged section of strand is cut away and a new piece of strand spliced onto the ends of the original strand using couplers. The simplest couplers are short tubes with two sets of wedges, while others allow the strand to be stressed at the splice location (Fig. 5.1, 5.2, and 5.3). The original anchorages do not need to be exposed when an in-line stressing coupler is used, and the stressing process is a limited form of proof test. Some designers specify that for this type of repair the strand must be stressed as usual, to 80 percent of ultimate before lock-off. Others acknowledge that long-term losses have already occurred and require that a lower initial tension be applied; the repaired tendon stressed in this manner will have approximately the same effective stress as the original tendons.

Replacing a strand for its full length and using the original anchors is also possible, but dislodging the old wedges is sometimes difficult and the anchors can be damaged in the process. It is usually advisable to replace the anchors with new ones since this gives the opportunity to improve the system’s durability. Once free of its anchorage, strand extraction is normally not difficult. In some cases a jack can be used to pull the strand out, but this method, while reliable, is slow. Usually the loose strand is pulled out by hand or with the assistance of a come-along or a vehicle. If binding is a problem and jacking is required, the applied stress should not exceed \(0.80 f_{pu}\). If the tendon cannot be moved, then the cause of the binding has to be located (usually by using the achieved elongation to calculate the distance back to the bound location) and removed.
Prior to inserting the new strand in the old sheathing remaining in the structure, moisture inside the sheathing should be removed (i.e., pulling rags through the sheathing until dry). A quantity of grease should be pumped into the sheath ahead of the new strand. The goal of this procedure, or any alternative procedure, should be to fill all voids inside the sheathing so that water cannot again enter into the sheathing and start a new cycle of corrosion. The new strand used can be either bare, or pre-greased and encased in a plastic sheathing from which it is pulled out to be inserted. In any tendon, only one type of grease should be used. In general, pre-greased strand, preferably one covered with an extruded sheathing, is desirable for use as replacement strand. The grease on a factory-greased strand is applied under pressure during fabrication so the coverage and penetration between the wires is better than can be achieved if the grease is applied in the field to a bare strand. The sheathing should be resealed at all inspection and repair locations to provide encapsulation of the strand.

Epoxy-coated strand meeting ASTM A 822/A may be considered for strand replacement. A smaller diameter strand must be used to accommodate the thickness of the coating (30 to 40 mils, or 0.7 to 1.0 mm). Special anchorages and wedges are required for use with epoxy-coated strand, so existing anchorages have to be replaced. There have been problems in the past with slipping of strand through anchorages due to inadequate bite of the chucks through the coating.

New strand is either pulled into the existing sheath, usually by first threading through a smaller diameter strand that has been welded to the end of the new strand, or pushed by means of a jack that has been reversed and tied off to provide the necessary reactive force. The strand buckles more readily at the jack as the embedment friction increases, so shorter strokes become necessary, but most strands can be replaced by this means.

Replacing strands is relatively expensive, intrusive, and time-consuming. However, it can be an effective remedy when architectural considerations are important because there is no change in the appearance of the structure. This work should be performed by experienced contractors who understand all aspects of proper corrosion protection for tendons.

5.4—Tendon replacement

In some structures or portions of structures where many strands are damaged in many places, it may be more cost effective to abandon some or all of the existing tendons and install new ones, rather than to replace strand and anchorages. This is generally practical only for slabs or wide beams since it requires that either intermittent or full-length slots be cut in the concrete. The procedure, termed “trenching,” as shown in Fig. 5.4 and 5.5, is a patented process. Detailing at new anchorages and along the length of the new tendon must ensure that the compression, uplift, and downward forces are transferred from the new concrete, in which the new tendons are encased, to the existing concrete.

5.5—External post-tensioning

Externally applied post-tensioning systems can be effectively used to strengthen large portions of existing structures. External post-tensioning systems have found application in the retrofit of a wide variety of framing systems, including two-way flat plates, flat slabs, one-way slabs, beams, and girders.

Typically these systems consist of adding straight tendons along the tensile zone of existing slabs or beams, or building a tendon truss under or alongside the existing slab or beam. The lower (tension) chord of the truss is the post-tensioned tendon or tendon group. The vertical truss member is usually a structural steel shape (normally a tube) with a bearing plate which bears on the soffit of the existing structure and applies the beneficial upward force. The top (compression) chord of the truss is the existing structure itself, to which the post-tensioning anchors are attached (Fig. 5.6 and 5.7). Advantages of external post-tensioned retrofits include:

• The ability to apply large upward forces to the existing structure with minimal headroom requirements;
• Little or no interference with existing utilities;
• Minimal disruption of the existing function of the
building; and

- Possible cost savings over methods such as tendon replacement.

One possible disadvantage of externally applied post-tensioning is that it is visible and in some cases (when it is not hidden by a ceiling or other feature) can change the architectural appearance of the structure. Fire protection requirements must be in accordance with the governing building code. Corrosion protection is also required.

5.6—Continuous acoustic monitoring

A proprietary acoustic monitoring system has been recently developed that provides continuous monitoring for post-tensioned structures. When a strand or wire fails, there is a sudden release of energy. Sensors mounted at various locations on the structure detect the acoustic response transmitted from the location of the break. Information collected by the sensors is recorded by terminals at the site and downloaded to a central computer where the information is analyzed to determine the probable location of the break.

The monitoring system does not determine the condition of the post-tensioning system at the outset of installation, but rather provides a record of the number and location of strand breaks in the structure subsequent to installation. The system is intended to assist with managing evaluation and repair of structures with known or suspected tendon corrosion problems. The goal is to minimize repair expenditures and still ensure structural integrity.

5.7 — External non-prestressed reinforcement or support

Strengthening can be accomplished with non-prestressed elements such as structural steel or reinforced concrete beams and columns. If architectural requirements permit, the most cost-effective method to strengthen an existing structure may be to add a permanent vertical support such as a column. Most structures, however, cannot accommodate additional columns, and require some form of flexural retrofit (beams and girders, or bonded external reinforcement) that carry loads to existing columns. If non-prestressed retrofits are designed to share loads with the existing floor system (that is, the retrofit is designed only for the strength deficiency, not as a total replacement for the flexural and shear strength of the floor framing), then analysis and design must be performed based on deflection compatibility to confirm that stresses in the existing concrete remain within allowable limits and/or that deflections remain within acceptable limits. Pre-jacking the retrofit system against the existing structure can cause it to share in supporting existing dead loads as well as applied live loads, and can reduce deflections in the system. Composite action between the retrofit and existing structure can also be used to promote load sharing.

Installation of external non-prestressed reinforcement or supports can involve major disruption in use of the building and may require rerouting of existing utilities, depending on the type and details of the external reinforcing system.

5.8—Total demolition and replacement

Demolition procedures for post-tensioned buildings (in their entirety) are similar to those for non-prestressed concrete buildings, but the collapse mechanisms are different. The tendons cause significant catenary action to develop; this can be helpful or detrimental, depending on the procedures and equipment used.

Generally, it is neither necessary nor advisable to cut tendons before demolition begins, since the tendons can be made to assist in controlling the collapses. Selective removal of concrete from parts of slabs and beams is usually needed, as is the weakening of walls, stair towers and columns. Precautions are sometimes needed to prevent broken tendons from exiting the edges of the structure and damaging adjacent property, but some studies have shown this to be an unlikely event for single strand systems. The amount of projection will depend on many factors, such as the type of sheathing, condition and quantity of grease, amount of drape, and so on. The potential for tendons exiting the structure should be evaluated for each project.
CHAPTER 6—SUMMARY

During the past 40 years, unbonded tendons have become a significant form of reinforcement for concrete building structures. Tendon types have changed in response to an industry-wide recognition of the vulnerability of the earlier tendons to corrosion damage. As the causes of tendon corrosion became known, specifications by PTI and ACI were changed to implement requirements for the materials, fabrication, installation, and design details to protect the tendons from deterioration. Adoption of requirements to improve the durability of unbonded tendons began only 10 years ago when PTI published its first comprehensive specification, and it took a few years for this document to be recognized by the design and construction community. Structures using single strand unbonded tendons that had poor quality grease, inadequate grease filling, water in the sheath, and poor corrosion protection details are more likely to suffer strand failures. Concern for the evaluation and repair of these unprotected and deteriorating structures was the primary impetus for preparing this document.

The most common type of unbonded tendon to be used for buildings during the past 30 years has a single 7-wire strand with cast anchors and two-piece wedges. Techniques for repairing these tendons are known by many contractors and engineers, as is the cost of doing this work. While there are many methods available to evaluate the condition of concrete and bonded reinforcement in a post-tensioned structure, there is no method available to determine the amount that an unbonded tendon has been weakened by corrosion.

Not every building can be economically repaired by replacing all, or parts of, the tendons. Sometimes it is best to disregard the existing tendons and replace them with an externally applied system. This method is adaptable to many circumstances and can be used for beams as well as for slabs.

Total demolition is the last resort for a deteriorated structure of any type. The decision to demolish a post-tensioned structure should be made after a realistic assessment has been made of the condition of the concrete and the bonded reinforcement, and not just on an estimate of the condition of the tendons.

CHAPTER 7—REFERENCES

7.1—Recommended references

The documents of the various standards-producing organizations referred to in this document are listed with their serial designation. Since some of these documents are revised frequently, the user of this report should check for the most recent version.

American Concrete Institute

American Concrete Institute

201.1R Guide for Making a Condition Survey of Concrete in Service
201.2R Guide to Durable Concrete
222R Corrosion of Metals in Concrete
224R Control of Cracking in Concrete Structures
224.1R Causes, Evaluation and Repair of

ASTM

B 117 Standard Test Method of Salt Spray (Fog) Testing
C 597 Standard Test Method for Pulse Velocity Through Concrete
C 876 Standard Test Method for Half-Cell Potentials of Uncoated Reinforcing Steel in Concrete
D 4748 Standard Test Method for Determining the Thickness of Bound Pavement Layers Using Short Pulse Radar

Post-Tensioning Institute

Field Procedures Manual
Manual for Certification of Plants Producing Unbonded Single Strand Tendons
Post-Tensioning Manual
PTI Committee for Development of the Field Procedures Manual for Unbonded Single Strand Tendons
Specifications for Unbonded Single Strand Tendons

Prestressed Concrete Institute

PCI Post-Tensioning Manual (1st ed.)

Transportation Research Board

NCHRP Syntheses No.140 Durability of Prestressed Concrete Highway Structures

NCHRB Report 313 Corrosion Protection of Prestressing Systems in Concrete Bridges

Other Publications


Precast/Prestressed Concrete Institute (Jul.-Aug. 1993), "Guideline for the Use of Epoxy Coated Strand," *Precast/Prestressed Concrete Institute Journal*, V. 38, No. 4.


**Referenced publications are available from the following organizations:**

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

**ASTM**
100 Bar Harbor Drive
West Conshohocken, PA 19428

**Precast/Prestressed Concrete Institute**
1717 West Northern Avenue
Suite No. 218
Phoenix, AZ 85021

**Post-Tensioning Institute**
1717 West Northern Avenue
Suite No. 218
Phoenix, AZ 85021

**Transportation Research Board**
National Research Council
2010 Constitution Avenue, NW
Washington DC 20418

**7.2—Cited references**


