Some Recent Corrosion Embrittlement Failures of Prestressing Systems in the United States

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FOREWORD

This paper is based on a paper presented by the authors at the FIP Symposium on Stress Corrosion Cracking of Prestressing Steel in Madrid, Spain, September, 1981.

Having been asked to report on the incidence of embrittlement failures of prestressing steels in the United States in the recent past, the authors have highlighted this type of failure in this paper. Evidently, any failure of a prestressing steel tendon due to corrosion of any kind is undesirable. The need to exercise extreme care to protect prestressing steels in any application cannot be overemphasized.

For the purposes of this paper, corrosion, or ordinary corrosion, is an electrochemical phenomenon in which the steel is affected by a particular environment, resulting in a measureable loss of metal. The failure of a given steel element occurs when the loss of material is so great that the remaining cross section is no longer capable of carrying the applied load. In failure, however, the metal generally still exhibits its normal ductility.

On the other hand, stress corrosion cracking and hydrogen embrittlement are distinctly different manifestations of corrosion of prestressing steel, stressed generally to over 50 percent of its ultimate strength. Without attempting to belabor the metallurgical definition of and distinction between the two phenomena, the noticeable effect in either case is a drastic loss of ductility on the part of the affected steel. Failures, when they do occur, are typically brittle, often with little or no loss of metal in the steel element, with possibly no structural warnings.

INTRODUCTION

Corrosion and, in particular, stress corrosion cracking and hydrogen embrittlement of prestressing steels, has always been a concern of engineers in the United States and indeed throughout the world. As a contribution to the understanding of the problem of corrosion of prestressing steel, and in order to put the magnitude of the problem in proper perspective, particularly from the viewpoint of the structural engineer, the authors have attempted to gather information on cases of corrosion of prestressing steel which may have occurred in the United States in the past 5 years or so.

To that effect, a survey was made in an effort to reach as many individuals as possible, throughout the country, who might have knowledge of such cases. This paper presents the results of the survey, a general discussion of the information received, and a description of four cases of embrittlement of prestressing tendons on which sufficiently detailed information was available. This report is also an update of a previous paper prepared by Schupack in 1978.[1]

SURVEY FORMAT

The limited survey consisted in sending more than 250 letters to selected individuals throughout the United States who, in the opinion of the authors, should be aware, collectively, of most cases of corrosion affecting prestressing steel. These individuals in-

Synopsis

A limited survey was made to gather as much data as possible on incidents of corrosion of prestressing steel which may have occurred in the United States during the past 5 years. From the authors' experience and information gathered from close to 100 responses to the survey, 50 structures were found in which tendon corrosion has occurred.

Corrosion incidents were usually found to be associated with poor details and/or execution, generally in the presence of an aggressive environment.

Considering the fact that about 200,000 tons (approx. 180,000 metric tons) of prestressing steel per year are used in the United States, the number of incidents reported is small, particularly since about 40 percent of them have occurred in post-tensioned parking structures exposed to high concentrations of chlorides from deicing salts.

Of the 50 corrosion incidents reported, 10 cases of probable brittle failures related to stress corrosion or hydrogen embrittlement were cited. It is likely that a few more may have occurred.

In most cases, the exposure of the prestressing steel to an aggressive environment, particularly with sulphides present, has triggered the loss in ductility. Those incidents involving embrittlement, in which more details were available, are described herein.

Recommendations are put forth whereby corrosion embrittlement failures of prestressing steels can be avoided.

cluded practicing structural engineers and representatives of state highway departments and other government agencies, prestressing steel and post-tensioning materials suppliers, contractors, pipe manufacturers and tank constructors. Enclosed with the letter of explanation and request for information was a detailed questionnaire which could be returned with the pertinent data.

Also, through the valuable cooperation of the Prestressed Concrete Institute, which included it in one of its Newsletters, the questionnaire reached all members of the PCI, including Company, Associate and Professional Members.

The limitations of the survey must be recognized. Even though about 50 mentions of corrosion were included in the nearly 100 responses received, it is unlikely that every case of embrittlement was reported. Unfortunately, not every key individual contacted nor, in particular, every prestressing materials supplier, responded.

SURVEY FINDINGS

General

The types of prestressing steels most prevalent in the United States have been: seven-wire stress-relieved strands (ASTM A416), used exclusively in pretensioning; the same strands, cold drawn and stress-relieved wires (ASTM A421), and cold stretched and stress-relieved bars (ASTM A722), making up post-tensioning tendons, both bonded and unbonded; and cold drawn wire, used to prestress cylindrical wire wound pipes and tanks.

All types of prestressing steels have been affected by corrosion at one time or another. Actual incidents of corrosion can generally be related to ill conceived details and/or poor execution, usually associated with an aggressive environment. In a few instances of steel failures, these could be related to manufacturing defects in the prestressing steel and not to corrosion.

The majority of reported cases of corrosion involved unbonded tendons. In the early days, these tendons were mostly grease coated and paper wrapped. The grease was sometimes not applied continuously nor uniformly. The paper sheath was also susceptible to possible tears which could contribute to further loss of the protective grease. For many years now, plastic sheaths enclosing the greased tendons are universally used. This has significantly improved the behavior of unbonded tendons, although improper detailing of an adequate protection system in the area of the anchorages and/or deficient methods of installation have been found to result in some corrosion problems.

No cases of corrosion of properly grouted tendons have been reported. In bonded construction, corrosion of tendons has been observed only when tendons have been found to be poorly grouted or not grouted at all. Cases have been reported where grout subsidence or sedimentation caused voids at high points.[2] When this happens, it is possible that a corrosive substance may find its way into these unfilled spaces. On the other hand, the actual behavior of properly grouted tendons in chloride environments has been surprisingly good. This parallels the findings on post-tensioned test beams which have been exposed in the tidal zone of the northeast coast of the United States for up to 21 years.[3]

Prestressed Pipes and Tanks

Several cases of corrosion of prestressing steel in wire-wound pipes and tanks were reported in general terms. The principal factors causing these failures were deficiencies in manufacture or in construction, resulting in insufficient or damaged mortar coat over the wires. The actual number of installations affected is not available although, from the general information received, the percentage of corrosion problems appears to be much lower for newer structures as workmanship and construction techniques have improved throughout the years.

Parking Structures

The most widely spread corrosion problem today[4] appears to exist in post-tensioned parking garage decks, particularly in areas of the country where chloride salts are used for winter deicing. This problem is basically the same as that of corrosion of reinforcing steel so prevalent in normal reinforced concrete parking decks and bridge decks. On all the approximately 15 post-tensioned concrete parking structures known to have problems with corroded tendons, the concrete had a chloride ion content, by weight of concrete, as high as 0.9 percent.

Generally, the chloride content varies from a high percentage at the top of the slab to a lower percentage at the bottom. This gradation is believed to be caused by salts being deposited externally on the surface as differentiated from chloride placed integrally with the concrete. Based on what has been reported and on the authors' own experience, no stress corrosion cracking of prestressing steel, but rather ordinary corrosion with significant loss of metal, has occurred in parking decks subjected to chloride.

Nuclear Containment Vessels

A rare opportunity of observing tendon performance exists in the nuclear power industry. Regulatory Guide 1.35, issued by the United States Nuclear Regulatory Commission, requires tendon inspection to take place 1, 3 and 5 years after the initial integrity test, and every 5 years thereafter. On vessels with unbonded tendons (protected by corrosion inhibiting grease), each inspection is performed on 2 to 4 percent of the tendons and includes:

- Removal of protective grease and visual examination of end anchorage hardware and surrounding concrete.
- Liftoff reading, detensioning and reloading tendons.
- Removal of one wire or strand from two or three tendons for visual examination for corrosion and to provide samples for tensile testing.
- Visual examination and chemical analysis of grease.

No unacceptable conditions have been reported on any of the approximately 30 prestressed containment vessels built to date in the United States. There have been a few instances of broken wires which were traced to material flaws or improperly formed buttonheads. None was attributed to corrosion. Potentially adverse conditions have been found in some tendons during surveillance, where water had seeped into the conduits. However, in no case (the oldest tendon was 10 years) has detrimental corrosion or embrittlement been found.

The two existing grouted containment vessels have been monitored by various means and nothing has been revealed to indicate any degradation of the tendons.

BRITTLE FAILURES

Of the total of approximately 50 structures in which corrosion of prestressing steel is known to have occurred, 10 cases involved brittle failure of one or more tendons. These affected only isolated areas in each structure and have not been of a catastrophic nature.

Most of these brittle failures have occurred in post-tensioned unbonded construction in commercial structures.

Five of these incidents can be definitely identified as some form of stress corrosion. The others are poorly reported and probably have been minimally researched. No known case of brittle corrosion has been revealed in pretensioned construction. The few incidents of corrosion in pretensioning have been generally related to inadequate concrete cover and/or poor concrete quality.

Four incidents of embrittlement failure, where more detailed information was available, are described below. The metallurgical information given was extracted from private reports by various consultants who are not identified to preserve the anonymity of the structures involved.

Case No. 1—Elevated Building Slab over a Parking Area, Post-Tensioned in Two Directions with Unbonded Greased and Paper Wrapped 0.6-in. (15 mm) Strands

Each one of two similar structures, part of a building complex, consists of a 230 x 330-ft (70 x 100 m) cast-in-place flat plate, 9.5 in. (240 mm) thick, post-tensioned in two directions. Constructed about 10 years ago, each slab is used as a building platform over a parking area, to support low-rise multiple dwellings. After the prestressed slab was constructed, wooden framed buildings were erected above. The slab, made of lightweight concrete, is supported on concrete columns which, in turn, are supported by concrete pile foundations.

The 0.6-in. (15 mm) monostrand tendons were protected by a coating of corrosion inhibiting grease, inside a sheath formed by spirally wound kraft fiber paper.

The 0.6-in. (15 mm) strand had a minimum ultimate strength of 270 ksi (1860 MPa) and met all the requirements of ASTM A416. The strands were initially stressed to 80 percent of ultimate to overcome friction and then relaxed and locked off at 70 percent. With time, the stress level in the strands, after losses, was expected to be about 60 percent of ultimate. Bundles of two to four tendons were spaced 18 to 42 in. (450 to 1060 mm) apart and were tied to supporting metal chairs and to reinforcing steel and other tendons in the transverse direction.

Some tendons failed about 40 days after stressing and additional sporadic failures continued to occur at reducing frequencies. Of the approximately 1200 tendons in the most affected slab, about 4 percent have failed to date. The failed 0.6-in. (15 mm) tendons have generally been replaced with ½-in. (13 mm) strands.

Failures of a distinct brittle nature tended to occur more frequently over columns, in areas of high negative moments and where the geometry of the tendons resulted in the sharpest curvatures. Samples, taken from various points along the length of a failed tendon removed from the structure, were tested in tension. Test results ranged from 100 percent of specified ultimate strength and a typically normal elongation at ultimate in excess of 6 percent to less than 85 percent of ultimate and under 1 percent of elongation. These differences sometimes occurred between samples removed from within 15 ft (5 m) of each other.

Concrete core samples removed from the slab were shown to contain 180 parts per million of sulphide sulphur. (Analyses of 500 to 900 parts per million were also obtained, but were subject to question.) Some differences in potential were observed between tendons in the slabs where failures occurred. Possibly related to this, about 90 percent of the failures also occurred in tendons arranged in the east-west direction, in preference to the north-south direction.

Samples of lightweight aggregate were found to be quite variable. Some

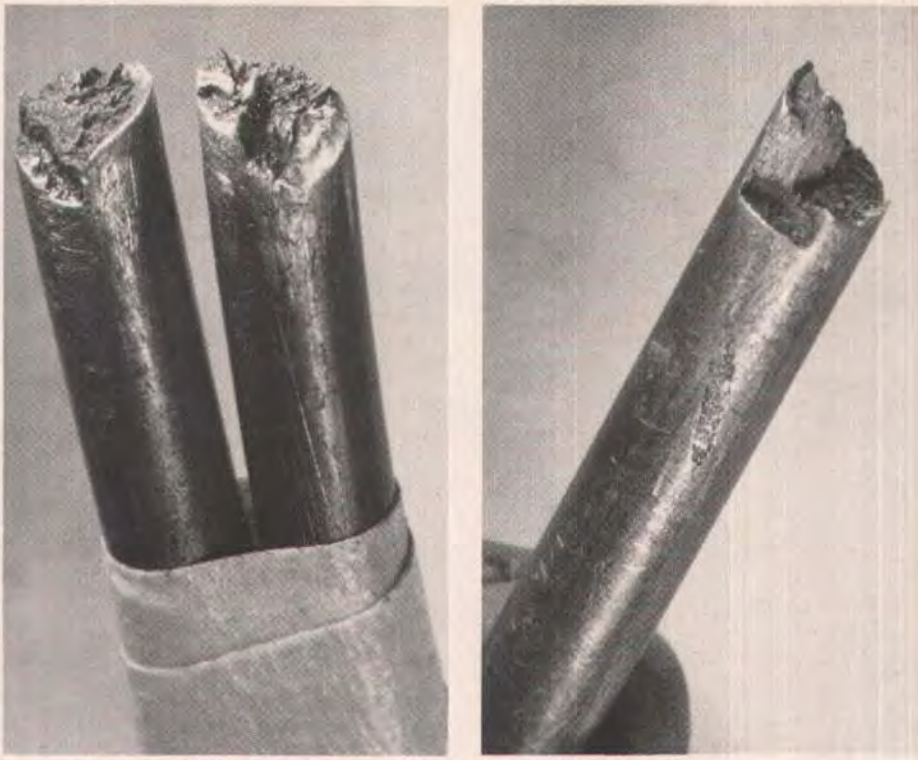

Fig. 1. Typical failure mode without surface attack.

aggregate particles were reddish in color, others brown with a darker or even black interior. The aggregate varied in hardness and in the size of its pores. Some samples of aggregate, with dark interiors, gave an odor of hydrogen sulphide when moistened with hydrochloric acid solution. The aggregate generally had a hard glazed exterior, but in bulk it showed dusting, and there were broken pieces of aggregate present.

Paper-wrapped tendons present a corrugated exterior to the concrete surrounding them. When tendons are tensioned, the movement of the strand, particularly with a small radius of curvature, may cause it to cut through the paper tape and crush aggregate protruding into the corrugation. This would expose high strength steel directly to pulverized aggregate. A concrete test beam was made and draped tendons, stressed to 80 percent of ultimate strength, were moved about 9 in. (230 mm) under sustained load. The beam was then sectioned and it was found that some of the aggregate had been crushed.

Metallurgical examinations, by many different laboratories, failed to give definitive answers. Some samples of broken wires had enough evidence of corrosion to warrant labeling the failures stress corrosion cracking. Other samples did not show evidence of corrosion pits or surface attack. Fig. 1 shows a typical failure mode without visible surface attack. Many cracks, primarily oriented in the transverse direction, formed near the point of failure, as can be seen in Fig. 2.

An interesting feature of some of the brittle failures was the fracture occur-

Fig. 2. Etching in acid disclosed many surface cracks.

ring at an angle of about 45 deg. to the longitudinal axis of the wire. It has been shown in the laboratory[5] that high strength steel wire stressed in H_2S solutions forms cracks which tend to be aligned at 45 deg to the direction of the tensile stress. The preferred crystallographic orientation of the steel in the wire causes this unusual crack orientation. It was, therefore, assumed that hydrogen sulphide embrittlement of the high strength steel most likely caused these delayed failures.

With or without a better explanation for these tendon failures, it is evident from this experience and the one cited later (Case 2 below), that H_2S must be kept away from the prestressing steel. It is obvious that a sheath that will not permit the sulphide penetration would be a solution. However, with the exigencies of construction, it is necessary to avoid altogether the presence of sulphides in concrete containing prestressing steel.

As a matter of interest, isolated failures due to embrittlement under very similar conditions to those described above are also known to have occurred on at least one, possibly two other structures (information is not sufficiently precise). Samples taken from different locations along the length of the same strand, after removal from the structure, again showed widely different behavior under standard tensile tests.

It must be realized that most, if not all, unbonded monostrand tendons installed in the United States today are encased in plastic tubing. Experiments and experience have confirmed that such a sheath is less likely to be damaged during construction than paper wrapping. Its smooth interior and exterior surfaces make it less susceptible to be cut by the strand moving under tension. This does not insure, however, that a plastic sheathed monostrand tendon is trouble free. The area near the end anchorage remains vulnerable, as its metal components remain exposed and must be effectively protected against corrosion. It is not unusual that the plastic sheath is cut short and the strand itself may lack adequate protection.

Several cases of corrosion, generally with heavy pitting and loss of metal, have been reported, where end anchorage pockets have been improperly filled or not at all, allowing water to gain

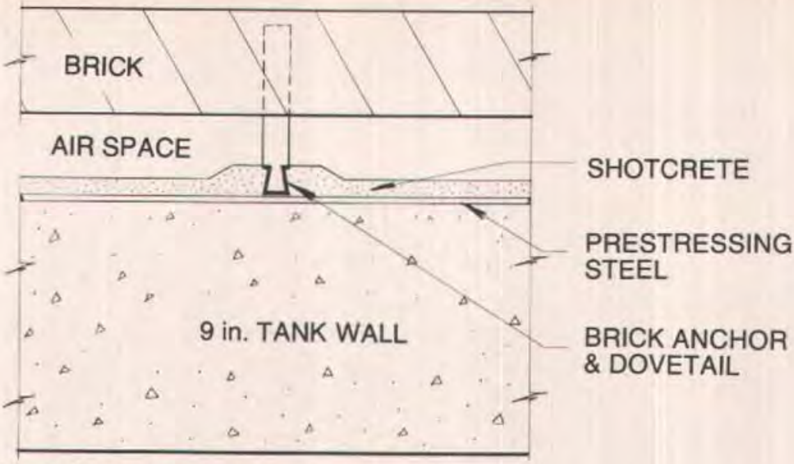

Fig. 3. Horizontal section through tank wall.

access and accumulate inside the plastic sheathing. Even cases of failures due to embrittlement (see Case 4 below) have been known to occur. This suggests the possible coincidence of a locally exposed strand because of a damaged sheath, together with electrical potential differentials between these exposed areas and the exposed end anchorages or other tendons.

Case No. 2 — Sewage Digesters

Four prestressed concrete digesters, which are 80 ft (24 m) in diameter and 32 ft (10 m) high, were built in 1950, using the wire wound system in which the wire is pulled through a die to induce the prestress. These tanks manifested two distinctly different corrosion problems, ordinary corrosion and stress corrosion cracking, both resulting from inadequate shotcrete protection caused by poor details. Similar conditions have been observed before in other sewage tanks, including the Owl's Head tank in Brooklyn, New York, which suffered a catastrophic failure in 1961.[6] This information is presented because of the embrittlement corrosion found in 1980.

The brick anchor detail of these tanks (see Fig. 3) was the same one which had been used in the Owl's Head tank. Apparently to facilitate construction, the galvanized brick dovetail anchor was wired directly onto the wrapped wire and then the shotcrete was placed. This whole problem would have been avoided if ¼ in. (6 mm) of shotcrete separated the dovetail from the prestressing wire.

These tanks were recently reevaluated by the authors' firm. Some random openings were made in the brick to examine the wire under the galvanized dovetail. From this cursory examination, it appeared that some wires exhibited only slight surface corrosion. Since this sampling was considered insufficient and the past experience with this brick anchor detail was poor, the brick facade on one tank was completely removed so that a detailed inspection of the wires could be made. After removal of the brick in order to inspect the wire, it was necessary to remove the galvanized steel dovetail and some minimal shotcrete that managed to get between the dovetail and the prestressing wire. A close inspection of the wires revealed several major

Fig. 4. Corroded wires bundled around a pipe. Shotcrete protection did not penetrate bundle and was not bonded to tank wall.

vertical lines of corrosion extending the full 16-ft (5 m) height of the dovetail. Of the 103 0.212-in. (5.4 mm) wires installed in that height, anywhere from 20 to 55 wires had failed or were so far corroded that they could not be counted on to provide any dependable ultimate strength. Frequently the top 6 ft (2 m) of wall had no effective reinforcement remaining.

In a number of dovetail locations, where virtually no effective prestressing was left, vertical cracks up to 10 ft (3 m) in length had formed in the concrete wall. In effect, the upper 6 to 10 ft (2 to 3 m) of the cylinder had failed. Fortunately, because of the earlier concern about the condition of the tank (some evidence of corrosion had been noticed in 1970), it had been restricted to a low operating level. Thus, the upper part of the tank had not been used for some time.

The nature of the corrosion was mixed. In places obvious brittle failures occurred as manifested by transverse cracks with no necking. In other areas, longitudinal splitting was observed but always associated with corrosion products. In general, the corrosion products were black.

A horizontal crack existed through the wall approximately 5 ft (1.5 m) above the bottom of the dovetail. It is postulated that sewage gas found its way through the crack and mixed with moisture. Since the black corrosion occurred both above and below the crack, it is assumed that sulphides had access to the wires.

The only corrosion protection available where the prestressing wire crossed under the dovetail slot was, in come rare cases, a maximum of $\frac{3}{16}$ in. (5 mm) of cement mortar. In many places, however, no protection was present. Where the mortar was dense, even as thin as $\frac{1}{16}$ in. (2 mm), protection was

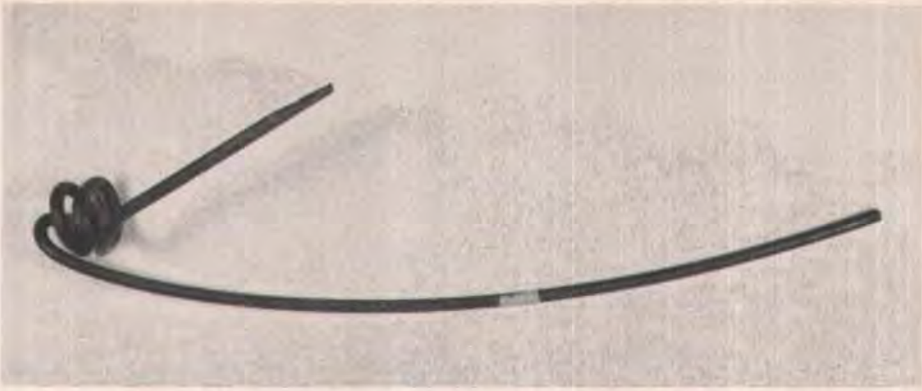

Fig. 5. Failed end of a longitudinally cracked wire with a bend test. This wire actually has a longitudinal crack going through the bend and for most of its length.

offered. Since these tanks were floating cover digesters, roof surface drainage did not spill over the wall. The wires, where exposed at the dovetail, were not in as aggressive an environment as the one that existed at the Owl's Head tank. This is a possible explanation for its 30-year survival.

A detailed inspection was also made of the lower portion of the tank walls, not covered with a brick facade, where accessible from inside the control house. Areas of delaminated, drummy sounding shotcrete were identified and removed. Here it was found that, where the wires were bundled around a penetration (see Fig. 4) and where the shotcrete was loose, extensive corrosion existed. It was also found that some longitudinal cracking of the wires had occurred. A short distance away, where they had been covered by good, sound shotcrete, the same wires did not appear to suffer any significant amount of corrosion.

The corrosion of the wires was induced by leakage of sewage material, liberating hydrogen sulphide, which came through the pipe openings. It was evident that corrosion occurred where the sludge material had access to the wire because it was not protected by sound shotcrete.

This illustrates the fact that, if the wire is properly protected and there is no direct access of the corrosive medium to it, prestressing steel does not tend to corrode. By avoiding the bundled wires, by making sure that the construction procedures were such that bond could be obtained between the shotcrete and the core wall, and by having the wires spaced adequately so that each one could be properly encapsulated in dense shotcrete, this problem could have been prevented.

Fig. 5 shows a piece of a wire which had failed in the structure. The top end of the wire exemplifies a typical brittle failure associated with a longitudinal split. The bend test performed on the wire indicates that no embrittlement occurred away from the splitting failure. However, by examining the wire closely with a 20-power magnification, it was found that the wire had a continuous longitudinal crack extending practically from one end to the other. The tape near the end of the wire of Fig. 5 indicates how far the crack was visible.

Similar cracked wire samples, also taken from next to the point of failure, when tested in tension, have exhibited good ultimate strength and elongation. It was also evident from these and other wires with longitudinal cracks that corrosion tends to concentrate along the crack, causing crevice type corrosion.

It is not known whether the longitudinal cracks found in a number of wires existed in the original wire, were formed during or after their drawing through the die in the wire winding process, or propagated from a local stress corrosion incident. Microscopic studies were not made on this wire. However, from another tank incident, it seems that incipient seams or folds will trigger radial cracking, resulting in substantial longitudinal splitting of the wire. This has been reproduced in the laboratory, in the presence of sulphides, but not with nitrates or chlorides.

According to several wire manufacturers, longitudinal seams or folds in wires cannot be avoided altogether and, if not more than about 0.003 in. (0.075 mm) in depth, they should not be detrimental.

The authors' experience indicates that wire which suffers normal corrosion or even chloride corrosion does not appear to exhibit this type of splitting. Cases of longitudinal cracks in wire have been reported by Phillips.[7] Schupack, who has inspected more than 70 prestressed tanks, has not observed, by the naked eye, this type of splitting in corroded wire in prestressed water tanks but has observed this in sewage and industrial tanks.

In summary, this digester tank, some 30 years old, shows that where the shotcrete protection is adequate, the wires are in good condition. The key consideration for a designer is that if the structure will be possibly subject to an aggressive environment, the designer has to be certain that the contaminants cannot reach the prestressing wires. Certainly, the detail of the dovetail located at the upper part of the tank is a classic example of an irreverent disrespect for the behavior of prestressing steel.

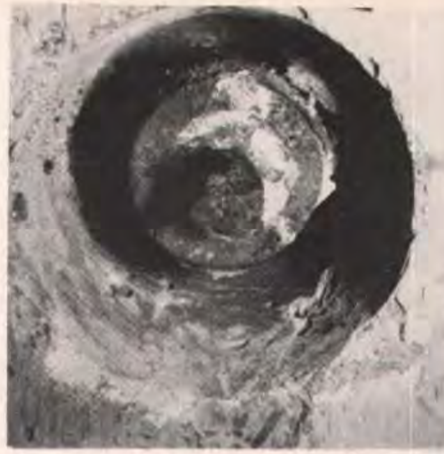

Fig. 6. Corroded end anchorage for 0.6-in. (15 mm) strand in a formed pocket.

Case No. 3 — Corrosion Failure of a Monostrand Tendon Anchorage in a Parking Structure

This multistory parking garage consists of cast-in-place, post-tensioned concrete slabs supported by a beam and column steel frame. The first indication of any problem with this structure was the very obvious protrusion of a strand about 15 ft (5 m) beyond the edge of the concrete slab. This occurred about 4 years after construction. The total length of the tendon was 165 ft (50 m). The anchorage holding the strand was a wedge type anchorage contained in a barrel housing. The barrel housing is a machined cylinder with a conical hole which takes the bursting force exerted by the wedges, putting the barrel housing in permanent circumferential tension. These anchorages have been used by the hundreds of thousands and there appears to be no prior incident of a time delayed sudden failure of the barrel.

In this particular incident, no significant corrosion was found on the prestressing strand after 5 years since the structure had been built, except where the strand protruded about ¾ in. (20

Fig. 7. Failed barrel of 0.6-in. (15 mm) strand anchorage showing longitudinal failure surface.

mm) beyond the anchorages. Strand taken from the dislocated tendon had more than the specified ultimate strength and elongation. The strand tendons were of the unbonded type protected with a corrosion inhibiting grease-like material and encapsulated in a plastic sheath.

The slabs in which the prestressing system was installed were of normal weight concrete with a strength of over 4000 psi (27.6 MPa). The concrete has a fair amount of surface scaling indicating a possibility of excessive water and/or over finishing. The concrete, analyzed after years in service, had an average chloride ion content by weight of concrete of 0.34 percent with a low of 0.09 percent and a high of 0.98 percent. All these values are higher than the now generally recognized limitation in the United States,[8] namely a chloride ion content of 0.014 to 0.025 percent (depending on the cement factor), by weight of concrete.

This structure is in the northern part of the United States where deicing is accomplished by using chloride salts. From other experiences, it is believed that the high chloride content is caused by saturation from surface deposited snow and ice, laden with salt, brought in by vehicles. This has become a major problem for many parking structures in northern climates in North America.

The anchorage pocket (see Fig. 6) was packed with a mortar which may or may not have contained calcium chloride. Analysis of the pocket filling material indicated chloride contents of up to 0.22 percent by weight of concrete. These pockets were filled mostly in the winter, and it is unknown whether any winter protection was provided. Some of the anchorage pocket mortar plugs appeared to have shrunk away and were loose. In some cases, the plug could be pulled out as a cone. Products of corrosion could be found at the surface of a number of anchorage pockets.

The barrels suffered longitudinal cracking (see Fig. 7). The appearance of the failure surface is also shown. It was reported by one metallurgical investigator that the failure was due to stress corrosion. Another investigator found no stress corrosion, but only a decrease in barrel area with the resulting higher bursting stresses causing failure.

Based on an analysis of the barrel, it is difficult to see how the amount of corrosion shown could have reduced the section to the point of causing a failure. The failure may have emanated from a corrosion stress raiser which may have propagated into a longitudinal crack. This may or may not have been a stress corrosion failure.

The reader may question the reason for including this case in a discussion of stress corrosion failures of prestressing steel. It is of particular interest because with all the reports on prestressing steel problems under elevated stress, practically none mentions failure of the anchorage. At the anchorage, the prestressing steel is subjected to the greatest abuse and the most complex stress patterns because of the gripping effects on the prestressing steel elements.

In this particular case, the environment was most conducive to the corrosion of the prestressing steel, but the

Fig. 8. Surface of wire sample adjacent to fracture showing elongated corrosion pits which intersect the fracture.

prestressing steel itself did not suffer any significant damage. Whether the barrel in this case was sacrificial in the corrosion cell is not known. If chloride induced stress corrosion was the cause of the failure of the barrel, this again reinforces the opinion most prevalent in the literature that prestressing steel in the presence of chlorides is generally not subject to stress corrosion.

There is a difference of opinion in this regard, but the experience reported and the authors' own experience appear to indicate this. A case of particular interest in this regard is the controlled test of 20 beams in the tidal zone at Treat Island, Maine, in which Schupack[3] has been involved for over 20 years. Post-tensioned beams have not indicated any forms of stress corrosion though there has been some very small amount of general corrosion.[9]

Case No. 4 — Unbonded Tendons in Roof of Hotel Structure

Two failed tendons in the structure were discovered when one of the strands projected about 3 ft (1 m) into an adjacent room. The failed 0.6-in. (15 mm) monostrand, greased and plastic sheathed, had been in service for about 5 years. It could not be determined by visual inspection, whether the plastic sheath was damaged prior to the tendon failing. The failure occurred a short distance away from a vertical opening in the concrete slab through which the two tendons crossed.

It is quite possible that the plastic sheath may have been damaged at the contact point with the sides of the slab opening or with the reinforcement of a column adjacent to the opening. In any case, having a tendon passing through an opening is not a desirable detail, since it is conducive to improper corrosion and fire protection, as well as to potential physical damage because of its accessibility.

A visual examination of the fractures disclosed that each wire in the strand had fractured transversely in essentially a brittle manner. Two of the samples exhibited some slight necking adjacent to the fractures. It is likely that these

Fig. 9. Photo micrographic of longitudinal cross section taken through corrosion pit on surface of sample. Typical branching and stress corrosion cracks emanating from the corrosion pit (top) of the wire are clearly evident.

two wires were the last wires in the strand to fail. Close examination of the strand wire fracture surfaces disclosed coarse, irregular, jagged profiles.

Irregularly shaped patches of localized corrosion were observed on the wire surfaces both at and remote from the fractures. Some of these corroded areas contained a corrosion product on the surface while others appeared essentially free of any corrosion product, appearing as sharp edged, shallow elongated pits.

Examination of a microspecimen in the as-polished, unetched condition disclosed that near the wire surface, the jagged fracture profile contained a corrosion product of appreciable thickness [approximately 0.006 in. (0.15 mm)], indicating that this portion of the fracture was exposed to a corrosive environment for an extended time period.

Fig. 8 shows typical corrosion observed on the surfaces of the wire samples.

The general appearance of the brittle fracture profile was characteristic of stress corrosion cracking. This was confirmed by examination of the microspecimen taken through pitted areas adjacent to the fracture which clearly exhibited branching cracks typical of stress corrosion cracking. These cracks are shown in Fig. 9.

The general microstructure of the strand wire as observed in transverse microspecimens appeared to be comprised of severely cold-worked pearlite with possibly some bainitic transformation products present. The wire surfaces showed no evidence of decarburization or intergranular oxidation.

Energy dispersive X-ray of the corrosion product on the wire surfaces revealed the presence of calcium, potassium, sulphur, and silicon; the silicon and sulphur in larger amounts than would normally be present in the steel strand. Additionally, chlorides were also detected.

With the exception of the normal and expected oxidation products of iron, the constituents observed in the corrosion products strongly suggest that the strand wire surfaces had either rubbed against concrete or accumulated a residue of concrete dust, since these elements are commonly observed in aggregate, and also implies that the encapsulating plastic conduit had been penetrated or perhaps fretted through at some point along the length of the strand. This scenario is compatible with the possibility of damage to the plastic sheath at or near the slab opening, as indicated above.

Examination of the submitted wire samples indicates that at least five of the seven strand wires failed transversely by stress corrosion cracking which cracking initiated at localized corrosion pits on the wire surfaces. Two of the submitted strand wires probably contained small stress corrosion cracks, but final failure occurred by tensile overload.

None of the wire samples showed any evidence of manufacturing defects in the nature of seams, laps or laminations which could have contributed to the failures. The metallurgical investigation also confirmed that the strand was not corroded prior to installation nor did the grease on the wire contain a corrodent.

Had these conditions existed at the time of the installation, failure would have occurred much earlier in the service life of the strands.

Although the strand failure occurred in an area containing heavy electrical equipment, the corrosion observed and the mode of failure were not characteristic of that which results from stray currents. Were stray current pitting the primary cause of failure, then the fractures would have been essentially ductile in nature and not brittle as that observed in the failed wires.

CONCLUSION

As structural engineers, it is the authors' opinion that the corrosion problems that have occurred are more a result of improper choice of details and poor execution than of any characteristics of the steels used. In all the corrosion cases cited, any type of prestressing steel would eventually have suffered distress. It is up to the engineer and constructor to select systems and construction methods which can more positively assure proper corrosion protection.

RECOMMENDATIONS

The following are a few important considerations that, if taken into account, will contribute towards building safe, problem-free structures.

Precast Pretensioned Concrete Members

Historically, there have been a minimum number of corrosion related problems with these structural members. The common use of high strength, dense, well compacted and properly cured concrete evidently provides the best corrosion protection to the prestressing steel. Any significant voids and honeycombs which are noticed, particularly in the vicinity of the prestressing strands should be filled in at an early date with a cement mortar using, preferably, materials similar to those used in the parent concrete.

Wire Wound Prestressed Cylindrical Structures (Pipes and Tanks)

Design details and construction procedures should be directed towards obtaining a complete encapsulation of each individual wire in sound, dense, portland cement mortar. Wires should be adequately spaced from each other, within the same layer and between layers. Bundling of wires should be avoided. If pipes, manholes, or other penetrations are present, placing circumferential bands on both sides of the penetration should be preferred over forcing a change of direction of the wires around the penetration, with the resulting bundling.

The application of the cover coat (typically shotcrete in the case of tanks) must be done with extreme care, by experienced personnel, to completely encapsulate each wire and to obtain a monolithic, well bonded, composite structure with the corewall. Auxiliary anchors or inserts of any kind should be embedded in the cover coat. Metal to metal contacts with the prestressing wires should be avoided.

Post-Tensioning Tendons — Bonded Construction

The most critical factors in this type of application are the quality of the grout and of the grouting procedures. Where these have been good, the structures have been trouble free. The choice of grouting materials, including admixtures, must be done carefully to relate to the particular application. Vertical tendons or any tendons whose geometry produces significant differences in elevation throughout its length, must be dealt with special care, as bleeding and water separation become more significant. Location of drains and vents, particularly those at or near the anchorage areas (grout caps, trumpets, etc.) is of the utmost importance, to prevent the formation of air pockets which may result in permanent voids within the grout volume.

The grouting operation itself, the sequence of opening and closing valves and vents, the choice of locking off all tendon outlets at the end of the grouting operation or, as is the authors' preference, of leaving the high point vents open, to allow for free expansion, must be carried out with care. Any of these efforts, in pursuit of the highest quality grouted tendon which is possible, will bear fruit in producing permanent, dependable corrosion protection to the prestressing steel at a minimum, if any, increase in cost.

Post-Tensioning Tendons — Unbonded Construction

The most predominant example of this type of construction is the monostrand tendon market, which has produced millions of square feet of cast-in-place post-tensioned concrete slabs and beams in all kinds of building structures. Because of its popularity, this market has always been highly competitive. This has resulted in many cases in compromising quality for a reduced cost.

The philosophy on the part of the post-tensioning supplier of providing the least costly installation which does not create any visible, immediate problems during construction has, unfortunately, prevailed in many a structure. Better quality control over present construction methods and slight improvement of some of the materials used would, at a minimal increase in cost, avoid many of the problems of corrosion which have occurred. The following are some of the things that can be done:

1. Better control of grease application during manufacture to obtain complete and uniform coverage of the strand.

2. Heavier wall thickness of plastic sheath to increase the resistance to damage and to crimping by tie wires.

3. Proper detailing of sheath to anchorage connection to insure that an effective seal exists at tendon ends. The common practice of allowing the plastic sheath to end several inches away from the anchorage should be forbidden.

4. Proper detailing and execution of the filling of the anchorage pockets, to insure an adequate sealed cover to the anchor so as to prevent the intrusion of any corrosive substances from the outside.

5. Control over concrete and mortar mixes to avoid any possibility of chlorides, sulphides or other potentially damaging materials being present.

6. Adequate field inspection.

Suggested Scheme to Provide a Higher Level of Corrosion Protection for Bonded and Unbonded Post-Tensioning Tendons

In order to cope with the realities of construction and the environment to which the structure may be exposed, the authors have developed the concept of an electrically isolated tendon (patent applied for). According to this concept, the complete tendon (including the prestressing steel elements), the anchorages and the corrosive protective medium (whether grease or cement grout), is completely encapsulated from end to end within a tough plastic enclosure.

The sheath itself can be smooth walled or corrugated, depending on whether it is an unbonded or a bonded application. It must be adequately sealed into the plastic cover enclosing the whole anchorage system. In this way, the tendon is protected, not only from the penetration of any aggressive materials such as chlorides or H_2S, but also from stray currents and the formation of long corrosion cells.

The use of this system will necessarily be more costly than what is common practice today. The authors estimate that it may amount, for example, to an increase of less than 1 percent of the total cost of a parking structure using ½-in. (13 mm) monostrand tendons. This additional cost is insignificant when one considers the tremendous cost of repair and disruption of the structure's use that result when tendons corrode.

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NOTE: Discussion of this paper is invited. Please submit your discussion to PCI Headquarters by November 1, 1982.