WHEN BAD THINGS HAPPEN TO GOOD [BUILDINGS]

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Synopsis:

The title of this paper borrows from the 1981 book by Harold Kushner entitled "When Bad Things Happen to Good People". In his book, Kushner attempts to explain why a universe created by a deity who is of a good and loving nature still holds so much pain and suffering for good people. In the context of this paper, the title is meant to be an epigraph that suggests that although a building may be meeting its intended structural purpose, bad things, at least as they are perceived, can happen during design, construction and service of the building that bring its safety into question. One of the main circumstances that can bring into question the integrity of an unbonded post-tensioned building is corrosion of the strands and anchorage components. This paper will highlight the unnecessary demise of a modern high-rise post-tensioned structure due to corrosion, and contrast that outcome to several existing unbonded post-tensioned buildings that experienced corrosion and were successfully repaired and continue to function.

Keywords

Concrete, corrosion, pitting, post-tensioning, reinforcing steel, strand corrosion

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INTRODUCTION

The title of this paper borrows from the 1981 book by Harold Kushner entitled "When Bad Things Happen to Good People". In his book, Kushner attempts to explain why a universe which is created by a deity who is of a good and loving nature still holds so much pain and suffering for good people. In the context of this paper, the title is meant to be an epigraph that suggests that although a building may be meeting its intended structural purpose, bad things, at least as they are perceived, can happen during design, construction and service of the building that bring its safety into question. It is not meant to be presumptuous that the pain of human tragedy can in any way be compared to problems experienced with inanimate objects such as a building, but the comparison does provide a meaningful way to contrast to the economic "pain" that can be caused when a good building is labeled suspect or unsafe.

For the specific case of unbonded post-tensioned concrete structures, one of the circumstances that can bring into question the integrity of a building is corrosion. In a paper published in 1996, Kesner and Poston[1] reported on the many extremes that were taking place at the time in the investigation and repair of post-tensioned concrete buildings experiencing corrosion. The authors concluded at that time that myths and misconceptions significantly affected the manner in which the integrity of unbonded post-tensioned concrete buildings was perceived differently by the various parties involved be it the owners, engineers or contractors. It was further concluded that acceptance of these myths and misconceptions as truths resulted in a misunderstanding of the integrity of an unbonded post-tensioned building experiencing corrosion, resulting in unnecessary and excessive spending on both investigation and repair.

There are hundreds of millions of square feet of unbonded post-tensioned construction in North America that have successfully served their intended purpose for decades. It is true that there have been selected cases in which corrosion of unbonded tendons has occurred, but with deliberate intervention, corrosion can be mitigated and the safety of the building ensured. Fast forward to the latter part of the first decade of the new millennium, and it would seem that the lessons learned from corrosion of unbonded post-tensioned concrete buildings referred to by Kesner and Poston have been largely forgotten or ignored.

This paper highlights the case of a 25-story unbonded post-tensioned concrete apartment building completed in the late 1990's that was demolished with corrosion of the unbonded tendons being attributed as a principal reason. It is understandable that corrosion can be perceived as a "bad thing," but the incontrovertible truth is that all ferrous metals experience corrosion as defined by the second law of thermodynamics. As with all things made of metal, including reinforced and prestressed concrete structures with embedded steel, it is not a matter of if corrosion occurs, but at what rate it occurs. For steel in direct contact with concrete, a passivating film forms on the surface due to the high pH environment, and thus the steel is well protected from corrosion. For unbonded post-tensioning tendons, the steel is directly protected by sheathing and grease. However, with the intrusion of aggressive elements such as water and oxygen, corrosion will initiate and can be sustained depending on a myriad of factors.

The analysis discussed below is based on the industry standards and practices for analyzing post-tensioning systems in a building. Based on those standards and practices, the analysis clearly demonstrated that the subject building's post-tensioning system tensile strength was significantly above the building code and design requirements and consequently, the demolition of the building was completely unjustified.

CORROSION AND PITTING

Corrosion engineering has developed into a mature field as the science and art of providing solutions when corrosion initiates and measures are needed to mitigate the effects. The following discussion is based upon principles and information found in Jones 1996[2].

In most circumstances, corrosion of steel requires the presence of both oxygen and water. The corrosion process occurs as a combination of an anodic (oxidation) half-reaction and a cathodic (reduction) half-reaction. In the anodic half-reaction, iron is dissolved, $Fe \rightarrow Fe^{2+} + 2e^-$, producing iron ions and electrons. This reaction can only proceed if the electrons produced are consumed by the cathodic half-reaction which, in steel corrosion, predominantly involves reduction of oxygen, $O_2 + H_2O + 4e^- \rightarrow OH^-$. Iron ions liberated by the anodic reaction and hydroxide ions produced by the cathodic reaction combine to form ferrous hydroxide, $Fe(OH)_2$ which deposits around the active corrosion site. The ferrous hydroxide is further oxidized by oxygen reduction to ferric (Fe^{3+}) hydroxide or red rust.

As stated previously, for steel in concrete, the high alkaline environment of concrete creates a passivating film on the steel that affords robust corrosion protection. But in the case of unbonded post-tensioning tensioning, there is not direct contact of concrete with the steel – rather corrosion protection is provided by plastic sheathing and grease. As long as this protection is maintained at all locations from end to end, there is little risk of corrosion.

The corrosion product deposits thus formed create a layer (or scale) over the actively corroding areas. Depending upon a variety of environmental conditions, the form of the corrosion can be uniform and general over the steel surface, or localized in the form of pitting corrosion. In uniform corrosion, the surface area ratio of cathode to anode is relatively the same. When a localized area of corrosion initiates surrounded by a larger cathodic surface area, pitting develops that typically proceeds at a much higher rate than uniform corrosion. Pitting corrosion is the form of corrosion generally of greatest concern with regard to loss of cross-sectional area and corresponding strength of a post-tensioning tendon.

Variable-to-poor grout quality and/or workmanship at post-tensioning live end anchorages can allow moisture and oxygen access to the steel components of the anchorage, including the tendon pigtail as well as behind, or inboard, of the anchor. Under those circumstances, active corrosion can develop, first at the exterior portion of the anchorage and subsequently along the path of the tendon to the area behind the anchorage. If that happens, active strand corrosion inboard of the anchor can become an aggressive anodic area supported by moisture and oxygen reduction at the exterior of the anchorage closest to the edge of the structure. Via hydrolysis reaction of ferrous iron ions liberated at the active corrosion sites with water, ferrous hydroxide forms as a corrosion product and, in the process, liberates hydrogen ions (H^+) according to the reaction, $Fe^{2+} + H_2O \rightarrow Fe(OH)_2 + 2H^+$. If chloride ($Cl^-$) also happens to be present in the anchorage grout or moisture, the $H^+ + Cl^-$ present create hydrochloric acid and a very aggressive corrosion condition for the strand.

There is a very important point to be emphasized regarding the onset of active aggressive corrosion inboard of the anchorage described above. That is, for corrosion inboard of the anchorage to both initiate AND continue, a renewable source of moisture and oxygen must be present at the exterior of the anchorage. The electrons produced by the consumption of the tendon steel inboard of the anchorage MUST be consumed by a cathodic reaction which, as noted previously, for steel corrosion will typically be oxygen reduction. As long as oxygen remains available at the exterior face of the anchorage and moisture is present on the anchorage and along the path of the tendon from outboard-to-inboard of the anchor, active inboard corrosion will be supported. IF, however, the source of oxygen and moisture can be excluded at the exterior face of the tendon by removal of existing corrosion products, application of coatings, and replacement of the poor grout with sound grout, the active corrosion process will be slowed and finally stopped as all available moisture and oxygen are consumed. As long as the amount and extent of corrosion on the inboard location of the tendon was not so severe as to compromise the structural integrity of the tendon, repairs at the "live end" anchorage will halt the corrosion process and restore the integrity of the post-tensioning system.

In response to these corrosion problems, the post-tensioning industry has significantly improved the corrosion protection measures used for unbonded strands. Figure 1 shows the evolution of corrosion protection systems over time, which demonstrates movement towards improved corrosion protection provided by watertight strand sheathing and encapsulated anchorages.

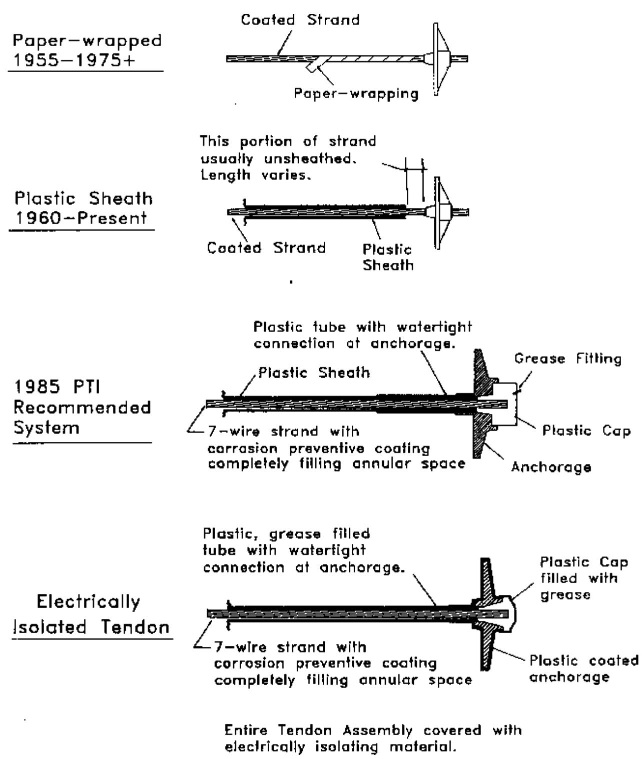

Figure 1—Evolution of corrosion protection for unbonded single strand tendons for buildings (reprinted from Reference 3).

The severity of pits that can form and the approximate strength loss associated with each is summarized in Figure 2. As can be seen, even though pits develop on the surface of the prestressing strand of an unbonded tendon, there is residual strength as measured by mechanical tension testing of the strand. Even in the most severe case (deepest pits, Figure 2d), the residual strength can be as high as 75 percent of the guaranteed ultimate tensile strength (GUTS).

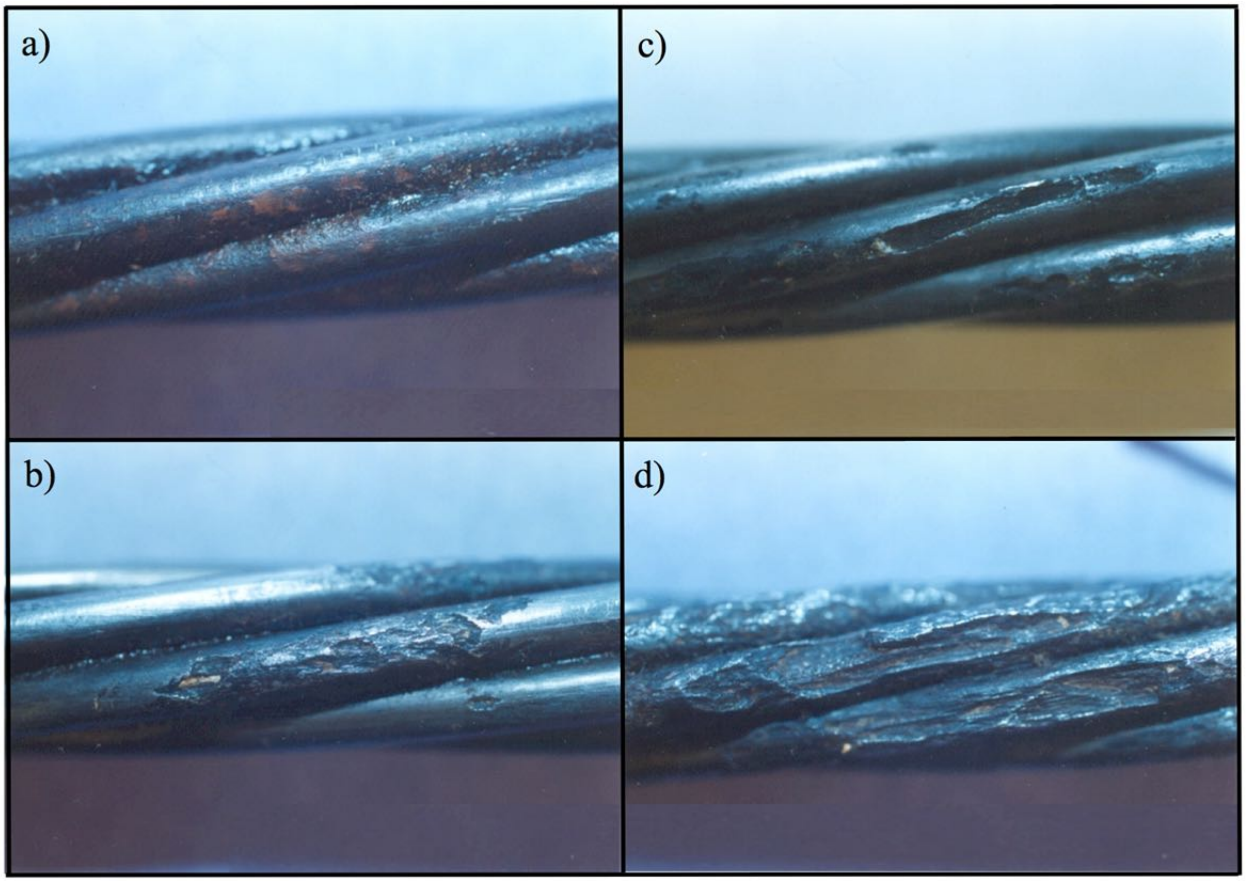

Figure 2— Relative corrosion pitting assessment (adapted from Reference 4): a) surface corrosion, no strength loss; b) mild pitting: no strength loss; c) intermediate pitting: approximately 10% strength loss; d) severe pitting: approximately 25% strength loss.

TENDON INVESTIGATION

Both the building owner's consultant team and the authors' team removed post-tensioning tendons from the subject building for inspection and evaluation pursuant to an agreed investigation protocol. The other team removed portions of about 80 tendons ("live end" anchorages and several feet of strand) and the authors removed fifteen. Figure 3 below summarizes the extent of pitting corrosion that the authors measured on the tendons removed by both parties. The authors measured pits on tendons removed by both parties using ASTM G 46 – Standard Guide for Examination and Evaluation of Pitting Corrosion[4]. The International Concrete Repair Institute (ICRI) classifies the severity of pits for in-service post-tensioning tendons (ICRI 03736[5]), such as those at the subject building. The various levels of pitting have been overlaid in shades below. Note that only one tendon fell within the "severe" (darkest shade) category and was subject to the greatest strength reduction of approximately 25 percent.

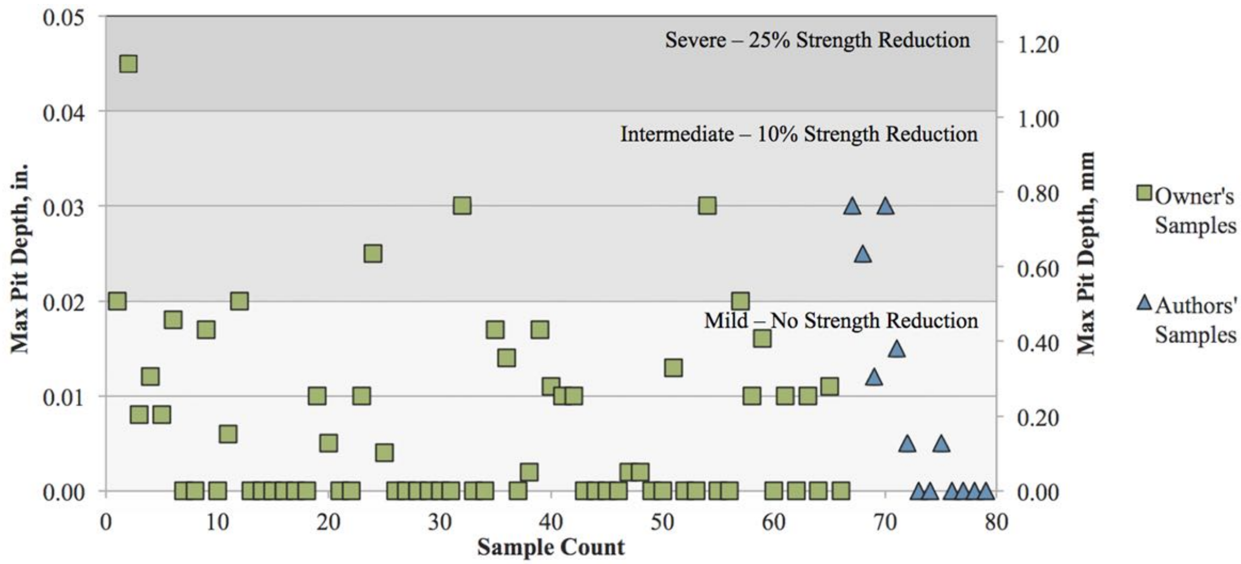

Figure 3—Measured pit depths for tendon samples removed from subject building; all measurements performed by the authors.

The building owner's consultants used a rating system as presented by Sason[6] that is intended solely for new construction in a bonded, pretensioned system with emphasis on the fatigue life of strands, such as would be experienced by pretensioned bridge girders subjected to thousands of cycles per day of truck loading. Hence, any rating system based on Sason's paper was not relevant to this structure. ACI 423.7-07[7], "Specification for Unbonded Single-Strand Tendon Materials and Commentary" in a section covering acceptance criteria for prestressing strand, states in R2.1.4, *"For further information, refer to Sason (1992). These criteria are not intended for use in evaluating tendons that are in service in existing buildings*." Furthermore, the Sason paper itself states that *"…a heavily rusted strand with relatively large pits will test to an ultimate strength greater than specification requirements. However, it will not meet the fatigue test requirements."* The guiding principle for the Sason pit-depth criterion is that the tendons will exceed the specified strength requirements in tension, but may have poor resistance to metal fatigue.

Conversely, the ICRI pitting categories are expressly intended for use in evaluating tendons that have been removed from in-service structures and directly address tensile strength, which is dependent on cross-section, not metal fatigue in a dynamic loading condition such as truck loading on a bridge. When using these more directly appropriate categorizations, only one (1) strand falls into the "severe" category and six (6) fall into the "intermediate" category. Three (3) of those six (6) were specimens selected, inspected, and tested by the authors. Results of strength testing are described in a later section of this paper. Additionally, the average pit depth measured by the authors on the strands removed by the other consultants (0.007 in./0.18 mm) was slightly lower than the average pit depth measured by the authors (0.009 in./0.23 mm) on the 15 tendons removed by the authors. Thus, it is clearly evident that the 15 post-tensioning tendons removed by the authors' team were truly representative of the range of existing conditions of the tendons in the building.

CASE STUDIES

The authors have encountered, and successfully repaired, numerous post-tensioned buildings with much more severe corrosion than was encountered at the subject building. A brief description of the investigation and repair of three buildings is presented: the University of Virginia School of Law Withers Building; and two Florida Gulf Coast Condominium Buildings.

University of Virginia School of Law Withers Building

The Withers Building, constructed in 1963, has one-way post-tensioned slabs over girders. The older post-tensioning system used pushed-through monostrand tendons which allowed for movement of water along the strand trajectory. During the course of adding new stairwells to the building, broken tendons were observed in the structure and repairs began shortly thereafter. The testing and repairs included:

- Tendon extraction
- Metallurgical analysis
- Tensile strength analysis
- Service load survey
- Structural evaluation
- Replacement of failed tendons
- Urethane injection
- Acoustic monitoring system

Strength testing of extracted tendon assemblies indicated that the minimum measured strength was 77 percent of GUTS or 208 ksi (1435 MPa). It should be noted that this value (208 ksi/1435 MPa) is only marginally greater than the maximum stress at nominal flexural strength of about 205 ksi (1415 MPa) in an unbonded tendon for slabs per the 2011 ACI 318 Building Code[8]. Corrosion in this building was severe and widespread, but was repaired in 1997 and the building has been performing successfully since implementation of the repairs.

An acoustic monitoring system was installed in 1998 to detect future wire breaks. If any such breaks occur, the necessary parties are informed immediately so that precautionary action can be taken as needed. The monitoring system also serves as a quality control measure on the repairs. The structure had thirteen individual wire breaks in the period from 1998 to 2006. However, it has not experienced a single wire break since 2006 and the University of Virginia is currently considering removal of the monitoring system due to the success of the corrosion mitigation system that was implemented.

Florida Gulf Coast Condominium Building

The investigation of the Florida Gulf Coast Condominium, which is a 12-story residential building, was triggered by visible rust staining on concrete surfaces and exposed “live-end” anchors that were visibly corroding. The building was constructed in the early 1980’s and used a loose-fit sheathing system. The corrosion was first observed about twelve years after construction. Fourteen (14) tendon/anchorage assemblies were removed from the building for strength testing. The average breaking strength of the assembly was 96.2 percent GUTS (259 ksi/1785 MPa) with the minimum measured assembly strength of 87.2 percent GUTS (235 ksi/1620 MPa). Examination of the tendon failures indicated ductile cup-cone type failures with no signs of embrittlement.

The repair of this building consisted of selected repositioning of live end anchorages inboard of the edge; the removal and replacement of grout pocket protection; and application of a waterproof coating on the exposed balcony slabs. The total cost of repairs was $550,000 whereas the cost of total tendon replacement was estimated to be $4,000,000. The building has continued to perform without any post-tensioning tendon failures for the last 15 years.

Second Florida Condominium Building

Finally, a second Florida Condominium Building constructed circa 1985, was built within yards of the high-tide line on the Atlantic coast of Florida. The building was oriented such that the “live-end” anchorages were on the side of the building facing the ocean and subjected to maximum chloride exposure. Moreover, the stressing pockets were filled with stucco instead of a good-quality grout. Fifteen (15) tendon/anchorage assemblies were tested with the average strength determined to be 95.4 percent GUTS (258 ksi/1780 MPa). Therefore, it was determined that the structural integrity had not been compromised and that simple repairs to the grout pockets was an effective repair strategy to arrest corrosion by mitigating the intrusion of oxygen, water, and chlorides.

MECHANICAL TESTING OF SUBJECT BUILDING TENDON ASSEMBLIES

The authors selected 15 tendons representing the range of in-situ conditions for removal from the subject building pursuant to the post-tensioned investigation protocol agreed to by the parties. These tendons were removed in a controlled manner resulting in minimal disturbance from their in-place state. The details of the tendon tests are summarized in Table 1. Each tendon was removed with the “live-end” anchorage intact and that anchorage was used in all laboratory tensile strength testing. The cut ends of the tendons were gripped with long barrel wedges such that stress raisers were eliminated from that end. Six (6) of the 15 tendons showed signs of corrosion and pitting and were tested to failure in tension. Two "clean" tendons were tested to failure to evaluate the effect of the anchorage and wedges on strength. The other seven tendons, two of which were removed from the adjacent parking structure, showed no evidence of corrosion. It is worth noting that the owner's consultant chose not to perform any strength testing, and instead relied solely on visual assessment of the tendons.

Table 1—Subject building strand sample locations, measured strengths, and failure modes.

Sample	Assembly Breaking Force, kips (kN)	Assembly Breaking Strength, ksi (MPa)	% GUTS	Location and Mode of Failure During Tensile Strength Testing
E9-0.6	39.5 (176)	259 (1785)	95.8	1 wire, shear break at live end anchor wedge
E18-15	37.7 (168)	246 (1695)	91.2	2 wire breaks in location of corrosion behind anchor
E8-47-1	39.0 (173)	254 (1750)	94.2	1 wire shear break at live end anchor wedged tooth
E10-26	38.0 (169)	248 (1710)	91.9	1 wire shear break at live end anchor wedge tooth
N21-0.6	37.4 (166)	244 (1680)	90.5	One wire tensile break in location of corrosion behind anchor
N18-0.6	35.2 (157)	230 (1585)	85.1	Three wire tensile breaks in location of corrosion behind anchor
N23-0.6	40.5 (180)	264 (1820)	97.9	Two wedge tooth shear breaks at live end
N16-0.6	39.6 (177)	258 (1780)	95.7	One wire shear break at live end anchor wedge
Average	**38.3 (170)**	**251 (1730)**	**92.8**	

Selected photographs of specimens after testing are shown in Figure 4. In each of these photographs, wire fractures adjacent to the anchors and wedges can be seen.

The ACI 318 Building Code limits stresses at nominal flexural strength in unbonded tendons to a maximum of f_{se} + 30,000 psi. For the subject building, the value of f_{se} was specified by the design engineer to be 175 ksi (1205 MPa); therefore, the maximum stress at nominal strength in the tendons per the ACI Code is 205 ksi (1415 MPa). The minimum tested value of assembly strength was 230 ksi (1585 MPa, Specimen N18-0.6). Thus, with the *maximum* permissible tendon stress at factored loads per ACI 318 and the *minimum* measured strength from testing, there remains a margin against rupture of at least 25 ksi (170 MPa, 9 percent GUTS). This maximum stress value represents the maximum permissible stresses under factored loads of 1.2DL (dead load) and 1.6 LL (live load) and 1.0 Secondary Moments. Under service load level, the tendon stress is significantly less than 205 ksi (1415 MPa). The evaluation clearly indicated that there was significant reserve capacity compared to code-required strength.

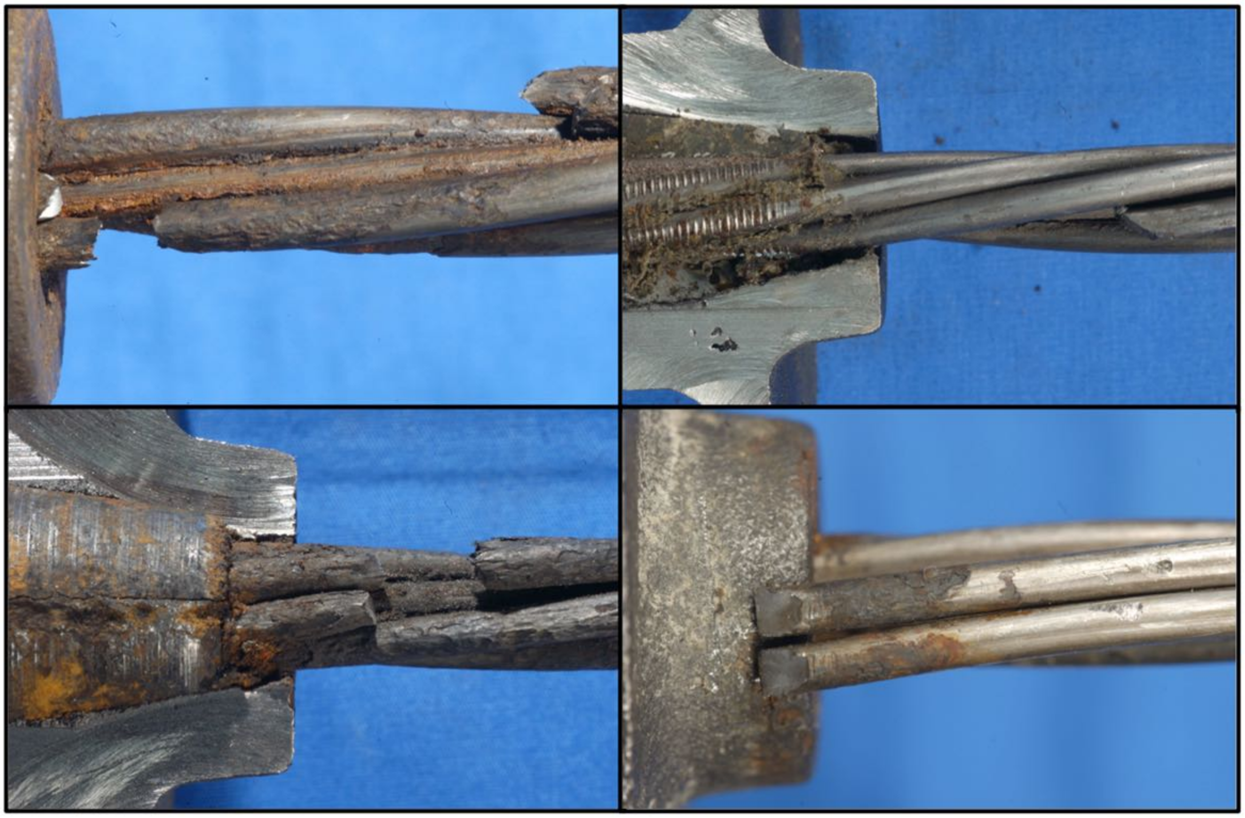

Figure 4—Selected photographs of fractures.

This reserve capacity is evident in Figure 5. The plot depicts the average and minimum assembly strengths as tested by the authors and contrasts them to the code-level required strength. Note that the required nominal strength of the tendon assembly (205 ksi/1415 MPa) is significantly less than the average measured strength (250 ksi/1725 MPa) and the minimum measured strength (230 ksi/1585 MPa). Based on these values, a substantial amount of reserve capacity remained, and the safety of the structure had clearly not been compromised to date.

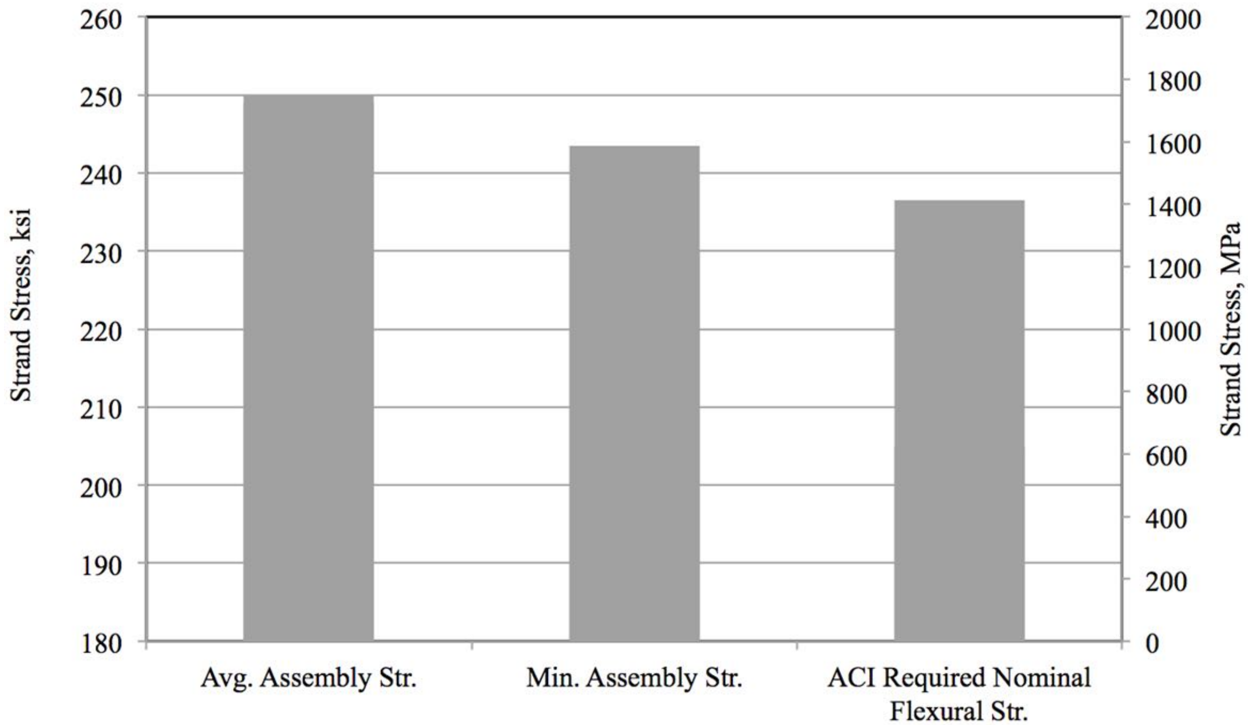

Figure 5—Required and measured tendon assembly strengths.

To further illustrate the magnitude of the alleged deficiencies in the subject building, it is possible to draw comparisons with past projects. A dramatic comparison can be made between this structure and the Withers Building that was investigated and successfully repaired by the authors. The Withers Building had severe corrosion present at the time repairs were implemented. Since the completion of repairs the structure remains in-service. The building has been continually monitored since it was repaired more than 13 years ago. Recall that there have been 13 individual wire failures to date, (there are seven wires to a strand) but thus far the University of Virginia has elected not even to replace the tendons associated with those wire breaks. The repair and monitoring program has been so successful that the owner plans to abandon the monitoring system in the near future. Figure 6 compares the observed tendon corrosion on these two structures. Tendon corrosion categorizations for these charts are the same as those used in Figure 2 and Figure 3. Clearly, the Withers Building had a far more advanced state of tendon/anchorage corrosion than was present in subject building. Despite the advanced state of corrosion, the Withers Building was successfully repaired and remains in service; therefore no reason exists to believe that the subject building could not have been easily repaired.

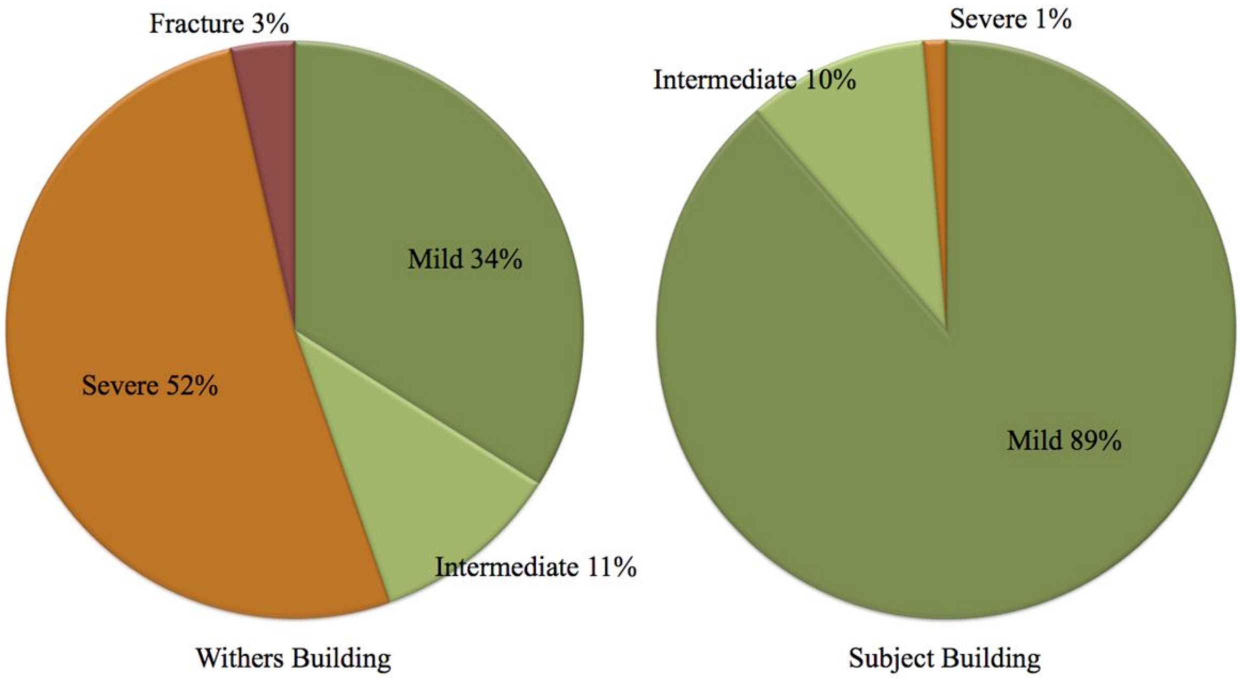

Figure 6—Comparison of corrosion in Withers Building and Subject Building.

In Figure 7, two photographs of tendons and wedges are shown. On the left is a sample removed from the "subject" building and on the right is a sample removed from the second Florida Condominium building. These two photographs were taken after the specimens were tested to failure. Both exhibited the same type of failure: shear fracture of a single wire. However, the sample from the subject building is less corroded and had a greater tensile strength (258 ksi/1780 MPa) than the Second Florida Condominium Building sample (238 ksi/1640MPa). This building was successfully repaired more than 16 years ago and remains in service.

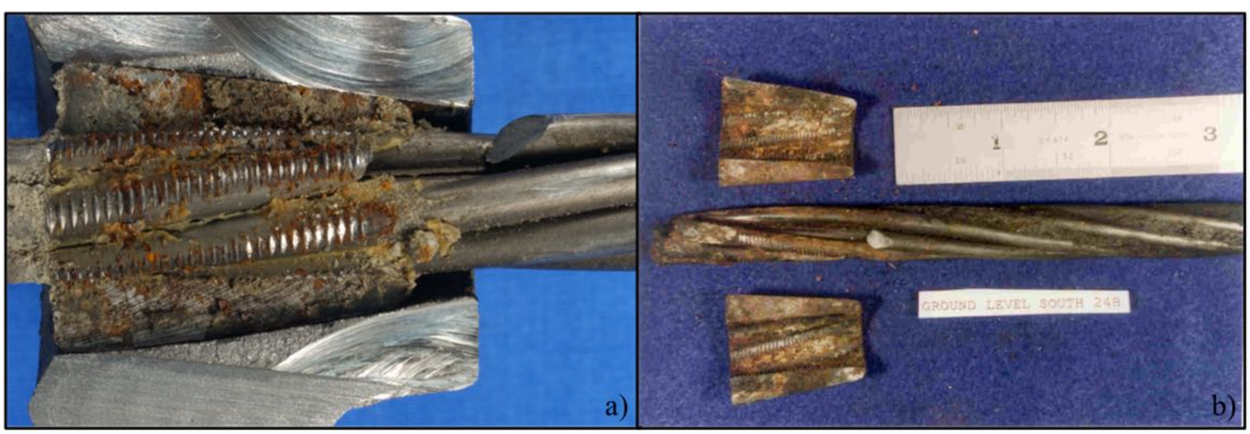

Figure 7—Wedge assemblies from a) Subject Building sample and b) Second Florida Building sample.

STRUCTURAL ANALYSIS OF TYPICAL FLOORS

Structural analysis was performed on typical floors for the subject building using commercially available software. These analyses were based on an ultimate strength load case in the building code for gravity loads (1.2 Dead Load + 1.6 Live Load + 1.0 Secondary Moments). For statically indeterminate structures, the moments due to reactions induced by prestressing forces, referred to as secondary moments, can be significant in both the elastic and inelastic states and are explicitly considered within the analyses.

Since the evaluation of the post-tensioning tendons clearly indicated that the stress in the tendons at factored loads could still be achieved by a comfortable margin, the results from the analysis indicated that the strength of the slab system for flexure and punching shear had not changed from the originally designed state.

PROPOSED REPAIR STRATEGY

Despite the fact that some of the subject building's post-tensioning anchorages were exposed by the Owner's investigation for more than a year, which likely resulted in additional corrosion, the post-tensioning tendons could have been remediated with relatively easy measures. To mitigate corrosion, it is necessary to stop the ingress of oxygen and water into the anchorages. All anchorages need to be protected from the elements to arrest corrosion. It is a well-known corrosion principle that eliminating oxygen diffusion will slow the corrosion rate to essentially zero. Once any existing oxygen inside and inboard of the anchorage and its components has been consumed the corrosion process essentially stops.

All metal surfaces of the anchorages, wedges, and strand tails must be sandblasted to remove existing corrosion products. All grout pockets should be re-packed with non-shrink grout that is prepared and cured per manufacturer's specifications. Additionally, an elastomeric coating over the concrete surface and grout pockets would provide an additional barrier to oxygen and water. The authors evaluated the conditions at the building and submitted engineered repair drawings including these measures that, if implemented, would have arrested corrosion.

Finally, simply as a precautionary measure, an acoustic monitoring system was proposed for installation throughout the building. Such a system would provide assurance that in the event that a wire does fail due to corrosion, its location will be known and replacement can occur if structurally necessary.

Cost estimates were obtained to implement the proposed repairs. The estimates indicated that the repairs could be implemented for a fraction of the building Owner's anticipated repair costs. Unfortunately, the repair recommendations were not implemented and the building was demolished.

SUMMARY

Despite the occurrence of some corrosion and pitting of the post-tensioning tendons as determined by the authors, the subject building exceeded the building code requirements for strength and did not present an imminent danger to its occupants or to the general public. Compared to many structures the authors have investigated and repaired, the corrosion level at the subject building was modest and could have been easily arrested at its then-current level by recognized and tested repair methods. The cost of these measures was a fraction of the sum the Owner stated it would cost to replace or splice 75 percent of the tendons in the building. There was simply no sound engineering basis for taking the Owner's course of action (demolition), which is unprecedented in the history of post-tensioned building repairs. It is necessary to arrest the corrosion in a repair program and to replace the grout anchorage protection at the live ends. Over the course of many years, the authors observed in other buildings much worse corrosion and have been able to successfully repair these structures. These repaired structures have almost a two-decade history of successful performance despite the incidence of corrosion significantly worse than observed and evaluated at the subject building. These repairs are based on sound engineering principles widely recognized and used in industry. Given the proven history of the repair strategy, there is no reason to doubt that, if repaired, the subject building would have continued to function with full integrity for the remainder of its service life.

EXTERNAL FACTORS

It is important to highlight the consequences of razing a building that could easily be repaired. Aside from the monetary costs associated with the drawn-out litigation, demolition, cleanup, and (presumably) replacement construction, a number of external factors should be noted in an effort to fully recognize the consequences of our decisions as engineers. Business owners were forced to temporarily close and re-locate on extremely short notice. These owners had to face loss of employees and loss of income while they desperately searched for new places to operate their businesses. Their customers had to be notified of their moving, and the businesses likely lost customers as a result.

At a glance, the environmental costs are staggering: the amount of embodied energy in a 25-story building and an adjacent 5-story parking garage are far from trivial; considering the additional energy required to demolish the structure, haul off the remains, and (presumably) build a replacement structure, what is left is an environmental travesty.

These consequences serve as a sobering reminder of our roles as engineers: our decisions have real-world economic, social, and environmental impacts. We have the knowledge to successfully repair corrosion in post-tensioned concrete structures, but if we forget, bad things can happen to good buildings.

ACKNOWLEDGEMENTS

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