Lateral capacity and repair of corrosion-damaged pile bents, part 3: Carbon-fiber-reinforced polymer application and strength testing

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- This is the final paper in a three-part series that explores the effects of corrosion damage on pile bents and demonstrates the effectiveness of carbon-fiber-reinforced polymer (CFRP) repair of severely corroded piles.
- This paper reports a study of the lateral load capacity of a one-third-scale pile bent bridge constructed with five piles that had exhibited greater than 96% cross-sectional steel strand loss and were repaired with CFRP material to restore bending resistance.
- Test results also demonstrated the suitability of numerical modeling to predict the strength gain from CFRP wrap repairs.

This paper is the third in a three-part series addressing the effect of corrosion damage on pile-bent-type bridge piers. The first paper covers the casting of one-third-scale prestressed concrete piles, the simulation of field damage conditions using an accelerated corrosion scheme, and laboratory testing of 4 five-pile bents at varying degrees of steel strand section loss.[1] The second paper describes numerical modeling of damaged piers and the comparison of modeling and laboratory results. That study demonstrated that a carbon-fiber-reinforced polymer (CFRP) repair could fully restore lateral capacity.[2] This paper describes research in which CFRP wrap repairs were applied to the same type of one-third-scale piles used in the first two investigations but with nearly all steel strand missing. This third study demonstrated the effectiveness of the repair by testing a five-pile bent constructed with repaired piles.

Background

A large percentage of overwater bridges in the United States have bent-type piers with piles vulnerable to corrosion. Therefore, having the ability to repair corrosion-damaged reinforced and prestressed concrete piles is an important aspect of maintaining or increasing the service life of existing overwater bridges. Prestressed concrete piles are particularly problematic components in overwater bridges because these piles have the thinnest concrete cover of any substructure element. In extremely aggressive environments, the concrete cover for prestressed piles specified in section 5.12.3 of the American Association of State Highway and

Transportation Officials' *AASHTO LRFD Bridge Design Specifications*[3] and Table 1.4.2-1 of the Florida Department of Transportation's (FDOT's) *Structures Design Guidelines*[4] is 76 mm (3.0 in.), whereas the concrete cover for other substructure elements ranges from 102 to 152 mm (4.00 to 6.00 in.). Further, when piles are in pile bent configurations, the concrete surface above the high-water level (in the splash zone) is subjected to cycles of wetting and drying, and the high chloride concentration buildup diffuses chloride more quickly into the above-water concrete surface than into any other portion of the pile. Piles that remain fully submerged to support water-level footings are far less prone to corrosion damage, and the footings with a thicker concrete cover requirement take the brunt of the splash-zone effects.

The FDOT *Bridge Inventory 2021 Annual Report*[5] provides a detailed accounting of all FDOT bridges and acknowledges that the cost to replace the bridges beyond their anticipated service lives (48% of all bridges in the FDOT inventory) is exorbitant. Given the costs of replacement, the report emphasizes the importance of continued condition monitoring, timely maintenance, and the use of new repair technologies to extend the useful service lives of existing bridges.

Like many areas of civil engineering, pile repair strategies have evolved to reflect owners' experiences of successes and failures, as well as a steady stream of research findings. In the late 1990s, FDOT classified pile jacket repairs as structural or nonstructural. In nonstructural repairs, a removable or stay-in-place form was installed around the pile, and the annulus that was formed around the pile was filled with concrete, mortar, or epoxy. Structural repairs included a reinforcing cage, whereas nonstructural repairs did not.[6] At that time, an overwhelming majority of repairs to FDOT bridges were nonstructural; however, very soon thereafter, it was found that stay-in-place forms masked the true state of the repair and continued corrosion deterioration went undetected. Nonstructural pile jackets were also found to fall off easily without the necessary confinement to withstand corrosion expansion forces beneath the unreinforced jacket (**Fig. 1**).

Although structural-type repairs included additional reinforcement, the concrete cover over the new reinforcing steel was thin, and those repairs also failed quickly. Today, structural pile jacket repairs are still used, but they incorporate zinc mesh electrically connected to the reinforcing steel to cathodically protect the new and existing steel. These types of repairs

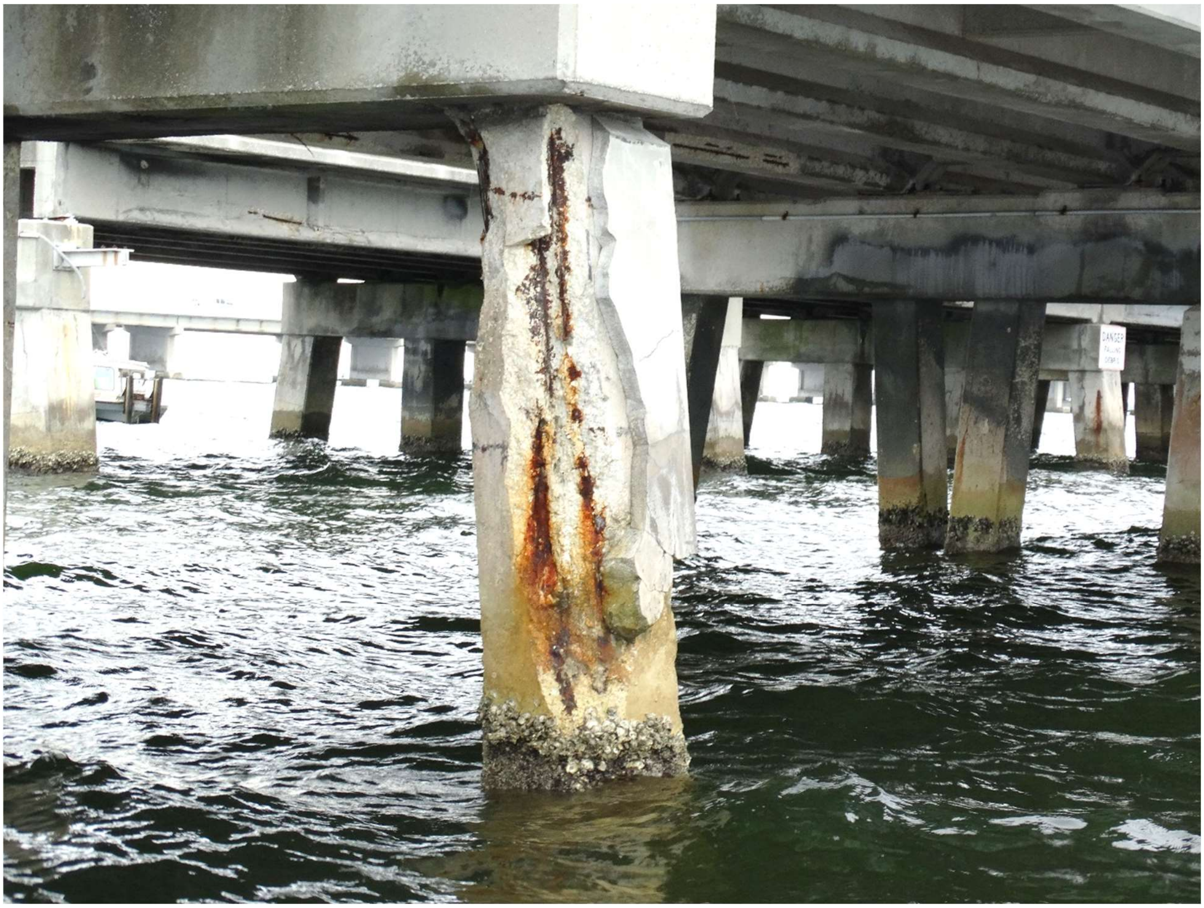

Figure 1. Failed nonstructural pile jacket repair. This repair method was commonly used from the 1970s through the 1990s.

Figure 2. Structural pile jacket repair installed circa 2021 and the same pile, three years later.

have an approximate life of 10 years,[7] but they do not always last that long. **Figure 2** shows a structural pile jacket with cathodic protection approximately three years after installation with the segment exposed to tidal flow missing.

An alternative repair strategy uses noncorroding fiber-reinforced polymers. This type of repair does not have the same lifespan limitations.[8,9]

Objectives and scope

The primary objective of this phase of the research was to determine the effectiveness of CFRP repairs in restoring the full lateral capacity of a pile bent with severely corroded piles. In the first research phase, one-third-scale pile bents lost 30% of their lateral capacity when the steel strand cross-sectional area loss was 50%. These losses were accompanied by catastrophic collapse.[1] The numerical modeling of phase 2 predicted a 50% reduction in lateral capacity when all steel in the corrosion damage zone was assumed to be lost; collapse would also be expected.[2]

For this third phase, piles that had been cast at the same time as the first phase in 2000 and stored in an outdoor research compound after the original study were recovered in 2020. At that time, the strands in the piles had corroded to at least 50% steel loss. Twenty years later, with continued corrosion, the piles were repaired with the CFRP wrap design presented in part 2. The repaired piles were then used to build another pile bent in the laboratory that was identical to bents in the original study. The new pile bent was load tested in the exact fashion used in the first study, and the results are compared herein to findings from the original undamaged, uncorroded pile bent test. In addition, the results from testing the new specimen are compared with the numerical model-predicted load response from part 2.

CFRP repair of the damaged piles

The design of the CFRP repair for the 152 mm (6 in.) square concrete prestressed piles established in part 2 of the research program called for one layer of a commercially available uniaxial fabric aligned longitudinally, followed by one layer of the same material applied as a transverse spiral wrap (**Fig. 3**).[2] The properties of the CFRP wrap material incorporated the manufacturer-recommended two-part epoxy as the adhesive; use of this adhesive requires the fabric to be fully saturated. **Table 1** lists the properties of the repair material. Confinement provided by the spiral wrap material was based on a transverse fiber slope of 1:4 or a 38 mm (1.5 in.) drop along each of the 152 mm sides. Hence, a 152 mm wide strip of fabric would touch side by side with subsequently applied spiral wraps once a full revolution was completed around the 610 mm (24 in.) pile top perimeter (4 sides × 38 mm drop/side = 152 mm).

Wrap repair procedures can be categorized by two field conditions of the pile: visible cracks in the piles but the pile shape is intact and where the concrete cover has not become dislodged or has missing or spalling concrete cover. At the

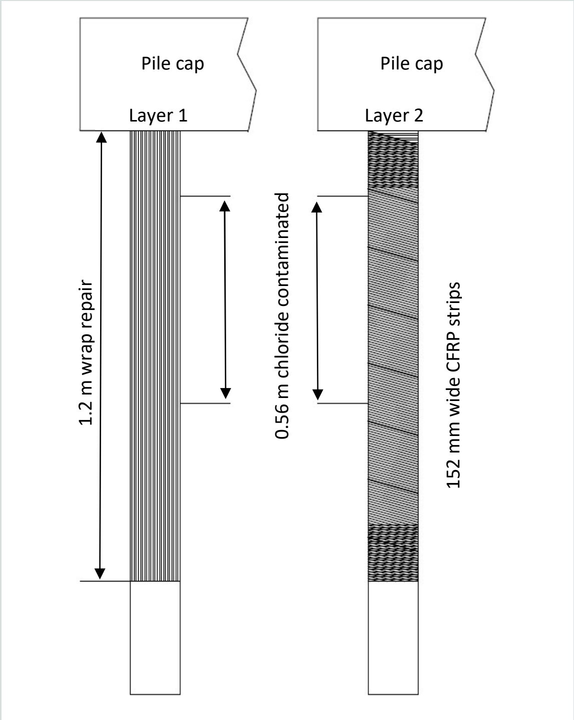

Figure 3. Repair scheme for severely corroded one-third-scale piles. Note: CFRP = carbon-fiber-reinforced polymer. 1 mm = 0.0394 in.; 1 m = 3.281 ft.

Table 1. Carbon-fiber-reinforced polymer repair material specifications

Property	Average ultimate	Design value
Tensile strength, MPa	1241	1055
Tensile modulus, MPa	64,828	64,828
Tensile strength per unit width, kN/mm	1.25	1.06
Nominal ply thickness, mm	1.016	1.016
Area density, g/m2	610	610

Note: 1 mm = 0.0394 in.; 1 m = 3.281 ft; ; 1 g = 0.0353 oz; 1 kN = 0.225 kip; 1 MPa = 0.145 ksi.

onset of any repair program, a crack pattern map should be developed and a cover soundness check should be performed to decide which repair approach is needed. The first case requires that the integrity of the existing cover be restored before wrapping begins. Restoration can be done by sealing and/or injecting the cracks. The aim is to ensure continuity and ensure that shear transfer can be developed between both the CFRP and the cover, and the cover and the core concrete. In the second case, the dislodged cover is completely removed, the remaining steel is cleaned and treated, a bond enhancement compound is applied to the concrete core, and the lost cover regions are formed, new cover is poured, and restored to their original dimensions. At that point, application of the FRP material can commence.

Pile condition survey

The condition of each pile was assessed using crack mapping, acoustic hammer tap testing, and surface potential measurements. In part 1 of the research program, the average maximum crack width in the piles increased progressively from 0.5 mm (0.02 in.) in the specimens with 10% steel loss to 3 mm (0.1 in.) in the piles with 50% steel loss.[1] Twenty years later, the average maximum crack width in all five piles used in part 3 of the research program had increased to 15 mm (0.59 in.) with a range of 8 to 28 mm (0.3 to 1.1 in.). The number of cracks had not increased, but the length of cracks had increased from 760 mm (30 in.) in the piles with 50% steel loss to 910 mm (36 in.) in the piles used in part 3. As discussed in the original paper in this series, the original chloride-contaminated damage zone was only 560 mm (22 in.).[1] **Figure 4** shows the condition of the five piles used in part 3 prior to repair. **Figure 5** shows example maps of cracks for the piles in the best and worst conditions (piles 5 and 2, respectively). Black regions denote open cracks or missing regions of cover. Each side of the piles was divided into 50.1 mm (2 in.) square regions (3 regions wide and 20 long) for the assessments.

Acoustic testing was performed on all four faces of each pile in general accordance with ASTM D4580, *Standard Practice for Measuring Delaminations in Concrete Bridge Decks by Sounding*.[10] The test is subjective. A technician listens as the concrete is tapped lightly with a hammer; if the technician hears the sound change to a hollow-sounding thud, that is considered to be an indication of cover delamination. The approach is much like finding a wooden stud behind drywall. In this case, test findings for the faces of the piles were compared with findings for the lower portions of the pile well below the damage zone, and all faces were found to be less sound than the lower portions. This finding meant that cover delamination in the damaged zone could be expected in all piles.

The investigators determined that piles 3 and 5, which had narrower crack widths than the other three piles, could be candidates for either cover removal or epoxy crack injection prior to wrapping because the overall shapes of these two piles had not changed by expanding corrosion products. The other three piles were deemed unsuitable for epoxy crack injection; hence, cover removal with subsequent form-and-pour cover restoration would be required.

Though analysis of the crack maps and soundness testing results identified a course of action for repairs, investigators performed surface potential measurements in accordance with ASTM C876, *Standard Method for Corrosion Potentials of Uncoated Reinforcing Steel in Concrete*,[11] to fully assess the initial pile conditions. The ASTM C876 method entails mapping the surface millivolt potential using a standard

Figure 4. Condition of piles 1, 2, 3, 4, and 5 at the start of part 3 of the research program, 20 years after they were cast. The piles are shown from left to right, looking at the east side.

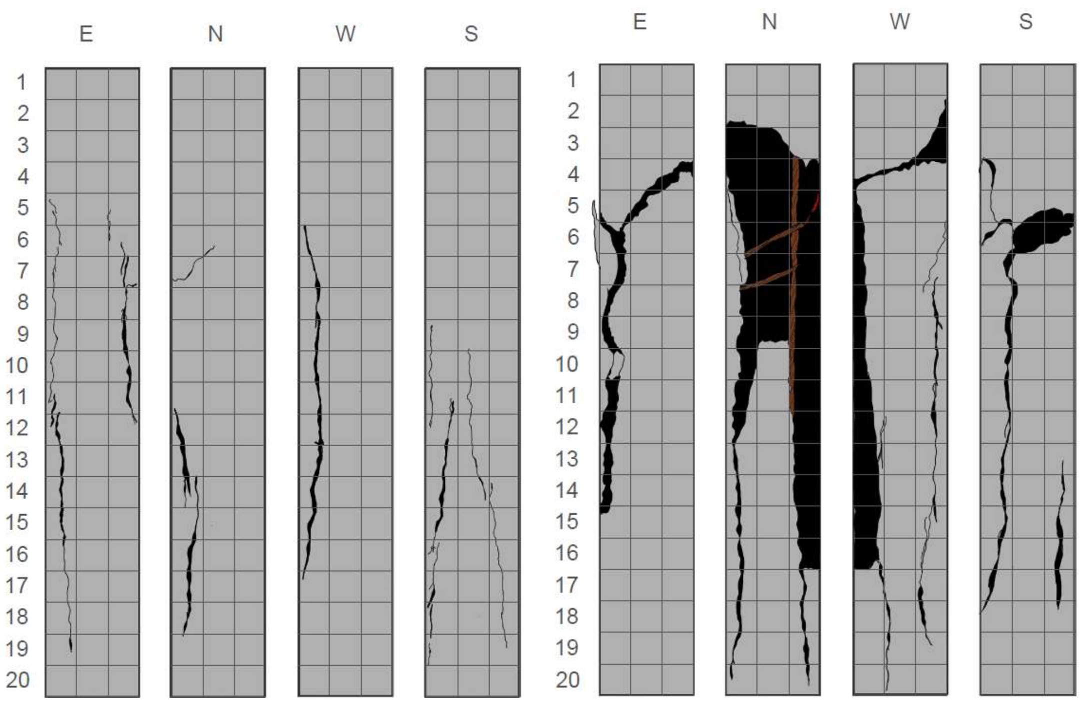

Figure 5. Crack surveys of pile 5 and pile 2. Black regions denote open cracks or missing regions of cover. Of the five piles used in this study, pile 5 was in the best condition and pile 2 was in the worst.

voltmeter connected to the reinforcing steel with the positive lead and the negative lead connected to a copper/copper sulfate ($Cu/CuSO_4$) reference electrode (**Fig. 6**). The reference electrode is touched onto a wetted concrete surface, and the electrical connection to the surface is ensured with a damp sponge. Measurements were conducted using the same 51 mm (2.0 in.) grid used for crack mapping (Fig. 5). Surface potential contours were then prepared along with average potential profiles for each side of the piles. Surface potential readings more negative than -350mV indicate a 90% probability of active corrosion, and surface potential readings above -200mV indicate no corrosion. Values between these thresholds indicate that the corrosion state in the measured area is uncertain.[11] In all piles and all faces, the contours were similar to those in **Fig. 7**, where surface potential values were far below -350 mV in the damage zone and were above -200 mV in the undamaged regions. At the bottom of all piles, surface potential values fell below -350 mV, but no cracks were observed. This finding was not surprising given that the piles had been in exposed conditions with moisture dripping down them since 2000 and that the strands were exposed as they extended below the bottoms of the piles. Figure 7 shows a typical surface potential contour plot, where dark-colored regions were corroding and light-colored regions were not. Figure 7 also shows a typical surface potential profile where the average of three surface potential measurements at each elevation for each face of the pile is plotted versus vertical position. All profiles showed active corrosion in the damage zone and at the pile bottom and nowhere else. Full details about the corrosion and crack mapping can be found elsewhere.[12]

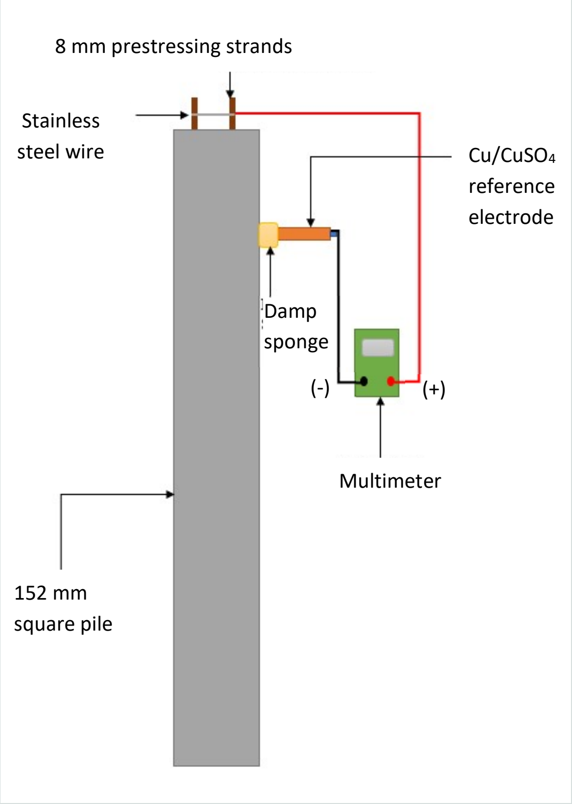

Figure 6. Electric surface potential test schematic. Note: $Cu/CuSO_4$ = copper/copper sulfate. 1 mm = 0.0394 in.

Given the vertical extent of the damage found in the pile condition survey, the length of the CFRP repair had to be increased from the initial assumption of 0.9 to 1.2 m (3 to 4.0 ft) to fully encapsulate the damage and provide sufficient longitudinal bond development.

Cover removal and steel assessment

For the piles where cover removal was required (piles 1, 2, and 4), investigators used a hammer and chisel to lightly tap on a corner crack in a direction that pushed the cover away from the center of pile. The chisel was never pushed directly into the crack toward the center of the pile nor was a prying action applied because such actions might stress or damage the core. The cover on sides with larger cracks was removed first, thereby releasing any interlocking of the cover from adjacent sides of the pile that might be holding the cover on the pile. Even on sides that showed fewer cracks (which were removed last), the cover was removed with little effort.

Piles 3 and 5 were candidates for either cover removal or epoxy crack injection prior to wrapping. When epoxy crack injection is used, the cracks are sealed at the surface with an epoxy putty. Then, an injection port is installed at the lowest point of each crack and a low-viscosity epoxy is pumped through the port to fill the crevices. The tops of the cracks are left open to identify when the cracks are filled and to prevent hydraulic pressure, which can dislodge delaminated portions of the cover. Investigators considered the number of epoxy injection ports that would be needed (one per crack) as well as the ease with which all cover material had been removed from piles 1, 2, and 4, and decided to remove the covers from piles 3 and 5 instead of pursuing epoxy crack injection. **Figure 8** shows the first two blows used to remove the cover on pile 3. Complete delamination of the concrete cover was confirmed in all piles, so the option to form and pour the cover replacement was appropriate.

Figure 9 shows the state of the piles immediately after cover removal. The remaining reinforcing steel was sparse. Stirrups were mostly gone, and none were continuous around a pile. In most cases, one of the seven prestressing strand wires remained but was badly corroded. Vestiges of the other six wires were rare. Investigators estimated that less than half of one wire diameter in each strand represented the remaining steel cross section, making the steel loss greater than 96%.

Pressure washing to remove loose debris was the only mechanism used to prepare the core of the piles. The steel was not removed unless it came off easily. This decision was based on three considerations: First, the CFRP wrap on the form-and-pour section was to be tested in relatively short order, so further corrosion would not be a factor. Second, the piles were not under a stabilizing service load as they would be in field conditions and

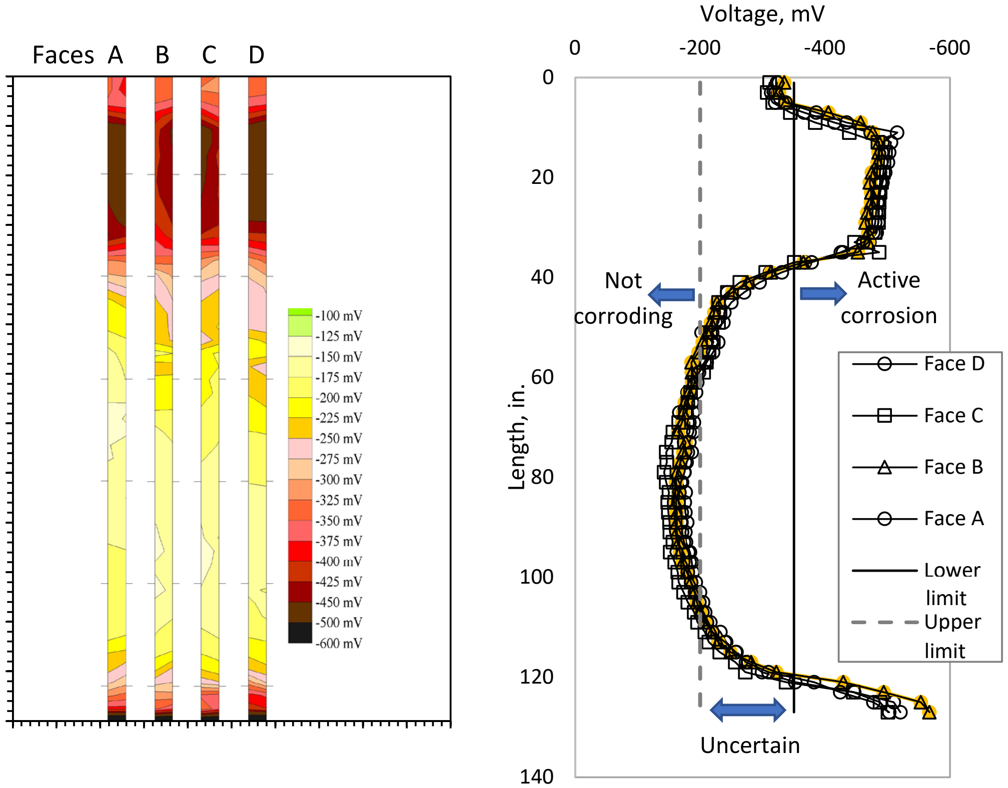

Figure 7. Typical surface potential contours and profiles. Note: 1 mm = 0.0394 in.

Figure 8. Concrete cover removal of pile 3, which was originally intended to be repaired by epoxy injection.

Figure 9. Condition of piles immediately after cover removal.

uneven steel removal could have induced an eccentric loading to failure during the repair. Last, the remaining core concrete was only 95 mm (3.7 in.) square, which constitutes only 39% of the original cross section, whereas the core in field pile sizes constitutes a larger fraction of the pile cross section.

Restoration of cross sections

Before CRFP could be applied, the missing concrete cover had to be restored to the full original cross section so external reinforcement could bond and transfer forces to the concrete above and below the damaged region. Each pile width and section depth was measured. Because the dimensions of the piles varied only slightly, investigators decided to use the largest measured dimension in each direction to fabricate a single sheet metal form to be used for restoring the covers on all piles. The form was equipped with a fill port/funnel (often referred to as a bird's mouth because the shape resembles a bird beak) and a sliding sheet metal door behind the fill port to serve as a debonding separation between the volume of grout in the fill port and the newly shaped pile side. **Figure 10** shows the form around a pile before it was poured.

A repair mortar was placed into the form after the pile was first wetted for 24 hours and left to dry for 2 to 3 hours. This process achieved a saturated surface-dry condition for each pile before the mortar was placed. The patch material was a rapid-setting, cement-based concrete repair mortar with high strength (55 MPa [8.0 ksi]), which is ideal for vertical concrete repairs. The five piles were systematically repaired, one at a time, within 10 days. The form was attached to a pile and sealed at the base to prevent leakage of the highly fluid mortar, flooded with water for 24 hours, drained, and allowed to dry for 2 hours. Then mortar was placed and allowed to cure overnight. The form was then stripped and the process repeated for the next pile. Upon removal of the form, a soaker hose assembly at the top of the pile (**Fig. 11**) was used to keep the newly placed repair material wet for an additional 72 hours, in accordance with the mortar manufacturer's recommendations.

To facilitate the CFRP wrap around the piles, surface roughness and discontinuities in the mortar caused by the form fitment were ground flat and the corners of the piles were rounded. Flat surfaces were ground with a 178 mm (7.0 in.) diameter diamond-impregnated grinding cup. The corners were rounded with a 9.5 mm (3⁄8 in.) radius demi-bullnose diamond grinder attachment (commonly used for finishing granite countertops). Both attachments (**Fig. 12**) were effective and faster than aluminum oxide alternatives. **Figure 13** shows the sharp corners and discontinuities in the mortar behind the fill port, as well as a pile after surface preparations. Corner radius preparations extended slightly beyond the anticipated CFRP repair regions but not into the 102 mm (4 in.) cap embedment region at the tops of the piles.

Figure 10. Sheet metal form secured to a pile and filling the form.

Figure 11. Restored cross section kept wet with soaker assembly dripping down all faces.

Figure 12. Grinder attachments.

Figure 13. Restored section before and after surface finishing.

CFRP application

The CFRP repair design called for one layer of a uniaxial material longitudinally aligned followed by one layer of the same material transversely aligned (Fig. 3). Longitudinal material was precut into pieces measuring 1.2 m (4 ft) long and 0.63 m (2.1 ft) wide. The length was based on the extent of crack damage, and the width was based on the sizes of the pile sides, which in some cases were slightly more than the nominal dimension of 152 mm (6 in.). Some lateral overlap of the longitudinal fibers was expected. The transverse material was precut into pieces measuring 4.9 m (16 ft) long and 152 mm wide. The transverse wrap fabric length allowed for one complete revolution around the pile top and then a downward spiral dropping one-fourth of the wrap width for every side of the pile. At the bottom of the repair length, the spiral was discontinued and one full revolution at the same elevation was applied. In the field, the width of the spiral material is often set to equal the width of the pile side and the length is limited to the amount that can be handled by the installers, who might be divers for underwater repairs. In this case, a one- or two-piece spiral wrapping method was reasonable. For a two-piece wrap, the second piece must slightly overlap the first piece along the spiral to develop the fibers.

The CFRP application was performed in three steps:

1. First, the precut fabric was saturated with epoxy.
2. Next, the epoxy-saturated fabric was applied to the piles (in two layers).
3. Last, the freshly placed fabric was held in place with a plastic stretch wrap, which also consolidated the saturated fabric and pushed out air voids.

This process is often performed by two teams: one to mix the epoxy and saturate the fabric and one to install the material on the concrete surface and apply the stretch wrap. In this case, three teams were used, with separate teams for the installation of the material and application of the stretch wrap.

The wet layup method was used where epoxy is rolled onto the concrete surface and fabric is saturated with epoxy before the material is applied to the pile. This method is faster than dry layup methods and ensures that there are no dry regions with inadequate epoxy saturation; however, there is a risk that too much epoxy will make the material slide off vertical surfaces. Thickening additives can be used to minimize material sag, but they were not used in this study. Saturation was ensured by laying out the dry material on the work surface, fully wetting one side using the recommended coverage [2] of 1.5m^2/L (60 ft^2/gal.) and then rolling the fabric onto a transfer dowel. This method allowed the dry bottom face of the fabric to be wetted by the top side and helped prevent the fabric from becoming too wet and heavy. This method is particularly effective for strips of fabric that are longer than the work surface because rolling helps process the material uniformly in limited-space conditions. The rolled material is also easy for the installation team to handle and apply. Dry layup entails wetting the concrete surface, applying the dry fabric to the surface, and then rolling epoxy onto the dry fabric outer surface until it is fully saturated and all air voids are expelled. This process can be done by one person or team because the fabric adheres so well to the surface; however, the dry layup process is two to three times longer than the wet layup process, which can be problematic when epoxies that have a short pot life are used. In this case, the manufacturer indicated that the epoxy has a long pot life, with reactivity of 6 to 7 hours, and would dry to a tack-free condition in 14 to 16 hours.

The installation team applied the longitudinal material first, all in one 0.63 m (2.1 ft) wide × 1.2 m (4 ft) long piece covering all four sides. The fabric was aligned with the 1.2 m edge and longitudinal fibers along the centerline of one pile side. By starting at the centerline of one side the edges of the fabric do not pull away as easily as if initiated or terminated at a rounded corner. For field repairs of larger piles, four (or two) longitudinal fabric strips the width of one (or two) pile sides would be applied separately, so the saturated pieces would be easier to handle; strips would extend from the center of one side around the corner to the center of the adjacent side, and so on. In underwater repairs in streams or tidal flowing-water conditions, individual longitudinal strips will tend to pull away from the pile surface prior to transverse wrapping, which then holds them in place. Bidirectional woven fabrics mitigate this issue because they can be used to spiral wrap the pile first from top to bottom and then back up to the top with the spiral crisscrossing, ensuring that longitudinal fibers develop across the adjoining spiral seams.

Next, for four of the five piles, the installation team applied the 4.9 m (16 ft) long × 152 mm (6 in.) wide transverse strips in a spiral fashion using the 1:4 slope described previously. For the remaining pile, the team used eight individual strips measuring 0.71 m (2.3 ft) long by 152 mm wide. In this case, the strips were applied horizontally with no spiral. The spiral layup was found to be faster.

Plastic stretch wrap was applied immediately after the transverse layer was applied. Stretch wrap is intended to promote intimate contact between the saturated fabric and the concrete surface during the curing period. Previous studies showed that wider square piles tended to achieve less contact pressure from the stretch wrap at the center of the pile side compared with the corner regions. This finding led to pressure-bagging and vacuum-bagging FRP repair schemes.[13,14] In three of the five piles, an additional layer of 6 mm (¼ in.) thick air bubble packing was loosely secured to the stretch-wrapped surface with tape and a second layer of stretch wrap was applied. This approach provided a subtle curvature along the pile side, allowing the stretch wrap to apply lateral pressure at the centers of the pile sides. Variations in the transverse fiber installation and use of air bubble packing were introduced to evaluate possible effects of different layup schemes. **Figures 14** through **18** show the steps in the repair process.

Figure 14. Saturating and rolling the 1.2 × 0.63 m (48 × 25 in.) longitudinal fabric (fibers run from left to right).

Figure 15. Longitudinal and transverse spiral carbon-fiber-reinforced polymer fabric application.

Figure 16. Application of stretch wrap.

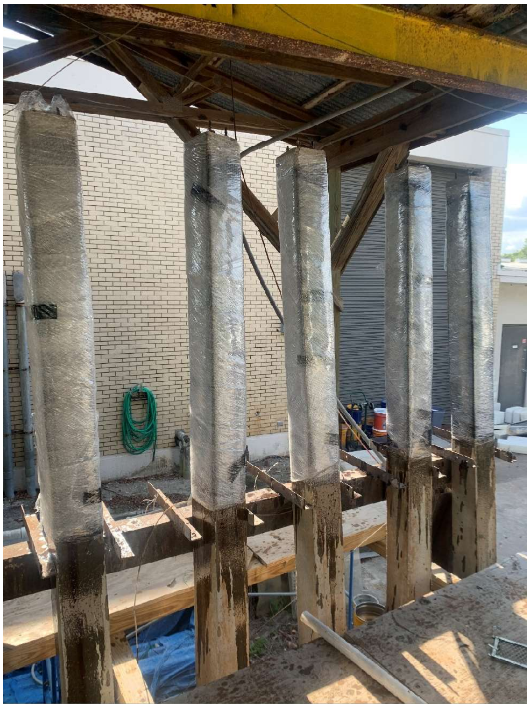

Figure 17. All piles immediately after repairs were completed.

Figure 18. Repaired piles with stretch wrap removed (24 hours after repair).

Pile bent construction

To construct the CFRP-repaired pile bent, the investigators followed the exact steps used in 2000[1,15] to construct the original one-third-scale pile bents in the laboratory. For the repaired pile bent, the only departure from field conditions was that the piles were repaired before the bent was constructed and not while in the bent. This change was necessary because the piles were too fragile to handle or transport by trailer to the load testing facility. Similar concerns arose in 2000 for the piles with 50% steel loss; at that time, four steel angles were bolted around the piles to provide a structural splint.[15] In this case, the piles had even less remaining steel and were deemed too fragile for the structural splint approach; piles were repaired in the outdoor storage compound prior to transport.

The model bent was constructed directly under a high-capacity laboratory reaction frame and consisted of an unrestrained pile cap, CFRP-repaired piles, and a fixed floor-level footing. Formwork for the pile cap was built to match the original 2000 plans.[1] The floor-level form made of 13 mm (½ in.) thick steel plates in 2000 was reused and bolted to the laboratory floor anchors. The reinforcement cages for the pile cap and floor footing were identical to those used in the other pile bents (part 1 of this paper series[1]). Piles were positioned vertically into the floor footing on 102 mm (4 in.) chairs and held in the vertical position by temporary falsework. Steel all-thread rods were used to clamp two full-length wood beams on opposite sides of the piles 114 mm (4.5 in.) below the tops, thus maintaining spacing and verticality. The wood beams also served to support the premade 13 mm (0.5 in.) thick plywood pile-cap form. **Figure 19** shows the floor footing and pile cap prior to concreting.

Concreting was performed with a 0.4 m^3 (0.5 yd^3) side discharge concrete bucket specially built in 2000 to place concrete under the laboratory reaction frame (**Fig. 20**). The 28-day compressive strength of the cap and footing concrete was 47.3 MPa (6860 psi). Forms were stripped when compressive strength achieved 44.9 MPa (6510 psi).

Instrumentation and test setup

Instrumentation consisted of twelve 120Ω PL-60 surface-bonded strain gauges per pile (60 gauges total) mounted at three positions along the pile length:

- directly beneath the cap

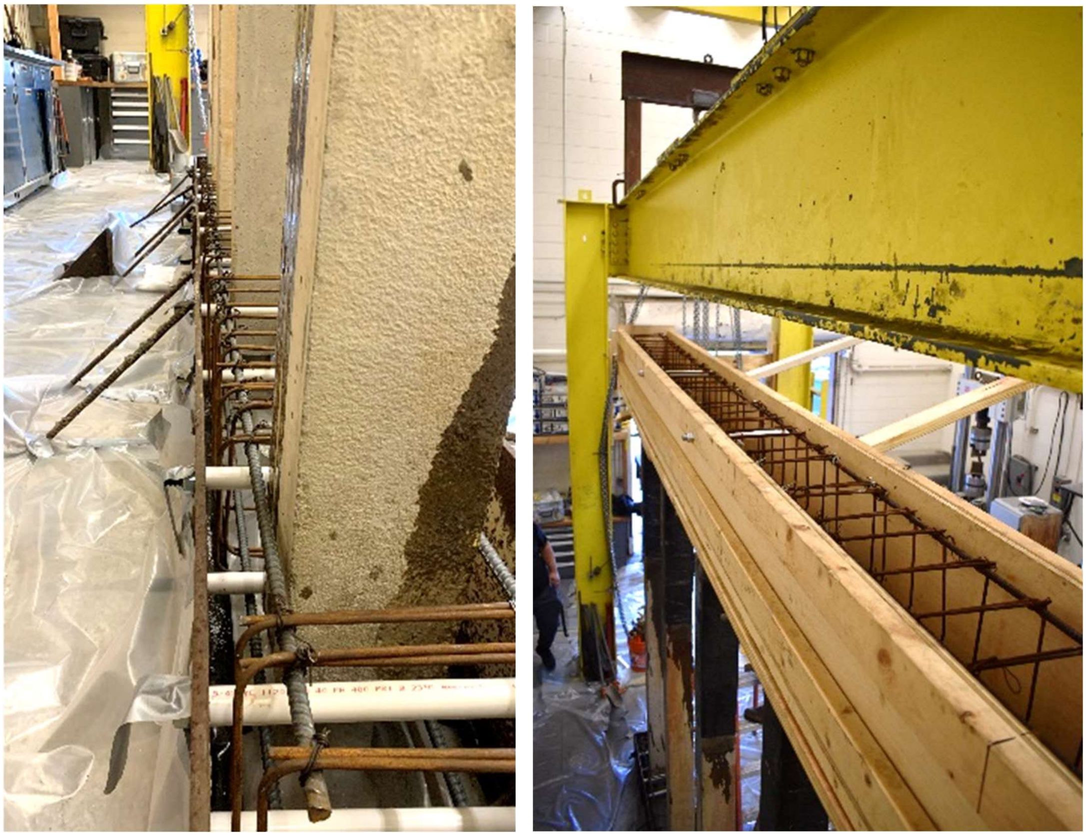

Figure 19. Forms for floor-level footing and pile cap prior to concreting.

Figure 20. Side discharge concrete bucket suspended above the pile cap.

- directly above the footing
- at midheight

One gauge was mounted at each position on each face. The PL-60 has a gauge length of 60 mm (2.4 in.), so the effective center of the gauge was approximately 30 mm (1.2 in.) below the cap or above the footing. Stringline displacement transducers were attached at the centerline of the cap at the leading and trailing ends to measure out-of-plane movement. Displacement transducers were also located at the pile cap and quarter points down the pile length to measure in-plane movement of the trailing pile where loads were applied.

Lateral loading was displacement controlled at 7.6 mm/min (0.30 in./min) applied at the center of the pile cap using a hydraulic actuator. Vertically aligned hydraulic actuators were positioned above each of the five piles to represent service load from girders. Vertical loads were maintained at 44.4 kN (10 kip) per pile and were free to translate with the pile cap during the lateral loading event using high-capacity rollers pushing up against the overhead reaction frame flanges. A shear force cell was placed between each of the hydraulic actuators and the pile cap. All strain gauges, displacement transducers, shear cells, and load cells were monitored and recorded at 1 Hz. **Figure 21** shows the fully instrumented pile bent prior to loading.

Figure 21. Fully instrumented pile bent.

Repaired pile bent test results

Loading was continued to the extent of the 152 mm (6 in.) hydraulic actuator stroke, which resulted in 142 mm (5.59 in.) of pile cap movement. No pile failures were observed in the repair zone. The bottom side of the pile cap directly adjacent to piles 2 and 4 broke away from the cap on the leading (tension) side of the pile, leaving a swept-out appearance that indicated partial loss of pile–to–pile-cap fixity. The footing with more embedment retained fixity, and all piles exhibited concrete crushing and spalling on the leading (compression) side of the piles. No concrete failures in the floor footing were observed. **Figure 22** shows failures in the cap and in a pile above the footing.

With the load fully removed, the pile bent elastically recovered 118 mm (4.65 in.) of lateral movement, which was more than the lateral movement recovered in any of the pile bents in the original study,[1] including the undamaged control with 0% steel loss. **Figure 23** shows the response to loading for the CFRP-repaired pile bent as well as the responses for the pile bents from the original study. The peak lateral load of the repaired pile bent was 43 kN (9.7 kip), which is slightly greater than the peak lateral load of the undamaged control (41.5 kN [9.33 kip]). The peak lateral loads in the repaired pile bent and the control occurred at similar displacements, 74 and 70 mm (2.9 and 2.8 in.), respectively. The linear-load response in the repaired pile bent was slightly less stiff than the response in the undamaged control bent, but stiffness was not less than the unrepaired damaged pile bents.

Investigators also compared the repaired pile bent response to the model-predicted pile bent response from part 2 of the research program.[2] **Figure 24** shows close agreement between the measured and modeled lateral load responses up to 73 mm (2.9 in.). At this load, the model piles start to reach the bending moment limit of the unrepaired pile, which exists at an infinitesimally small region between the cap and the top of the CFRP repair. Without this limit, the model assumes that the CFRP pile sections extended into the cap. Model response was terminated when three of the five piles reached the moment limit.

Discussion

The load-testing scheme established in part 1 of the research program focused on replicating the bending moment distribution in the piles down to an inflection point 10*D* below the pile cap, which is similar in location to what would be found in field conditions (6.3*D* to 13.2*D*), where *D* is pile size. This portion of the bending moment diagram encompassed the corrosion-damaged regions in the laboratory pile bents and the field surveys of bridge damage. The region below the inflection point down to the floor-level footing was not

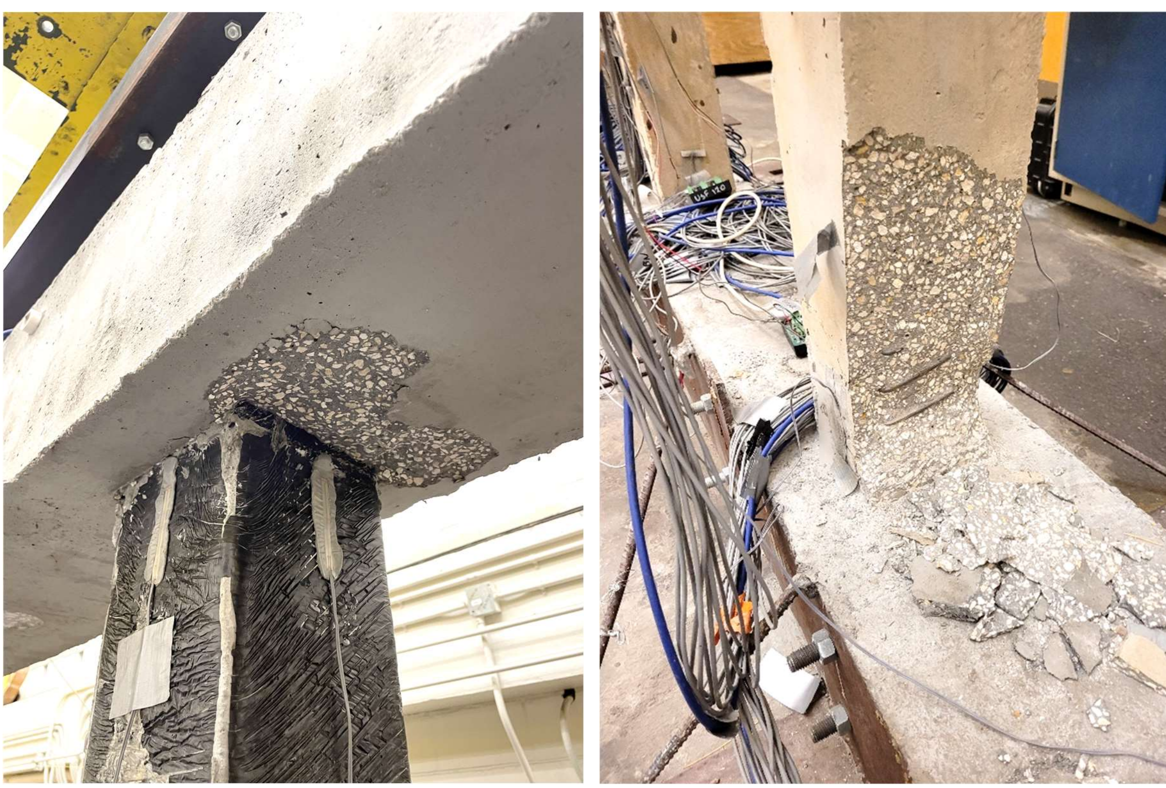

Figure 22. Leading face of pile 4 after loading, cap and pile spalling.

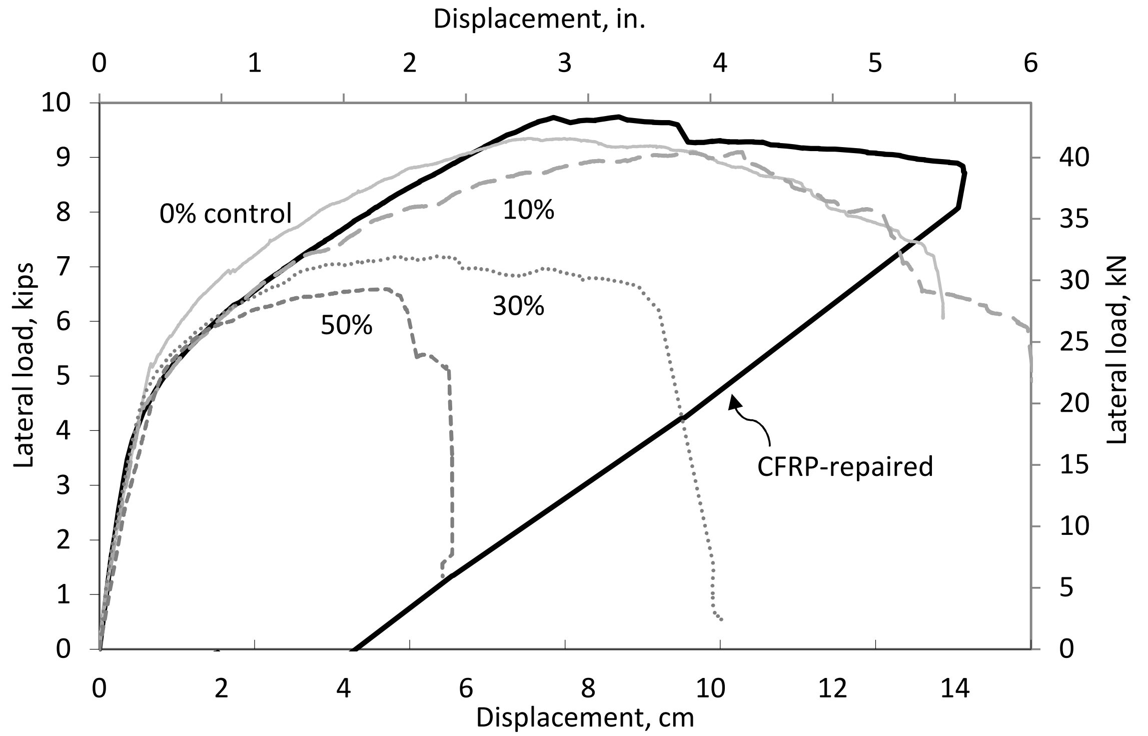

Figure 23. Comparison of measured displacement in response to lateral loading for the repaired pile bent and the unrepaired pile bents. Note: CFRP = carbon-fiber-reinforced polymer.

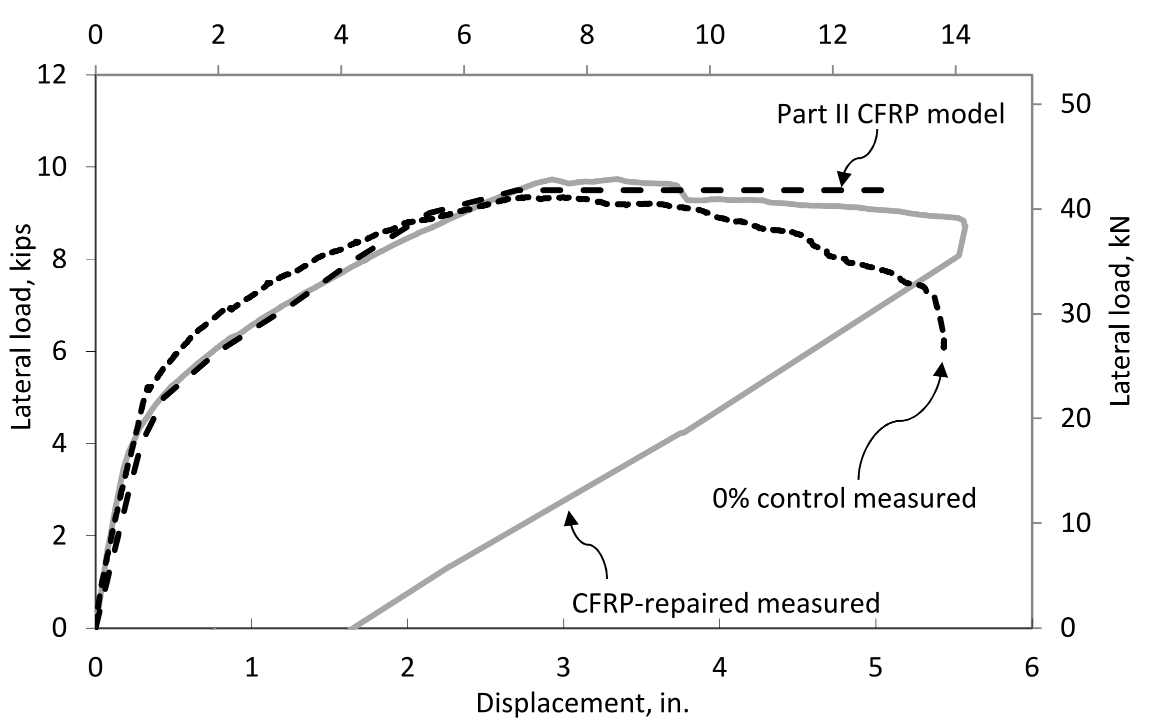

Figure 24. Measured versus model-predicted displacement of the repaired pile in response to lateral loading. Note: CFRP = carbon-fiber-reinforced polymer.

intended to replicate soil restraint. Instead, it was the most straightforward means to produce the bending moment distribution and inflection point at the correct location beneath the cap. Theoretically, a symmetric bending moment distribution would result from the top and bottom of pile fixity conditions. To a large extent, this assumption was correct, with the exception that the section properties of the corroded region of the piles tested in part 1 were weaker than the corresponding position in the piles below the inflection point. This weakening resulted in the zero-bending-moment inflection point moving upward from the midpile position in the unrepaired corroded pile bents as lateral load increased. In contrast, the 1.2 m (4.0 ft) long repaired regions of the piles were stronger than the regions below the inflection point, which caused the inflection point to move downward during loading; however, the embedment length of the pile into the pile cap at the top (102 mm [4.00 in.]) was less than the floor-level footing embedment of 203 mm (8.00 in.), which in part offset the effects of the upper-region pile strength gain. Nonetheless, the inflection point did move downward, indicating an overall stiffer pile and pile-fixity state in the upper half of the pile. **Figure 25** shows the strain distribution in the pile bent with no corrosion loss and in the CFRP-repaired pile bent where the upward and downward movement of the inflection point was noted as lateral load increased in increments of 5.56 kN (1.25 kip). In the last load increment shown, no pile cracking on the tension side beneath the pile cap was detected in the CFRP-repaired piles (+312 µe); data for the unrepaired pile section indicated cracking and a drop in strain at the same load level in the same region.

The fixity afforded from the minimum required embedment of prototype piles (for example, 0.3 m [1 ft]) is substantial where, in the case of the model pile bents, the cap concrete failed before the pile bending resistance of the CFRP-repaired piles. The undamaged control specimen from part 1 of the research program demonstrated both cap and pile failure modes.

Numerical model results matched closely with the measured pile bent response, but all models were run with a fixed-head condition and not the pinned-head condition as defined by FDOT specifications for an embedment length of 0.3 m (1 ft). Failure to assign some level of fixity results in an unconservative prediction of bending moment in the region most prone to corrosion damage beneath the cap.

The CFRP-repaired pile bent exhibited a large degree of ductility and elastically recovered 82% of the applied displacement. In field conditions where the mudline region cannot develop the highest bending moments, the concrete crushing of the piles (at the floor-level footing) would not occur, and a more elastic response of the piles would be expected.

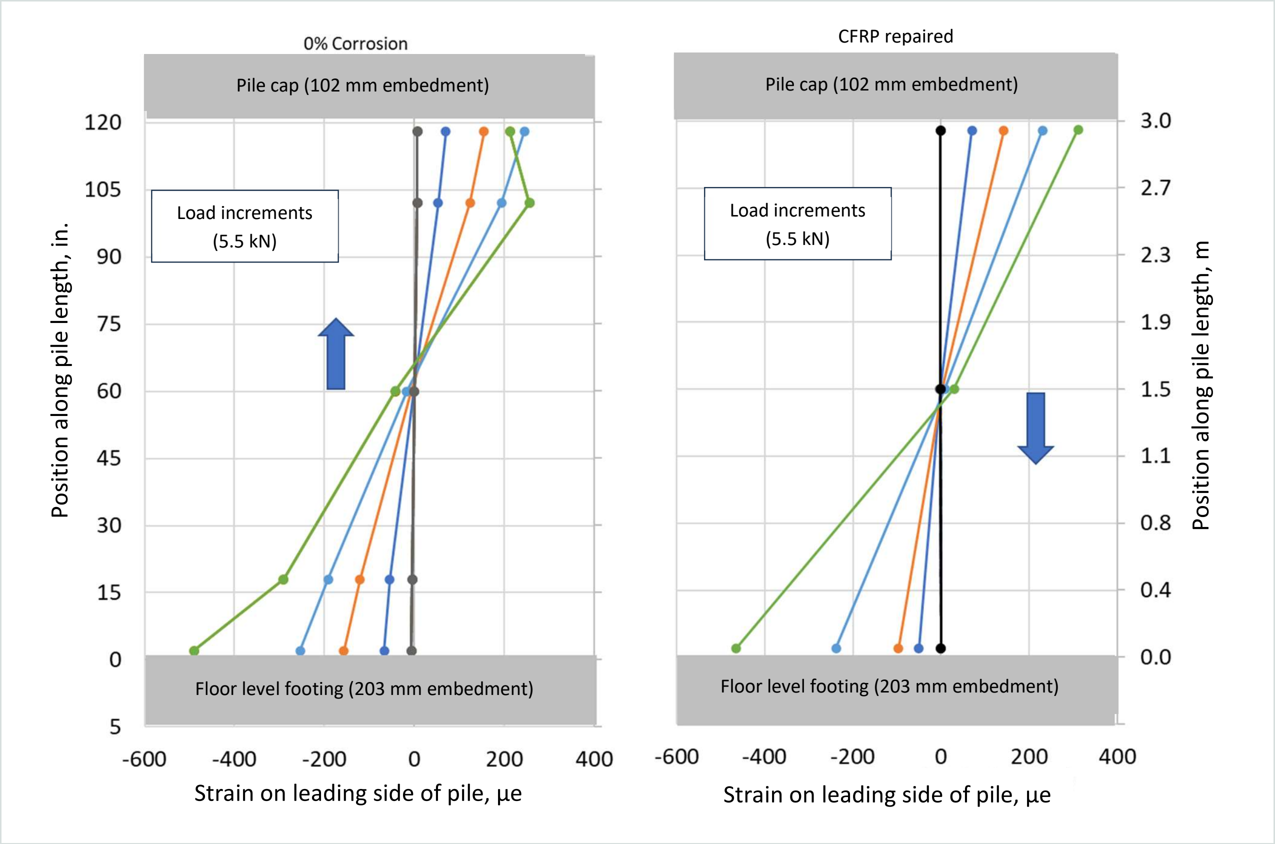

Figure 25. Strain distribution on leading face of pile 1. Note: CFRP = carbon-fiber-reinforced polyer. 1 mm = 0.0394 in; 1 kN = 0.225 kip.

However, large lateral displacements of piles in soil would also not be fully elastic due to the plastic deformation of the soil from lateral loads. Assuming half of the lateral displacement was distributed to the upper and lower halves of the tested piles, only about 76 mm (3 in.) of the displacement can be attributed to the region above the inflection point. Below the inflection point, a pile embedded in soil would require far more lateral movement to develop the same bending moment distribution in the upper half, below the cap. Scaling effects of the one-third-scale models translates into an equivalent lateral displacement in the field of greater than 0.46 m (1.5 ft) to impose bending failure. An unrepaired pile bent with 50% to 100% steel loss developed only half the original lateral resistance, and catastrophic collapse occurred at displacements 70% less than those for the CFRP-repaired alternate.[2]

Conclusion

This paper reports on the final part of a 20-year research program to assess the effects of corrosion damage on lateral pile bent capacity, the efficacy of numerical modeling in replicating CFRP repairs for field applications, and the performance of CFRP repairs on severely corroded prestressed concrete piles under lateral bending conditions. Using recommended materials and guidance, investigators demonstrated the ease with which CFRP repairs can be made. Subsequent testing of a one-third-scale, CFRP-repaired five-pile bent found that the lateral load capacity of the original uncorroded/undamaged pile bent was completely restored. In fact, a slightly higher ultimate load was realized by the CFRP-repaired alternate when compared with the undamaged pile bent, and the elastic rebound exceeded all other pile bents tested. In short, no corrosion-damaged, CFRP-repaired region failed. No debonding or loss of confinement was observed, so the effects of the two wrap-installation variables introduced—air bubble packing and transverse fiber installation/alignment—could not be determined.

No cost evaluation was presented, but the availability of quick and effective repairs of corrosion-damaged bridge piers is an essential aspect of maintaining a large inventory of bridges well past the anticipated service life. The results demonstrate that CFRP repairs could be considered for corroded piles to extend the useful lifespan of these structures; however, because testing was performed shortly after repair, durability was not considered.

Acknowledgments

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Notation

D = pile diameter

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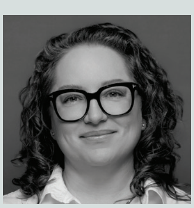

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Abstract

Pile-bent-type bridge piers support a large fraction of today's overwater bridges. In aggressive environments, the thin concrete cover typical of prestressed piles makes the piles vulnerable to corrosion damage not encountered by submerged piles supporting water-level footings. This vulnerability is due to the repetitive cycles of wetting and drying in the regions above and within the tidal sea level through which the piles extend.

This paper is the final paper in a three-part series that explores the effects of corrosion damage on pile bents and demonstrates the effectiveness of carbon-fiber-reinforced polymer (CFRP) repair of severely corroded piles. This paper reports a study of the lateral load capacity of a one-third-scale bridge pile bent constructed with five piles that had greater than 96% cross-sectional steel strand loss and were repaired with CFRP material to restore bending resistance. The lateral load capacity of the repaired pile bent met and exceeded that of an undamaged, uncorroded pile bent built from the same lot of prestressed concrete piles, which had been tested 20 years earlier. Test results also demonstrated the suitability of numerical modeling to predict the strength gain from CFRP wrap repairs. The findings are encouraging and support the applicability of new technologies to maintain aging infrastructure.

Keywords

Bridge pier, bridge pile, carbon-fiber-reinforced polymer, CFRP, CFRP wrap repair, corrosion damage, lateral capacity, pile, pile bent, steel loss.

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