CONSTRUCTION, STRENGTH, AND DRIVING PERFORMANCE OF CFRP-PRESTRESSED CONCRETE PILES

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ABSTRACT

Prestressed concrete piles are often used for bridge foundations in Florida, but they are subject to deterioration from corrosion, especially in saltwater environments. The Florida Department of Transportation (FDOT) supported a study on concrete piles, prestressed with non-corroding carbon-fiber-composite cable (CFCC). Five (5) 610-mm [24-in.] square piles were constructed with 15.2-mm [0.6-in.] diameter CFCC strands. Strains were measured during strand detensioning and during flexural strength tests, to evaluate the strand's transfer and development lengths. The flexural test on a 12.2-m [40-ft] pile resulted in a strength that was 8% higher than theoretical and a mid-span deflection of over 229 mm [9 in.] at failure. Two (2) 30.5-m [100-ft] piles, monitored under hard driving conditions at a bridge construction site, behaved well with no major damage or loss of prestress. CFCC-prestressed concrete piles show promise for use in foundations and should result in prolonged bridge life.

Keywords: Carbon-fiber-composite cable (CFCC), CFRP, Prestressed pile

INTRODUCTION

Prestressed concrete piles – a common foundation type for bridges because of their economy of design, fabrication, and installation – deteriorate when their steel prestressing strands corrode. This is prevalent in harsh marine environments, particularly in the splash zone where the water level fluctuates; periodic wet and dry spells slowly deposit salt that penetrates the concrete and corrodes the strands. Deteriorated piles can be retrofitted or replaced, but these solutions are expensive and not long-term. An alternative to steel strands is needed, to prolong the life of the foundation and therefore the bridge structure. One such alternative is carbon-fiber-composite cable (CFCC), which does not corrode. The cost of CFCC is currently higher than steel strands; however, the cost of prestressing strand materials is a relatively small percentage of a bridge's overall cost. The higher initial cost of CFCC would likely be paid back with the long-term benefit of prolonged maintenance-free bridge life. This paper presents on efforts to construct CFCC piles, as well as on test results to determine the transfer length, development length, flexural strength, and driveability. Five (5) 610 mm x 610 mm [24 in. x 24 in.] square CFCC-prestressed concrete piles were constructed at Gate Precast Company (GATE) in Jacksonville, Florida. Three (3) of the piles were 12.2-m [40-ft] long, were monitored for transfer length at GATE, and were tested for development length and flexural strength at the Florida Department of Transportation (FDOT) Marcus H. Ansley Structures Research Center in Tallahassee, Florida. The two (2) 30.5-m- [100-ft-] long piles were driven and monitored at a construction site in Volusia County, Florida.

CFCC MATERIAL

Fiber-reinforced plastics (FRPs) are high-strength, composite materials that are impregnated with a resin material. FRP can be made from carbon, such as carbon-fiber-composite cable (CFCC). CFCC is poly-acrylonitrile based, where individual wires are twisted and wrapped with synthetic yarns to protect the fibers from ultra-violet radiation and mechanical abrasion. From the manufacturer's technical data on CFCC, pull-out tests show that it has bond strength to concrete of 6.67 MPa [967 psi] – more than twice that of steel. The thermal coefficient of expansion is $0.62\text{x}10^{-6}$/degree Celsius, about 1/20th that of steel. Relaxation is less for CFCC than for steel. CFCC is also lightweight, and the twisted-strand configuration makes it easy to handle and coil. However, disadvantages are its low impact resistance, high cost, and brittle behavior which means that it does not yield like steel does before failing (the stress-strain relationship is linear up to failure). Care must be taken to protect the strands from damage, deformation, and sudden shocks caused by heavy or hard objects.

CFCC is produced in cables with diameters ranging from 5 to 40 mm [0.2 to 1.57 in. Cables are made from one (1), seven (7), 19, or 37 wires. In the piles that were cast and tested for this research, a 7-wire strand diameter of 15.2 mm [0.6 in.] was used for longitudinal prestressing, and a single wire with a diameter of 5.0 mm [0.2 in.] was used for transverse spiral reinforcement; these diameters are nominal and per the manufacturer. The manufacturer's reported effective cross-sectional areas of the CFCC strand and wire are

115.6 mm^2 [0.179 in^2] and 15.2 mm^2 [0.0236 in^2], respectively; these areas were used in the research calculations. Measured by immersion testing per *ASTM D792-13*[1], the CFCC strand's cross-sectional area is 143.5 mm^2 [0.222 in^2]. The manufacturer's guaranteed ultimate tensile strength (GUTS) is 270 kN [60.7 kip] for the 15.2-mm [0.6-in.] strands and 38 kN [8.54 kip] for the 5.0-mm [0.2-in.] wire. The strands' nominal modulus of elasticity is 155 GPa [22,480 ksi], and the ultimate tensile strain is 1.6%; the wires' nominal modulus of elasticity is 167 GPa [24,221 ksi].

LITERATURE REVIEW

In the past couple of decades, several research projects have been performed on carbon-fiber-reinforced polymer (CFRP) for structural applications. ACI[2] provides an exhaustive narrative on previous research and the historical development of the material, as well as material properties and design recommendations. Previous evaluations on using CFRP as a replacement for traditional steel reinforcement or prestressing have focused on concrete beam members. For columns, CFRP has mostly been used for retrofitting applications.

Bridge Street Bridge was the first bridge in the U.S. to use CFRP. Before it was constructed, Grace[3] performed flexural testing on the beam design. Grace[4] reported on long-term monitoring results and concluded that the performance of CFCC is comparable to steel. Tests by Grace et al.[5] on T-beams prestressed and reinforced with CFCC and CFRP showed that CFCC performance is comparable to that of steel. The flexural load carrying capacity and deflection were 107% and 94%, respectively, relative to a beam with steel.

Other research has focused on CFRP's transfer and development lengths, which are important parameters for design of structural members. Tests by Issa et al.[6] on glass-fiber-reinforced polymer (GFRP) strands resulted in a transfer length of 254 to 279 mm [10 to 11 in.] — much less than for steel strands. Beam tests by Domenico[7] showed that the transfer length of CFCC strands is shorter than for steel, and a prediction equation was proposed. Mahmoud et al.[8] tested 52 concrete beams that were pretensioned with Leadline bars, CFCC strands, and steel strands. They recommended a prediction equation for transfer length and concluded that CFCC has a shorter transfer length than steel. This was attributed to CFCC's modulus of elasticity being less than steel's, which causes more friction between the strand and concrete during prestress release because of the larger lateral strains caused by longitudinal strains. Mahmoud and Rizkalla[9] tested 24 pretensioned concrete beams and proposed an equation for calculating the development length for CFRP strands.

Testing on column-type members has focused mostly on using CFRP to strengthen or repair existing concrete columns by wrapping them with sheets or by mounting strips along the member's length, such as the recent work by Hadi et al.[10] and by Gajdosova and Bilcik[11]. Most relevant to the study presented herein is the research by Abalo et al.[12], who performed tests on a 610-mm [24-in.] square prestressed concrete pile, to evaluate the use of CFRP mesh in place of conventional spiral ties. Cast for comparison, the control pile had 16 15.2-mm [0.6-in.] diameter low-relaxation strands in a square pattern with W3.4 spiral ties. The control and CFRP piles were both 12.2-m [40-ft] long and tested similarly. The ratio of actual-to-theoretical moment capacity was 1.27 for the CFRP pile and 1.21 for the control pile. A similar test setup was used in the study presented herein to assess the flexural behavior of CFCC-prestressed piles. Additional research will be discussed below, as it becomes pertinent to the subject matter and discussion.

PILE CONSTRUCTION

PLACEMENT OF STRANDS, SPIRALS, AND COUPLERS

This research investigated the feasibility of replacing steel prestressing with CFCC prestressing in concrete piles for bridge foundations. Five (5) pile specimens were constructed over a one-week period in a 134-m- [440-ft-] long self-stressing form (Fig. 1). The CFCC strand and CFCC spiral patterns (Figs. 2 and 3) were based on FDOT's standard details for a 610-mm [24-in.] square pile with 20 15.2-mm [0.6-in.] diameter strands. Throughout construction, several differences were noted with respect to using CFCC as opposed to steel strands and spirals. To avoid damaging the CFCCs by abrasion when pulling them into position, the conventional steel headers were replaced with 12-mm- [0.5-in.-] thick plywood headers (Fig. 4). The headers were placed at every pile-end location. Headers were also placed at each end of the bed, for casting 1.5-m- [5-ft-] long concrete blocks that would secure the CFCC strands after stressing; this was done for safety during tying of the spirals and in case the stressed strands slipped from the couplers. The concrete blocks were to be cast 0.30 m [1 ft] away from Pile 1 at the stressing end and from Pile 5 at the non-stressing end. A 0.30-m [1-ft] space was provided between piles to leave room for cutting the CFCCs during detensioning.

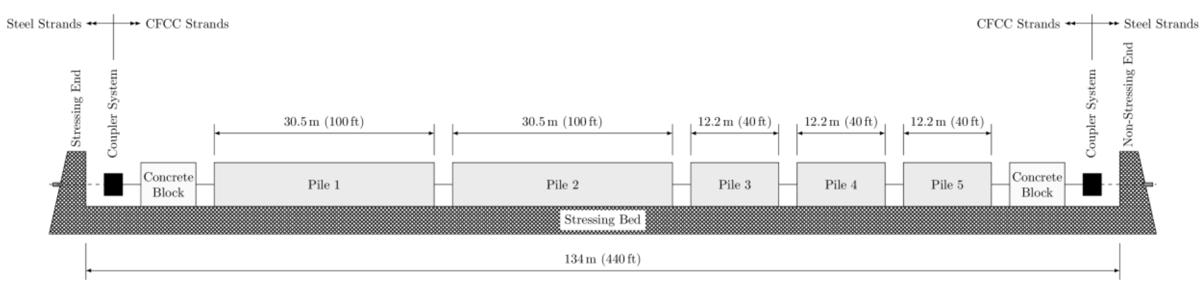

Fig. 1. Stressing bed and pile schematic

For each pile, a bundle of 5-mm [0.2-in.] diameter CFCC spiral was placed in the casting bed. Then the CFCC strands were pulled from spools and fed through the headers along the casting bed. One strand at a time was pulled by hand instead of machine pulling several at a time as is typically done. Each strand was cut to a length of 110 m [360 ft] before pulling another one from the spool. This length accounted for the total pile length, the concrete blocks, the headers, the strand elongation, and the additional length needed to avoid coupler interference during stressing.

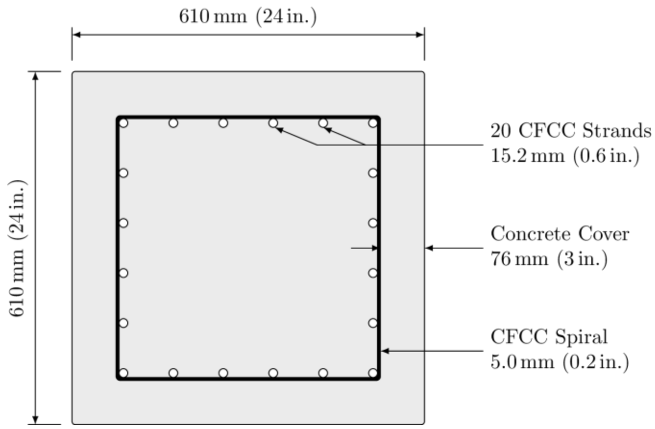

Fig. 2. Pile cross section

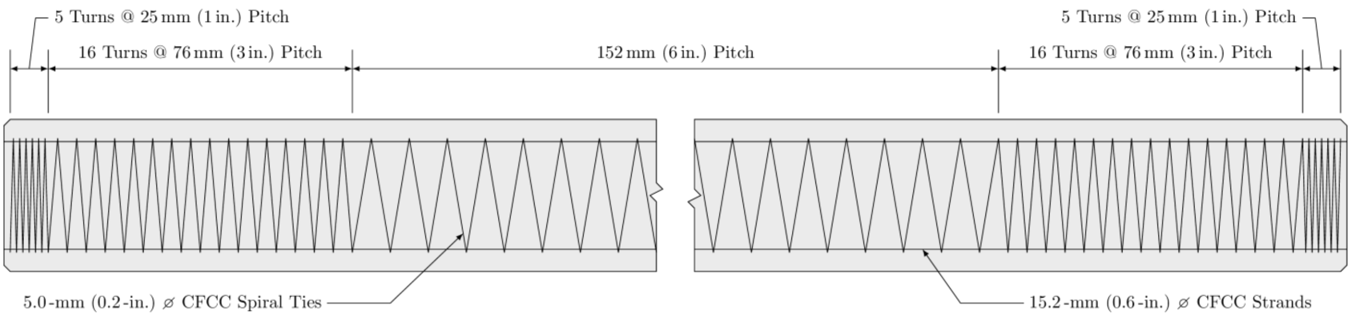

Fig. 3. Pile elevation

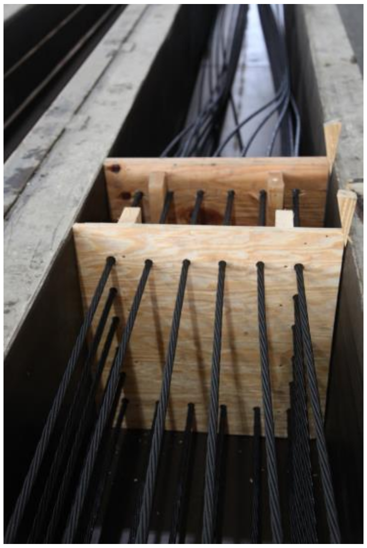

Fig. 4. Wooden header

Because CFCC is brittle and susceptible to abrasion, the conventional method of anchoring it for prestressing was not allowed. Instead, couplers, produced by the CFCC manufacturer, connected the CFCC to conventional, seven-wire, 1.86-GPa [270-ksi], low-relaxation, 15.2-mm [0.6-in.] diameter steel strands (Fig. 5), similar to a method used by Grace et al.[5]. The steel strands were anchored using conventional grips at the non-stressing and stressing ends of the precasting bed. The coupler consisted of two stainless steel pieces that screwed together: a sleeve for the CFCC and another for the steel strand. (Before the CFCC manufacturer produced this coupler, Mahmoud et al.[8] wrapped synthetic yarns around each strand to protect the CFCC from direct gripping.) A steel mesh sheet was wrapped around the CFCC strands to buffer the bite from the special wedges during seating. A braided grip provided a second layer of buffering while creating frictional forces against the wedges. The conventional steel strand was anchored to the coupler using a standard chuck.


Fig. 5. Staggered couplers after initial pretensioning

The CFCC manufacturer's personnel installed all 40 couplers using their recommended procedures. The couplers were staggered at 0.9-m [3-ft] increments to avoid their interacting when the strands elongated during tensioning. Figures 6 and 7 show the stagger pattern at the stressing end; Locations *A*, *B*, and *C* represent the couplers that extended 0.6 m [2 ft], 1.5 m [5 ft], and 2.4 m [8 ft], respectively, from the end of the concrete block.

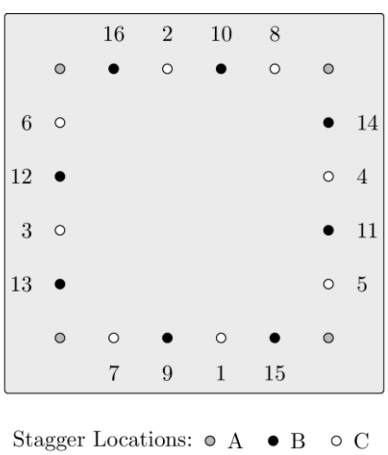

Fig. 6. Stressing end section view; CFCC strand stagger pattern and stressing sequence

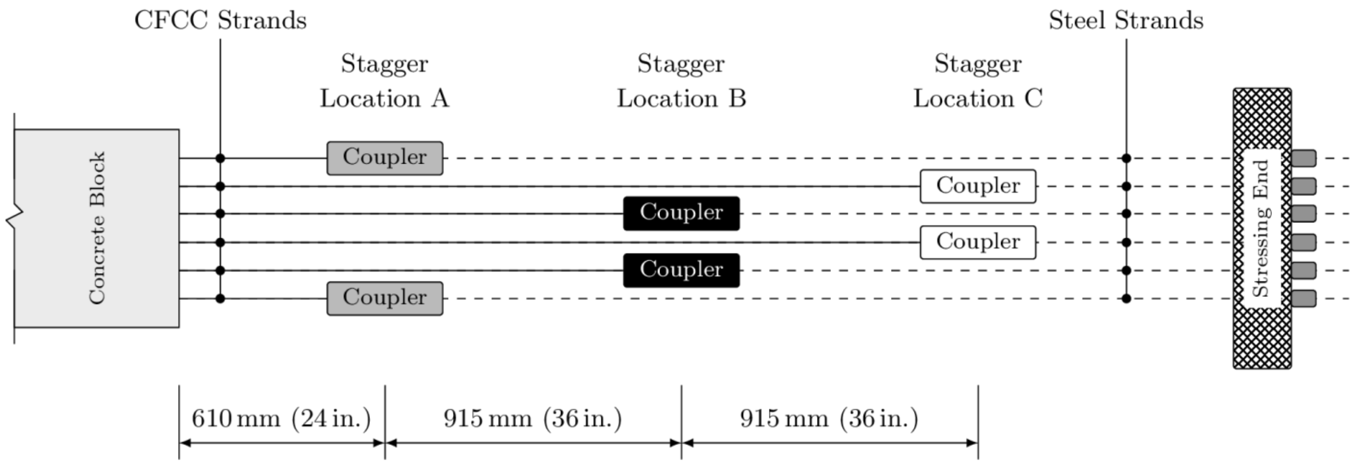

Fig. 7. Stressing end plan view; coupler stagger pattern

STRAND STRESSING

The CFCC strands were marked at the edge of the couplers to reveal slippage that might occur during stressing. The force was applied using a hydraulic monostrand jack, and the steel strands were locked using open grips at the stressing end. The stressing pattern differed from the precaster's typical pattern for steel strands, to ensure that the couplers would remain clear of each other as each strand elongated. All strands were initially stressed to 22 kN [5 kip] (Fig. 5). The corner strands were not stressed further, for reasons explained below. The remaining 16 strands were stressed in the sequence shown in Fig. 6, starting with the couplers closest to the stressing jack and extending 2.4 m [8 ft] from the end of the concrete block, proceeding to the couplers extending 1.5 m [5 ft], and finally to the couplers extending 0.6 m [2 ft].

According to ACI[2], CFCC should not be stressed to more than 65% of GUTS. For 15.2-mm [0.6-in.] diameter strands, GUTS is 270 kN [60.7 kip]. However, the casting bed would not have been strong enough for 20 CFCC strands stressed to 65% of GUTS. To keep the total compressive force less than the 3043-kN [684-kip] bed capacity, the four (4) corner strands were not stressed beyond the initial force of 22 kN [5 kip] (8.2% of GUTS), and the remaining 16 strands were stressed to 175 kN [39.45 kip] (65% of GUTS) – for a total compressive force of 2897 kN [651.2 kip]. This force ensured a minimum compression of 6.9 MPa [1 ksi] on the pile cross section to overcome tensile stresses during driving.

The CFCC manufacturer advised the precaster to gradually stress each strand to its 175-kN [39.45-kip] force over 3 minutes, to allow wedge seating in the coupler without strand slippage. The expected combined elongation of the CFCC strands and steel strands was 1200 mm [47.25 in.]; this includes an assumed abutment rotation of 6.4 mm [0.25 in.] and anchor sets of 3.2 mm [0.125 in.] and 9.5 mm [0.375 in.] for the non-stressing and stressing ends, respectively. Wedge seating losses in the coupler were assumed to be 3.2 mm [0.125 in.] for the steel strand and 55.0 mm [2.165 in.] for the CFCC strand. During the stressing process, after each strand tensioning was complete, elongation of strands was recorded by measuring from the pre-marked spots on the strands to the end of the jack. The measured elongations ranged from 1187 mm to 1270 mm [46.75 in. to 50 in.], which was close to the expected 1200 mm [47.25 in.]. PCI[13] prestressing loss equations predicted a total prestress loss of 8.8% for each of the 16 strands. The four (4) corner strands that were initially stressed to only 22 kN [5 kip] had much greater losses (61.6%) because the elastic shortening, creep, and shrinkage losses due to all the strands being stressed were disproportional to the small initial stress in those strands.

FINAL PREPARATIONS AND CASTING

After stressing was complete, fast-curing concrete blocks were cast around the CFCC strands between the pile ends and casting bed ends, as a measure of safety to secure the stressed strands. The CFCC spirals were placed in their final position and tied to the CFCC strands with plastic zip ties (Fig. 8). The spirals would provide confinement for the concrete during driving operations. Steel lifting loops were installed in accordance with FDOT standards.

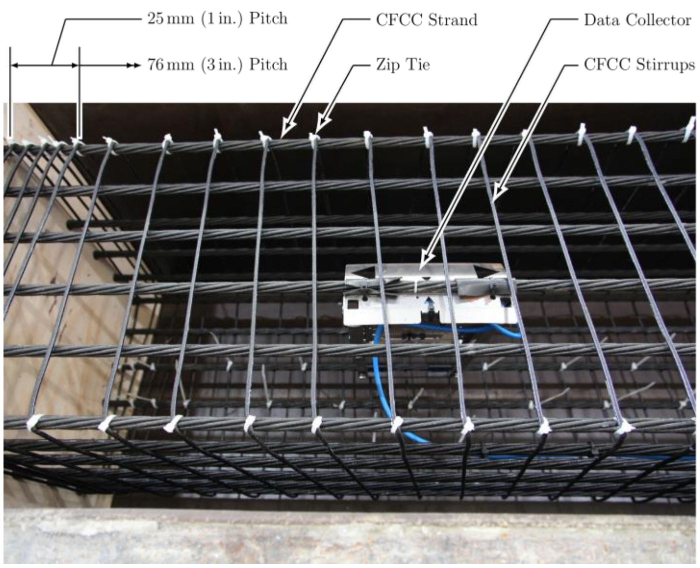

Fig. 8. Installation of stirrups

Because CFCC strands are susceptible to abrasion and damage from conventional mechanical vibrators, the manufacturer recommends the use of a rubber-tipped vibrator to consolidate the concrete. A mechanical vibrator with no rubber wrapping may be used with caution if the spacing between the CFCCs is larger than the diameter of the vibrator head. However, in this research, self-consolidating concrete (SCC) was used so that a mechanical vibrator would not be needed and the potential of impacting the CFCC would be avoided altogether. SCC is a highly-workable concrete that flows under its own weight through densely-reinforced or complex structural elements. (Andrawes et al.[14] researched the bond of SCC with steel strand and concluded that SCC does not affect the strand's transfer or development length and is comparable to conventional concrete). The mix design included #67 limerock, sand, flyash, high-range water reducer, and accelerator (for faster curing). The water-to-cement ratio was 0.34, the density was 2279 kg/m^3 [142.3 lb/ft^3], and the 28-day cylinder strength was measured at 59.6 MPa [8640 psi]. Four (4) truckloads of concrete were used to cast the five (5) piles. Once the casting and finishing were complete, the bed was covered with plastic to ensure a uniform curing temperature. Steam curing was not preferred because the temperature could have affected the couplers; according to the manufacturer, slippage of a strand in the coupler occurs at around 60^{o}C [140^{o}F].

Seven (7) 102-mm x 203-m [4-in. x 8-in.] cylinders were made for strength testing after 24 hours (to check the release strength) and at the times of the flexure tests and pile driving tests. The concrete strength at 24 hours after casting was 37.0 MPa [5370 psi] based on an average of two (2) cylinder tests.

LESSONS LEARNED

Several months before the five (5) concrete piles were cast, a first attempt was made to precast the piles. The casting setup and layout were similar to that described above, except that 12.7-mm [0.5-in.] diameter strands were used instead of 15.2-mm [0.6-in.] diameter strands. Hence, the coupler dimensions also differed. In this first attempt, one of the CFCC strands that had been fully stressed slipped out of the coupling device. Prestressing operations were stopped, and the manufacturer performed a detailed inspection of the coupling device. Thereafter, improvements were made to the device and installation procedures. (This resulted in the successful casting of the five (5) piles reported herein.) The improvements will help precasters to consistently install the couplers in accordance with manufacturer's recommendations.

TRANSFER LENGTH TEST PROGRAM

BACKGROUND

In a pretensioned member, stress is transferred from the prestressing strands to the surrounding concrete through bond. The length over which the stress is transferred is inversely proportional to the bond strength. For design, it is necessary to predict this transfer length, since it signifies the location at which the effective prestress has been fully transferred to the member's cross section. The additional length required to develop the strand strength from the effective prestressing stage to the ultimate stage is called the flexural bond length. The development length is the sum of the transfer length and the flexural bond length.

AASHTO[15] states that the transfer length for a steel strand should not exceed 60 times its diameter, while ACI[16] recommends using the equation

$$L_t = 1/3\, f_{se}\, d_b \qquad (1)$$

where

L_t = transfer length (in.)
f_{se} = effective stress in strand after losses (ksi)
d_b = strand diameter (in.)

INSTRUMENTATION

After the concrete cured for 24 hours, both ends of the three (3) 12-m [40-ft] piles were instrumented with 60-mm [2.36-in.] electrical resistance foil strain gages designated according to pile number (3 through 5) and orientation in the bed (N for North (stressing end) and S for South): 3N, 3S, 4N, 4S, 5N, and 5S. The strain gages were adhered to the top face (as positioned in the bed) of each pile (Fig. 9). The non-stressing (dead) end of the pile was instrumented with eight (8) strain gages along its centerline, and the stressing (live) end had 18 strain gages installed approximately along the top corner strands. The detensioning operations commenced the following day.

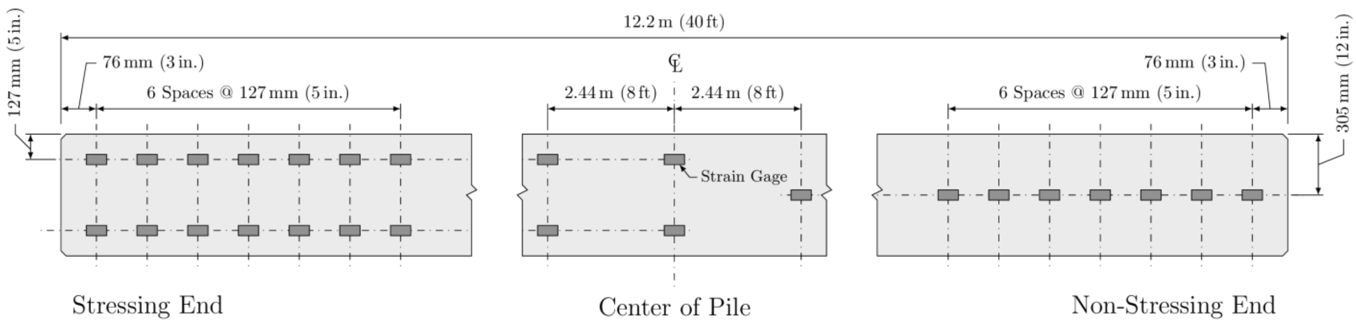

Fig. 9. Strain gage layout on top of pile for transfer length test (Not to scale)

PRESTRESS RELEASE

For a conventional pile, the precaster torch-cuts the strands in a routine pattern. However, in this study, the cutting sequence was governed by the coupler positions, to prevent the couplers from interacting when they pulled in towards the pile during strand cutting. Similar to a conventional sequence, the cuts were alternated in a symmetrical pattern about the axes of the cross section, to reduce unnecessary (although temporary) tension on the pile's outer surfaces. First, the corner strands that extended 0.6 m [2 ft] from the end of the concrete block at the stressing end were cut, followed by the strands that extended 1.5 m [5 ft] and the strands that extended 2.4 m [8 ft].

Conventionally, torches were used to cut the steel strands simultaneously at both the stressing and non-stressing ends. Then the CFCC strands between the pile headers were cut using a side grinder because CFCCs are bonded with epoxy and should not be torched. The header width between the pile ends was only about 0.3 m [1 ft], but this distance should be increased for future projects to provide a greater space for lowering the grinder to the bottom strands.

TEST RESULTS

The transfer length for CFCC strands was determined by continuously measuring concrete strains during strand detensioning in the casting bed. Figure 10 shows the strain profile along the length of pile '3', with each line representing the strains after a particular strand was cut.

The data shows the gradual transfer of prestress to the surrounding concrete throughout the strand cutting operations.

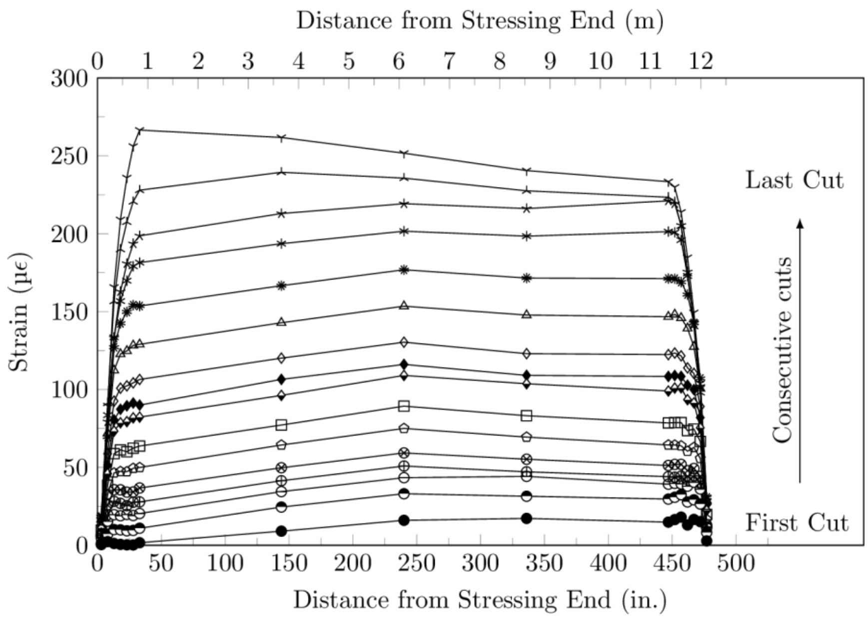

Fig. 10. Strain profile for pile 3 at release

Two methods can be used to measure the transfer length of a strand: (1) the 95% Average Maximum Strain (AMS) method (Russell and Burns[17]) which uses the measured strains along the transfer zone of a prestressed member and (2) the "draw-in" or "end-slip" method (Balazs[18]; Logan[19]). The AMS method, used in this study, is based on the assumption of an idealized theoretical linear increase in strain in the transfer zone, followed by a uniform strain plateau. The AMS procedure is to plot the strain vs. longitudinal distance, extend the line where the strain increases in the transfer zone, visually estimate the strain plateau region, draw a line corresponding to 95% of that strain, and find the intersection of the 95% line and the extended line in the transfer zone.

The strain profiles for all six (6) pile ends after 75% and 100% release were analyzed using the AMS method. The 75% release condition was employed to provide additional data points, to supplement the 100% release data; 75% release refers to 15 strands being released, and 100% release refers to all strands being released. As an example, the 100% release results for the non-stressing end of pile 5 (end 5S) are shown in Fig. 11. For the stressing ends, e.g., pile end 5N which had two rows of gages, the averages of the strain gage pairs were plotted. (For pile end 4N, the transfer length was evaluated visually because a distinct strain plateau was difficult to define.) Then, the transfer lengths from the AMS method for the 75% and 100% stress release were averaged for each pile end (Table 1). The lengths for the two (2) conditions (75% and 100%) did not vary from the average by much and were typically within an inch of the average.

The transfer length was predicted with the equation from ACI 440.4R-04[2]:

$$L_t = f_{pi}\, d_b \;/\; \alpha_t\, f'_{ci}{}^{0.67} \quad (2)$$

The predicted transfer length was 947 mm [37.3 in.], using the factor α_t from Grace[20] of 11.2 (for psi and in. units) or 2.12 (for MPa and mm units) and f_{pi} of 1517 MPa [220 ksi]. The observed transfer length was 635 mm [25 in.] – 33% lower than predicted by Equation 2. Mahmoud et al.[8] proposed a value for α_t of 25.3 (for psi and in. units) or 4.8 (for MPa and mm units), which resulted in a predicted transfer length of 419 mm [16.5 in.] – 34% lower than observed. Furthermore, the transfer length observed in this study was 31% less than the AASHTO[15] provision of 60 d_b (914 mm or 36 in.). Equation 1 from ACI[16] resulted in a predicted transfer length of 1021 mm [40.2 in.], using an effective prestress f_{se} of 1386 MPa [201 ksi] after all prestress losses as calculated per PCI[13]. The observed transfer length was 38% less than predicted.

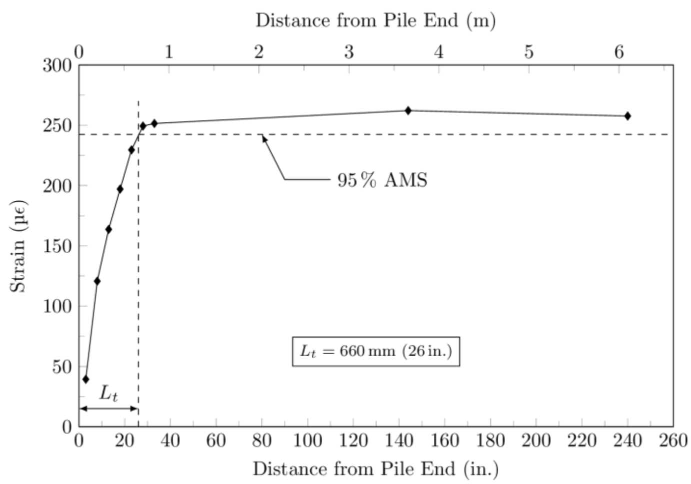

Fig. 11. Strain profile for pile end 5S at 100% stress release

Table 1. Transfer length for specimen pile ends

Pile End	Transfer Length mm (in.)
3N	737 (29.0)
3S	546 (21.5)
4N	648 (25.5)
4S	559 (22.0)
5N	711 (28.0)
5S	622 (24.5)
Average Transfer Length	635 (25.0)

From Table 1, the transfer lengths at the stressing ends, N, are higher than at the non-stressing ends, S. The average ratios of non-stressing to stressing end transfer lengths ranged from 0.74 for pile 3 to 0.86 for piles 4 and 5. According to Pozolo[21], transfer lengths might be influenced by factors such as concrete casting location, cutting location, and the use of multiple batches of concrete.

DEVELOPMENT AND FLEXURAL STRENGTH TESTS

Development length is the total embedment length of the strand that is required to reach a member's full design strength at a section. Development length for a steel strand may be calculated per ACI[16] and AASHTO[15]:

$$L_d = 1/3\, f_{se}\, d_b + (f_{ps} - f_{se})\, d_b \tag{3}$$

where
L_d = development length (in.)
f_{ps} = prestress in steel at the time for which the nominal resistance of the member is required (ksi)
The first term is the ACI[16] expression for the transfer length of a steel prestressing strand, and the other terms are its flexural bond length.

Development length and flexural strength tests were performed at the FDOT Structures Research Center between 45 and 50 days after casting, using similar experimental setups, instrumentation layout, and test procedures. The test setup was similar to that used by Gross and Burns[22]. For each test, the pile was simply supported; parameters were varied as given in Table 2. The piles were placed on elastomeric bearing pads, supported by two (2) steel I-beams leveled and grouted to the floor. Load was applied at a rate of 1112 N/s [250 lb/s] until flexural cracks formed, and then the rate was changed to 890 N/s [200 lb/s] until a bond or flexural failure occurred.

Table 2. Test matrix

Test No.	Test Type	Pile No.	Simple-Supp. Span m (ft)	Shear Span m (ft)	Cantilever Length m (ft)	Embedment Length m (ft)
1	Development Length	1	6.71 (22)	1.52 (5)	5.18 (17)	1.83 (6)
2	Development Length	1	8.23 (27)	2.74 (9)	1.68 (5.5)	3.05 (10)
3	Flexural	2	11.6 (38)	4.05 (13.3)	N.A.	4.36 (14.3)

Sixty-mm [2.36-in.] strain gages were used to measure concrete top fiber strains around the load point (for the development length tests) and in the constant-moment region at mid span (for the flexural strength test). Non-contact, laser displacement gages measured vertical deflections at several points along the span. Strand end slip, the longitudinal displacement of the strand relative to the pile end, was monitored throughout the tests by linear variable displacement transducers (LVDTs), anchored to four (4) CFCCs in the bottom of the pile.

DEVELOPMENT LENGTH TESTS

Development length tests (Tests 1 and 2) were performed on one (1) 12.2-m [40-ft] pile specimen. The concentrated load was applied to the pile by a hydraulic actuator. Because the predicted development length was less than 3.0 m [10 ft], the load was applied close to the support (see Fig. 12). After the first test was completed, approximately 2.0 m [6.5 ft] of the pile's tested/damaged end was separated from the specimen and discarded. The remaining 10.2-m [33.5-ft] length was used for the second test; the damaged end was cantilevered approximately 1.68 m [5.5 ft], and the undamaged, opposite end of the pile was loaded. The Test 1 instrumentation layout is shown in Fig. 13. Six (6) deflection gages were used to monitor vertical deflections. Four (4) electrical resistance foil strain gages monitored the top surface strains near the load point.

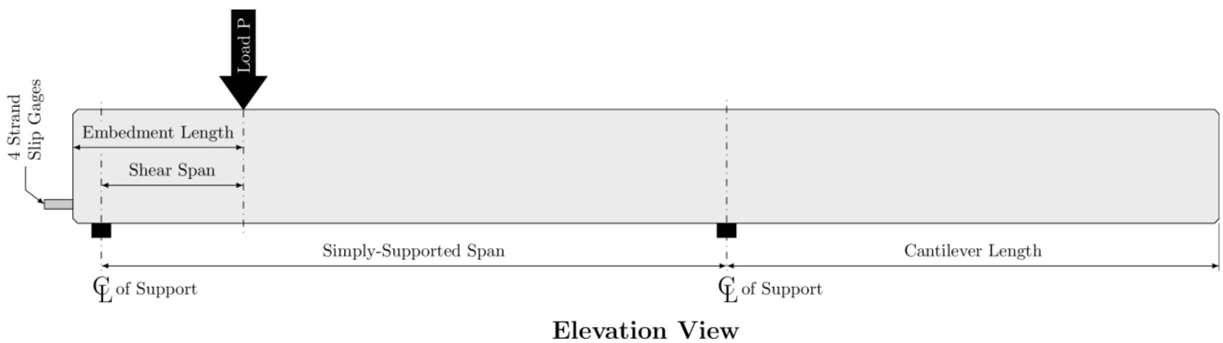

Fig. 12. Test setup for development length tests

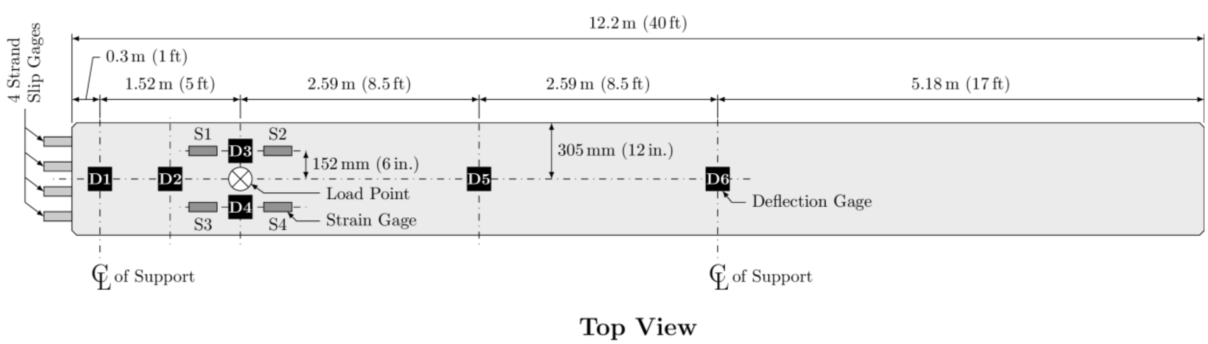

Fig. 13. Gage layout for development length Test 1

For Test 1, the embedment length was 1.8 m [6 ft], the simply-supported span length was 6.7 m [22 ft], and the cantilever length was 5.2 m [17 ft]. Figure 14 plots the applied load versus the deflection (average of gages D3 and D4), and Figure 15 is the applied load versus the top strain (average of gages S1 – S4). The first flexural crack was observed at a load of 778 kN [175 kip] and extended up to 0.6 m [2 ft] from the load point towards the end of the pile. Failure occurred at a load of 912 kN [205 kip] with a final crack pattern illustrated in Fig. 16. The flexural cracks propagated to 102 mm [4 in.] from the top fiber. The maximum top-fiber (compressive) strain in the vicinity of the load point at failure was 0.0012 (due only to bending and not including prestressing effects). The concrete began to visibly fail in compression before overall pile failure. There was no observable strand end slip of any of the CFCC strands throughout the test.

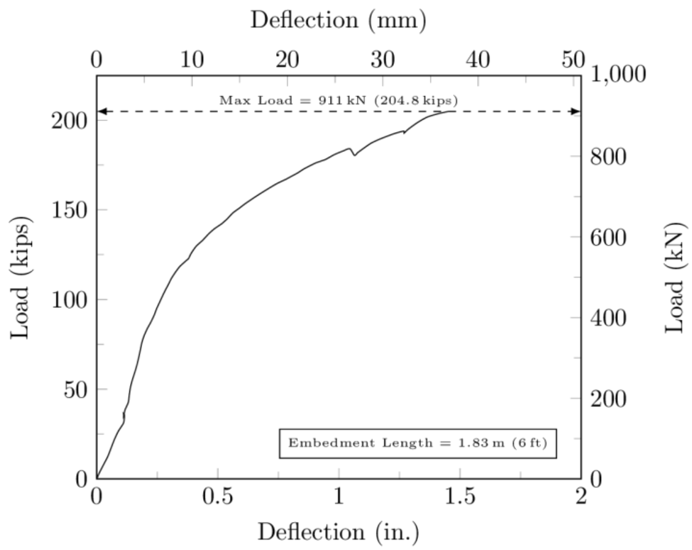

Fig. 14. Load vs. Deflection for Test 1

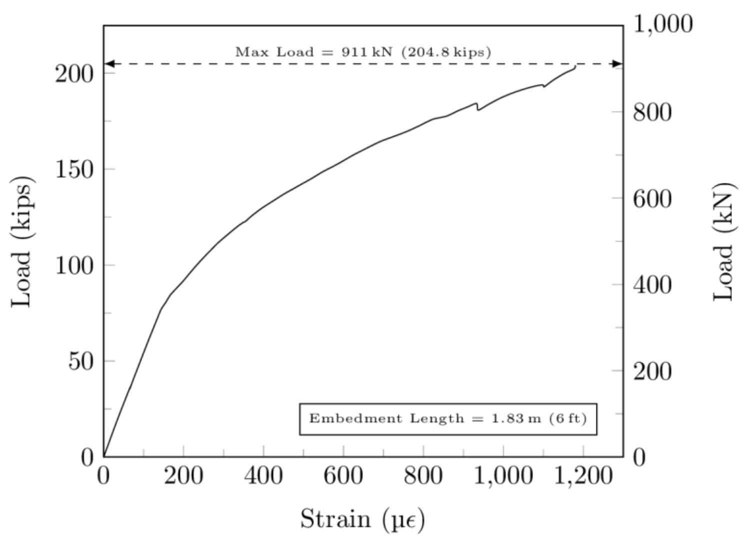

Fig. 15. Load vs. Strain for Test 1

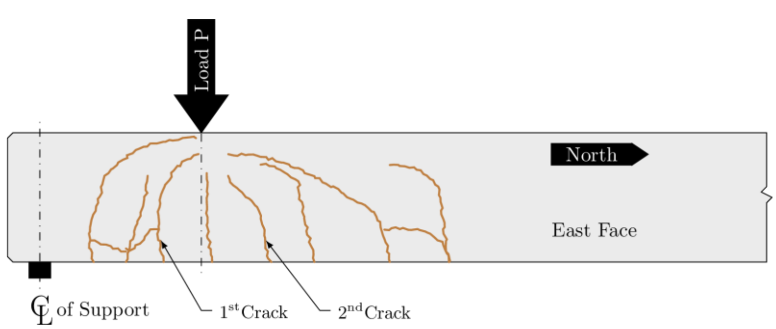

Fig. 16. Failure crack pattern on east face for Test 1

The structural integrity of the cantilevered end (opposite the tested end from Test 1) remained undisturbed throughout Test 1, so this pile end was used to perform Test 2. For this second test, the embedment length was 3.0 m [10 ft], the simply-supported span length was 8.2 m [27 ft], and the cantilever length was approximately 1.7 m [5.5 ft]. The load versus deflection plot was shaped similarly to Test 1. The first flexural crack occurred at a load of 449 kN [101 kip], on the bottom of the pile under the load application point. The cracks propagated up to 76 mm [3 in.] from the top fiber and extended up to 0.9 m [3 ft] from the load point towards the end of the pile. Flexural failure occurred at a load of 534 kN [120 kip] and a deflection of 71 mm [2.8 in.]. The maximum top-fiber (compressive) strain in the vicinity of the load point at failure was 0.00138 (due to bending and not including prestressing effects), where local concrete crushing occurred. The concrete began to visibly fail in compression before overall pile failure. The crack pattern was similar to Test 1.

Tests 1 and 2 both failed in flexure, as evidenced by the origination of vertical cracks that propagated upward from the bottom surface. The shortest embedment length used in these two (2) test setups was 1829 mm [72 in.]. In general, the development length is the shortest embedment length that develops the strand's flexural capacity without any bond slip. Therefore, these tests indicate that the strand was developed in less than 1829 mm [72 in.]. There was no observable strand slip of any of the four (4) instrumented CFCC strands throughout Tests 1 and 2.

Table 3 provides development length predictions per equations from ACI[16], AASHTO[15], Mahmoud and Rizkalla[9], and Lu et al.[23]. Lu et al.[23] predicts development length as follows:

$$L_d = 1/3\, f_{se}\, d_b + 3/4\, (f_{pu} - f_{se})\, d_b \qquad (4)$$

This results in a predicted development length of 2591 mm [102 in.], which is 42% higher than the shortest embedment length tested in this study. The predicted development length according to ACI[16] and AASHTO[15] is 3124 mm [123 in.] – 71% higher than the shortest embedment length tested. The low value of the measured development length might be due to the characteristic properties of CFCC or the result of using high-strength, self-consolidating concrete.

Table 3. Development length predictions

Prediction Equation	Predicted Length mm (in.)
Lu et al. (2000)	2591 (102)
ACI (2011) and AASHTO (2011)	3124 (123)
Mahmoud and Rizkalla (1996)	737 (29)
Mahmoud and Rizkalla (1996) with Grace (2000) α_t	1245 (49)

FLEXURAL STRENGTH TEST

A flexural strength test (Test 3) was performed on a second 12.2-m [40-ft] pile specimen, supported at its ends (Fig. 17). The instrumentation layout (plan view shown in Fig. 18) included 14 strain gages and ten (10) non-contact deflection gages. The single point load from the hydraulic actuator was transferred to the pile via a spreader beam, made from two (2) steel I-beams, to generate two (2) load points on the pile and a constant-moment region in approximately the middle third of the pile. The weight of the spreader beam and its bearing plates was approximately 13.3 kN [3000 lb].

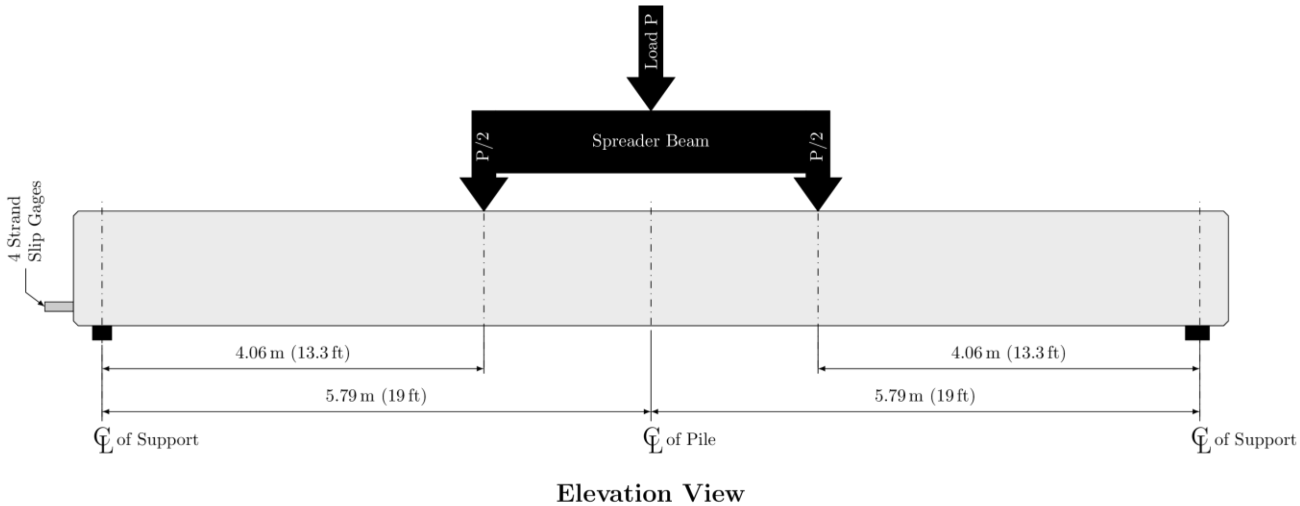

Fig. 17. Test setup for flexure test

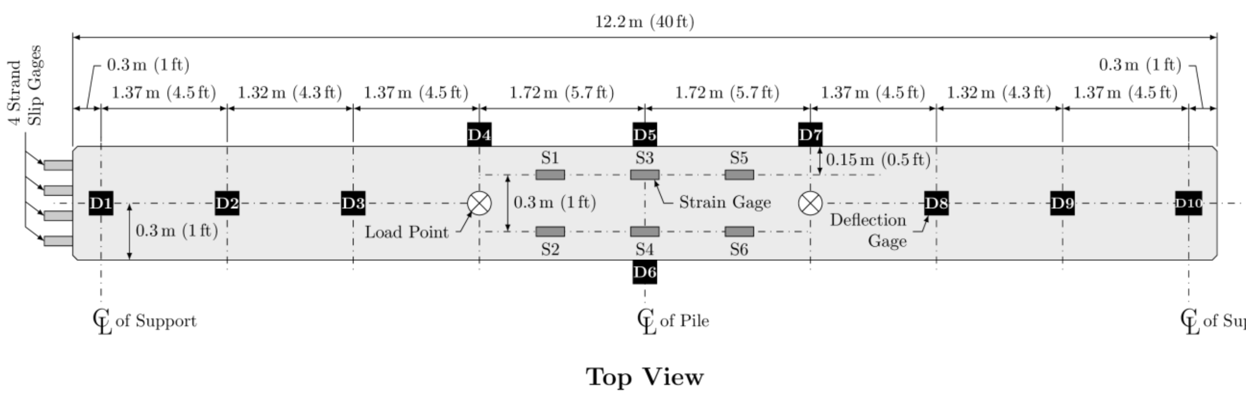

Fig. 18. Gage layout for flexure test, Plan view (Not to scale)

Three (3) 102-mm x 203-mm [4-in. x 8-in.] concrete cylinders, tested on the day of the flexural strength test (50 days after pile casting), had an average compressive strength of 65.5 MPa [9500 psi]. Figure 19 is a plot of the applied load versus mid-span deflection (average of gages D5 and D6). Failure occurred near the spreader beam support (Fig. 20) at a load of 503 kN [113 kip] and a mid-span deflection of 245 mm [9.63 in.], not including the effects of the pile or spreader beam weight. The maximum top-surface compressive strain was 0.0013 (due only to bending and not including prestressing effects), an average of gages S3 and S4 at mid span. The concrete began to visibly fail in compression before overall pile failure.

The cracks, sketched in Fig. 21, were uniformly distributed in the constant-moment region and extended up to 1.5 m [5 ft] from the load points toward the ends of the pile. At the maximum load, the flexural cracks propagated up to about 76 mm [3 in.] from the top fiber. There was no strand end slip in any of the strands throughout the test.

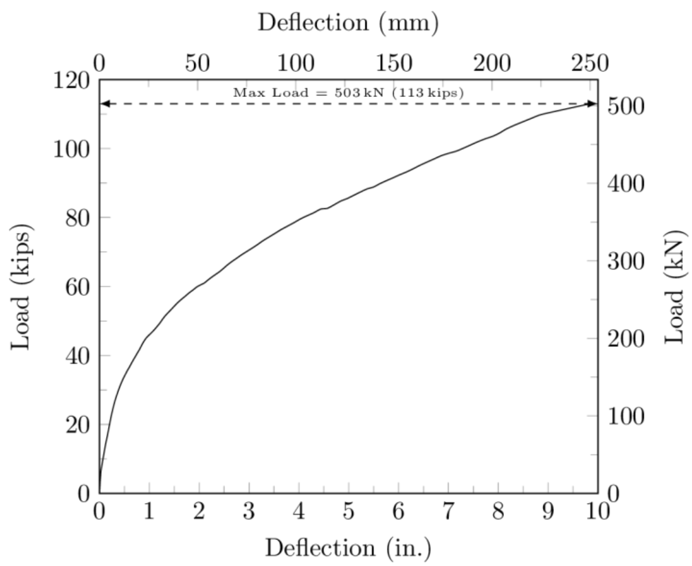

Fig. 19. Load vs. Deflection for flexure test

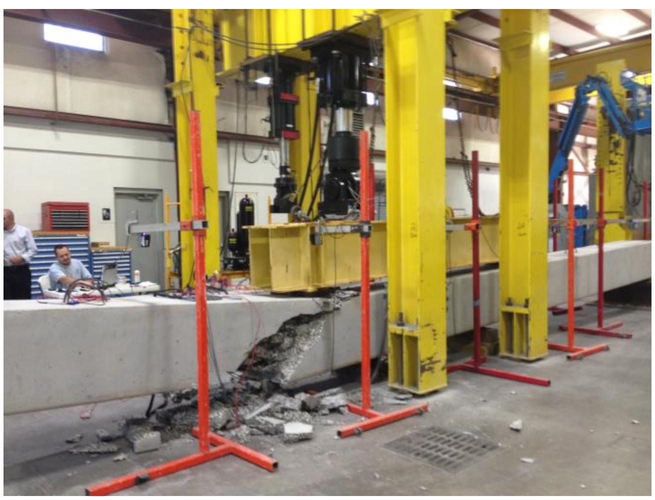

Fig. 20. Failure under one of the load points

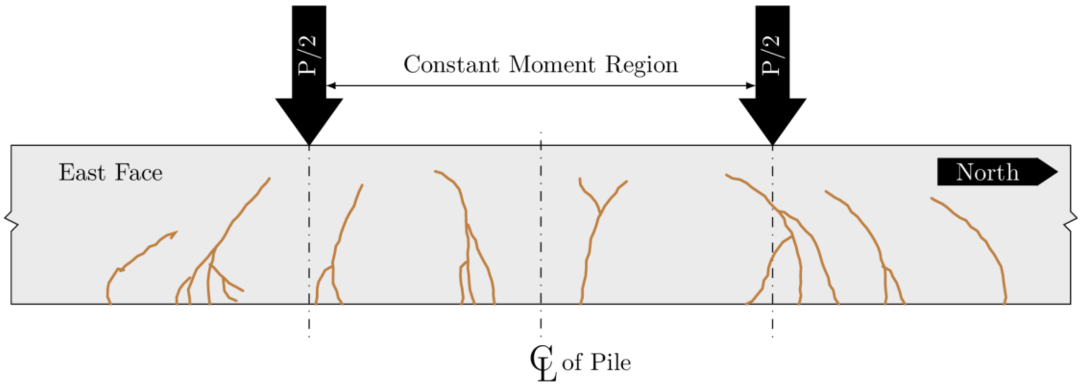

Fig. 21. Failure crack pattern on east face for flexure test

The applied failure load of 503 kN [113 kip] equates to a calculated moment of 1021 kN-m [753 kip-ft]. This generated a total test moment at mid span of 1186 kN-m [875 kip-ft], including an initial calculated moment of 165 kN-m [122 kip-ft] due to the pile and spreader beam weights. The calculated theoretical pile capacity was 1097 kN-m [809 kip-ft], so the flexural strength was 8% higher than theoretical. Furthermore, the 245-mm [9.63-in.] mid-span deflection at failure indicates some ductility.

PILE DRIVING TESTS

During pile installation, the pile driver hammer imposes high impact forces. The hammer blow causes a compression wave that propagates along the pile and reflects once it reaches the pile tip. Depending on the soil resistance, the reflecting wave can cause compressive or tensile stresses in the pile. This wave can cause damage to the concrete, high stresses in the prestressing strands, and possible rupturing of the bond between the strands and concrete. To counteract tensile stresses, the piles were designed to have a permanent compression of 6.89 MPa [1000 psi] at the effective prestress level, after losses.

The final purpose of this research was to evaluate the behavior of the pile during driving for a bridge foundation. The two (2) 30.5-m [100-ft] piles were installed, by a contractor, on the Deer Crossing Bridge construction site on Interstate 4 and west of U.S. Highway 92. The piles were driven adjacent to production piles. Measured four days later, the concrete compressive strength was 10,080 psi, averaged from two (2) 102-mm x 203-mm [4-in. x 8-in.] cylinders.

The goal was to “test the limits” of the piles. The first pile was conventionally driven, as determined by FDOT personnel on site, and then subjected to hard driving during the latter part of installation. The second pile was also installed under hard driving conditions to further test the limits and to test for repeatable behavior. Both piles were driven to refusal: Piles 1 and 2, respectively, were subjected to 2765 and 3139 hammer blows. Pile Driving Analyzer® (PDA) and Embedded Data Collectors (EDC) were both used to monitor the stresses in the piles throughout driving. FDOT also provided geotechnical expertise and assessed the performance of the pile based on observations and PDA and EDC results. After testing, the piles were cut off at 0.6 m [2 ft] below grade, covered by soil, and abandoned in place.

PILE DRIVING ANALYZER

The Pile Driving Analyzer® (PDA) system was used to monitor the two (2) piles during driving operations. The PDA uses accelerometers and strain transducers to continuously measure pile-top forces and velocities during driving, and it allows the engineer to make adjustments during the drive (e.g., reduction of hammer stroke or increase in pile cushion) to prevent damage to the pile. Measurements recorded during driving are also used to calculate the pile driving resistance, as well as the pile's static bearing capacity. FDOT provided and installed the PDA system and interpreted the results.

EMBEDDED DATA COLLECTORS (EDC)

EDCs are strain transducers and accelerometers that are embedded in a concrete member. A summary of an evaluation program carried out by FDOT to compare EDC results to PDA and the associated signal matching software (CAPWAP) is included in Herrera et al.[24]. The EDC system estimates soil damping for every blow during driving and could potentially be used to monitor the pile specimen over a long period of time (several months or years) if accessibility to the instruments is provided. EDCs, pre-installed in the piles before casting, were placed at mid-height, at a distance of two (2) pile widths from the head of the pile, and at one (1) pile width from the bottom of the pile. After the concrete was cast, the strains were recorded through a wireless receiver; this continued throughout the strand cutting operations and then during driving.

ANALYSIS OF RESULTS

Both piles performed well during installation, even though they were subjected to hard driving conditions and high levels of stress. No major damage was observed, other than local concrete spalling at the pile heads, which was likely due to intentional hard driving and the use of thin driving cushions. The piles' resistances were approximately twice the 4000-kN [900-kip] suggested driving resistance per FDOT's Structures Design Guidelines[25] for a conventional 610-mm [24-in.] prestressed pile. The data also suggests that there was no significant loss of prestress.

The aim was to observe the piles' behavior when subjected, at minimum, to the hardest conditions that FDOT would allow for standard piles. Although driving and subsurface conditions prevented the development of maximum compression stresses, based on *measured* concrete compressive strength, of 43.1 MPa [6.25 ksi], per FDOT Specification 455-5.11.2[26], the stresses in the piles *did* exceed the typical limit used in production pile driving (which is 24.8 MPa [3.6 ksi], assuming a *nominal* 41.4 MPa [6000 psi] concrete strength and 6.89 MPa [1000 psi] for initial prestress). In addition, the theoretical limit on *tension stress*, 9.51 MPa [1.38 ksi] based on *measured* concrete compressive strength, was exceeded during driving. These are indications that the desired hard driving was indeed produced.

SUMMARY

The following research activities were performed:

- Five (5) 610-mm [24-in.] square prestressed concrete piles were cast using 20 15.2-mm [0.6-in.] diameter CFCC prestressing strands. Produced at Gate Precast Company in Jacksonville, Florida, two (2) piles were 30.5-m- [100-ft-] long, and three (3) were 12.2-m- [40-ft-] long.

- Transfer length tests were performed at Gate Precast Company in Jacksonville, Florida, on the three (3) 12.2-m [40-ft] piles, by measuring concrete strains during strand detensioning.

- Two (2) development length tests were performed on one (1) of the 12.2-m [40-ft] piles at the FDOT Structures Research Center in Tallahassee, Florida.

- One (1) of the 12.2-m [40-ft] piles was tested for flexural strength at the FDOT Structures Research Center.

- The two (2) 30.5-m [100-ft] piles were driven at a bridge construction site in Volusia County, Florida, to monitor the static resistance of the piles and the pile behavior during driving.

CONCLUSIONS

SPECIMEN PRODUCTION

There are unique challenges associated with using CFCC strands in a prestressed concrete pile. The precaster has to adapt to a new technique of stressing the strand with respect to:

- Use of a different header material (e.g., wood instead of steel) to prevent damage to CFCC strands while installing them in the precasting bed

- Proper handling of the CFCC to prevent damage

- Coupler installation

- The stressing method of CFCC strands, with regard to a slower-than-normal recommended stressing rate

- Concrete consolidation during placement, preferably without a vibrator or with a rubber-tipped vibrator to prevent damage to strand

TRANSFER LENGTH OF CFCC

An analysis of the strain gage data from the transfer length tests suggests that the CFCC strands have a 635-mm [25-in.] transfer length, which is 38% and 31% less than that predicted by ACI[16] and AASHTO[15], respectively, for steel strands. The observed transfer length is 33% lower than the transfer length calculated from ACI 440.4R-04[2] and using the alpha factor by Grace[20]. Testing of more pile specimens could be performed to determine an alpha factor for CFCC strand transfer length predictions. Nonetheless, the observed transfer length is less than the predicted values. This is conservative for bond and shear considerations. On the other hand, the short transfer length could be detrimental and would need to be accounted for in situations where top tensile stresses in the concrete are high at release, such as for a beam member. This is not a concern for a pile member, where theoretically the entire cross section is compressed when all the strands are stressed.

DEVELOPMENT LENGTH OF CFCC

The strands in the Test 1 pile had an embedment length of 1829 mm [72 in.]. Because the pile failed in flexure, rather than by failure of the strand-to-concrete bond, the development length could not be determined in this study. However, it can be concluded that the development length of the tested CFCC strands is less than 1829 mm [72 in.] and therefore less than the AASHTO[15] prediction of 3124 mm [123 in.] for steel strands (and using CFCC's value for GUTS).

FLEXURAL STRENGTH OF CFCC-PRESTRESSED PILE

The flexural strength of the CFCC-prestressed concrete pile was 8% higher than theoretical. The test results suggest that the flexural performance of CFCC-stranded piles is comparable to that of steel-stranded piles. The cracking patterns in the two (2) development length tests and the flexural test resembled flexural failure. In all tests, there was no end slip in any of the strands, which indicates a good bond characteristic of CFCC with concrete. In addition, the pile's mid span had deflected over 229 mm [9 in.] at failure, which indicates some ductility. This is consistent with the approximate 254-mm [10-in.] deflection of concrete piles with similar dimensions but prestressed with steel and tested by Abalo et al.[12]

PILE DRIVING

Two (2) 30.5-m [100-ft] piles were subjected to hard driving conditions and high internal compressive and tensile stresses. Both performed well, with no major damage or loss of prestress.

ACKNOWLEDGMENTS

The Florida Department of Transportation (FDOT) provided the funding for this project. William Potter provided much insight, and Chris Weigly contributed time and instrumentation expertise. Gate Precast Company in Jacksonville, Florida, deserves much recognition for producing the piles. Tom Newton and Scott Henning were exceptionally professional and accommodating, while the unique details required for constructing the precast concrete piles were worked out. Wendell Crews and Zulfin Masinovic's enthusiasm and patience made the work enjoyable. Thanks also go to Mohamad Hussein at GRL Engineers, Inc., for providing the PDA system and on-site analysis and expertise, and Don Robertson and Harold Dohn at Applied Foundation Testing, Inc., for providing EDC pile driving testing services.

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