Construction, strength, and driving performance of carbon-fiber-reinforced polymer prestressed concrete piles

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- A study was conducted on carbon-fiber-composite cable (CFCC) in prestressed concrete piles for bridge foundations.
- Five square piles that were prestressed with CFCC behaved better than predicted in terms of transfer length, flexural strength, and ductility.
- Two piles subjected to hard driving conditions at a bridge construction site behaved well with no major damage or loss of prestress.

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 wetting and drying 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 greater than that of steel strands; however, the cost of prestressing strand materials is a relatively small percentage of a bridge's overall cost. The greater initial cost of CFCC would likely be paid back with the long-term benefit of reduced maintenance during the bridge's service life. This paper presents efforts to construct CFCC prestressed concrete piles, as well as test results to determine the strand transfer and development lengths, pile flexural strength, and pile drivability. Five 24 × 24 in. (610 × 610 mm) CFCC-prestressed concrete piles were constructed. Three of the piles were 40.0 ft (12.2 m) long, were monitored for transfer length, and were tested for development length and flexural strength. The two 100 ft (30 m) long piles were driven and monitored at the Deer Crossing Bridge in Volusia County, Fla.

CFCC material

Fiber-reinforced polymers (FRPs) are high-strength, composite materials that are impregnated with a resin material. FRP can be made with carbon (CFRP), glass (GFRP), or aramid (AFRP) fibers. One such CFRP is CFCC. CFCC is produced using carbon fibers of polyacrylonitrile and epoxy resin configured into individual "wires" that are twisted and wrapped with synthetic yarns to protect the fibers from ultraviolet radiation and mechanical abrasion. From the technical data on CFCC, pull-out tests show that it has a bond strength to concrete of 967 psi (6.67 MPa), more than twice that of steel. The thermal coefficient of expansion of CFCC is 0.34×10^{-6}/°F (0.62×10^{-6}/°C), about 1⁄20 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 that 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 has diameters ranging from 0.2 to 1.6 in. (5 to 41 mm). Cables are made from 1 wire, 7 wires, 19 wires, or 37 wires. In the piles that were cast and tested for this research, a seven-wire-strand diameter of 0.6 in. (15 mm) was used for longitudinal prestressing, and a single wire with a diameter of 0.2 in. (5 mm) was used for transverse spiral reinforcement. These diameters are nominal and are provided by the manufacturer. The manufacturer's reported effective cross-sectional areas of the CFCC strand and wire are 0.179 in.^{2} (115 mm^2) and 0.0236 in.^{2} (15.2 mm^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 0.222 in.^{2} (143 mm^2). The manufacturer's guaranteed ultimate tensile strength is 60.7 kip (270 kN) for the 0.6 in. (15 mm) strands and 8.5 kip (38 kN) for the 0.2 in. (5 mm) wire. The strands' nominal modulus of elasticity is 22,500 ksi (155 GPa), and the ultimate tensile strain is 1.6%. The wires' nominal modulus of elasticity is 24,200 ksi (167 GPa).

Literature review

In the past couple of decades, several research projects have been performed on CFRP for structural applications. ACI 440-04[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 as a wrap to provide confinement for retrofitting applications.

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

Other research has focused on CFRP's transfer and development lengths, which are important parameters for the design of structural members. Tests by Issa et al.[6] on GFRP strands resulted in a transfer length of 10 to 11 in. (254 to 280 mm), much less than that for steel strands. Beam tests by Domenico[7] showed that the transfer length of CFCC strands is shorter than that 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 strand. 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 in the recent work by Hadi et al.[10] and by Gajdosova and Bilcik.[11] Most relevant to this study is the research by Abalo et al.,[12] who performed tests on a 24 in. (610 mm) square prestressed concrete pile to evaluate the use of CFRP mesh in place of conventional steel spiral ties. Cast for comparison, the control pile had sixteen 0.6 in. (15 mm) diameter, low-relaxation steel strands in a square pattern with W3.4 spiral ties. The control and CFRP piles were both 40.0 ft (12.2 m) 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 this study to assess the flexural behavior of CFCC-prestressed piles. Additional research is discussed in the next sections.

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 pile specimens were constructed

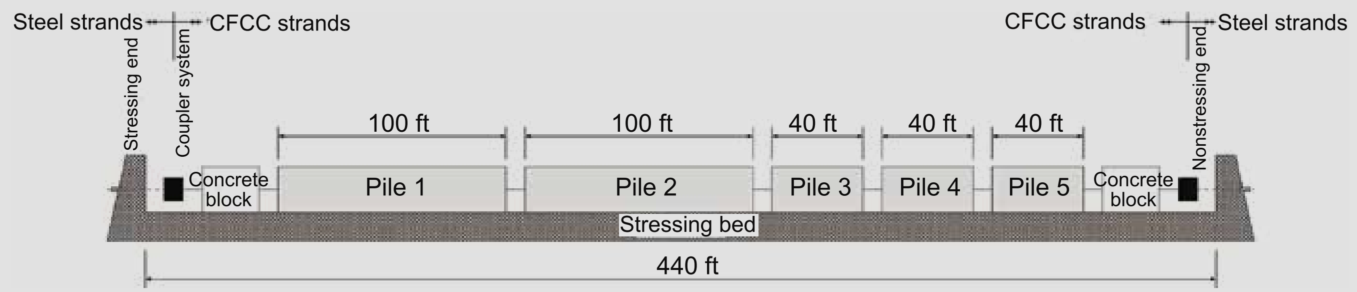

Figure 1. Five piles were cast in a 440 ft long self-stressing form (bed). Note: CFCC = carbon-fiber-composite cable. 1 ft = 0.305 m.

over a one-week period in a 440 ft long (134 m), self-stressing form (**Fig. 1**). The CFCC strand and CFCC spiral patterns (**Fig. 2** and **3**) were based on the Florida Department of Transportation's (FDOT's) standard details for a 24 in. (610 mm) square pile with twenty 0.6 in. (15 mm) 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 0.5 in. (13 mm) thick plywood headers. The headers were placed at every pile-end location. Headers were also placed at each end of the bed for casting 5 ft (1.5 m) 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 1 ft (0.3 m) away from pile 1 at the stressing end and from pile 5 at the nonstressing end. A 1 ft space was provided between piles to leave room for cutting the CFCCs during detensioning.

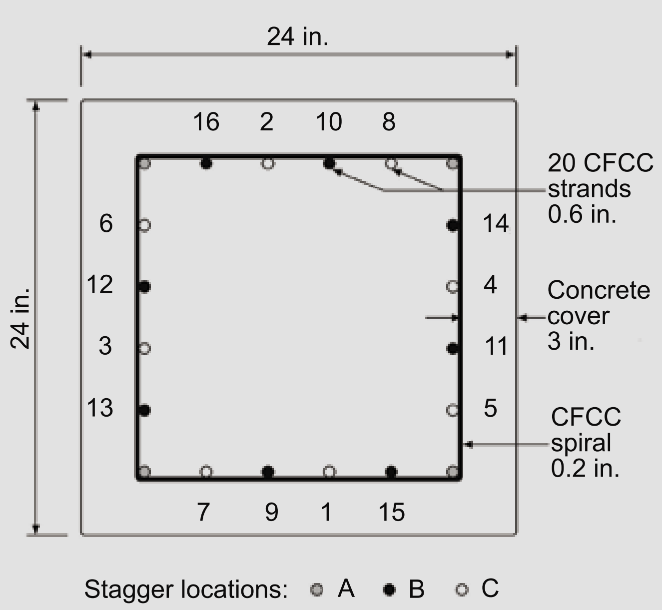

Figure 2. The 24 in. square piles had CFCC spiral transverse reinforcement and twenty 0.6 in. diameter CFCC strands and were stressed in a symmetric pattern. The couplers were staggered to prevent interference during stressing and release. Note: CFCC = carbon-fiber-composite cable. 1 in. = 25.4 mm.

For each pile, a bundle of 0.2 in. (5 mm) 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. Strands were pulled by hand one at a time instead of pulling several at a time by machine, as is typically done. Each strand was cut to a length of 360 ft (110 m) before another one was pulled 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.

Because CFCC is brittle and susceptible to abrasion, the conventional method of anchoring it for prestressing was not viable. Instead, couplers connected the CFCC to conventional, seven-wire, 270 ksi (1860 MPa), low-relaxation, 0.6 in. (15 mm) diameter steel strands (**Fig. 4**), similarly to a method used by Grace et al.[5] The steel strands were anchored using conventional grips at the nonstressing 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. (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.

All 40 couplers were installed by the manufacturer using their recommended procedures. The couplers were staggered at 3 ft (0.9 m) increments to avoid their interacting when the strands elongated during tensioning. Figure 2 shows the stagger pattern at the stressing end. Locations A, B, and C represent the couplers that extended 2 ft (0.6 m), 5 ft (1.5 m), and 8 ft (2.4 m), respectively, from the end of the concrete block.

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

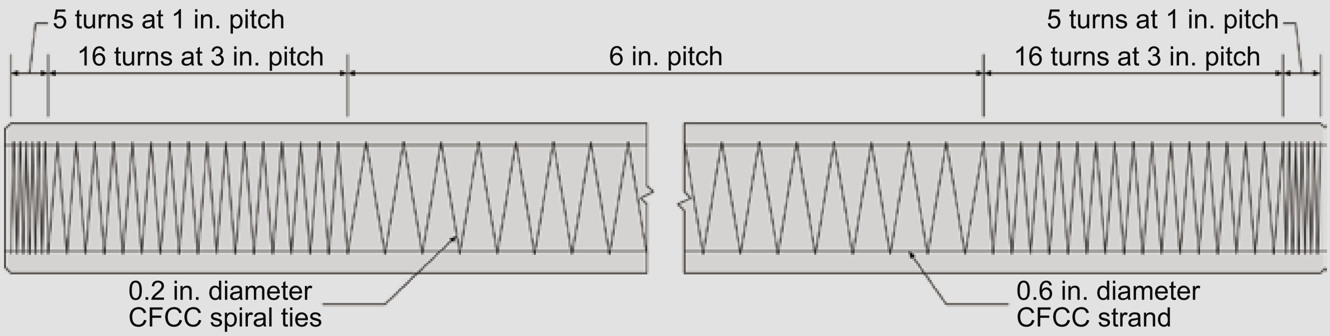

Figure 3. The piles were transversely reinforced with CFCC strands. Note: CFCC = carbon-fiber-composite cable. 1 in. = 25.4 mm.

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 5.0 kip (22 kN) (Fig. 4). The corner strands were not stressed further, for reasons explained later. The remaining 16 strands were stressed in the sequence shown in Fig. 2, starting with the couplers closest to the stressing jack and extending 8 ft (2.4 m) from the end of the concrete block, proceeding to the couplers extending 5 ft (1.5 m), and finally to the couplers extending 2 ft (0.6 m).

Figure 4. The couplers were staggered to prevent interference during stressing. The steel strands are in the foreground and the carbon-fiber-composite cable (CFCC) strands are in the background. Couplers connect the steel strands to the CFCC strands.

According to ACI 440-04, CFCC should not be stressed to more than 65% of guaranteed ultimate tensile strength. For 0.6 in. (15 mm) diameter strands, guaranteed ultimate tensile strength is 60.7 kip (270 kN). However, the casting bed's capacity was not adequate for 20 CFCC strands stressed to 65% of guaranteed ultimate tensile strength. To keep the total compressive force less than the 684 kip (3040 kN) bed capacity, the four corner strands were not stressed beyond the initial force of 5.0 kip (22 kN) (8.2% of guaranteed ultimate tensile strength), and the remaining 16 strands were stressed to 39.45 kip (175.5 kN) (65% of guaranteed ultimate tensile strength), for a total compressive force of 651.2 kip (2897 kN). This force ensured a minimum compression of 1.0 ksi (6.9 MPa) on the pile cross section to overcome tensile stresses during driving.

The CFCC manufacturer advised the precaster to gradually stress each strand to its 39.45 kip (175.5 kN) force over three minutes to allow wedge seating in the coupler without strand slippage. The expected combined elongation of the CFCC strands and steel strands was 47.25 in. (1200 mm). This includes an assumed abutment rotation of 0.25 in. (6.4 mm) and anchor sets of 0.13 and 0.38 in. (3.3 and 9.7 mm) for the nonstressing and stressing ends, respectively. Wedge seating losses in the coupler were assumed to be 0.13 in. (3.3 mm) for the steel strand and 2.17 in. (55.1 mm) for the CFCC strand. During the stressing process, after each strand tensioning was complete, elongation of strands was recorded by measuring from the marked spots on the strands to the end of the jack. The measured elongations ranged from 46.75 to 50.0 in. (1187 to 1270 mm), which was close to the expected 47.25 in. The *PCI Design Handbook: Precast and Prestressed Concrete*[13] prestressing loss equations predicted a total prestress loss of 8.8% for each of the 16 strands. The four corner strands that were initially stressed to only 5.0 kip (22 kN) had much greater losses (61.6%) because the elastic shortening, creep, and shrinkage losses due to all of the strands being stressed were disproportional to the small initial stress in those strands.

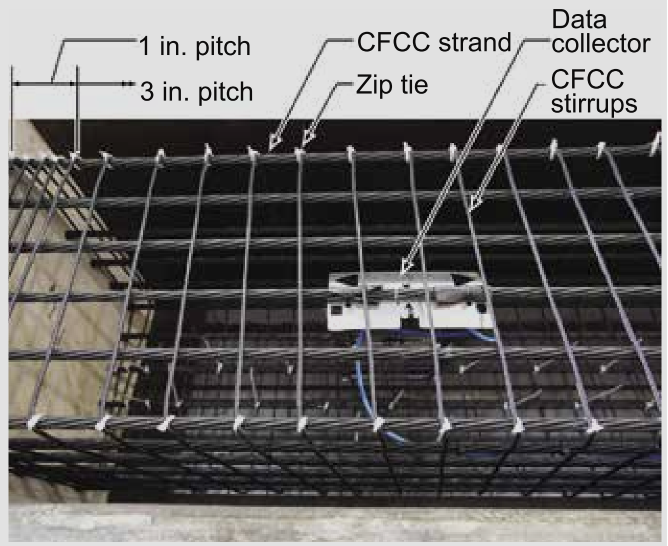

Figure 5. The stirrups were connected to the strands with plastic zip ties. Note: CFCC = carbon-fiber-composite cable. 1 in. = 25.4 mm.

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. 5**). The spirals would provide confinement for the concrete during driving operations. Steel lifting loops were installed in accordance with FDOT standards.

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 mixture design included no. 67 limestone, sand, fly ash, high-range water-reducing admixture, and accelerator (for faster curing). The water-cementitious materials ratio was 0.34, the density was 142.3 lb/ft^3 (2279 kg/m^3), and the 28-day cylinder strength was measured at 8640 psi (59.6 MPa). Four truckloads of concrete were used to cast the five 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 around 140°F [60°C].)

Seven 4 × 8 in. (100 × 200 mm) cylinders were made for concrete compressive strength testing after 24 hours (to check the release strength) and at the times of the flexure tests and pile-driving tests. The concrete compressive strength at 24 hours after casting was 5370 psi (37.0 MPa), based on an average of two cylinder tests.

Lessons learned

Several months before the five concrete piles were cast, a first attempt was made to precast the piles. The casting setup and layout were similar to that described previously, except that 0.5 in. (13 mm) diameter strands were used instead of 0.6 in. (15 mm) 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, resulting in the successful casting of the five piles discussed in this paper. The improvements will help precasters consistently install the couplers in accordance with manufacturer 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 because it signifies the location at which the effective prestress has been fully transferred to the concrete 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.

The American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications*[15] says that the transfer length for a steel strand should not exceed 60 times its diameter, while ACI 318-11[16] recommends using Eq. (1).

$$L_t = \tfrac{1}{3} f_{se} d_b \quad (1)$$

where

L_t = transfer length

f_{se} = effective stress in strand after losses

d_b = strand diameter

Instrumentation

After the concrete cured for 24 hours, both ends of the three 40 ft (12 m) long piles were instrumented with 2.4 in. (60 mm) electrical-resistance-foil strain gauges 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 gauges were adhered to the top face (as positioned in the bed) of each pile. On the stressing (live) end, seven pairs of gauges were placed symmetrically about the pile's longitudinal axis and 5 in. (130 mm) from the pile corners. On the nonstressing (dead) end, seven gauges were placed along the centerline of the pile's longitudinal axis. These 21 gauges were longitudinally spaced at 5 in. beginning 3 in. (76 mm) from the end of the pile. An additional pair of gauges was placed at midspan, a pair was placed at 8 ft (2.4 m) from midspan, and a single gauge was placed at 8 ft on the other side of midspan. The detensioning operations commenced the following day.

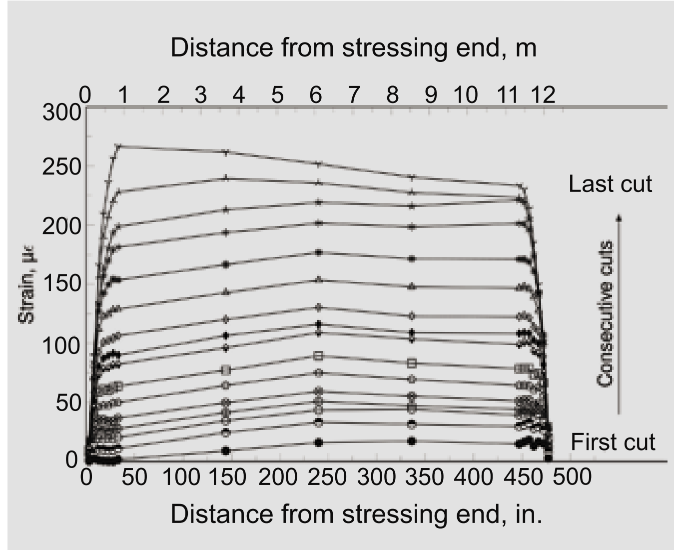

Figure 6. The measured strains at several locations along the pile length increased as each strand was detensioned. Pile 3 data during release are shown.

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 toward 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 (though temporary) tension on the pile's outer surfaces. First, the corner strands that extended 2 ft (0.6 m) from the end of the concrete block at the stressing end were cut, followed by the strands that extended 5.0 ft (1.5 m), and finally the strands that extended 8.0 ft (2.4 m).

Conventionally, torches were used to cut the steel strands simultaneously at both the stressing and nonstressing ends. The CFCC strands between the pile headers were then 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 1 ft (0.3 m), 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 6** shows the strain profile along the length of pile 3, with each line representing the strains after a particular strand was cut. The data show the gradual transfer of prestress to the surrounding concrete throughout the strand cutting operations.

Two methods can be used to measure the transfer length of a strand:

- 95% average maximum strain method,[17] which uses the measured strains along the transfer zone of a prestressed member
- draw-in or end-slip method,[18,19] which involves observing the strand slip at the free end of the transfer length

The average maximum strain method, which was 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 average maximum strain procedure is to plot the strain compared with 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 pile ends after 75% and 100% release were analyzed using the average maximum strain method; 75% release refers to 15 strands being released, and 100% release refers to all strands being released. For example, **Fig. 7** shows the 100% release results for the nonstressing end of pile 5 (pile end 5S). For the stressing ends—for example pile end 5N, which had two rows of gauges—the averages of the strain-gauge 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 75% and 100% stress release cases were averaged for each pile end. The transfer lengths ranged from 21.5 in. (546 mm) to 29.0 in. (737 mm), with an average of 25.0 in. (635 mm) for the six pile ends.

The transfer length was predicted with Eq. (2) from ACI 440.4R-04:

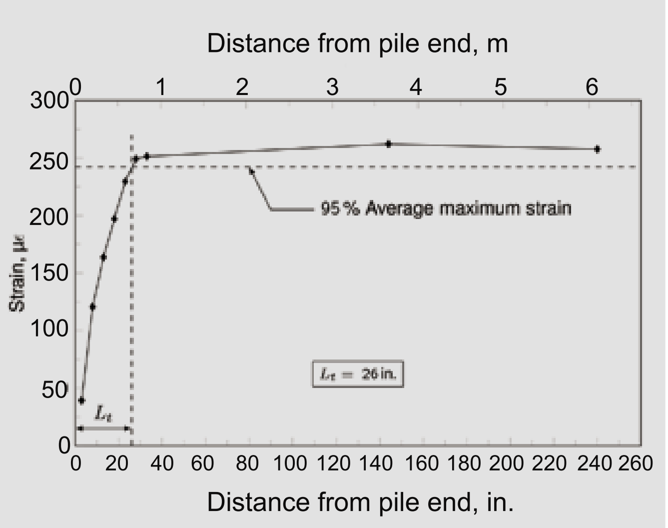

Figure 7. The average maximum strain method was used to determine the transfer length. Shown here are the measured strains on pile end 5S at 100% stress release. Note: L_t = transfer length. 1 in. = 25.4 mm.

$$L_t = \frac{f_{pi} d_b}{\alpha \left(f'_{ci}\right)^{0.67}} \tag{2}$$

where

f_{pi} = initial stress in strand

α_t = factor used in transfer-length equation

f'_{ci} = initial concrete compressive strength

The predicted transfer length was 37.3 in. (947 mm), using the factor α_t from Grace[20] of 11.2 for pounds per square inch and inch units (2.12 for megapascal and millimeter units) and f_{pi} of 220.0 ksi (1517 MPa). The average observed transfer length was 25.0 in. (635 mm), 33% less than predicted by Eq. (2). Mahmoud et al.[8] proposed a value for α_t of 25.3 for pounds per square inch and inch units (4.8 for megapascal and millimeter units), which resulted in a predicted transfer length of 16.5 in. (419 mm), 34% less than observed. Furthermore, the transfer length observed in this study was 31% less than the AASHTO LRFD specifications provision of $60d_b$ (36.0 in. [914 mm]). Equation (1) from ACI 318-11 resulted in a predicted transfer length of 40.2 in. (1020 mm), using an effective prestress f_{se} of 201.0 ksi (1386 MPa) after all prestress losses as calculated per *PCI Design Handbook*. The observed transfer length was 38% less than predicted.

Transfer lengths at the stressing ends (N) were greater than at the nonstressing ends (S). The average ratios of nonstressing end–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

Another research objective was to measure the CFCC strand development length and the pile flexural strength. To design a pile, an engineer must predict both of these values. 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 318-11 and AASHTO LRFD specifications using Eq. (3):

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

where

L_d = development length

f_{ps} = prestress in steel at the time for which the nominal resistance of the member is required

The first term in Eq. (3) is the ACI 318-11 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 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 and parameters were varied (**Table 1**). The piles were placed on elastomeric bearing pads, supported by two steel I-beams leveled and grouted to the floor. Load was applied at a rate of 250 lb/sec (1110 N/sec) until flexural cracks formed, and then the rate was changed to 200 lb/sec (890 N/sec) until a bond or flexural failure occurred.

Strain gauges, 2.4 in. long (60 mm), were used to measure concrete top fiber strains around the load point (for the development-length tests) and in the constant-moment region at midspan (for the flexural-strength test). Noncontact, laser displacement gauges 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, anchored to four CFCCs in the bottom of the pile.

Development-length tests

Development-length tests (tests 1 and 2) were performed on one 40.0 ft (12.2 m) long pile specimen. The concentrated load was applied to the pile by a hydraulic actuator. Because the predicted development length was less than 10 ft (3.0 m), the load was applied close to the support (**Fig. 8**). After the first test was completed, approximately 6.5 ft (2.0 m) of the pile's tested/damaged end was separated from the specimen and discarded. The remaining 33.5 ft

Table 1. Test matrix

Test number	Test type	Pile number	Simply supported span length, ft	Shear span, ft	Cantilever length, ft	Embedment length, ft
1	Development length	1	22.0	5.0	17.0	6.0
2	Development length	1	27.0	9.0	5.5	10.0
3	Flexural	2	38.0	13.3	n/a	14.3

Note: n/a = not applicable. 1 ft = 0.305 m.

(10.2 m) length was used for the second test. The damaged end was cantilevered approximately 5.5 ft (1.7 m), and the opposite, undamaged end of the pile was loaded. **Figure 9** shows the instrumentation layout for test 1. Six deflection gauges were used to monitor vertical deflections. Four electrical-resistance-foil strain gauges monitored the top surface strains near the load point.

For test 1, the embedment length was 6.0 ft (1.8 m), the simply supported span length was 22 ft (6.7 m), and the cantilever length was 17 ft (5.2 m). **Figure 10** shows the applied load compared with deflection (average of gauges D3 and D4). The first flexural crack was observed at a load of 175 kip (778 kN) and extended up to 2 ft (0.6 m) from the load point toward the end of the pile. Failure occurred at a load of 204.8 kip (911.0 kN). The flexural cracks propagated to 4.0 in. (100 mm) 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). There was no observable strand end slip of any of the CFCC strands throughout the test.

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 10 ft (3 m), the simply supported span length was 27 ft (8.2 m), and the cantilever length was approximately 5.5 ft (1.7 m). The load-versus-deflection plot was shaped similarly to that of test 1. The first flexural crack occurred at a load of 101 kip (449 kN) on the bottom of the pile under the load application point. The cracks propagated up to 3.0 in. (76 mm) from the top fiber and extended up to 3 ft (0.9 m) from the load point toward the end of the pile. Flexural failure occurred at a load of 120 kip (534 kN) and a deflection of 2.8 in. (71 mm). The maximum top-fiber (compressive) strain in the vicinity of the load point at failure was 0.00138 (due to bending), where local concrete crushing occurred. The crack pattern was similar to that in 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 test setups was 72.0 in. (1830 mm). 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 72.0 in. There was no observable strand slip of any of the four instrumented CFCC strands throughout tests 1 and 2.

The predicted development length from Mahmoud and Rizkalla[9] is 29.0 in. (737 mm); using the factor α_t from Grace[20] increases the prediction to 49.0 in. (1240 mm). Because these are both less than the shortest development length tested in this study where no strand slip was observed, one of these predictions might be reasonable. Lu et al. [23] predict development length as Eq. (4):

$$L_d = \tfrac{1}{3} f_{se} d_b + \tfrac{3}{4}(f_{pu} - f_{se}) d_b \tag{4}$$

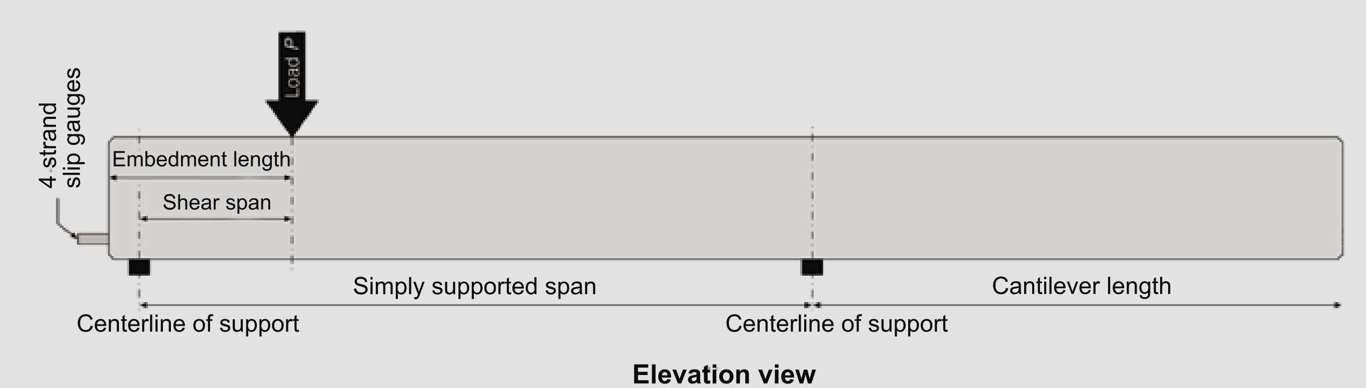

Figure 8. For the development-length tests, the load was placed near one of the supports.

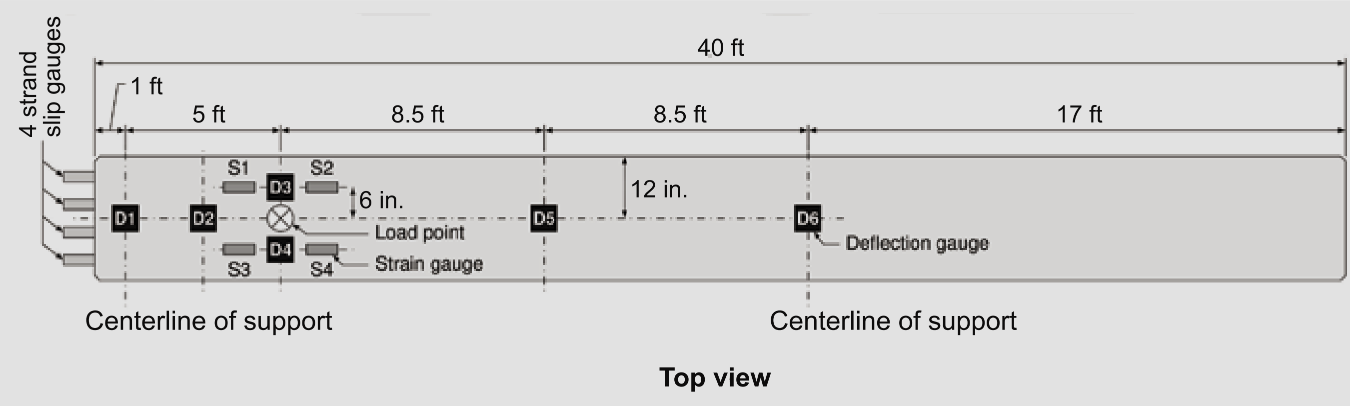

Figure 9. For the development-length tests, gauges were used to measure strains and deflections at several locations on the span. Test 1 locations are shown as an example. Note: not to scale. 1 in. = 25.4 mm; 1 ft = 0.305 m.

where

f_{pu} = ultimate tensile stress of prestressing strand

This results in a predicted development length of 102 in. (2590 mm), which is 42% longer than the shortest embedment length tested in this study. The predicted development length according to ACI 318-11 and AASHTO LRFD specifications is 123 in. (3120 mm), 71% more 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.

Flexural-strength test

A flexural-strength test (test 3) was performed on a second 40.0 ft (12.2 m) long pile specimen, supported at its ends. The instrumentation layout (**Fig. 11**) included 14 strain gauges and 10 noncontact deflection gauges. The single point load from the hydraulic actuator was transferred to the pile via a spreader beam, made from two steel I-beams, to generate two 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 3000 lb (13 kN).

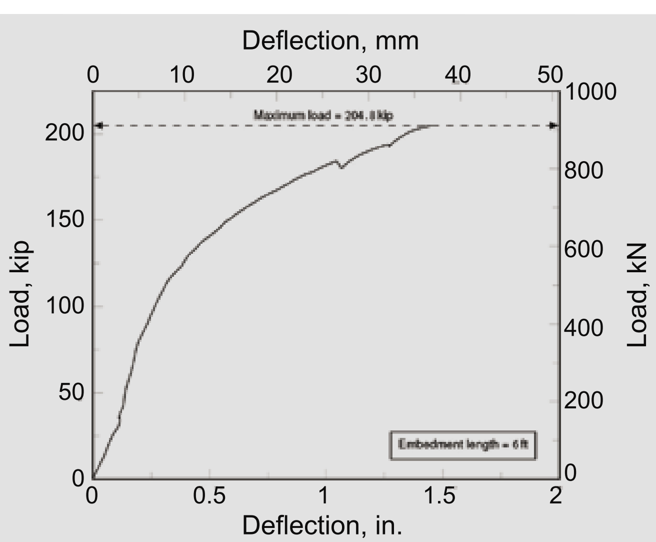

Figure 10. The applied load at failure was 204.8 kip for test 1, which had a 6 ft embedment length. Note: 1 ft = 0.305 m; 1 kip = 4.45 kN.

Three 4 × 8 in. (100 × 200 mm) concrete cylinders, tested on the day of the flexural-strength test (50 days after pile casting), had an average compressive strength of 9500 psi (66 MPa). **Figure 12** shows the applied load compared with midspan deflection (average of gauges D5 and D6). Failure occurred near the spreader-beam support (**Fig. 13**) at a load of 113 kip (503 kN) and a midspan deflection of 9.63 in. (245 mm), not including the effects of the pile or spreader-beam weight. The maximum top-surface compressive strain was 0.0013, an average of gauges S3 and S4 at midspan. The cracks were uniformly distributed in the constant-moment region and extended up to 5.0 ft (1.5 m) from the load points toward the ends of the pile. At the maximum load, the flexural cracks propagated up to about 3.0 in. (76 mm) from the top fiber. There was no strand-end slip in any of the strands throughout the test.

The applied failure load of 113 kip (503 kN) equates to a calculated moment of 753 kip-ft (1020 kN-m). This generated a total test moment at midspan of 875 kip-ft (1190 kN-m), including an initial calculated moment of 122 kip-ft (165 kN-m) due to the pile and spreader-beam weights. The calculated theoretical pile capacity was 809 kip-ft (1100 kN-m), so the measured flexural strength was 8% greater than the theoretical flexural strength. Furthermore, the 9.63 in. (245 mm) midspan deflection at failure indicates some ductility.

Pile-driving tests

During pile installation, the pile-driver hammer imposes large 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 counter-

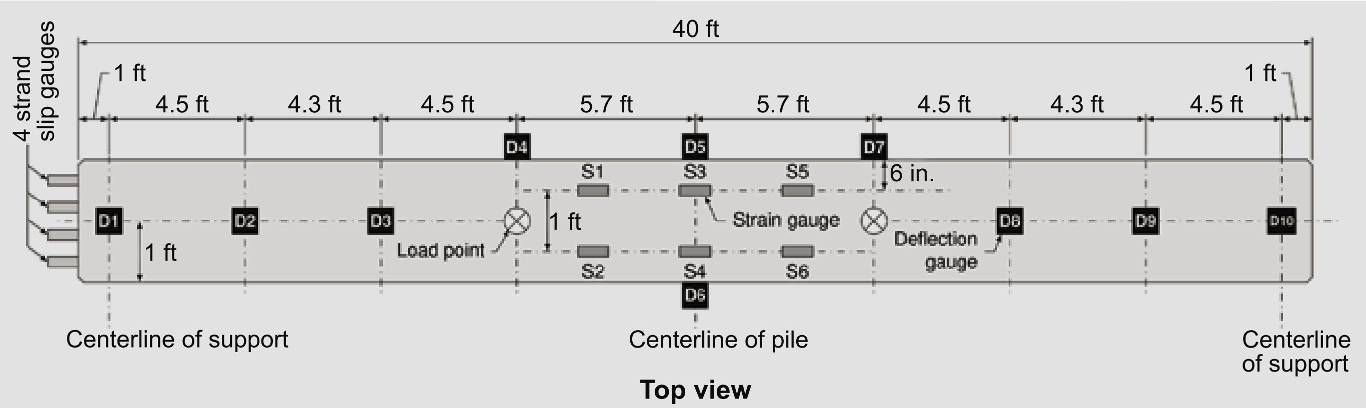

Figure 11. For the flexure test, gauges were used to measure strains and deflections at several locations on the span. Note: not to scale. 1 in. = 25.4 mm; 1 ft = 0.305 m.

act tensile stresses, the piles were designed to have a permanent compression of 1000 psi (6.9 MPa) 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 100 ft (30 m) long piles were installed by a contractor on the Deer Crossing Bridge on Interstate 4 and west of US Route 92 in Volusia County, Fla. The piles were driven adjacent to production piles. Measured four days later, the concrete compressive strength was 10,080 psi (69.50 MPa), averaged from two 4 × 8 in. (100 × 200 mm) 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 were subjected to 2765 and 3139 hammer blows, respectively. The pile-driving analyzer system and embedded data collectors 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 the pile-driving analyzer system and embedded data collector results. After testing, the piles were cut off at 2 ft (0.6 m) below grade, covered by soil, and abandoned in place.

Pile-driving analyzer

The pile-driving analyzer system was used to monitor the two piles during driving operations. It 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 (for example, 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.

Embedded data collectors

Embedded data collectors are strain transducers and accelerometers that are embedded in a concrete member. A

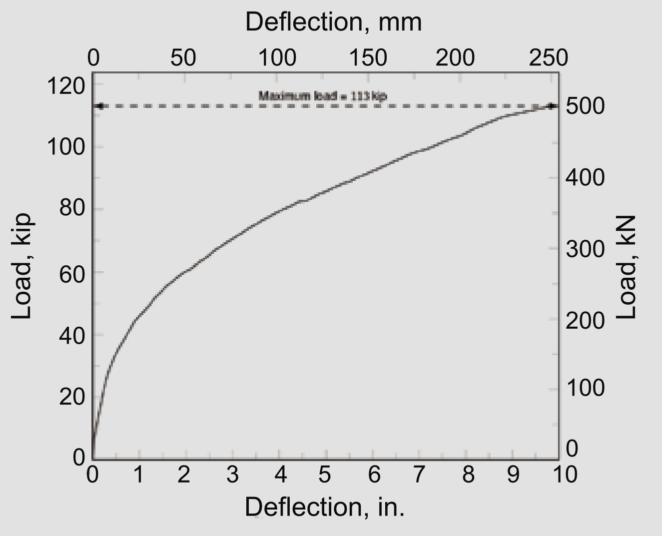

Figure 12. The applied load at failure was 113 kip for the flexure test. Note: 1 kip = 4.45 kN.

Figure 13. In the flexure test, the pile failed near one of the load points.

summary of an evaluation program to compare embedded data collector results with the pile-driving analyzer system and the associated signal-matching software is included in Herrera et al.[24] The embedded data collector 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. Embedded data collectors installed in the piles before casting were placed at midheight at a distance of two pile widths from the head of the pile and at one 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, 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 suggested 900 kip (4000 kN) driving resistance per FDOT's *Structures Design Guidelines*[25] for a conventional 24 in. (610 mm) prestressed pile. The data also suggest 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 the FDOT *Structures Design Guidelines* would allow for standard piles. Although driving and subsurface conditions prevented the development of maximum compression stresses based on the *measured* concrete compressive strength of 6.25 ksi (43.1 MPa) 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 3.60 ksi [24.8 MPa], assuming a *nominal* 6000 psi [41 MPa] concrete strength and 1000 psi [6.9 MPa] for initial prestress). In addition, the theoretical limit on tension stress, 1.38 ksi (9.51 MPa) 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 24 in. (610 mm) square prestressed concrete piles were cast using twenty 0.6 in. (15 mm) diameter CFCC prestressing strands. Two piles were 100 ft (30 m) long, and three were 40.0 ft (12.2 m) long.

- Transfer-length tests were performed on the three 40.0 ft (12.2 m) piles by measuring concrete strains during strand detensioning.

- Two development-length tests were performed on one of the 40.0 ft (12.2 m) piles.

- One of the 40.0 ft (12.2 m) piles was tested for flexural strength.

- The two 100 ft (30.5 m) long piles were driven at the Deer Crossing Bridge in Volusia County, Fla., to monitor the static resistance of the piles and the pile behavior during driving.

Conclusion

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 the following:

- use of a different header material (for example, 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

- increased inspection needed to ensure compliance with material handling requirements and proper installation of couplers

Transfer length of CFCC

An analysis of the strain-gauge data from the transfer-length tests suggests that the CFCC strands have a 25 in. (635 mm) transfer length, which is 38% and 31% less than that predicted by ACI 318-11 and AASHTO LRFD specifications, respectively, for steel strands. The observed transfer length is 33% less than the transfer length calculated from ACI 440.4R-04 and using the alpha factor from 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 conservative in that it is less than the predicted values.

Development length of CFCC

The strands in the test 1 pile had an embedment length of 72.0 in. (1830 mm). 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 72.0 in. and therefore less than the AASHTO LRFD specifications prediction of 123 in. (3120 mm) for steel strands (using CFCC's value for guaranteed ultimate tensile strength).

Flexural strength of CFCC-prestressed pile

The measured flexural strength of the CFCC-prestressed concrete pile was 8% greater than the theoretical flexural strength. 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 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 midspan had deflected more than 9.0 in. (230 mm) at failure, which indicates some ductility. This is consistent with the approximate 10.0 in. (254 mm) deflection of concrete piles with similar dimensions but prestressed with steel and tested by Abalo et al;[12] however, CFCC is a brittle material. If the design requires significant ductility, then the designer needs to fully understand the material properties and behavior.

Pile driving

Two 100 ft (30.5 m) long 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. This demonstrates that implementing the use of CFCC in prestressed concrete piles is feasible.

Further research

Additional research is needed on prestress loss effects for CFCC strands because their material properties differ from those of steel. Also, CFCC strands do not corrode, and if they are used in conjunction with CFCC spirals for transverse reinforcement, then it may be possible to reduce the concrete cover. More testing is needed to assess whether reduced cover is feasible. Deterioration of CFCC strands due to environmental exposure, including ultraviolet radiation and saltwater, also needs more study. Reliability of the coupling system that is used for stressing should be investigated.

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FDOT provided the funding for this project. William Potter provided much insight, and Chris Weigly contributed time and instrumentation expertise. Gate Precast Co. in Jacksonville, Fla., deserves 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 being 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 pile-driving analyzer system and on-site analysis and expertise and Don Robertson and Harold Dohn at Applied Foundation Testing Inc. for providing embedded data collector pile-driving testing services.

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Notation

d_b = strand diameter

f'_{ci} = initial concrete compressive strength

f_{pi} = initial stress in strand

f_{ps} = prestress in steel at the time for which the nominal resistance of the member is required

f_{pu} = ultimate tensile stress of prestressing strand

f_{se} = effective stress in strand after losses

L_d = development length

L_t = transfer length

P = load

α_t = factor used in transfer-length equation

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Abstract

Carbon-fiber-reinforced polymers are advantageous because of their resistance to corrosion, particularly in aggressive or saltwater environments. This study examined carbon-fiber-composite cable (CFCC) in prestressed concrete piles for bridge foundations. Five 24 in. (610 mm) square piles were constructed and prestressed with 0.6 in. (15 mm) diameter CFCC. Strain measurements during detensioning indicated that the transfer length was less than design code predictions for steel prestressing strands. Additional tests showed that the development length was also less than code predictions. A bending test on a 40.0 ft (12.2 m) long pile resulted in a flexural strength that was 8% greater than the theoretical flexural strength, and the pile had good ductility with a midspan deflection of more than 9.0 in. (230 mm) at failure. Two 100 ft (30 m) long piles, monitored under hard-driving conditions at a bridge construction site, behaved well with no major damage or loss of prestress.

Keywords

Carbon-fiber-composite cable, carbon-fiber-reinforced polymer, CFCC, CFRP, pile, transverse spiral.

Review policy

This paper was reviewed in accordance with the Precast/Prestressed Concrete Institute's peer-review process.

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