# Bridge Case Study: What a Contractor Needs to Know on an FRP Reinforcement Project

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**Abstract:** Civil works construction is now facing a transformational change with the growing implementation of fiber-reinforced polymer (FRP) materials as rebars and tendons for concrete structures. To avoid surprises and decrease risk, a contractor must be cognizant of FRP's differences with traditional steel reinforcement in terms of procurement, quality control, and installation. A bridge replacement project under construction in Florida is presented in this paper as a case study to address the technology's constructability and adaptability in a number of structural elements for both substructure and superstructure. The paper also provides some productivity considerations for the case of FRP reinforcing cage assembly in terms of work hours, including a comparison with the traditional steel solution. The case study points out critical issues such as procurement, acceptance, and workforce experience that can provide guidance for both implementation and standardization of the technology. **DOI: 10.1061/(ASCE)CC.1943-5614.0000998.** 



## Introduction: The Halls River Bridge

The Halls River Bridge (HRB) is a bridge replacement project of an existing structure that reached functional deficiency. Fig. 1 shows the schematic plan and elevation views consisting of reinforced and prestressed concrete (RC and PC) elements. The proposed two-lane roadway consists of 3.66-m (12-ft) lanes, 2.44-m (8-ft) shoulders, and 1.52-m (5-ft) sidewalks on both sides. This section of roadway has been classified as a rural major collector, with a design speed of 80 km/h (50 mph). In order to consistently guarantee one open lane to traffic due to this being the sole access to the Homosassa Springs community, the bridge was designed to be rebuilt in three phases:

- Phase I consists of an initial realignment of approach roadway and installation of traffic control devices and temporary signing for switching traffic [Fig. 2(a)].
- Phase II consists of the northern bridge portion demolition and construction of half of the new structure alongside the existing [Figs. 2(b and c)].
- Phase III consists of demolishing the remaining southern portion of the existing bridge and the completion of the project [Fig. 2(d)].

During Phase II, the two-lane traffic is limited to one travel lane, phased by traffic lights and assisted by trained flaggers during critical construction activities. The decision of having circulating traffic during construction is vital for the location, since the Halls River road is a dead-end road where no alternative detours are available, making the existing bridge the only path to reach a small town located on the west side. The double continuous line in Fig. 1 represents the centerline and axis of symmetry of the bridge, while the dashed line represents the construction joint that indicates the phasing construction line, which is offset approximately 1.22 m (4 ft) from the centerline of the bridge. Fig. 2 shows the construction sequence and phasing of the bridge.

During Phase II, the following major equipment was deployed to the site: two 230 Manitowoc 888 crawler cranes (Manitowoc, Milwaukee, Wisconsin) (one at each end of the bridge); one loader Deere 644K (John Deere, Moline, Illinois); one double-axle trailer; one welder 340 AMP; one compressor 185 CFM; three barges of different sizes [$3 \times 12 \times 0.6$ m ($10 \times 40 \times 2$ ft), $3 \times 6 \times 0.6$ m ($10 \times 20 \times 2$ ft), $3 \times 3 \times 0.6$ m ($10 \times 10 \times 2$ ft)]; one excavator 336E L CAT (Caterpillar, Deerfield, Illinois) (equipped with rotary rock grinder and bucket); one hydraulic impact hammer APE Model 7-3 (American Piledriving Equipment, Kent, Washington); and one vibratory hammer APE 44 vibro (American Piledriving Equipment, Kent, Washington) (or similar) for both bridge crews.

The 230-ton cranes and the hammers were selected according to site access and to the large size of the PC elements (piles and sheet piles) to be driven from the shores. The hammer was equipped with a variable throttle control unit that allowed it to operate initially at low levels and then gradually increase to the necessary power required for pile installation (ramp-up measure) to minimize the impact on the aquatic wildlife. The pile installation activities consisted of predrilling 0.56-m (22-in.) diameter starter holes through the embankment fill material and cap rock, down to the required preform elevation at $-8.2$ m ($-27$ ft) for intermediate bents, using an APE Model 50 Top Drive Auger. Upon reaching the desired depth with the auger, the drilling was stopped, and the auger was lifted. The material remaining within the auger flight was removed from the hole and this activity was repeated until the hole was clean to the desired depth. The piles were subsequently driven to the cutoff elevation. Close monitoring for vibrations on existing bridge and adjacent elements was conducted to avoid any damage and assure traffic safety.

Once piles and sheet piles were set in place, the following activities could be conducted with lighter equipment. The characteristic light weight of FRP reinforcement was expected to be

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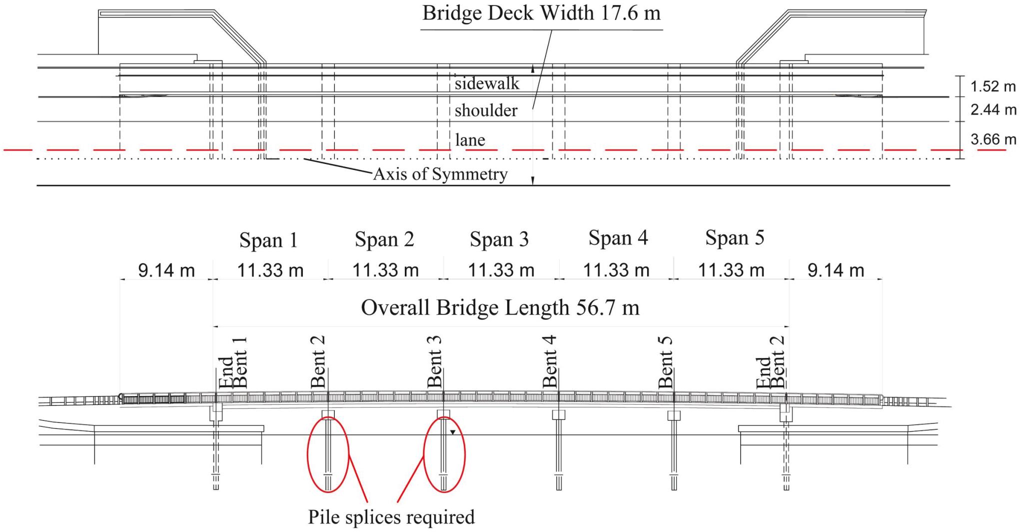



**Fig. 1.** Plan and elevation views of the new Halls River Bridge.

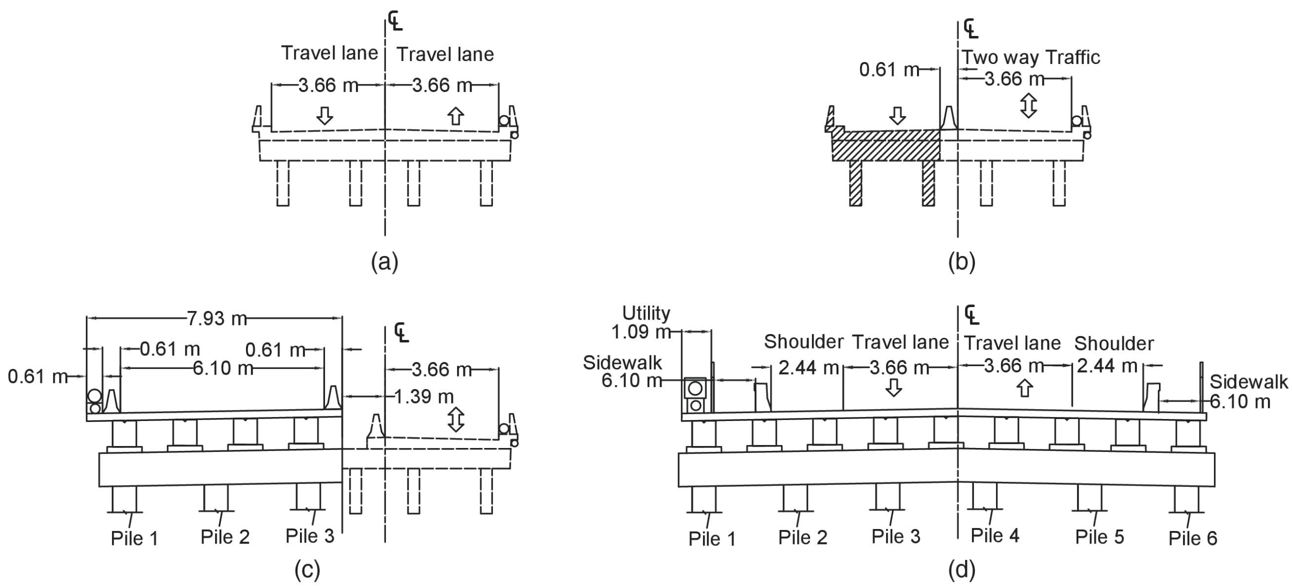



**Fig. 2.** Construction sequence and phasing: (a) Phase I; (b) Phase II; (c) Phase II completed; and (d) Phase III completed.

advantageous, especially during superstructure construction (Hastak et al. 2004), requiring less labor force and a smaller crane.

For this project, the Florida Department of Transportation (FDOT) standardized several innovative structural elements utilizing carbon FRP (CFRP) tendons and glass FRP (GFRP) rebars, sometimes in combination with stainless steel (SS) bars and traditional carbon-steel (CS) strands.

The 56.7-m (186-ft) bridge included, among other structural elements,

- 36 CFRP-PC 0.46-m (18-in.) square bearing piles;
- 86 Hybrid CS-PC/GFRP-RC 305 × 762-mm (12 × 30-in.) sheet piles;
- six GFRP-RC pile-bent caps;
- 998-$m^2$ (10,742-$ft^2$) GFRP-RC bridge deck, 0.22 m (8.5 in.) thick, two-way slab top and bottom reinforced;
- 150 m (492 ft) of GFRP-RC traffic railings; and
- two 9.1-m (30-ft) GFRP-RC approach slabs.

The embankment at both approaches is supported by 149 CFRP-PC/GFRP-RC 305 × 762-mm (12 × 30-in.) concrete sheet piles, GFRP-RC bulkhead and deadman caps, and a 19.5-m (64-ft) GFRP-RC gravity wall. The use of such innovative materials and structural solutions targets a reduced life-cycle cost and environmental impact and an extended service life of 100 or more years (Cadenazzi et al. 2018, 2019a, b).

The implementation of SS and hybrid CS/GFRP alternatives in PC sheet piles was included in elements for which proposers advocated a synergy among high strength, ductility, and environmental resistance, given the chloride-ion-rich subtropical environment of the HRB site (Nolan et al. 2018).

The deployment of the innovative technology in the case study faced several challenges, such as unforeseen situations requiring on-site adaptation and modifications commonly experienced in FDOT projects that use traditional carbon steel materials, but that, at the time of writing, still need to be addressed in the specifications for FRP materials. Examples of some of the challenges included traffic railing constructability, unforeseen pile splicing, and sheet pile wall modification and redesign. These challenges were met and overcome providing an opportunity to field proof the technology's constructability and adaptability. This case study may result in additional constructability considerations and guidance for designers and contractors.

## Significance

All of the material presented in this paper was gathered thanks to the constant presence at the HRB site of the first author. Through direct observation and constant monitoring of the construction activities, the first author was able to collect the present data to field proof the technology's constructability and adaptability. Being a DOT project, the challenges encountered at HRB and the respective solutions adopted were key for the widespread implementation and standardization of the technology. Some of the innovative solutions adopted contributed to the advancement of the technology's construction knowledge. To the best of the authors' knowledge, this paper investigates specific circumstances that the FRP technology never faced before, and thus the respective solutions are important to the literature and to the FRP construction sector. The ultimate scope of this paper is thus to provide more resources, including construction knowledge and experience, for planning and designing in advance a quality project that uses FRP materials.

## FRP Rebar and Tendon General Considerations

General recommended practices for the use of FRP rebars and tendons are discussed with consideration for the construction activities that required handling, storing, installing, cage lifting, and on-site concrete casting.

### Handling

Given their lower toughness and stiffness compared to traditional materials, FRP bars (and tendons) require care in ensuring that they are not damaged or excessively flexed during handling or lifting. Generally, the use of straps or spreader bars helps distribute the lift force and avoids excessive deformation (ACMA 2016). In addition, FRP products should not be dragged, dropped, or thrown to avoid damaging the bar surface and exposing the fibers.

### Storage

Containers are the preferable means for transportation and storage. When stored outdoors, bars should be covered by opaque plastic fabrics to avoid mishandling and guarantee protection from direct sunlight (ACMA 2016). If stored outdoors for more than 4 months, the coverage becomes a requirement for protection from ultraviolet ray exposure.

Long-term exposure of FRP bars to temperatures above 120°C (248°F) is harmful to the resin component [ACI 440.1R (ACI 2015)]. While in storage or any time prior to installation, FRP bars should be placed on firm, level, clean, nonstaining surfaces similar to traditional reinforcement (ACMA 2016).

### Installation

As nonductile materials with low toughness, there are currently limited FRP methods available for connecting precast concrete structural members and FRP elements on site. Despite some technology currently under development, FRP bars cannot be bent, welded, meshed, joined, or threaded after placement. Splicing of FRP elements is done usually by overlapping.

It is always good practice to have trained labor to ensure correct FRP installation and minimize the risk of damage and movement of reinforcement during concreting operations (ACMA 2016).

Given the FRP's noncorrosive nature, it is good practice, and required in FDOT specs (2016b), to always use plastic-coated wires or plastic zip ties when tying FRP reinforcement to avoid corrosion (Zoghi 2013) or potential damage to the bars' surface.

### Cage Lifting

The lifting process should guarantee the reinforcing cage integrity and safety at any time, avoiding large deformations of the cage that can result in displaced bars or even in breaking open of the cage when the bars are not well tied (Schürch and Jost 2006). To avoid such incidents, contractors can use supporting frames or steel beams that stiffen the GFRP cage during handling and lifting (Schürch and Jost 2006). At the HRB, a four-point pick-up steel spreader beam was used to lift and place the GFRP cage.

### Casting

Because of its relatively low density, FRP reinforcement may float during casting, especially under vibration (Zoghi 2013). For this reason, it is important to have the bars properly secured to the formwork at several locations and provide additional plastic chairs as compared to traditional steel practice (CRSI 2009) due to FRP lower stiffness. The HRB experience revealed the importance of using a rubber-tipped vibrator that protected the FRP rebar from any surface damage during concrete vibration activities.

Fig. 3 shows the rubber-tipped vibrator used at HRB for any cast-in-place activity.

## Challenges and Solutions

This section provides general considerations prior to construction and challenges encountered during construction along with the adopted solutions.

### Procurement Time and Acceptance Testing

Procuring the FRP material at the HRB required time and adjustments to the project schedule that would not typically have been experienced with traditional materials. FDOT specifications require that a manufacturer be preapproved and listed as a qualified vendor. In addition, each lot of FRP reinforcement delivered to the site must undergo specific testing for its acceptance (FDOT 2016b). Required tests per ASTM standards must be performed by an approved laboratory (FDOT 2016b). Such tests include

- degree of cure per ASTM E2160 (ASTM 2018a),
- fiber content per ASTM D2584 (ASTM 2018b),
- moisture absorption per ASTM D570 (ASTM 2010),
- measured cross sectional area per ASTM D792 (ASTM 2013), and
- ultimate tensile strength and tensile modulus of elasticity per ASTM D7205/D7205M (ASTM 2016).

It should be noted that among the preceding tests, the moisture absorption test can take up to 11 weeks. These testing requirements need be considered in the project schedule as an activity on the critical path.

To further aggravate the lead-time procurement of material for this project, the manufacturers of FRP bars and tendons for this project were based abroad as there are a limited number of FRP suppliers in the US approved by FDOT. For fabrication of the precast PC bearing piles and the PC sheet piles, the CFRP strands

**Fig. 3.** Rubber-tipped vibrator. (Image by Thomas Cadenazzi.)

and spirals were produced in Japan and shipped to the precast yard in Jacksonville, Florida. Similarly, the GFRP rebars for all cast-in-place RC elements (bulkhead and deadman caps, pile-bent caps, bridge deck, approach slabs, gravity wall, and traffic railings) were manufactured in Italy and shipped to the site in Homosassa, Florida, via four different shipments, each requiring a month-long surface transportation time. For this reason, the procurement must consider in the schedule the lead time for manufacturing and shipping, as well as procurement of additional quantities of reinforcement to ensure immediate replacement in case of damages during shipping or on site. The general contractor of this project selected an Italian manufacturer for a number of reasons beyond the scope of this paper. However, given the growing market, there are an increasing number of domestic suppliers as well as nondomestic suppliers that are setting up distribution centers in the US, in order to overcome lead-time issues.

The industry is attempting to define what constitutes a *lot*, in order to ensure quality control and at the same time be realistic with the associated testing activities. Existing documents define a lot as any bar produced from start to finish with the same constituent materials used in the same proportions without changing any production parameter (ACI 2015). However, there is also a trend in defining a lot in terms of linear meters (or linear feet) of bars produced. This is a current debate taking place from a quality control standpoint.

On the other hand, from a quality assurance standpoint, this project involved testing activities for the material acceptance. FDOT specifications did not provide a clear definition of what represented a lot of FRP products, in order to determine the testing frequency. Initially, the lot definition reverted to the *FDOT Materials Manual* (FDOT 2014b), which states that any time a raw component of the manufacturing process is changed, a new lot begins. This would have resulted in an unreasonable number of lots that would consequently have required individual acceptance testing. As a result, the contractor was permitted to randomly sample each delivery to verify compliance with acceptance requirements. FDOT is in the process of revising its specification to define what constitutes an FRP lot.

### Traffic Railings

In Phase II, the north side of the bridge had a temporary traffic railing that was replaced with a permanent GFRP-RC railing during Phase III. The permanent railing was positioned over the Phase II temporary travel lane of the newly constructed deck. In order to insert the permanent north side traffic railing after the casting of the permanent deck and to give a structural connection between the deck and traffic railing, it was necessary to drill the rebars inside the newly constructed deck. For this reason, the north side of the deck was designed to include PVC inserts or drillable blockouts (Rocchetti 2017). Such blockouts were installed with the placing of the deck reinforcement during Phase II.

This process was intended to allow accurate embedment of the GFRP bar insert from the railings without damage to the deck reinforcement. Additionally, the process required transverse supporting rebars to ensure stability of the blockouts during construction. Fig. 4 shows the traffic railing reinforcement along with blockouts (coded as 5i and 5d in the figure) and additional bars, displayed as solid-filled elements.

This solution proved to be problematic as the lack of deep surface deformation in the GFRP bars allowed for the tied deck reinforcement to move and slightly shift under the weight of the workers' boots. As the GFRP bars shifted, the blockouts also moved out of alignment. After several unsuccessful attempts, the contractor proposed an alternative solution to use a shear key blockout (Fig. 5). As indicated in Fig. 6, each shear key blockout was tied to the longitudinal bars of the top mat so that its location and the locations of adjacent bars were known based on bar spacing. This shear key blockout was made of four 12.7-mm (0.5-in.) thick sheets of hard insulation panels of extruded polystyrene. That means, in total, that the 51-mm (2-in.) thickness of the blockout matched the entire concrete cover of the deck in this location. The shear key blockout also allowed for the location of transverse bars, so that they were not damaged during drilling in Phase III. At the time of casting of the permanent railing, the temporary keyway was removed and drill holes for the dowel bars were positioned to avoid existing deck reinforcing, by having open deck reinforcement [Fig. 6(b)]. At this stage, a plywood template was also set every 229 mm (9 in.) to facilitate drilling for the inclined 5d bars. Holes for the 5i bars were instead spaced every 114 mm (4.5 in.). Fig. 6(b) only shows longitudinal reinforcement because the transverse bars were not present in such locations.

At the time of installing the traffic railing reinforcement, epoxy was poured and bars 5i and 5d were inserted inside the drilled holes [Fig. 6(c)]. After epoxy was cured, the traffic railing concrete was cast, as well as the shear key [Fig. 6(d)].

### Cracks on Cutoff Piles

During pile cutoff activities, some cracking between the CFRP strands was observed on the faces of several piles. Cracks lay perpendicular to the spiral reinforcement [Fig. 7(a)] and between strands [Fig. 7(b)]. In some cases, longitudinal cracks extended from the cutoff end of the pile as well [Fig. 7(c)].

Cracks were caused by the transverse pressure exerted by the CFRP tendons. The majority of the strands on the cutoff face of the piles had slipped in the order of 9.53 mm (3/8 in.), as indicated in Fig. 7(b), resulting in the formation of cracks perpendicular to

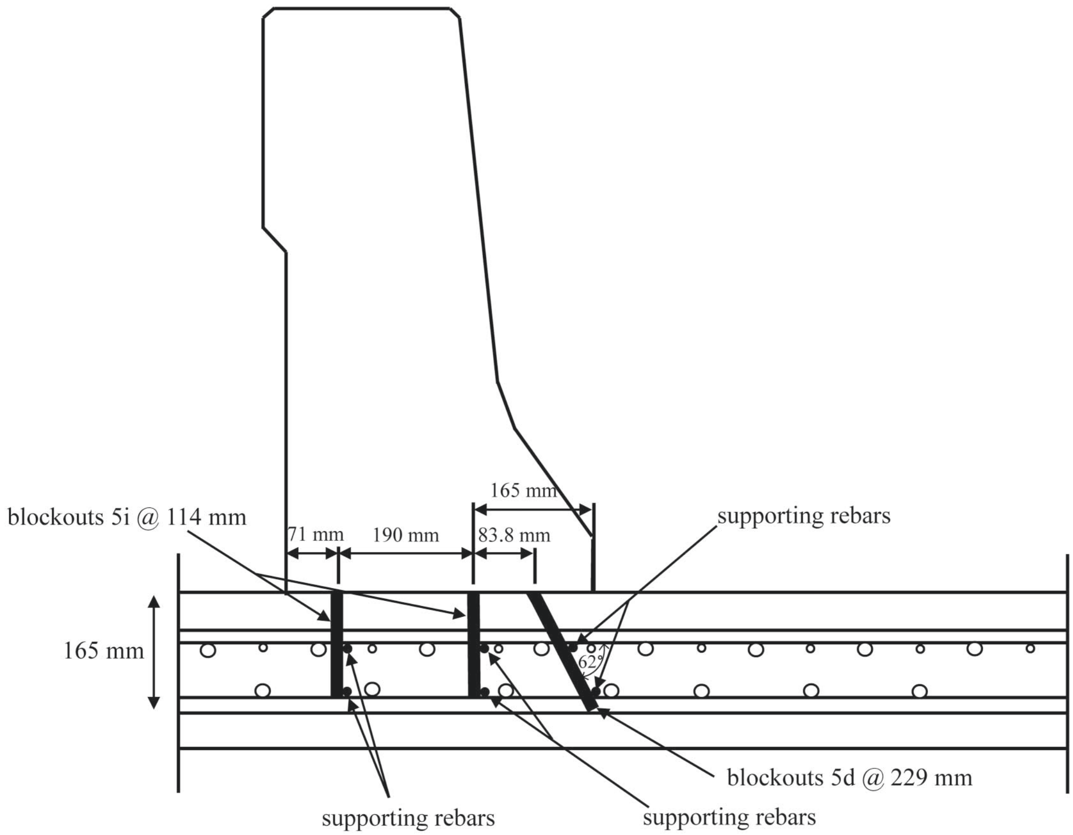



**Fig. 4.** North side traffic railing original construction. (Data from Rocchetti 2017.)

**Fig. 5.** Shear key blockout. (Image by Thomas Cadenazzi.)

the spiral reinforcement [Fig. 7(a)]. The maximum crack size measured in one of the piles was 0.28 mm (0.011 in.) with others equal to or lower than 0.18 mm (0.007 in.). Given that the length of most cracks was within 305 mm (12 in.), no remedial action was necessary as the cracks were encapsulated in the pile cap pour. Cracks that did extend beyond the cap were repaired per FDOT subarticle 400-21.5.2 (2016a) based on crack width.

## Pile Splices

Fig. 8. (FDOT 2014a) shows a cross section and side view of the typical CFRP-PC pile used for the bridge substructure.

Given unexpected soil conditions, pile splicing was required during construction for four piles in Bent 2 and one pile in Bent 3 (Fig. 1 for location). The length of the splices was based on an exploratory H-pile monitored and driven to capacity for 45.7 m (150 ft). The exploratory pile was driven halfway between Pile 3 of Bent 2 [pile identification is shown in Figs. 2(c and d)] and the existing bridge.

The total length of the splice was calculated as the difference between the length of the exploratory pile at capacity and the length of the original 20.1-m (66-ft) long PC pile. The pile splice length thus calculated was 25.6 m (84 ft). By means and methods of construction, such a splice was impractical to handle. Typical pile splices are approximately 6.4 m (21 ft) to 9.1 m (30 ft). Since the total splice length was longer than the installed pile, it would be difficult and also quite expensive to safely support a single 25.6-m (84-ft) splice next to active bridge traffic. Therefore, the pile splices needed to be divided into two segments of 12.8 m (42 ft) each.

Fortunately, each of the five installed piles needed one drivable unforeseen 12.8-m (42-ft) long pile splice, while an additional drivable preplanned pile splice was only required for Pile 3 in Bent 2 (close to the location of the exploratory H-pile). Figs. 9(a and b) (FDOT 2014a) show reinforcement detail of the drivable unforeseen pile splice and the drivable preplanned pile splice, respectively. Fig. 9(c) shows the lengths and splice sequence of the pile requiring two splices.

For the dowels, it was required to use #6 CFRP solid bars. Unfortunately, the FRP industry could not supply #6 CFRP bars without long lead times for large order quantities. The manufacturer of the CFRP-PC piles offered the DOT 19-mm CFRP strand (CFCC) to use as pile splice dowels, but the lack of experience or test data for nonprestressed application of stranded CFRP required

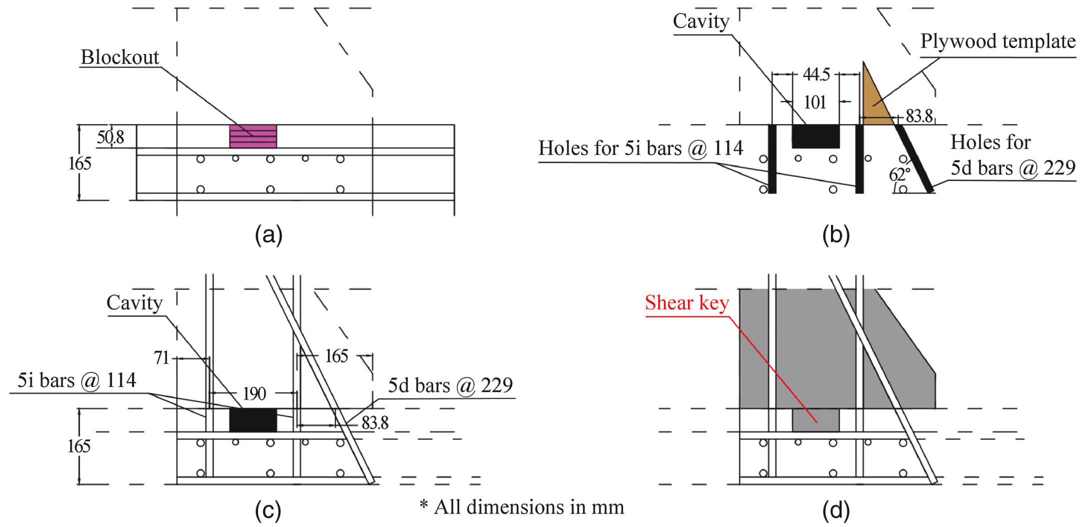



**Fig. 6.** (a) North side traffic railing construction sequence. Blockout insert: (b) holes for dowel bars; (c) dowel bars insert; and (d) casting of traffic railing and shear key.

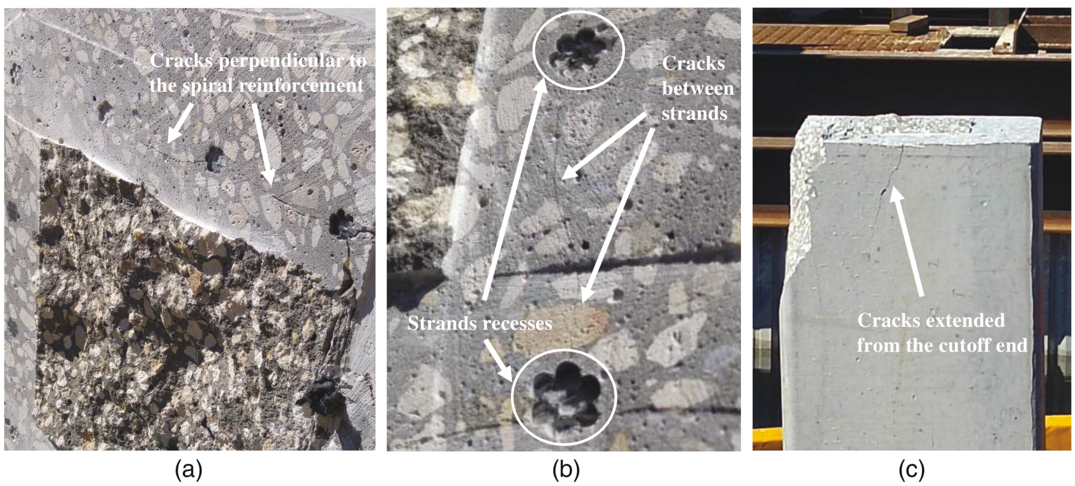



**Fig. 7.** Pile cracks (a) perpendicular to the spiral reinforcement; (b) between strands; and (c) extended from the cutoff end of the pile. (Images by Thomas Cadenazzi.)

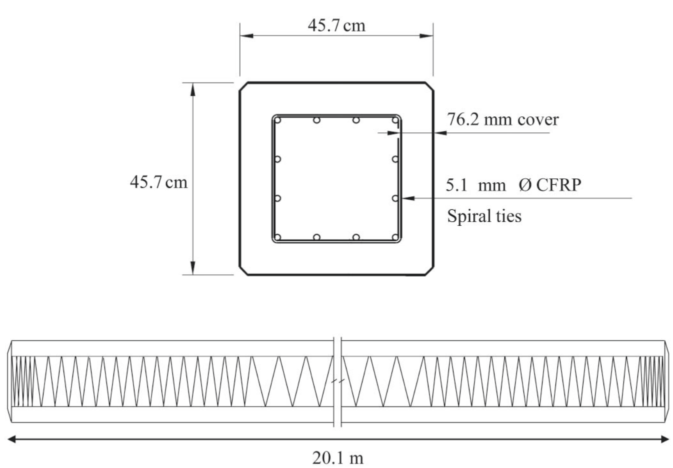



**Fig. 8.** 45.7-cm$^2$ CFRP-PC pile cross section and side view. (Data from FDOT 2014a.)

a test confirmation for equivalent shear resistance and development length. Additionally, the contractor and precaster were also wary of the flexibility of nontensioned strands and potential installation complications for CFCC, so SS solid bar was perceived as the lower risk and expedited solution. To resolve the issue, FDOT allowed the use of no. 10 SS bars (2205 alloy).

On top of the existing pile and the drivable unforeseen pile, eight holes were drilled [Fig. 10(a)] to accommodate the male section of both the drivable unforeseen pile and drivable preplanned splice [Fig. 10(b)]. Given the outside diameter of the no. 10 stainless steel bar being 41.27 mm (1.625 in.), holes 823 mm (2.7 ft) deep and 47.6 mm (1.875 in.) in diameter were drilled to allow tolerance. In order to complete the splice connection on site, a plywood form was assembled, nailed, and secured to the pile head using metal flashing [Fig. 10(a)]. On the top of the pile, five spacers were liquid-nailed in order to provide a gap for the epoxy to flow evenly inside the predrilled holes [Fig. 10(a)]. Using an air compressor, the holes were first cleared of any dust. About 26.5 L (7 gal.) of epoxy

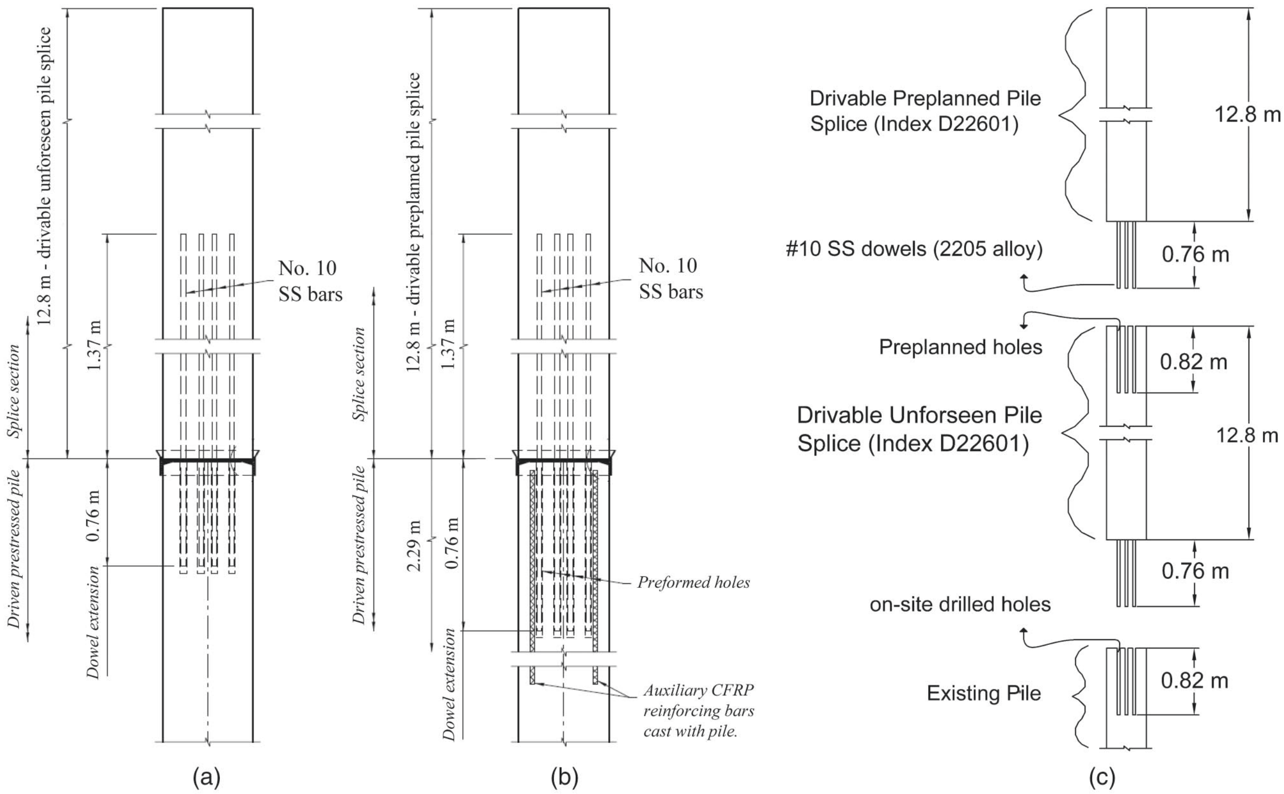



**Fig. 9.** (a) Drivable unforeseen PC pile splice detail; (b) drivable preplanned prestressed/precast pile splice detail (data from FDOT 2014a); and (c) pile splice installation sequence.

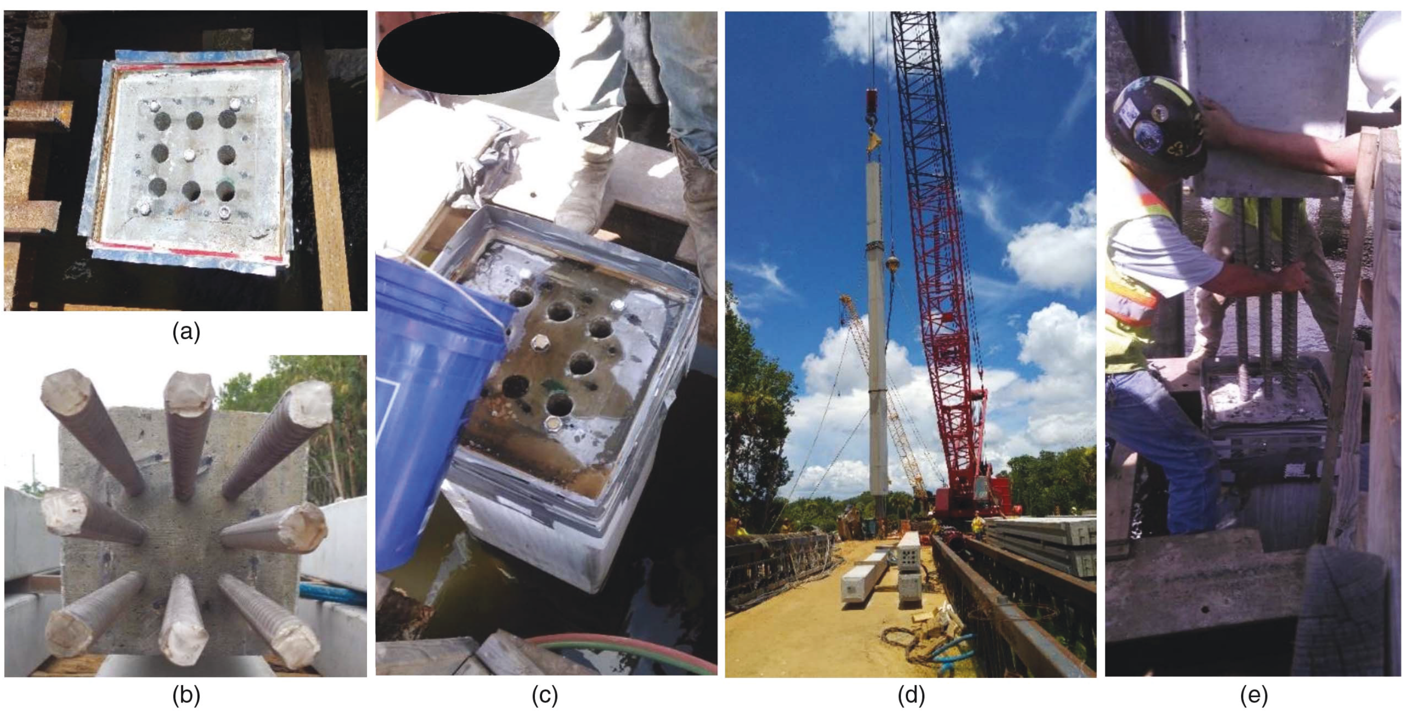



**Fig. 10.** Construction sequence of CFCC prestressed splice piles. From left to right the figures display the splice piles, the lifting of the piles, and the epoxy splice connection. (Images by Thomas Cadenazzi.)

(two-part Pilgrim EM CBC IV epoxy) were poured into each splice connection [Fig. 10(c)]. The male splice section was then lowered onto the installed pile [Fig. 10(d)], ensuring proper placement of the SS bars [Fig. 10(e)]. Once the epoxy had cured, the spliced pile was driven to bearing, or spliced again with the preplanned section in the case of Pile 3.

Pile driving was monitored with a Pile Driver Analyzer (PDA) system. Given ground conditions, pile integrity, hammer

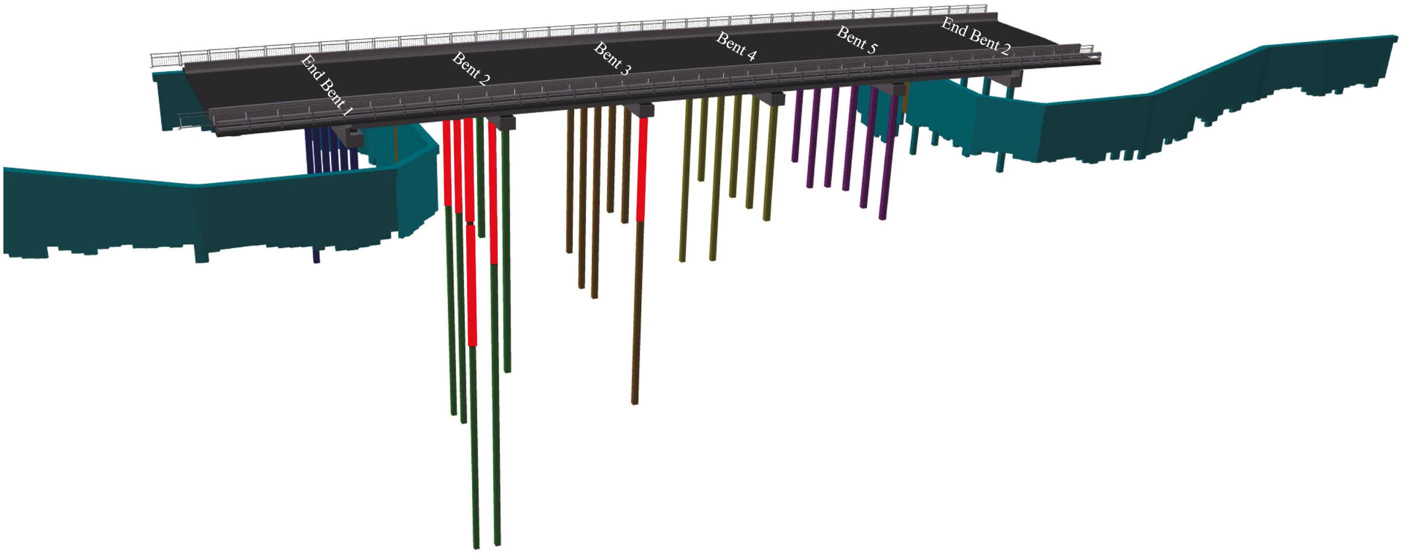



**Fig. 11.** 3D bridge model.

performance, and driving stresses along the length of the pile, a minimum end bearing (EB) of 1,779 kN (400 kip) was required. Pile splices were driven until such capacity was reached.

Fig. 11 shows the three-dimensional (3D) model of the newly constructed bridge, representative of the revised design and as-built elevations; marked in red are the locations of the pile splices in Bent 2 and Bent 3, respectively.

### Sheet Pile Wall Redesign

The PC sheet pile retaining wall in direct contact with saltwater includes the use of GFRP reinforcing bars along with CFRP prestressing strands. The PC sheet pile cross section is rectangular with a width of 762 mm (30 in.) and a depth of 305 mm (12 in.), as shown in Fig. 12. Each pile is 8.2 m (27.5 ft) long and was initially designed to reach a tip elevation of −7.8 m (−25.5 ft).

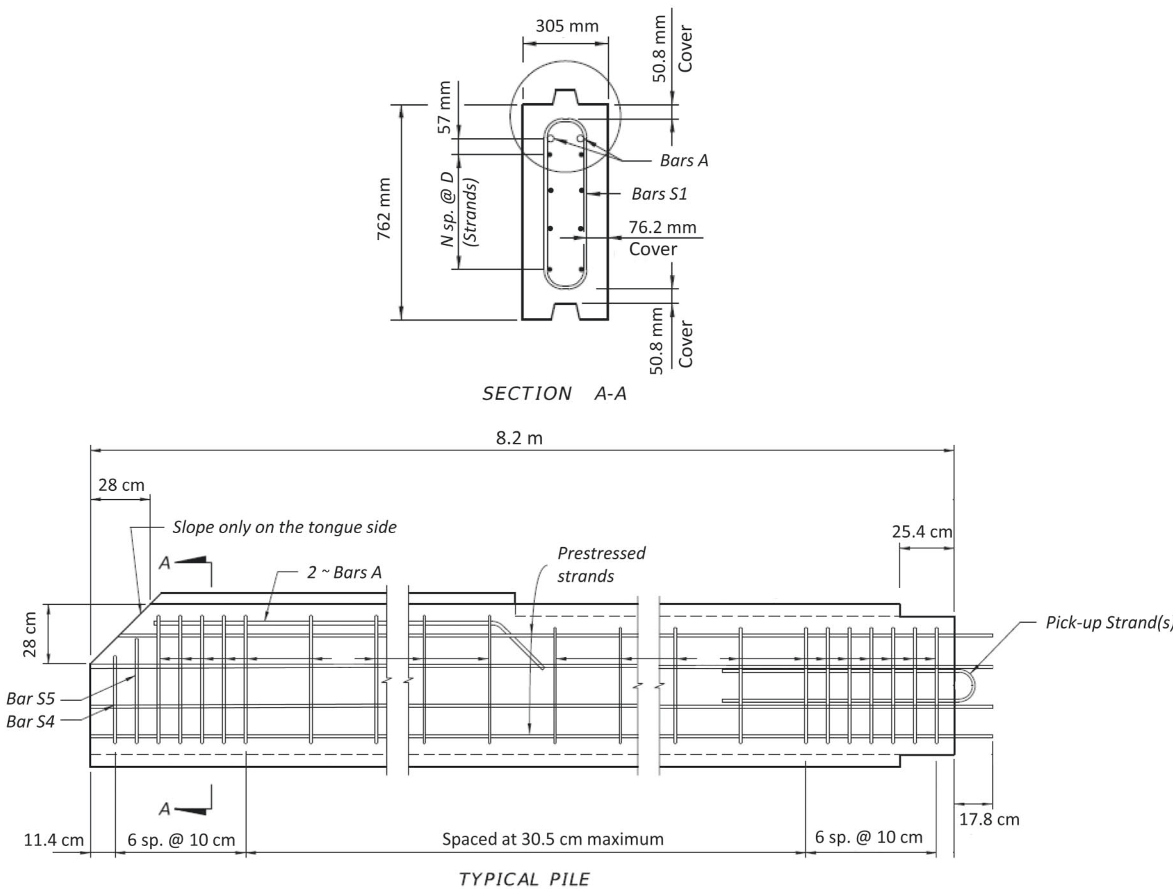



**Fig. 12.** 76.2 × 30.5-cm PC sheet pile cross section and side view. (Data from FDOT 2014a.)

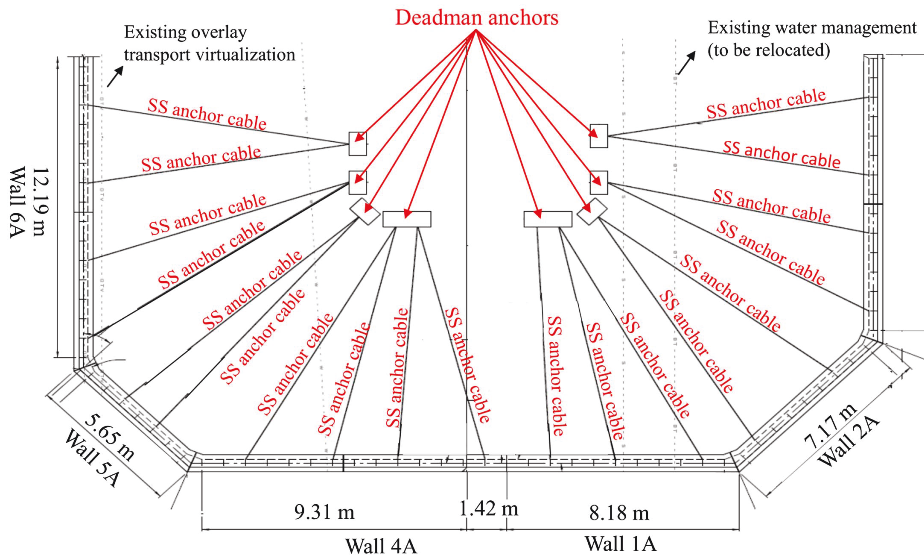



**Fig. 13.** Tie-back sheet pile wall design plan view (FDOT Bridge plans).

The retaining seawall system is modular with male-female interlocks. Reinforcing in the bulkhead cap also exclusively utilized GFRP reinforcing. High strength concrete Class V-Special 41.4 MPa (6 Ksi) was specified for the fabrication of the PC sheet piles, while Class IV Concrete 37.9 MPa (5.5 Ksi) was used in the cast-in-place sections of the bulkhead cap.

During PC sheet pile installation activities in Phase II, a hard layer of weathered limestone was encountered. The unexpected soil conditions forced the construction crew to attempt several excavation methods. Among the others, the contractor tried jetting, hydraulic hammer driving, prepunching, and preboring. Ultimately, the contractor had to resort to trenching. For an easier installation, the PC sheet piles were redesigned to accomplish a shorter elevation of approximately 3 m (10 ft), which required a tie-back sheet pile wall design (Fig. 13).

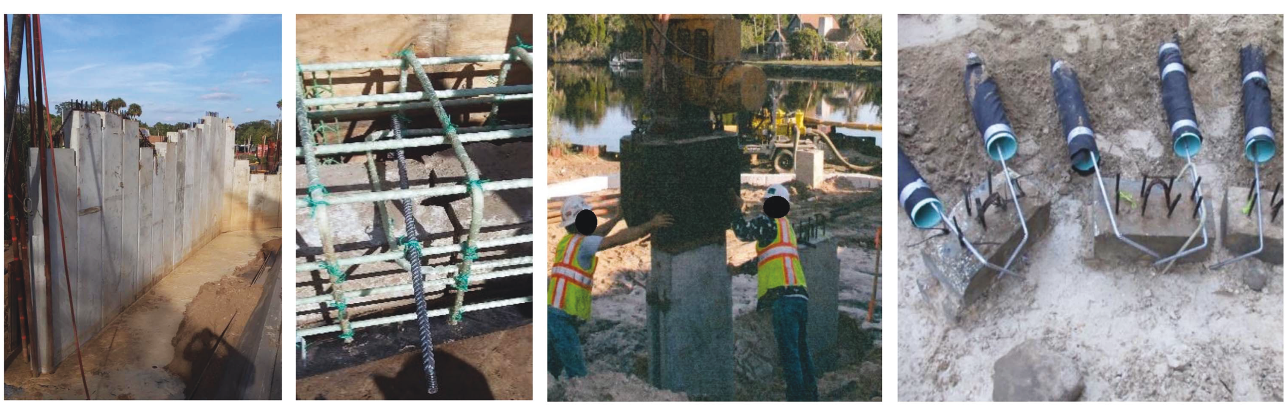
Fig. 13 shows the west end sheet pile wall design. For symmetry, the same design has been adopted on the east end. In doing so, the sheet pile structure was modified from a cantilevered [Fig. 14(a)] to an anchored design [Fig. 14(b)]. The new design reimplemented part of the sheet piles' cutoffs as deadman anchors, which are indicated in Fig. 13 and shown in Fig. 14(c).

The deadmen carry a substantial portion of the wall loading through threaded SS anchor rods [indicated in Fig. 13 and shown in Fig. 14(d)], installed within a 102-mm (4-in.) diameter PVC perforated pipe (for construction operation protection and drainage reasons). The connectors used were no. 5 SS rods ASTM A955, Grade 75, with length ranging from 8.2 m (27 ft) to 12.5 m (41 ft). The bar ends were threaded for coupling at one end and had a 25.4-m (10-in.) tail bend at 90° at the other. After installation and tightening, tie-backs and deadman anchors were backfilled with A-3 granular material as per AASHTO M-145 (AASHTO 2008).

### Sheet Pile Interlocking

Given that the redesigned sheet pile wall was tipped to a shallower elevation, most of the sheet piles of Phase II needed to be saw-cut after installation [Fig. 15(a)]. GFRP bars were easily, safely, and rapidly cut with light cutting equipment. The relative ease of drilling through GFRP-RC elements has been revealed to be safer to operators and faster than steel-RC elements.

After bulkhead cap pour, due to a missing V-groove contraction joint along the seawall bulkhead cap, a distinct crack was noticed



(a) (b) (c) (d)

**Fig. 14.** (a) Cantilevered sheet pile wall; (b) anchored sheet pile wall through SS rods; (c) deadman anchor installation; and (d) SS rods anchored to deadmen prior to casting of the deadmen cap. (Images by Thomas Cadenazzi.)

(a) (b) (c)

**Fig. 15.** (a) RC-GFRP sheet piles saw-cut; (b) extended crack at corner cap; and (c) RC-GFRP bulkhead cap remedial action. (Images by Thomas Cadenazzi.)

several weeks after casting. Operators did eventually saw-cut the crack in order to control it, but an extended crack manifested several days after, as shown in Fig. 15(b). The solution involved removing the entire portion of the bulkhead cap at the sheet pile interlock line, doweling existing bars, and recasting [Fig. 15(c)]. The remedial action was accomplished with no delays as the material was cut easily with hand saws or light grinding and drilling equipment. This avoided damage to drill bits and did not delay the work progress. In addition, given the noncorrosive nature of GFRP, there was no further need of additional superficial concrete patching.

**Fig. 16.** GFRP no. 6 stirrups placement in diaphragm sections. (Image by Thomas Cadenazzi.)

### *Diaphragms*

To reaffirm quality control importance, during construction it was noticed that the no. 6 stirrups in the diaphragm sections were fabricated too wide: they were fabricated between 210 mm (8.25 in.) to 222 mm (8.75 in.) wide, while the plans required 190 mm (7.5 in.). Therefore, the stirrups were placed skewed (Fig. 16), since the GFRP cannot be field bent and reordering new stirrups with such a long lead time would have been punitive for the contractor. This solution did not require any additional validation because the acting reinforcement was constituted by the vertical legs.

## Productivity

During Phase II of construction, in the process of installing GFRP reinforcement cages for the bent cap, it was possible to quantify productivity. Table 1 presents a reinforcement list for the bent cap, in terms of the length, number, and unit weight of each bar. The collecting activity duration data for the bent cap element accounted for the handling and cage assembling activities only, as the cages were preassembled in a yard and set in place through the aid of a crawler crane.

The productivity was calculated in terms of length of reinforcement installed per work hour. In total, for each bent in Phase II of construction, 688 m (2,258 ft) of bars were installed by five laborers in 4.5 h. Productivity is then calculated in terms of linear feet, per hour, per laborer (ft/h/laborer) as

**Table 1.** Pile bent cap reinforcement list

| Size | Length (m) | Numbers | Total length (m) |
|---|---|---|---|
| #4 | 1.91 | 2 | 3.81 |
| #4 | 1.17 | 2 | 2.34 |
| #4 | 1.47 | 3 | 4.42 |
| #5 | 2.06 | 164 | 337.41 |
| #5 | 1.80 | 18 | 32.46 |
| #5 | 1.85 | 16 | 29.67 |
| #5 | 1.70 | 20 | 34.04 |
| #8 | 8.41 | 22 | 184.96 |
| #8 | 4.04 | 12 | 48.46 |
| #8 | 1.78 | 6 | 10.67 |
| Total | | | 688 |

$$
\begin{aligned}
&\text{Productivity}_{\text{RC-GFRP Bent Cap}} \\
&\quad = \frac{\text{Length TOT bars}}{\text{Laborers} \times \text{Hours}} = \frac{688}{5 \times 4.5} \\
&\quad = 30.6\ \text{m/h/laborer}(100.4\ \text{ft/h/laborer}) \qquad (1)
\end{aligned}
$$

Productivity of on-site placement of steel by weight is usually estimated as 50 kg/h/laborer (100 lb/h/laborer) (Forsythe 2014). Given the mass per unit length of each steel bar size, it has been possible to calculate similar productivity for a similar steel reinforced cage, which resulted in 25.5 m/h/laborer (83.6 ft/h/laborer).

Assuming the same reinforcement quantity for steel as for FRP, this study revealed that in deploying FRP there is approximately a gain of 20% in terms of time. The same installation time saving has been noticed for the Phase II bulkhead cap, where, based on the same assumptions, the gain reached a value of 23.6%. Similarly, assuming that the amount of GFRP reinforcement can be 20%–25% more than the corresponding steel reinforcement for the same capacity, it is concluded that there is no cost of installation penalty when using GFRP rebars.

Given HRB construction sequences, the crane used to set GFRP cages on bent caps was the same 250-ton crane used to drive piles. The assumptions made on productivity do not consider the fact that a smaller crane size could have been deployed, resulting in possible cost savings.

Poor site organization and lack of experience in deploying suitable equipment can affect optimization of FRP productivity, the same way that lack of labor experience can cause difficulties in FRP handling. For the case study, the labor force was properly trained.

## Conclusions

The Halls River Bridge revealed to be a true laboratory for the implementation of new construction materials, thus becoming a case study providing guidance to contractors intending to work with FRP-RC/PC structures. Through this case study, this paper has identified outcomes related to the procurement, testing, constructability, fabrication issues, and construction methods of FRP-RC/PC bridge elements that validated both CFRP-PC and GFRP-RC technology. The case study revealed that FRP materials may require a long lead time. The project scheduling should also consider that preordering materials may not be possible for federally funded projects. Furthermore, when unforeseen conditions or damage occur, there must be a sufficient reserve of FRP material available or there must be provision for additional time to supply materials. Generally, preapproving material suppliers will minimize delays while ordering FRP materials.

Additionally, the quantity, frequency, and responsibilities of sampling and testing for QC and independent verification should be clearly identified in the contract documents and accounted for when the contractor orders FRP material.

Regarding pile splicing, no. 10 SS bars were used as dowels. This is because there was a lack of responsiveness in the supply chain of #6 CFRP solid bars and a lack of test data for nonprestressed applications of stranded CFRP. On the other hand, the SS rebar industry was able to intervene and provide a rapid solution due to synergy with traditional construction practices. However, stranded CFRP dowels are currently undergoing an experimental campaign for validation, to be certified and available for future uses in lieu of traditional or SS rebar dowels.

As for the installation of sheet piles, the trenching method was utilized as the last resort after trying other methods. Even though the trenching method was the only means by which it was possible to install the sheet piles to the design tip elevation, the method was revealed to be expensive, and the resulting lateral resistance may be difficult to quantify, especially in nongranular soils.

Ultimately, this study demonstrates the on-site improved productivity implications of GFRP-RC elements through direct monitoring of GFRP reinforcement installed per work hour. Based on such results, this paper may help contractors in estimating in advance the productivity of FRP elements, while bidding on FRP projects. In doing so, future contractors can maximize crew efficiency and, ultimately, profits.

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