Inspection and Monitoring of Fabrication and Construction for the West Halls River Road Bridge Replacement

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Submitted to:

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TASK 1 Deliverable: End of Construction Report
(Part A – Fabrication and Construction)

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CHAPTER 1
ABSTRACT

Corrosion is a major concern for bridge design and maintenance in Florida. Due to the aggressive environments in Florida, bridge maintenance costs can become excessive, leading the Florida Department of Transportation (FDOT) to research new technology and designs that can reduce bridge life-cycle costs. Implementing the use of fiber reinforced polymer (FRP) bars and Hybrid Composite Beams (HCB) on West Halls River Bridge will allow FDOT to determine the viability of using these new materials in future bridge construction projects. The West Halls River Bridge replacement project will be the first bridge in Florida to use HCB beams, and if successful may lead to increased service life of future bridges.

This report was prepared at the end of construction of the new West Halls River Bridge. It describes the research team's efforts on Task 1 of the research project, objectives for which are explained in the following chapter, Chapter 2. Then, in Chapter 3, the activities for Research Task 1 are outlined. Chapter 4 is a technical summary of the observations made during fabrication and construction of the bridge. It is organized by the bridge components, which include:

a. CFCC Prestressed Concrete Piles
b. Temporary Sheet Piles
c. Prestressed Concrete Sheet Piles
d. GFRP Reinforced Pier Caps
e. Hybrid Composite Beams
f. GFRP Reinforced Diaphragms
g. Casting of RCA and RAP Gravity Walls
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CHAPTER 2
RESEARCH OBJECTIVES

Because of the aggressive coastal environment in Florida, corrosion damage leads to reduced bridge service life and is a main concern for bridge maintenance. Maintenance costs account for a large portion of the capital expenditures for FDOT and have led to research, design, and implementation of less corrosive materials than traditional steel.

The existing West Halls River Bridge, located in Homosassa Springs (Citrus County), Florida, is a multi-span reinforced concrete slab bridge and was marked for replacement. The replacement bridge includes Carbon Fiber Composite Cable (CFCC) prestressed concrete piles, CFCC-prestressed sheet pile walls, and Glass Fiber Reinforced Polymer (GFRP) reinforced pier caps and deck slab. Another experimental aspect of the bridge is the use of Hybrid Composite Beams, which incorporate a concrete compression arch, galvanized steel strands for tension reinforcement, and an FRP shell. According to FDOT, the addition of these experimental aspects is anticipated to increase the bridge service life to 100 years and decrease the annual maintenance costs.

The main objective of the research project was to determine the constructability and performance of the experimental aspects of the bridge. Fabrication and construction were observed in the manufacturing facility, precast yard, and construction site.

In addition, test blocks were cast on top of the sheet pile walls, for later extraction so that durability testing on the reinforcement could be performed after construction, to monitor deterioration. Durability tests will continue for several months after construction. These test results are being submitted to FDOT in separate reports prepared by the research subcontractor, University of Miami.

CHAPTER 3
RESEARCH TASK 1

The main purpose of Task 1 was to observe, document, and summarize the fabrication and construction methods used to build the West Halls River Bridge. Fabrication of precast elements was observed and documented at the precast yard, and construction of the bridges was observed and documented at the bridge construction site. The following subtasks were performed:

A. Collected information and observed and documented by digital photographs, videos, and written description the precast elements fabrication. These activities included but were not limited to:
   a. collect material data for concrete, reinforcing bars, and prestressing strand;
   b. observe and evaluate Hybrid Composite Beam (HCB) fabrication including the fiberglass tub shell casting and internal concrete arch casting;
   c. observe and evaluate CFCC/CFRP prestressed pile fabrication with emphasis on prestressing operation;
   d. observe and evaluate CFRP prestressed sheet pile fabrication;
   e. obtain copies of plant QC/QA documents from the plant that confirms dimensional tolerances, concrete voids, and final product quality
B. Collected information and observed and documented by digital photographs, videos, and written description the erection, casting, and installation of new material construction. These activities included but were not limited to:
   a. collect material data for concrete and reinforcing bars;
   b. observe CFCC/CFRP prestressed pile driving visually and with PDA and/or EDC equipment;
   c. observe installation of CFRP prestressed sheet piles;
   d. observe GFRP pile bent cap reinforcement handling, placement and casting;
   e. observe HCB erection and placement on supports;
   f. evaluate HCB vertical and lateral dead load deflections before and during deck casting;
   g. observe GFRP concrete deck reinforcement handling, placement and casting;
   h. observe casting of gravity walls with RCA and RAP;
   i. obtain test blocks cast by contractor

CHAPTER 4
TECHNICAL SUMMARY

4.1 Introduction

The existing Halls River Bridge, built in 1954, had a bridge deck width of 30 ft that included two (2) 12-ft-wide lanes. The replacement bridge cross section consists of six (6) CFRP prestressed concrete piles per intermediate bent, GFRP reinforced pier caps, nine (9) hybrid composite beams per span, and a GFRP-reinforced deck with a width of 57 ft 9 ¾ in., as shown in **Figure 1**. The two (2) 12-ft-wide travel lanes with 8-ft-wide shoulders are separated from a 5-ft-wide sidewalk by a concrete barrier. The bridge is 185 ft 10 in. long and has five (5) spans, each 37 ft 2 in. long. From the centerline of the middle beam, the northern beams are spaced at 6 ft 7 ⅝ in., and the southern beams are spaced at 6 ft 4 ⅝ in. The bridge deck is reinforced with GFRP No. 6 bars on top and bottom spaced at 4 ½ in. longitudinally, GFRP No. 4 bars on top and No. 6 bars on bottom at the end bents, and GFRP No. 5 bars on top at the intermediate bents.

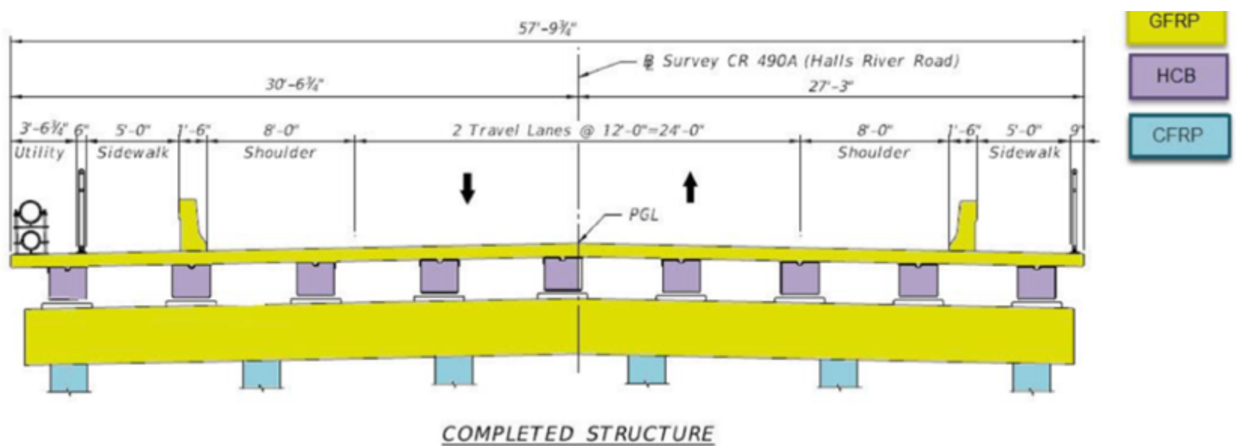

Figure 1: Profile view of Halls River Bridge replacement (from FDOT (Surarez, Siddiqui, and Pelman, 2016))

The bridge replacement occurred in stages to allow for continual traffic flow. The northern half (westbound traffic lane) of the existing bridge was demolished, and both lanes of traffic were moved to the southern half of the bridge (Phase II-Stage 1). After construction of the northern section of the new bridge (Phase II-Stage 2), consisting of four (4) beams per span and a deck width of 26 ft, traffic was shifted to the newly-constructed structure, and the southern half (eastbound traffic lane) of the existing bridge was demolished (Phase III-Stage 1). The southern section of the new bridge includes five (5) beams per span with a deck width of 31 ft 9 ¾ in. During the final phase of construction, traffic was relocated to the newly-constructed southern section of the bridge (Phase III-Stage 2). After completion of the post-installed traffic barrier and pedestrian/bicycle railing on the northern section of the new structure, traffic was shifted to the final position.

The original contractor for Halls River Bridge was Astaldi Construction Corporation, but the contractor defaulted in March 2019 during construction of the southern section of the new structure (Phase III-Stage 1). In April 2019, Watson Civil Construction assumed responsibility for completing the construction of the bridge. According to an FDOT bridge inspector, the transition between contractors was straightforward and amicable, with Astaldi manager working with Watson Civil Construction to reduce further construction delays.

4.2 CFCC Prestressed Concrete Piles

During Phase II of the construction project, eighteen 18-inch square CFCC-prestressed concrete piles were cast at Gate Precast Company in Jacksonville, FL. Typically, there is a three- to five-month lead time (depending on the method of shipment) to obtain CFCC materials from the supplier, Tokyo Rope. The contractor elected to cast the production piles to the test pile lengths, to avoid any delays. The production pile lengths were as follows: 55 ft for end bent 1; 66 ft for bents 2 and 3; 70 ft for bents 4 and 5; and 64 ft for end bent 6. During storage, the CFCC piles were supported on wooden cribbing, as shown in **Figure 2**, on four points.

The first six (6) 18-inch square CFCC piles that were fabricated were overstressed due to human error, on December 14, 2016. Overstressing resulted in an effective pile prestress of 1258 psi, more than the minimum target value of 1000 psi. With an initial strand stress of 189.1 ksi, final effective strand stress of 177.7 ksi, and a net in-service pile prestress of 1182 psi, the piles were within boundary conditions of 2018 IDS-22600 interaction diagram for 0.6-in.-diameter CFCC 18-in. square piles.

On February 15, 2017, the first test pile (R3) was driven at bent 3. The 27 ft x 6 ft pile driving template was set into position over the bent. Using an APE Model 50 Top Drive Auger, 22-in.-diameter starter holes were drilled to an elevation of -27 ft. The auger was lifted using the 230-ton Manitowoc 888 crawler crane (**Figure 3**). When drilling at bent 5, the auger encountered refusal, requiring the use of an H-pile to punch through the limerock to reach the required elevation.

Figure 2: CFCC piles stored at Halls River Bridge

Figure 3: Crane lifting 22-inch auger; Tip of 22-inch auger

After the preforming, the piles were lifted using steel cable wrapped around two points on the CFCC pile (**Figure 4**). As the pile was lifted, the crane operator slowly loosened one cable and positioned the vertical pile into the preformed hole (**Figure 5**). Pile Driving Analyzer (PDA) sensors or strain transducers were attached to predrilled holes at the top of the pile to monitor pile capacity, blows per inch, and stresses in the pile. The PDA operator from Foundation and Geotechnical Engineering (FGE), LLC, oversaw the installation of the transducers; the transducers were tightened if the PDA curve data was inconsistent after one blow.

Figure 4: Cables attached to CFCC piles

Figure 5: CFCC piles lifted by crane

After ensuring the transducers were correctly installed, pile driving commenced using an APE D30-42 single-acting diesel hammer at the lowest fuel setting. The diesel hammer has variable throttle control that allows for ramp-up measure or an increase in power and a rated energy of 74,419 ft-lbs. As the piles are driven, the fuel settings are increased as allowed by the blows per inch and pile stresses. If the maximum allowable of 20 blows/in. was reached, pile driving was stopped, and the fuel settings were reduced. When the pile reached practical refusal, pile driving was terminated.

While driving piles, a seismograph was used to monitor the vibrations of the existing bridge; the seismograph sensor was placed in a sandbag on top of the pile cap of the existing bridge. The maximum allowable vibration of the bridge was 0.5 in./sec; this limit was reached when driving pile R3 at bent 3. While driving piles, the turbidity of the mixing zone of Halls River was monitored; 41 NTU was the maximum allowable turbidity. Pile driving at bent 5 was delayed due to the sighting of manatees. If manatees were sighted within 50 ft of the bridge, pile driving was halted for 30 minutes from the last sighting.

FGE performed 100% dynamic monitoring of pile driving for piles on bents 3 and 5. Using a phi factor of 0.75, piles on bent 3 and bent 5 achieved the nominal bearing capacity of 385 k. Pile capacities for bent 3 were the results of set checks performed on the piles after driving. Pile driving was stopped within 12 in. of cut-off on pile 3 at bent 3 due to failure to reach required resistance. In this case, a set check was performed to determine pile acceptance. Pile 1 on bent 5 achieved refusal at the end of initial drive (EOID) with a capacity of 888 k. Piles 2 and 3 on bent 5 achieved 2 ft of bearing at the end of initial drive. **Table 1** provides a summary of the pile driving records for piles on bents 3 and 5.

Table 1: Summary of pile driving records for bents 3 and 5

Bent	Pile No.	Date Driven	CAPWAP Pile Capacity (k)	Pile Capacity (k)	Pile Tip Elevation (ft)	No. Blows	Date Set Check	No. Blows Set Check	Amount Set
3	1	2/17/17	388	411	-53.00	591	2/17/17	8	2”
3	2	2/17/17	390	413	-58.75	921	2/17/17	8	2”
3	3*	2/15/17	600	604	-57.90	1878	2/17/17	9	1”
5	1*	2/21/17	653	888	-31.2	754	n/a	n/a	n/a
5	2	2/23/17	n/a	595	-35.8	458	n/a	n/a	n/a
5	3	2/22/17	n/a	753	-31.8	401	n/a	n/a	n/a
*Test Pile									

On March 22, 2017, the piles at bent 2 were driven on fuel setting 2 to within a foot of pile cutoff, without reaching pile bearing capacity. Astaldi had proposed to use pile cutoffs as splice piles for bent 2, due to the lead time required to order CFCC material and produce additional piles, as stated in RFI 0030. Pile R5 (bent 6 pile 3), pile R17 (bent 6 pile 1), and pile R2 (bent 4 pile 2) were selected due to the length and condition of the cutoffs. Although Pile R2 did not have any excessive spalling, there were cracks within the CFCC strands on the cutoff end of the pile, and a hairline crack extended approximately 2 ft longitudinally along the pile. The pile cutoffs were stored on-site but were not treated as usable piles. Some piles were stacked on each other, and the piles were used as a platform for boat storage, as shown in **Figure 6**. The use of cutoff piles was later rejected as a feasible option, and splice piles were fabricated at Gate Precast Company for Bent 2.

Figure 6: Stored pile cutoffs

Extensive cracking between the CFCC strands was observed on the cutoff faces of piles R1 (bent 5 pile 1), R13 (bent 5 pile 2), and R2 (bent 4 pile 2). On pile R13, cracks were present between the strands on two sides of the pile end and tearing of the CFCC strand was visible, as shown in **Figure 7**. A longitudinal crack, approximately 2 ft in length, extended from the cutoff end. The majority of the CFCC strands on the cutoff face of pile R13 were recessed, as shown in **Figure 8**. Pile R1 had cracking between the CFCC strands on one side of the pile end, and all strands were recessed (**Figure 9**). The pile also had a longitudinal crack approximately 1.5 ft from the cutoff end. Pile R2 had slight cracking between CFCC strands on one side of the pile end, as shown in **Figure 10**.

Figure 7: Cracking on Pile R13 cutoff

Figure 8: Pile R13 cutoff recessing of strands

Figure 9: Pile R1 cutoff cracking between strands

Figure 10: Pile R2 cutoff cracking between strands

During the cutoff of piles 1, 2, and 3 at bent 5, the pile heads were damaged; spalling of pile 2 at bent 5 is shown in **Figure 11**. In all three piles, the spall lengths did not extend beyond the area that will be covered by the pile cap, and therefore, Astaldi proposed to remove loose material from the pile heads and pour the pile cap as planned. Excessive cracking was noted on pile 2 at bent 5 after pile cutoff procedures occurred. A 23.25-in. crack was present on the west face of the pile and a 28.0-in. crack on the east face. The crack also extended through the CFCC strands on the top of the pile. The cracks were 0.011 in. wide at the top of the pile and decreased to 0.001 in. wide at the end of the crack.

Figure 11: Spalling of Pile 2 Bent 5

After pile cutoff on bent 6, cracking was evident on pile 1. A crack extended from the north face of the pile, through the CFCC strands, and continued down the south face of the pile. Recessing of the CFCC strands was also evident on piles 1 and 3 at bent 6. **Figures 12** and **13** show the pile cracking and the recessing of the strands on pile 1 at bent 6.

Foundation & Geotechnical Engineering, LLC (FGE) performed PDA testing on piles at bent 2 during driving and set checks. On April 18, 2017, a set check was performed on the three piles at bent 2 to determine if the piles had gained the required nominal bearing capacity of 444 k. Due to the water level, the piles were only driven 3 ft, from elevation 59 ft 6 in. to 62 ft 6 in. All piles at bent 2 failed to reach capacity and required splicing. Astaldi decided against using pile cutoffs as drivable piles and therefore contracted Gate Precast Company to fabricate splice piles for bent 2.

Figure 12: Cracking of Pile 1 at Bent 6

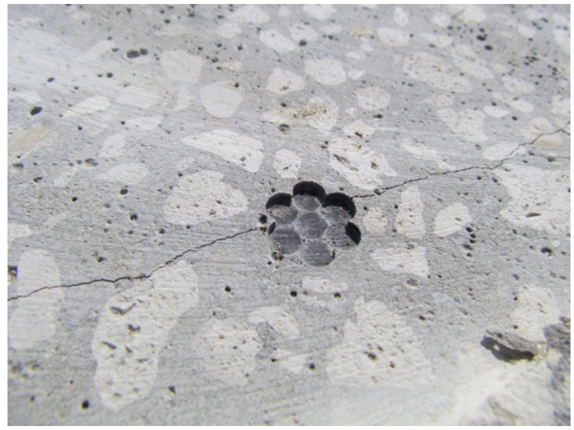

Figure 13: Recessing of CFCC strand at Pile 1 Bent 6

A 14x90 exploratory H-pile was driven on April 24, 2017, to determine the location of the rock layer required to reach capacity at bent 2. The pile was driven to the east of bent 2 and halfway between bent 2 pile 3 and the existing bridge. The location was chosen to provide more data for Phase III pile lengths at bent 2. Nominal bearing capacity was not reached until a depth of 150 ft. Based on the collected data, FGE suggested an authorized pile splice length of 84 ft for all three piles at bent 2. Per RFI 0037, Astaldi's request to divide the required splice length of 84 ft into two segments, 30 ft and 54 ft, was approved. Astaldi planned to use pile splice detail Index D22601 sheet 1/1. Per RFI 0035, #6 CFRP bars or #10 stainless steel dowels could be substituted in the Developmental Design Standard Index D22601 and D22618.

Piles 2 and 3 at bent 1 were cut on April 19 and 20, 2017 (**Figures 14** and **15**). The piles were prepared for cutting by removing the pile driving form and surveying and marking a pile cutoff line. Using a hand-held concrete saw, the piles were cut one side at a time, slowly moving around the pile. A loop was positioned over the pile head, and as the pile was cut free it was lifted using the crane.

Figure 14: Cutting Bent 1 Pile 1

Figure 15: Cutting Bent 1 Pile 2

Bent 1 pile 2 exhibited cracking between the CFCC strands from the west to the east on the northern row of strands, and a few of the strands recessed slightly. The north-western edge of the pile spalled, and one of the CFCC strands tore, as shown in **Figures 16-18**. In bent 1 pile 3, one strand recessed slightly, as shown in **Figure 19**.

Figure 16: Spalling of Bent 1 Pile 2

Figure 17: Cracking between CFCC (Bent 1 Pile 2)

On June 13-15, 2017, the six (6) 18-in. CFCC splice piles were fabricated at Gate Precast Company in Jacksonville, FL. Unlike originally planned, the splice pile length of 84 ft was divided into two (2) equal segments of 42 ft. The pile beds were scraped, cleaned, and sprayed with a releasing agent to prepare for pile fabrication. Plywood headers and footers were placed in the bed and secured with wedges.

Figure 18: Tearing of CFCC (Bent 1 Pile 2)

Figure 19: Recessing of CFCC (Bent 1 Pile 3)

CFCC spirals were individually tagged and labelled (**Figure 20**) by the supplier, Tokyo Rope; the labels were kept by the Quality Control department to document the spirals used for pile production. **Figure 21** shows the spirals in storage before installation. The CFCC spirals were placed in the bed (**Figure 22**) and temporarily secured near the plywood header using plastic coated tie wire. The spirals were secured near the headers to prevent abrasion between the spirals and the CFCC strands while the strands were being pulled into position. After securing the spirals, the CFCC strand was pulled from a large spool by two workers. The workers fed the strand through the plywood headers and spirals. When the two workers were further down the bed, other workers assisted in pulling the strand and ensured the strand was not being damaged by the spirals or the headers. When the strand was pulled the length of the bed, it was pulled taught and cut with a skill saw (**Figure 23**). Workers wore gloves when pulling the strands and also wore masks when cutting them.

After the 12 CFCC strands were pulled and cut, couplers were installed between the CFCC strands and the steel strands. The couplers were installed using the same process as for the sheet pile and square pile fabrication. A spiral pre-cut mesh was taped to the end of the CFCC strand, tightly spiraled around the strand, then taped on its end. A stainless-steel mesh was formed over the spiral mesh and taped (**Figure 24**); the excess mesh was cut with a side grinder.

Figure 20: Label on CFCC spirals

Figure 21: CFCC spirals

Figure 22: Secured CFCC spirals

Figure 23: Cutting of CFCC strand

Figure 24: Stainless steel mesh on CFCC strand

After spraying the coupler sleeve with molybdenum spray, the sleeve and the silicone wedge O-ring were slid over the mesh-wrapped CFCC strand. Four (4) wedges (**Figure 25**) were inserted into the O-ring, and then the wedges were set into the coupler sleeve using a pneumatic jack. The other half of the coupler was slid over the steel strand, a wedge was attached to the strand, and the coupler was slid back over the wedge. To complete the installation, the sleeve on the CFCC strand was screwed into the coupler on the steel strand (**Figure 26**). **Figure 27** shows several installed couplers lying in the precast bed.

Figure 25: Wedge seating CFCC strand

Figure 26: CFCC-steel coupler installation

Figure 27: Aligned couplers

The coupler was fully tightened and then backed off so it would not be damaged during tensioning. After coupler assembly, the strands were positioned to align the couplers in the desired locations. The steel strands were anchored to the bulkheads, at the ends of the precast bed, using standard open grips. The 12 strands were initially stressed to 8,000 lb, in the sequence shown in **Figure 28**. After all strands were initialed, the coupler locations were marked (**Figure 29**) so that any strand slippage due to stressing could be observed and measured. **Figure 30** is a top view of the casting bed, showing the couplers' positions after initial stressing.

The CFCC strands were then pulled to the target force of 34,310 lb, in the sequence shown in **Figure 31**.

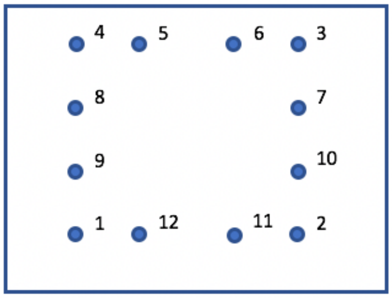

Figure 28: CFCC strand initialed sequence

Figure 29: Coupler marked after initial stressing

Figure 30: Coupler positions after initial stressing

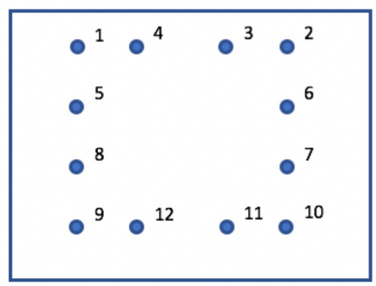

Figure 31: CFCC strand stressing sequence

After pulling the strand, the total strand elongation was measured. Projected strand elongation was 34 ½ in., with a tolerance of 1 ¾ in. The first six (6) strand elongations as recorded are shown in **Table 2**; the recorded elongations were within tolerance.

During stressing, CFCC strand No. 11 broke in the middle of the bed. After breaking and retracting, the CFCC strand broke at multiple header locations along the bed (**Figures 32** and **33**).

Table 2: Measured CFCC strand elongations

Strand No.	Elongation (in)
1	34 ½
2	34
3	34 ¾
4	34 ¾
5	34 ⅝
6	34 ½

Figure 32: Broken CFCC strand

Figure 33: Broken CFCC strand

Workers removed the broken CFCC strand, cut the coupler off each end, pulled a new CFCC strand, and installed the couplers on the new strand. After stressing strand No. 11 again, the strands were left to sit overnight, to ensure strand stability.

For steel strands, it is not uncommon for one wire in the strand to snap during stressing; if one wire breaks, the workers wrap the broken wire around the strand and continue fabrication. Unlike steel, when CFCC is damaged and one wire in the seven-wire strand breaks, the entire strand breaks. Some workers were wary when working with CFCC strands and stated the material is temperamental and requires careful handling.

After allowing the stressed CFCC strands to sit overnight, eight (8) No. 10 Carpenter 2205 stainless steel bars were inserted into predrilled holes in the headers (**Figure 34**). A piece of rebar was used to prevent abrasion between the CFCC strands and the stainless-steel bars during bar installation. On three (3) of the splice piles, the 13-ft stainless-steel bars were positioned with 10 ft 6 in. embedded and 2 ft 6 in. projected from the piles. On the remaining three (3) splice piles, the 15-ft stainless-steel bars were embedded 10 ft 6 in. and projected 4 ft 6 in. The stainless-steel bars were positioned and secured using plastic coated wire. After installing the stainless-steel bars, the 0.2- in.-diameter CFCC spirals were pulled to the correct pitch and tied using a plastic tie gun, as shown in **Figures 35 – 37**.

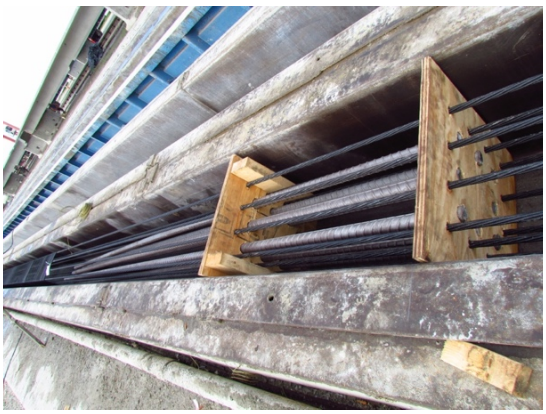

Figure 34: Stainless steel bar installation

Figure 35: Tying CFCC spirals

Figure 36: Tied CFCC spirals

Figure 37: Wires to secure stainless steel bars

Headers and footers were stabilized with 2x4 blocks hot glued to the forms. On the three (3) splice piles with 13-ft stainless-steel No. 10 bars at the footers, eight (8) 2-in.-diameter metal corrugated tubes and eight (8) No. 9 stainless steel bars were installed at the header of the piles (**Figure 38**). The metal corrugated tubes were 4 ft 8 in. long, and the No. 9 stainless steel bars were 7 ft 6 in. long. Small wooden blocks were screwed into the plywood headers (**Figure 39**) where the metal corrugated tubes were located to prevent concrete from entering the tubes during casting. Positioning and attaching the blocks was difficult for the workers because of interference from the stressed strands. As shown in Figure 39, the blocks were not leveled, which resulted in misaligned tubing; this will be discussed in more detail later.

Figure 38: Installation of metal tubes

Figure 39: Blocks to secure metal tubes and bars

Lifting rods (**Figure 40**) were installed to facilitate stripping the piles from the forms. After installing the stainless-steel bars, metal tubing, CFCC spirals, and lifting loops, the beds were cleaned with a blower to prepare for concrete casting. A self-consolidating concrete (SCC) mix was used to cast the splice piles (**Figure 41**). After casting, workers troweled the top to a smooth finish (**Figure 42**).

Upon arrival to the bridge site on Wednesday June 21, 2017, the splice piles were inspected. The position of the metal corrugated tubing was found to be out of tolerance. After a meeting between FDOT, Astaldi, and Gate Precast Company, two remedial options were submitted in

Request for Correction (RFC) 0002. In the first option, the centerline of the piles would be offset to maximize the location of the existing holes and the core drilled holes, as shown in **Figure 43**. The second option would maintain the existing centerline of the piles, and all holes would be oversized, as shown in **Figure 44**.

Ultimately, RFC 0002 was voided, and Gate Precast Company fabricated three (3) new splice piles on Saturday, June 24, 2017, and delivered them to Halls River Bridge on Tuesday, June 27, 2017 (**Figure 45**).

Figure 40: Lifting rod placement

Figure 41: Pouring of SC concrete

Figure 42: Troweling of SC concrete

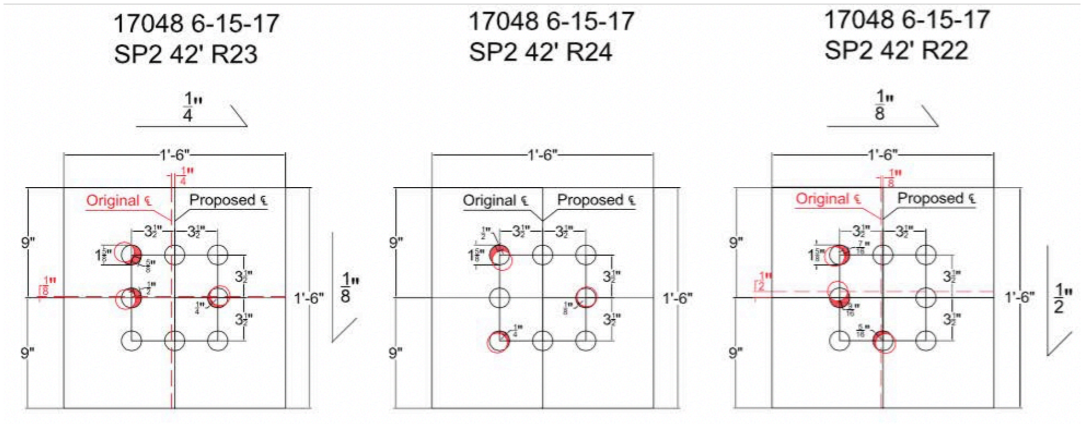

Figure 43: Option 1 correction of splice piles (Gate Precast Co.)

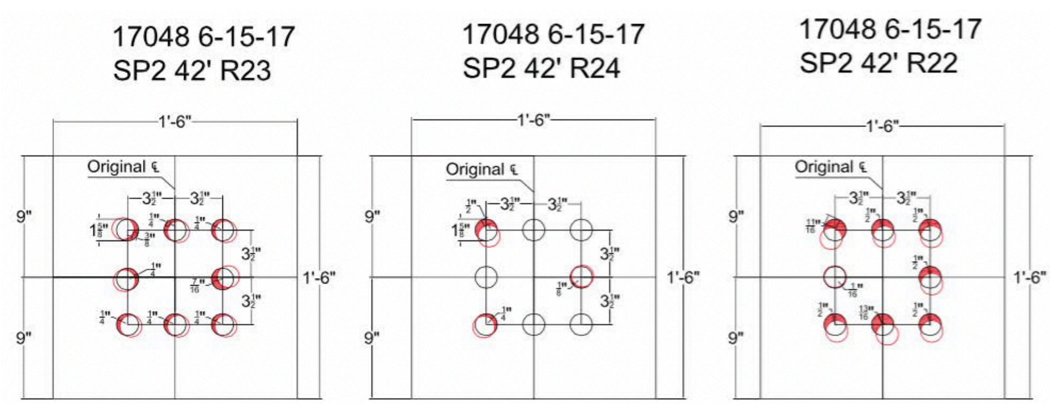

Figure 44: Option 2 correction of splice piles (Gate Precast Co.)

Figure 45: Marked splice pile at bridge site

On June 27, 2017, the first splice connections were made on piles 1, 2, and 3 at bent 2. To make the splice connection, foam was liquid nailed to the pile head and secured using duct tape. A plywood form was secured around the foam and tightened using Band-It bands (**Figure 46**). Pre-cut metal flashing was then liquid nailed to the plywood and secured using more Band-It bands (**Figure 47**). The holes in the form were filled with foam and caulked with liquid nail. Nuts were liquid nailed on the top of the pile to provide a gap for epoxy to flow between the two piles.

Using an air hose, the holes in the pile head were cleared of any material. The splice was then lowered and dry fit to ensure proper placement of the stainless-steel bars into the cored pile head (**Figure 48**). If needed, additional flashing was attached to achieve the required height of epoxy coverage.

Figure 46: Tightening of plywood form

Figure 47: Securing of metal flashing

Figure 48: Dry-fit of splice pile

A two-part Pilgrim EM CBC IV Epoxy was mixed for at least three (3) minutes before pouring the epoxy into the cored holes. The splice pile was lowered (**Figure 49**), and more epoxy was poured into the form until the epoxy height was at the top of the metal flashing. Approximately seven (7) gallons of epoxy were required per splice connection. After making the splice connections, the piles could not be driven for at least 24 hours to allow the epoxy to harden. On June 28, 2017, the three (3) splice piles were driven at bent 2 (**Figure 50**). Pile driving was scheduled for 2:00 p.m., but the sighting of a manatee at 1:57 p.m. caused the pile driving to be delayed until 2:30 p.m.

Figure 49: Placement of splice pile

Figure 50: First splice piles at Bent 2

Pile 3 was driven for 27 minutes at the lowest fuel setting to reduce the risk of overstressing the splice connection. Pile 2 was driven until the PDA data displayed overstressing of the splice connection and concern was raised about the slight deflection of the pile head. Pile 1 was driven until the PDA operator signaled that the pile reached an acceptable bearing capacity.

For bent 2, set-checks were planned for piles 1 and 2 to ensure adequate bearing capacity was reached. A second splice pile was required for pile 3; the splice connection was formed on June 29, 2017. After allowing the epoxy to set for 24 hours, pile 3 was scheduled to be driven on the morning of June 30, 2017. After driving the pile for a few minutes, the PDA operator stopped the pile driving due to overstressing of the splice connection.

The required Nominal Bearing Resistance (NBR) for Bent 2 is 384 k using a resistance factor of 0.75 and 100% dynamic instrumentation. The initial 66-ft piles were driven on March 22, 2017, with set checks performed on March 24, 2017, and re-drives on April 18, 2017. On June 28, 2017, the first splice on Piles 1, 2, and 3 was driven without reaching bearing capacity. During pile driving, the dynamic data for Piles 1 and 2 displayed reflections at the splice locations.

To lower the driving stresses, an APE 7-3 hydraulic hammer, with lower impact velocity, was used to drive the second splice pile of Pile 3 on July 10, 2017. Pile driving on Pile 3 was stopped at elevation -122.9 ft due to integrity issues indicated from the data. After geotechnical analysis, the consensus was that the integrity issues were due to unloading conditions which falsely indicate integrity issues. Unloading conditions are when the velocity becomes negative before 2L/C.

Set checks were performed on Piles 1 and 2, and Pile 3 was re-driven to an elevation of -134.3 ft on July 12, 2017. Stroke heights were carefully controlled to prevent pile damage. Piles 1 and 3 were close to reaching capacity, but Pile 2 did not reach sufficient capacity. Set checks were again performed on July 17, 2017; the dynamic data displayed upward reflections that indicate potential damage of the piles. The data indicated integrity issues, but a large upward reflection was evident at the toe of the piles suggesting the toe was still intact.

The maximum tension stresses achieved during driving were observed at the splice location for all three piles; the tension stresses for Pile 1, 2, and 3 were 350 psi, 810 psi, and 800 psi, respectively. The estimated CAPWAP capacity of Piles 1, 2, and 3 are 450 k, 360 k, and 525 k, respectively. FDOT performed an analysis on the piles assuming Pile 2 has an NBR of 360 k which equates to a factored load of 270 k using a resistance factor of 0.75. With the decreased capacity of Pile 2, the additional loading on Piles 1 and 3 were within the capacity of the piles. The axial loading on Pile 3 is an NBR of 394 k which equates to a factored load of 295 k. The decrease in the load of Pile 2 changed the moments and shears, but all values are within the original design limits. **Table 3**, provided by Foundation & Geotechnical Engineering, LLC, displays the length driven and the estimated ending capacity of the piles at Bent 2.

Table 3: Summary of pile driving at Bent 2 (Foundation & Geotechnical Engineering, LLC)

Pile	Length (feet)	Elevation Driven (feet) from	Elevation Driven (feet) to	Approx. Ending Capacity (kips)	Comments
APE D30-42 Open-Ended Diesel Hammer					
March 22, 2017 Initial Drive Piles 1, 2 and 3					
1	66	-9	-58.0	110	Pile did not achieve NBR. Integrity is 100%.
2	66	-8	-59.0	58	Pile did not achieve NBR. Integrity is 100%.
3	66	-10	-59.0	95	Pile did not achieve NBR. Integrity is 100%.
Set-check Pile 3 March 24, 2017					
3	66	-59.0	-59.3	161	Pile did not achieve NBR. Integrity is 100%.
April 18, 2017 Redrive Piles 1, 2 and 3					
1	66	-58.0	-62.0	255	Pile did not achieve NBR. Integrity is 100%.
2	66	-59.0	-62.0	150	Pile did not achieve NBR. Integrity is 100%.
3	66	-59.3	-62.0	185	Pile did not achieve NBR. Integrity is 100%.
June 28, 2017 Redrive Piles 1, 2 and 3					
1	66 + 42 = 108	-62.0	-99.9	328	Pile did not achieve NBR. Integrity is 100%.
2	66 + 42 = 108	-62.0	-96.8	128	Pile did not achieve NBR. Integrity is 100%.
3	66 + 42 = 108	-62.0	-101.9	139	Pile did not achieve NBR. Integrity is 100%.
APE 7-3 Hydrualic Hammer					
July 10, 2017 Redrive Pile 3					
3	66 + 42 + 42 = 150	-101.9	-122.9	264	Pile did not achieve NBR. Integrity is 100%.
July 12, 2017 Set-check Piles 1 and 2, Redrive Pile 3					
1	66 + 42 = 108	-99.9	-99.9	387	Pile achieved NBR. Integrity is 100%. Observed slight upwards reflection at splice.
2	66 + 42 = 108	-96.8	-97.2	280	Pile did not achieve NBR. Integrity is 100%. Observed upward reflection at splice. Stopped driving to evaluate reflection at splice.
3	66 + 42 + 42 = 150	-122.9	-134.3	398	Pile achieved NBR. Integrity is 100%.
July 17, 2017 Set-check Piles 1, 2 and 3					
1	66 + 42 = 108	-99.9	-100.0	450	Pile achieved NBR. Integrity is 83%. Observed upwards reflection 33 ft. above pile tip and slight upwards reflection at splice.
2	66 + 42 = 108	-97.2	-97.3	360	Pile did not achieve NBR. Integrity is 100%. Observed slight upwards reflection at splice. Did not want to continue driving due to reflection at splice.
3	66 + 42 + 42 = 150	-134.3	-134.3	525	Pile achieved NBR. Integrity is 100%. Observed slight upwards reflections at both splices.

Shown in **Table 4** is a summary of the pile data for Phase II, including the four splice piles required at Bent 2. A graphical representation of the pile data is shown in **Figure 51**, where the tip elevations, cut off elevations, and ground elevations (G.E.) of the piles are provided.

Table 4: Pile driving data – Phase II

Bent	Pile No.	MFR's Pile No.	Pile Length (ft)	Date Cast	Date Driven	Penetration Below Ground (ft)	Tip Elev.(ft)	Head Elev. (ft)	Cut Off Elev. (ft)	Ground Elev. (ft)	Length Driven (ft)
1	1	R6	55	12/14/16	3/24/17	49.70	-46.18	8.82	6.0	3.52	52.18
	2	R12	55	12/20/16	3/24/17	26.70	-23.18	31.82	6.2	3.52	29.38
	3	R18	55	12/24/16	3/24/17	26.70	-23.18	31.82	6.4	3.52	29.58
2	1	R4	66	12/14/16	3/22/17	N/A	-99.98	-33.98	N/A	N/A	N/A
	1	R-27 (Splice)	42	6/24/17	6/28/17	84.98	-33.98	8.02	6.1	-6.55	106.08
	2	R15	66	12/23/16	3/22/17	N/A	-97.18	-31.18	N/A	N/A	N/A
	2	R-25 (Splice)	42	6/24/17	6/28/17	82.18	-31.18	10.82	6.3	-6.55	103.48
	3	R10	66	12/20/16	3/22/17	N/A	-134.25	-68.25	N/A	N/A	N/A
	3	R26 (Splice)	42	6/28/17	6/28/17	N/A	-68.25	-26.25	N/A	N/A	N/A
	3	R (Splice)	42	6/28/17	7/12/17	119.25	-26.25	15.75	6.4	-6.55	140.65
3	1	R16	66	12/23/16	2/17/17	37.25	-52.25	13.75	6.2	-7.0	58.45
	2	R9	66	12/20/16	2/17/17	44.41	-59.41	6.59	6.2	-7.0	65.61
	3	R3	66	12/14/16	2/15/17	42.82	-57.82	8.18	6.6	-7.0	64.42
4	1	R8	70	12/20/16	3/16/17	44.85	-59.85	10.15	6.2	-5.35	66.05
	2	R2	70	12/14/16	3/16/17	14.85	-29.85	40.15	6.4	-5.35	36.25
	3	R14	70	12/23/16	3/16/17	16.78	-31.78	38.22	6.6	-5.35	38.38
5	1	R1	70	12/14/16	1/16/17	16.20	-31.20	38.80	6.0	-6.57	37.20
	2	R13	70	12/23/16	2/22/17	16.78	-31.78	38.22	6.3	-7.11	38.08
	3	R7	70	12/20/16	2/21/17	16.78	-31.78	38.22	6.5	-6.70	38.28
6	1	R17	64	12/23/16	3/20/17	30.71	-26.96	37.04	5.8	3.75	32.76
	2	R11	64	12/20/16	3/20/17	33.17	-29.42	34.58	6.0	3.75	35.42
	3	R5	64	12/14/17	3/20/17	26.42	-22.67	41.33	6.2	3.75	28.87

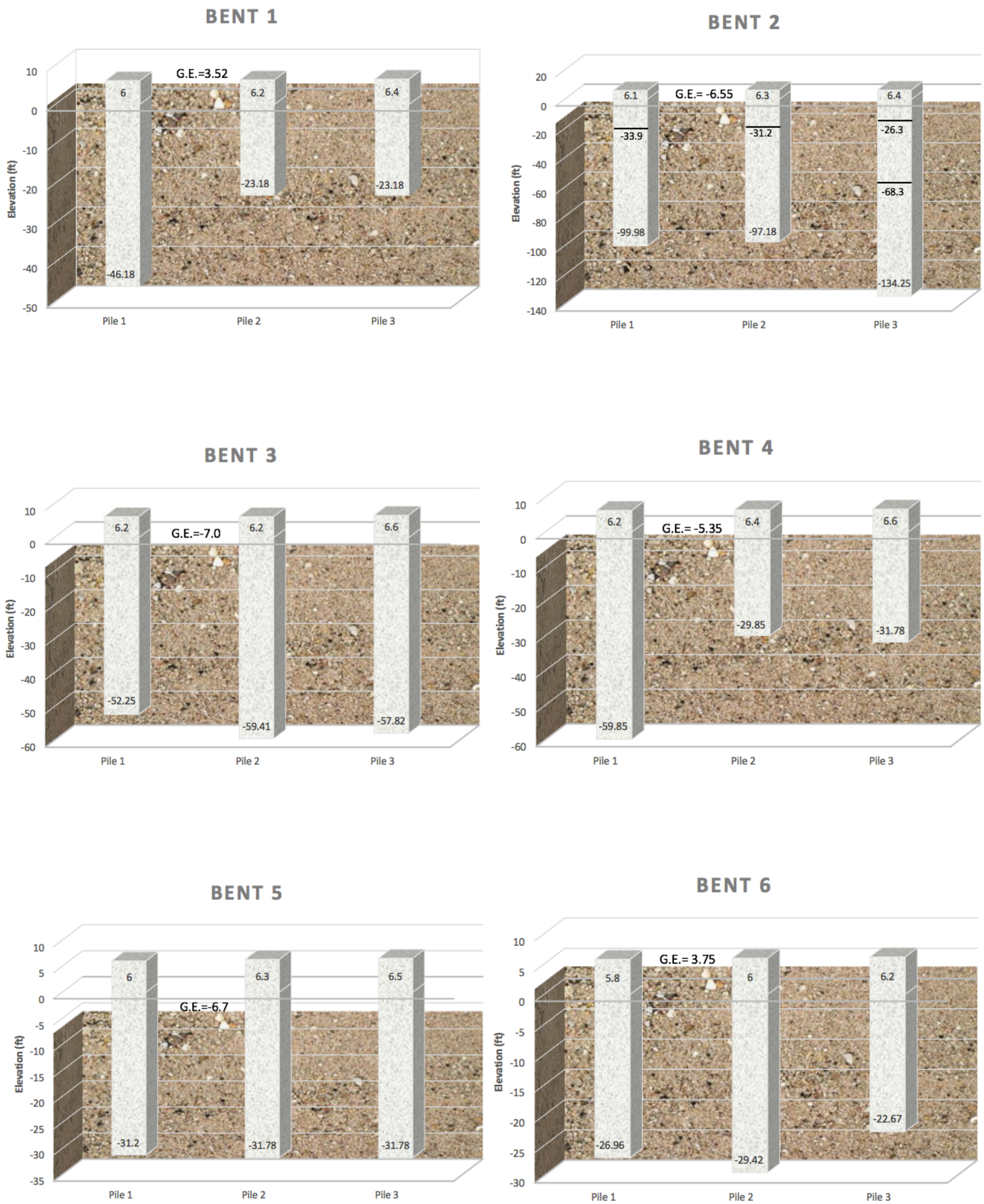

Figure 51: Phase II pile diagrams

Based on the pile driving data from Phase II, the recommended pile lengths for Phase III Bents 1, 2, 4, 5, and 6 are shown in **Table 5**. Soil borings were needed to determine the pile lengths required at Bents 2 and 3. The unknown soil conditions presented a challenge for Astaldi in ordering CFCC spirals from Tokyo Rope; CFCC spirals have a three-month lead time, and Tokyo Rope planned on manufacturing spirals in September.

Table 5: Pile lengths for Phase III

Bent	Pile Length (ft)
1	53
2	Additional Data Required
3	Additional Data Required
4	75
5	50
6	44

During Phase II of the construction project, splice piles were required for all piles at bent 2. To avoid the need for splice piles, Astaldi Construction hired Foundation & Geotechnical Engineering, LLC, (FGE) to take additional borings at bent 2 and 3 and recommend an acceptable pile length. After conducting additional borings and referring to the pile driving data from Phase II, FGE recommended production pile lengths of 85 ft for bent 2 and 75 ft for bent 3. Unfortunately, both pile 5 at bent 2 and pile 6 at bent 3 did not meet bearing requirements during pile driving or subsequent pile restrikes. During Phase II, six (6) 18-in. CFCC splice piles each 42 ft in length were fabricated at Gate Precast Company in Jacksonville, FL. Four (4) of the splice piles were used at bents 2 and 3 during Phase II. The two extra splice piles from Phase II were used on pile 5 at bent 2 and pile 6 at bent 3. Unlike the splice procedure used during Phase II, Astaldi used a 1 in. x 1 in. x ¼ in. plastic spacer between the piles instead of the ½-in. nut.

Displayed in **Table 6** is the Phase III pile driving data including the pile number, pile length, penetration below ground, pile tip and head elevation, pile cutoff elevation, and the length of the pile driven. The pile driving data is represented in a visual format in **Figure 52**, where the pile cutoff elevation and tip elevation are labeled on each pile along with the listed pile ground elevation. As shown in Figure 52, there is a large variation between the driven pile lengths on bents 2 and 3 due to the varying soil conditions. Piles at bent 1, 2, 3, 4, and 5 are displayed in **Figures 53**, **54**, **55**, **56**, and **57** respectively. As shown, the pictures are prior to splice pile installation. Piles stored on site for bent 6 displayed cracks between the CFCC strands on all three piles, shown in **Figure 58**.

Table 6: Pile driving data – Phase III

Bent	Pile No.	Pile Length (ft)	Penetration Below Ground (ft)	Tip Elev.(ft)	Head Elev. (ft)	Cut Off Elev. (ft)	Ground Elev. (ft)	Length Driven (ft)
1	4	55	23.06	-19.55	33.45	6.5	3.51	26.05
	5	55	20.92	-17.93	35.07	6.2	2.99	24.13
	6	55	22.87	-20.21	32.79	6.0	2.66	26.21
2	4	85	15.5	-30.5	54.5	6.6	-5.2	37.1
	5	85	N/A	-120.19	-35.19	N/A	N/A	N/A
	5(Splice)	42	105.19	-35.19	6.81	6.4	-5.24	126.59
	6	85	45.67	-60.67	24.33	6.2	-5.02	66.87
3	4	75	12.74	-27.74	47.26	6.7	-5.2	34.44
	5	75	32.85	-47.85	27.15	6.5	-5.2	54.35
	6	75	N/A	-76.71	-1.71	N/A	N/A	N/A
	6(Splice)	42	61.71	-1.71	40.29	6.3	-5.2	83.01
4	4	75	12.4	-27.4	47.6	6.7	-5.2	34.1
	5	75	12.2	-27.2	47.8	6.5	-5.2	33.7
	6	75	12.18	-27.18	47.82	6.3	-5.2	33.48
5	4	50	14.25	-29.25	20.75	6.5	-5.2	35.75
	5	50	16.21	-31.21	18.79	6.3	-5.2	37.51
	6	50	16.17	-31.17	18.83	6.1	-5.2	37.27
6	4	44	21.77	-19.58	24.42	6.3	2.19	25.88
	5	44	19.92	-18.58	25.42	6.1	1.34	24.68
	6	44	22.5	-20.58	23.42	5.9	1.92	26.48

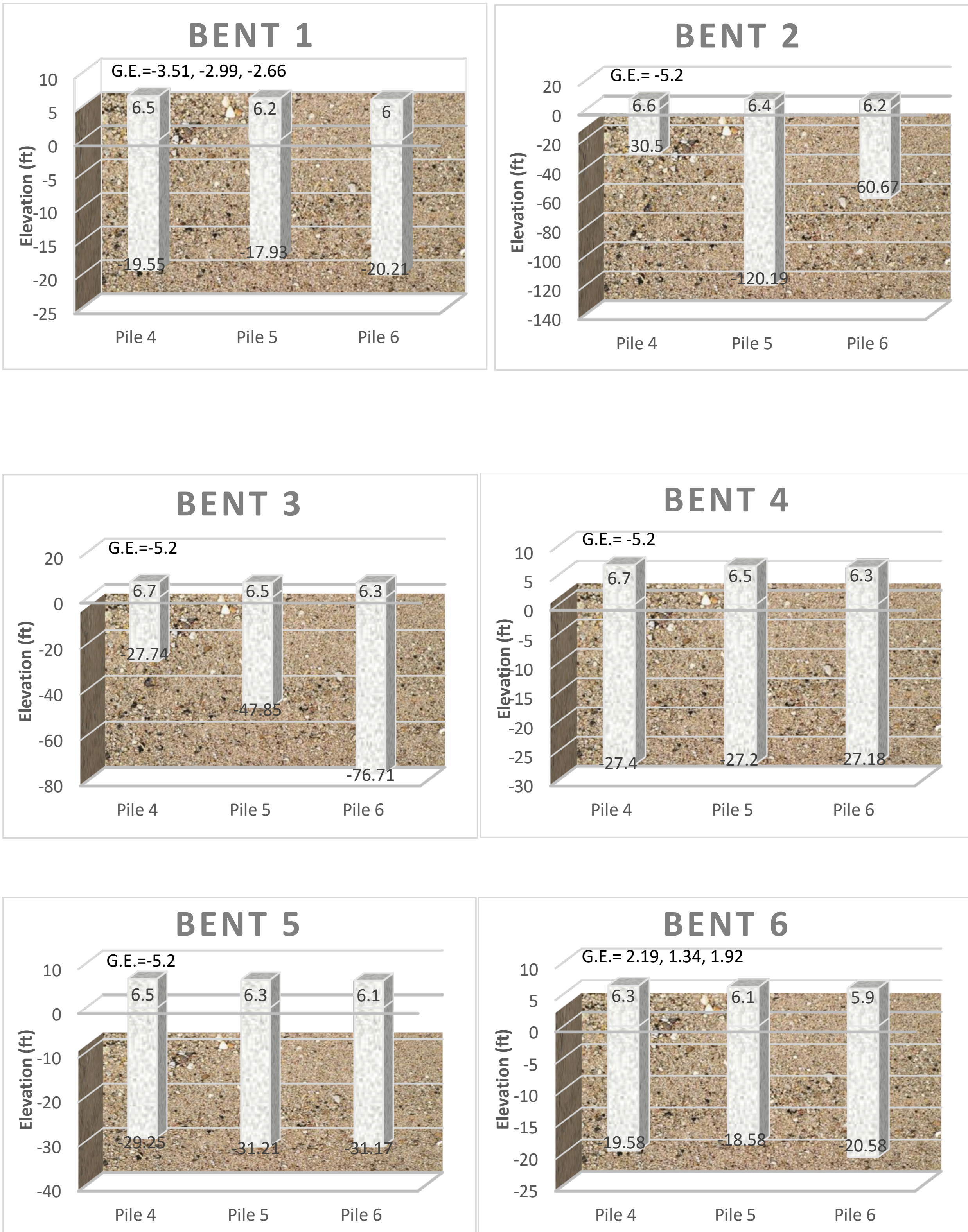

Figure 52: Phase III pile diagrams

Figure 53: CFCC piles at Bent 1

Figure 54: CFCC piles at Bent 2

Figure 55: CFCC piles at Bent 3

Figure 56: CFCC piles at Bent 4

Figure 57: CFCC piles at Bent 5

Figure 58: Cracks on piles for Bent 6

4.3 Temporary Sheet Piles

Temporary steel sheet piles were installed to allow for crane access during bridge construction and to control turbidity during sheet pile driving. PZ-27 Grade 50 steel sheet piles were used for the temporary walls. During Phase II, six walls were constructed: walls 1A, 1B, 1C on the west side of the bridge, and 2A, 2B, 2C on the east side (**Figure 59**).

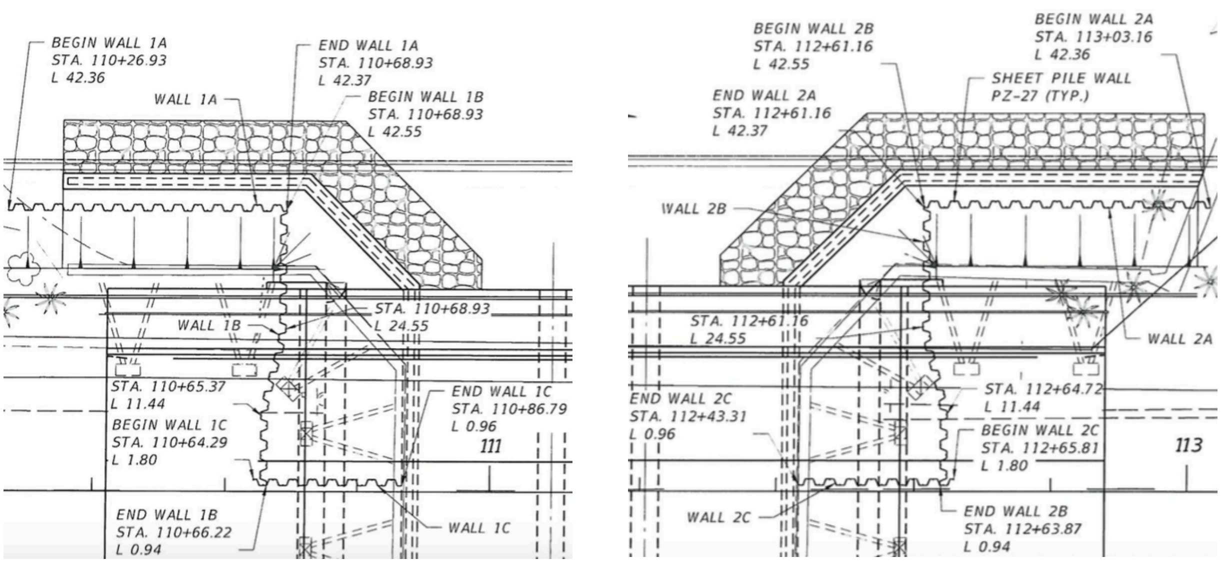

Figure 59: Temporary sheet pile wall layout (from ASA Consultants, Inc.)

The temporary sheet pile walls on the west side of the bridge were constructed before moving to the east side of the bridge. To lift the sheet piles, two holes were cut into the top of the pile, then the pile was lifted using the 230-ton Manitowoc 888 crawler crane (**Figure 60**). After each pile was lifted, workers interlocked the pile with the adjacent drilled pile. H-piles were driven and welded to form a temporary platform for workers to access the sheet piles. After the leads were removed from the pile, the APE pile driver was lowered onto the top of the steel sheet pile. The temporary sheet piles were driven to the required elevation, and then the pile driver was removed.

Figure 60: Temporary sheet pile installation

Temporary critical sheet piles were driven to contain soil during demolition of the eastbound lane of the existing bridge, as shown in **Figure 61**. Temporary sheet piles were also driven for seawall installation on the eastern side of the eastbound lanes.

Figure 61: Demolition on existing bridge (eastbound lane)

4.4 Prestressed Concrete Sheet Piles

FDOT Type "A" precast concrete CFRP/GFRP sheet pile wall was used for walls 1A, 1B, 2A, 2B, 3A, 3C, 4A, 4B, 5A, 5B, 6A, and 6B at Halls River Bridge. Gate Precast Company located in Jacksonville, FL, fabricated 137 Type "A" CFCC prestressed sheet piles with seven (7) different sheet pile designs. The sheet piles were marked based on the type of pile design and the wall the pile will be installed in – for example, sheet pile mark SH1-1A is sheet pile design 1 used at sheet pile wall 1A.

On February 23, 2017, the first set of CFCC sheet piles was prestressed at Gate Precast. Seventeen (17) sheet piles were fabricated on the 569 ft 5 in. long precast bed. After removing the steel prestressed sheet piles that were cast on February 21, 2017, from the bed, the bed was first prepped for the next set of sheet piles by spraying a release agent in the forms. The plywood sheet pile headers were installed, and GFRP stirrups were set out in groups. The GFRP stirrups and hooked bars were manufactured in their required shape and length, and overlapping legs were tied with zip-ties by the manufacturer. To form the ends of the sheet piles in the casting beds, plywood headers (**Figure 62**) were used instead of steel headers, to protect the CFCC strands from being damaged when pulling them through the headers.

A typical procedure for installing strands was as follows. CFCC strands were pulled from a spool (**Figure 63**) and fed through the headers and through the pre-placed bundled GFRP stirrups to the end of the bed. The light weight of CFCC allows for one worker to pull the strand as other workers ensure the CFCC is correctly passing through the headers and stirrups. If the bundle of stirrups were to be temporarily placed toward the middle of the sheet pile, abrasion between the stirrups and CFCC strand may occur when pulling, so the bundled stirrups are placed closer to the headers instead. When the CFCC is pulled the entire length of the bed, the workers pull the strand taut, then the CFCC strand is cut, and another strand is pulled.

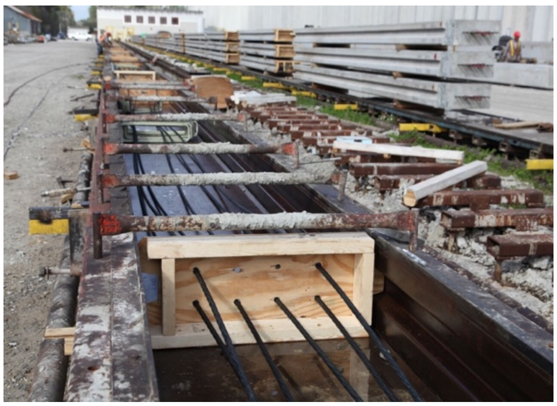

Figure 62: CFCC strands through plywood header

Figure 63: CFCC spool

After the eight (8) CFCC strands were pulled, the couplers between the CFCC strands and steel strands were installed. Gate Precast received training from Tokyo Rope on CFCC handling and coupler installation. The couplers, supplied by Tokyo Rope, were used at both ends of the bed.

The coupler installation procedure was as follows. A spiral pre-cut mesh was taped to the end of the CFCC strand, tightly spiraled around the strand, then taped on the end of the mesh; this is similar to CFCC pile production. A stainless-steel mesh was formed over the spiral mesh, then taped, and the excess mesh was cut with a side grinder. After spraying the coupler sleeve with molybdenum spray, the sleeve and the wedge O-ring were slid over the mesh wrapped CFCC strand. Four wedges were inserted into the O-ring and then the wedges were set into the coupler sleeve using a pneumatic jack. On the steel strand, a coupler was slid over the strand, a wedge was attached to the strand, and the coupler was slid back over the wedge. To complete the coupler installation, the coupler sleeve on the CFCC strand was screwed into the coupler on the steel strand. The coupler was fully tightened and then backed off to prevent damage to it during tensioning. The steel strands were attached to the anchoring blocks using standard open grips (**Figure 64**).

Figure 64: Open grips on steel strands

After installing couplers at both ends of the prestressing bed, the CFCC strands were pulled to an initial force of 8000 lb with a hydraulic monostrand jack. According to FDOT Specification 450-8.2.2, an initial force of 5% to 25% of the final force can be applied to a single straight strand during tensioning to remove the slack in the strands. In Request for Information (RFI) 0027, Astaldi requested the “initial pull” for CFRP strands in Type “A” sheet piles. The State Structures Design Office replied with an initial jacking force of 38.25 k. Excess length of the steel strands was cut from the live end before attaching the hydraulic jack for the final pull. The strands were then pulled to the target force. The bottom row of CFCC strands was stressed first; **Figure 65** shows the stressing pattern for all eight (8) strands.

For a bed length of 6833 in., a CFCC length of 6396 in., and a steel strand length of 437 in., the net theoretical elongation was calculated to be 53 in. The gross elongation is a combination of the elongation of the CFCC strand, steel strand, abutment rotation, dead end, live end, and steel and CFCC coupler elongation. Anchorage set loss occurs with movement of the strand before the wedges are properly seated. The anchorage set loss depends on the prestressing system, but a commonly assumed value is 0.25 in. for elongation computations. The CFCC strands and steel strands were marked where the coupler was attached (**Figure 66**) so that later any strand slipping due to tensioning could be measured.

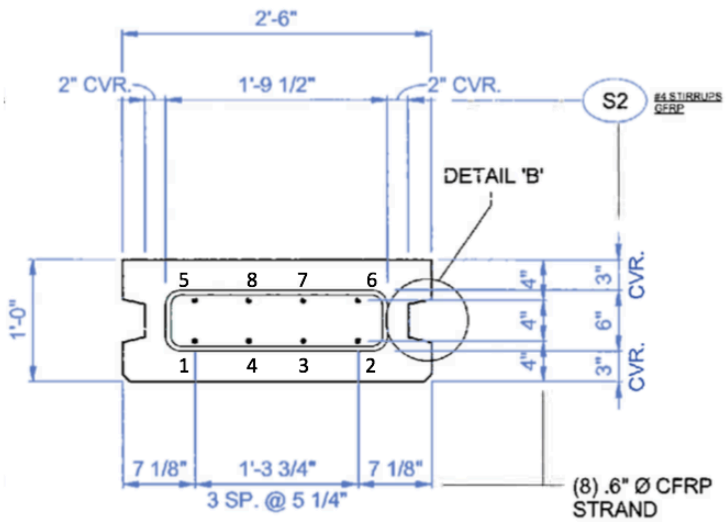

Figure 65: Strand stressing pattern (from Gate Precast Company)

Figure 66: Couplers for steel strands and CFCC strands

After tensioning, the strand's elongation was measured from the live (stressing) end. **Table 7** shows the measured elongation of the eight strands, where the elongations range from 53 in. to 53.75 in. The measured elongation or slippage of the coupler at the dead (non-stressing) end of the strands was around 1.125 in. for all strands. The stressed CFCC strands were left to rest overnight.

On February 24, 2017, the GFRP reinforcement was tied, and the sheet piles were cast using self-consolidating concrete (SCC). Epoxy coated wire and plastic ties were used to secure the

GFRP stirrups and bars (**Figure 67**). The epoxy coated wire was twisted by hand and cut using wire cutters, while the plastic ties were dispensed from an automatic tie gun.

Table 7: Elongation of CFCC strands

Strand	Elongation
1	53.75 in.
2	53.625 in.
3	53.25 in.
4	53.125 in.
5	53.25 in.
6	53.125 in.
7	53.25 in.
8	53.0 in.

Figure 67: Ties used to secure GFRP stirrups to CFCC strands

After aligning the GFRP bars, workers noticed that the hooked GFRP reinforcing bar was manufactured 1 ft too long, so the excess was cut using a side grinder, and the bar was positioned according to the design drawings. This was done for all sheet piles in the bed.

Two GFRP bars were used to stabilize the end stirrups, near the tapered edge of the sheet pile. The two extra GFRP bars were not required for the CFCC strand piles, but the workers installed them anyway because they were familiar with this procedure from the steel strand sheet pile fabrication. Two half-inch diameter 270 ksi black steel strands were installed as lifting loops with a minimum embedment of 2 ft 10 in. and a 5-in. minimum cover, as approved in RFI 0023. A wooden key template was inserted into the sheet pile form and stabilized using plastic chairs and wooden stakes. To form the tapered end of the sheet pile, a plywood template was inserted around the CFCC strands (**Figure 68**). The holes in the headers and tapered plywood forms were filled with foam and caulked. After tightening the sheet pile forms using a power drill, wooden blocks were glued between the headers using a hot glue gun. The concrete was cast (**Figure 69**), and the top was troweled to a smooth finish (**Figure 70**).

The prestressed CFCC sheet piles were stored on the bridge site using a two-point support system and stacked four piles high, as shown in **Figure 71**. FDOT Type "H" steel prestressed sheet wall piles were fabricated at Gate Precast for walls 7A, 7B, 8A, and 8B. On February 23, 2017, steel prestressed piles that were cast on February 21 were removed from forms after gaining strength. The sheet piles were stacked next to the precast bed to allow for CFCC sheet pile fabrication. The single layer of steel strands in the sheet piles (**Figure 72**) presented fabrication difficulties due to the strand layer location. Chairs were required on the sides and the bottom of the steel strands to stabilize the GFRP stirrups. The lifting loop on the sheet piles interfered with strand detensioning, requiring a shield to be placed between the steel strands and the lifting loop when cutting. The sheet piles were stacked in storage (**Figure 73**).

Figure 68: Tapered end of CFCC sheet pile

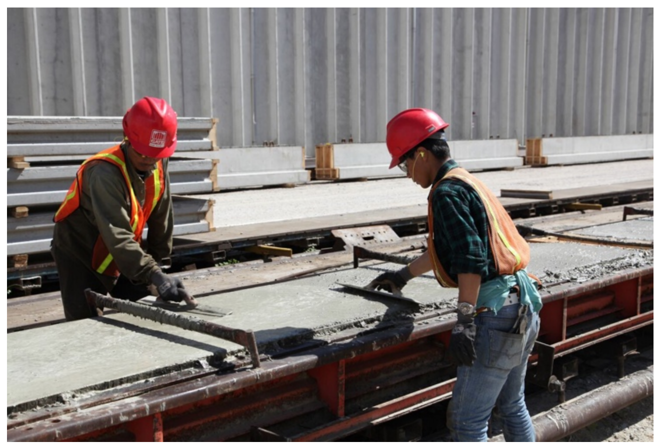

Figure 69: Placing SCC with hopper

Figure 70: Troweling of SCC

Figure 71: Storage of CFCC sheet piles at Halls River

Figure 72: Steel-stranded sheet pile

Figure 73: Steel-stranded sheet pile in storage

On March 10 and 13, 2017, jet pile driving was attempted on the sheet piles of Wall 1B, but a solid rock layer prevented driving operations. As stated in RFM 0002, jet pile driving was approved as a sheet pile installation method. A 14x120 H-pile was then used as a punch to attempt to break through soil layers. After jet driving installation failed, the contractor drilled preformed holes with a 26-in.-diameter auger and then attempted driving using a specialty made hydraulic impact hammer. Due to failed driving attempts, excavation was started using a hydraulic excavator. Excavation was halted due to concern for the existing bridge structure, and Astaldi requested FDOT to evaluate the need for temporary critical sheet piles, as stated in RFI 0034. Eight (8) to ten (10) truckloads of A3 embankment fill were brought in to compact the soil around the excavated area. Sheet pile number SH1 was the first pile attempted to drive, as shown in **Figures 74** and **75** there was slight cracking of the keyway edge due to driving operations. Also stated in RFI 0034 was FDOT's recommendation to move the starter pile further north, away from the existing bridge, to achieve entire installation depth.

Astaldi installed nine (9) CFCC sheet piles (**Figure 76**) by trenching on the east sea wall as of June 30, 2017. The sheet piles were installed to the lowest elevation possible, but they did not reach the required depth. Astaldi was able to trench through the first layer of dense soil, but they were unable to excavate soil after encountering a harder rock layer. A 42-in.-wide hydraulic drum cutter was ordered to excavate the solid rock layer. Trenching on the west side of the bridge encountered the same dense rock layer. A double layer of metal sheet piles has been used on the west side to contain soil during trenching operations (**Figure 77**).

Figure 74: Sheet Pile SH1

Figure 75: Sheet Pile SH1

Figure 76: Installed CFCC sheet piles

Figure 77: Double layer of metal sheet piles

As of September 28, 2017, fifteen (15) CFCC sheet piles were installed on the west side of Halls River Bridge (**Figures 78** and **79**). The soil conditions prevented timely sheet pile installation, and according to RFI 0040 all concrete sheet piles must be installed to plan elevations.

Standard Penetration Tests (SPT) were performed on the soil in front and behind the sheet pile wall to attempt to verify the existing installed sheet piles and establish a new minimum tip elevation for the remaining sheet piles. Also, an anchored wall design proposed by FDOT was done by Astaldi's engineer to expedite the sheet pile installation process. In RFM 0007, Astaldi requested to remove the 7-in. projection of the CFCC strand from the top of the concrete sheet piles due to the fragility of the strand and the constant effort required by the contractor to protect the strands. RFM 0007 was approved on the condition that if an anchored wall design is pursued additional dowel bars will have to be used to account for the loss of shear transfer.

Figures 78: Sheet piles (West HRB)

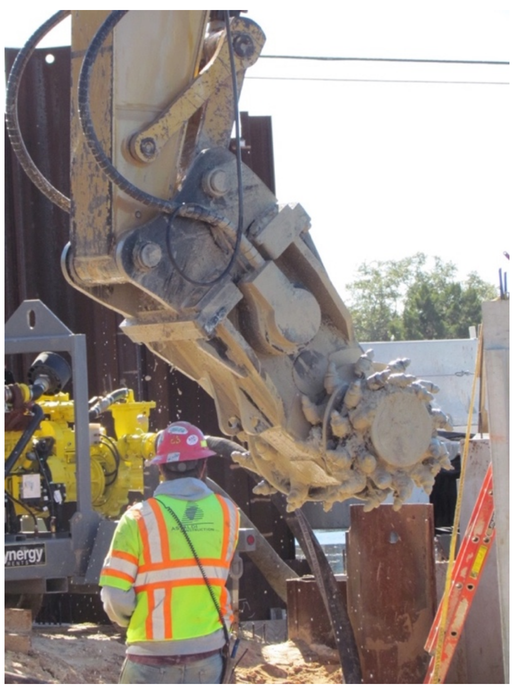

Figure 79: Trencher for CAT336E

On September 26, 2017, the A-3 fill material supplier in Hernando County was shut down for undisclosed reasons. The material supplier projected the closure to last for a few days, but FDOT and Astaldi searched for another approved supplier for A-3 fill material to continue sheet pile installation.

The soil conditions encountered when installing the prestressed concrete sheet piles caused setbacks in construction. The sheet piles were installed by trenching using a CAT 336E with a rotor trenching attachment, shown in Figure 79. The minimum tip elevation was unreachable due to the soil conditions, leading Astaldi to propose a new sheet pile design with deadman anchors, as shown in **Figure 80**. The Phase II sheet pile walls on the west side of the bridge (sheet pile walls 1A, 2A, and 3A) have been installed using the proposed deadman design; the same method was used for the sheet piles on the east side of the bridge for Phase II (sheet pile walls 1B, 2B, and 3B). Sheet pile installation progress is shown in **Figures 81** and **82**. The updated design uses #5 stainless steel deadman anchors with a 6-in. perforated PVC sleeve. The sheet piles were to be installed to the minimum tip elevations listed in **Table 8**. After reaching approved elevation, the sheet piles were cut using a concrete saw and covered with an approved geosynthetic material. The sheet piles on the west side were backfilled and tamped to prepare for bulkhead cap formwork. The completed sheet pile walls 1A, 2A, and 3A are shown in **Figure 83**.

During placement of the rubble rip rap, cracks on the sheet pile wall corners expanded. Crack widths on walls 2A-3A were in excess of ½ in. and extended the entire length of the pile cap, while the cracks on walls 2B-3B were approximately ¼ in. wide and also extended the length of the pile cap. The cracks on the top and face of walls 2B-3B are shown in **Figures 84**, **85**, and **86**. On walls 2A-3A, A2B Engineering LLC proposed to cut away the cracked pile cap (**Figure 87**), roughen the concrete surface, apply an APL approved epoxy, and form and pour the pile caps using the original concrete mix design. The proposed repair procedure for walls 2B-3B included injecting an FDOT approved epoxy into the cracks and restoring the control joint with a ¾ in. saw cut.

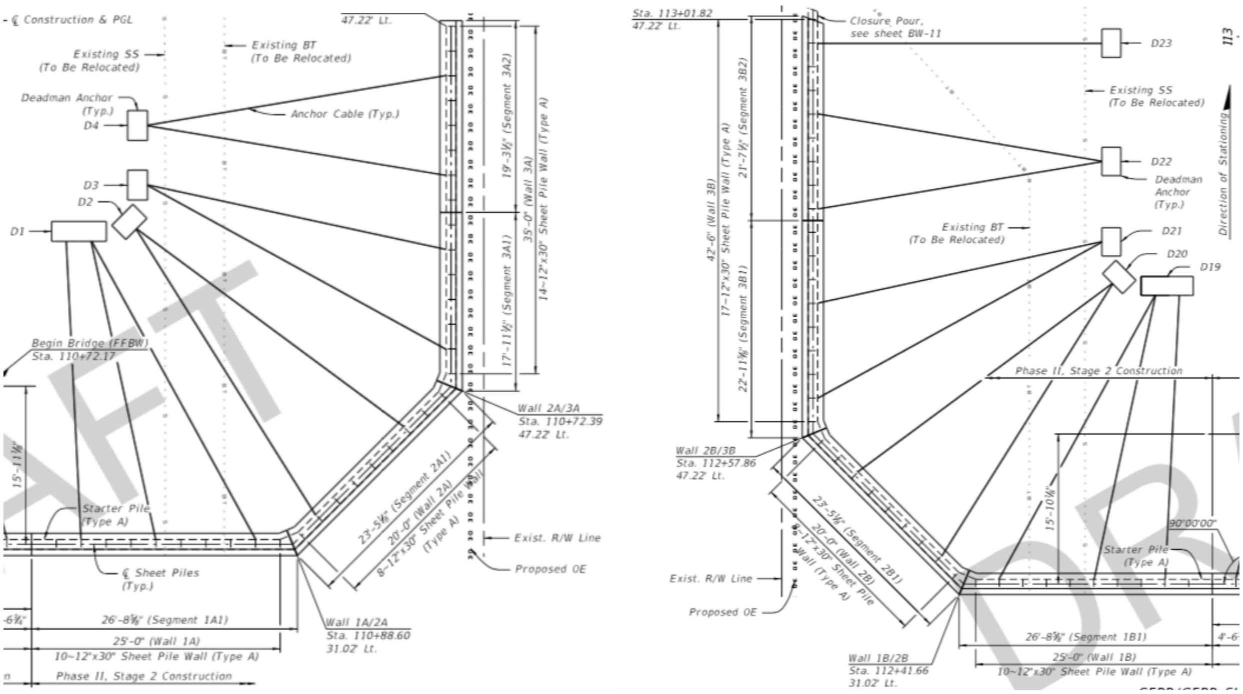

Figure 80: Phase II deadman anchor design

Figure 81: West side sheet pile walls (1A, 2A, and 3A)

Figure 82: East side sheet pile walls

Table 8: Phase II updated sheet pile wall elevations

Wall No.	Station	No. Piles Required	Min. Tip Elevation (ft)	Top Wall Elevation (ft)
1A	110+88.60	10	-12.3	2
1A/2A	110+88.60 to 110+87.57	1	-12.3	2
2A	110+87.57 to 110+73.43	8	-12.3	2
2A/3A	110+73.43 to 110+70.94	1	-9.7	2
3A	110+70.94 to 110+35.94	14	-9.4 (6 piles), -8.8 (8 piles)	2
1B	112+41.66	10	-12.3	2
1B/2B	112+41.66 to 112+42.69	1	-12.3	2
2B	112+42.69 to 112+56.83	8	-12.3	2
2B/3B	112+56.83 to 112+59.32	1	-12.3	2
3B	112+59.32 to 113+01.82	17	-10.8 (6 piles) -11.6 (6 piles) -12.3 (5 piles)	2

Figure 83: Sheet pile walls 1A, 2A, and 3A

Figure 84: Crack on top of seawall 2B-3B

Figure 85: Crack on face of seawall 2B-3B

Figure 86: Cut out crack on walls 2A-3A

Figure 87: Crack on face of seawall 2B-3B

As of September 2018, ten (10) sheet piles had been installed on the east side of Phase III construction. Similar to Phase II construction, the sheet piles were installed by excavating soil and using a deadman with anchor rod tieback. The concrete mix design (02-1511-04) is shown in **Table 9**; the mix has a design spread limit of 27 ± 2.5 in. and a maximum water-to-cementitious material ratio of 0.37. The compressive strength and concrete tests results including percent of air, spread, release strength, and 28-day strength of the 12-in. sheet piles are provided in **Table 10**. A minimum release strength of 4,000 psi and a 28-day strength of 6,000 psi were required.

Table 9: Concrete mix design 02-1511-04

Product	Quantity	Production Facility
921: Cement Type II (MH)	656 lb	CMT29 Suwannee American Cement
929: Fly Ash Class F	144 lb	FA11 Recovered Resources
901: C12 #67 Stone	1350 lb	87145 Titan America LLC
902: F01 Silica Sand	1347 lb	71132 Vulcan Materials Company
MasterAir AE 90	1.5 fl oz	BASF Construction Chemicals LLC
MasterSet R 100	8 fl oz	BASF Construction Chemicals LLC
MasterGlenium 7700	32 fl oz	BASF Construction Chemicals LLC
Water	295 lb	
Theoretical Unit Weight	140.4 pcf	
Theoretical Yield	27 cf	

As of November 07, 2018, the east side seawall walls 4B and 5B were complete and construction was progressing on walls 6B, 7B, and 8B. Shown in **Figure 88**, the east side seawall 6B was under construction where temporary steel sheet piles were used to contain the soil during sheet pile driving. On the west side, walls 4A and 5A of the seawall were complete and wall 6A was under construction. **Figure 89** displays the completed seawalls 4A and 5A along with the installed temporary steel sheet piles. After construction of wall 6A, shown in **Figure 90**, walls 7A and 8A were to be constructed on the west side.

Table 10: Compressive strength results and concrete tests 12-in. sheet piles

Product Serial #	Product Mark #	Date Cast	% Air	Spread	VSI	Release Age	Release Strength	28 Day 1st Cyl	28 Day 2nd Cyl	28 Day 3rd Cyl
A154-	4-SH10 5-SH11	8/9/17	6.0	26.0"	1	3 days	4120 psi 4080 psi	7790 psi	7550 psi	7880 psi
A171	4-SH12 5-SH16	8/9/17	6.0	26.0"	1	3 days	4120 psi 4080 psi	7790 psi	7550 psi	7880 psi
A172- A183	6-SH19 2-SH18 1-SH17 1-SH20	8/11/17	5.6	27.0"	0.5	3 days	4580 psi 4790 psi	7400 psi	8280 psi	7530 psi
A184- A200	8-SH02 9-SH05	1/12/18	4.8	27.0"	0.5	3 days	4890 psi 5550 psi	7860 psi	8070 psi	8310 psi
A201- A217	8-SH02 9-SH05	1/16/18	4.2	29.5"	0.5	3 days	4810 psi 4910 psi	8490 psi	8310 psi	8830 psi
A218- A234	7-SH02 10- SH05	1/23/18	3.0	29.5"	0.5	3 days	5285 psi 5410 psi	8940 psi	9150 psi	8730 psi
	SH05 SH02	6/14/18	3.2	29.5"	1	1 day	5100 psi 4960 psi	9610 psi	8940 psi	9300 psi

Figure 88: East side seawall (walls 4B, 5B, & 6B)

Figure 89: West side seawall (walls 4A & 5A)

Figure 90: West side seawall (wall 6A)

Sheet pile wall 8B is composed of twenty-one (21) sheet piles, of which twelve (12) of them did not meet minimum tip elevation, as shown in **Figure 91**. A2B Engineering LLC performed an independent analysis using Support IT to determine the sufficiency of the sheet pile walls. The analysis yielded an increased tie-back force from 1.523 kips/ft to 2.013 kips/ft. To account for the increase in tie-back force it was proposed to add additional deadmen anchors (with one stainless steel bar) every 5 ft where the sheet piles did not meet minimum tip elevation, as shown in **Figure 92**.

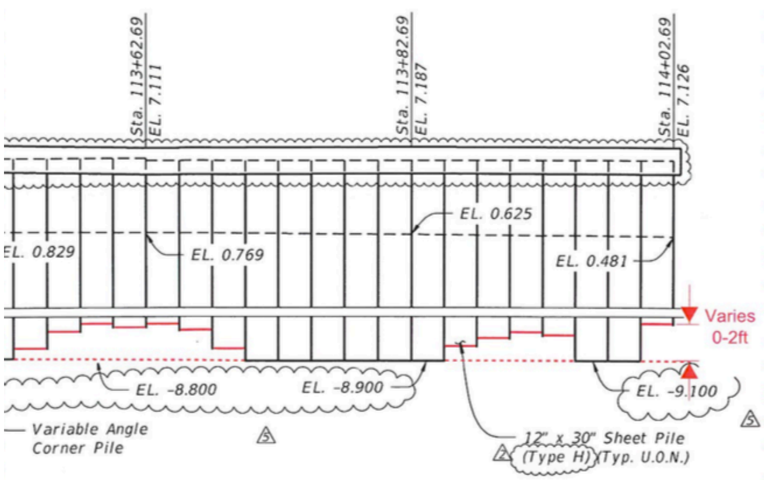

Figure 91: Wall 8B elevation

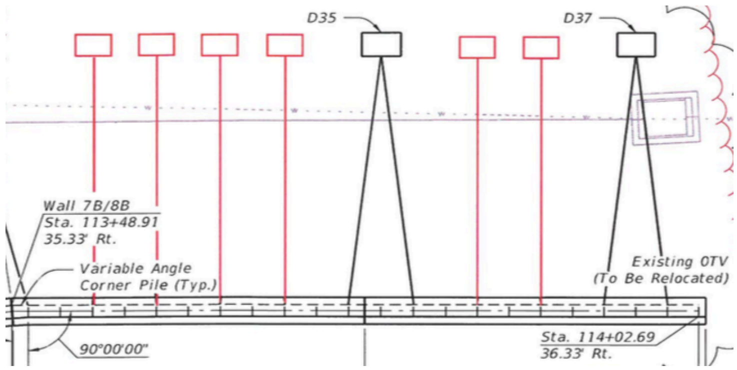

Figure 92: Wall 8B deadmen anchor design

As of June 2019, the sheet pile walls 7B and 8B, located on the southern half of the east side of the bridge, were under construction. The completed walls 4A, 5A, and 6A (shown in **Figure 93**) are located on the southern half of the west side of the bridge.

Figure 93: Sheet pile walls 4A, 5A, 6A (Figure provided by FDOT)

4.5 GFRP Reinforced Pier Caps

Denson Construction was subcontracted by Astaldi to construct GFRP cages for the concrete pier caps. On April 12 through April 14, 2017, three cages were constructed on the storage yard near Halls River Bridge. The cages were tied using plastic ties (**Figure 94**) and transported from the storage yard to the bridge on a flatbed truck. Using a four point pick up, the cages were lifted from the flatbed truck to the pier cap formwork, as shown in the **Figures 95** and **96** below.

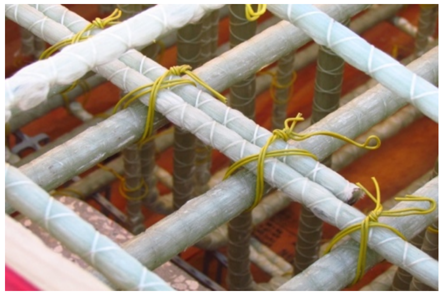

Figure 94: Plastic ties used on GFRP cage

Figure 95: Installation of GFRP pier cap cage

Figure 96: Crew members positioning GFRP cage

Crew members re-position GFRP bars to ensure proper concrete coverage between the GFRP bars, pile heads, and edge of the formwork. After ensuring the GFRP bars were in the correct position, the bars were re-tied by hand using plastic ties. The light weight of the bars allowed the workers more maneuverability than traditional steel bars, but gloves were required to protect workers' skin from glass fibers.

On April 18, 2017, FDOT held a pre-pour meeting to discuss the rules and regulations regarding concrete delivery, testing, and acceptance. Production limits discussed were regulations on temperature and transit time. Mixing and delivery of concrete items discussed included the use of a grate when concrete slump is greater than 6 in., proper ticketing of concrete trucks, calibration of concrete truck water measuring devices, and maximum drum revolutions. Sampling and testing of the concrete were performed for both quality control (QC) and verification testing (VT). The initial slump test taken before the addition of water must be within tolerance of +/- 1.5 in. The samples taken for testing must be at the final point of placement; in the case of the bent caps, the testing was taken from the truck before the concrete was flown to the bent caps in the hopper. Plastic concrete tests were performed on the initial load from each concrete plant. For the bent caps, the concrete plant used was Argos USA Corporation located in Brooksville, FL (a 45-minute drive from the bridge site). For quality control testing, three (3) cylinders were cast for all structural concrete used. VT labs collected six (6) cylinders of concrete, two (2) for testing at one (1) day, two (2) for testing at three (3) days, and two (2) for testing at seven (7) days. A cure box was constructed to hold the cylinders and maintain a temperature between 60 and 80 degrees (**Figure 97**). Three cylinders collected for QC and VT were to be tested at 28 days to determine concrete strength.

Bent caps 4 and 5 were cast on April 18, 2017. The plywood forms were sprayed with water to aid in form release. The concrete mix design includes 350 lbs of Type II Cement, 350 lbs slag, 1680 lbs of #57 stone, 1165 lbs of silica sand, 0.5 fl oz Darex AEA admixture, 21 fl oz WRDA 60 admixture, 42 fl oz Recover admixture, 70 fl oz Adva 140M admixture, and 267 lbs water. Initial concrete tests were taken for plastic properties and water/cement ratios. Concrete producer data is displayed in **Table 11**, and the test results from QC and VT are shown in **Table 12**.

Figure 97: Curing box with concrete cylinders

Table 11: Concrete producer data

Theoretical Unit Weight	141.1 pcf
Theoretical Yield	27.01 cf
Slump	7.25 in
Air Content	2.1%
Temperature	98 degrees F
Average Chloride Content	0.033 lbs/cyd
Water to Cementitious Materials Ratio	0.38
28 Day Compressive Strength	7030 psi

Table 12: QC and VT tested values

Value	Initial	Acceptance
Slump	7.25 in	7.25in
Air	2.1 %	2.0%
Concrete Temperature	81 degrees F	81 degrees F
W/C Ratio	0.36	0.36

After the concrete was accepted, the mix was poured through a grate and into the 2-yd concrete hopper. Using the crane, the hopper was "flown" to the bent, where crew members lifted the lever on the hopper to release the concrete into the bent forms (**Figure 98**).

Figure 98: Casting of Bent 4 pier cap

Using a vibrator with a rubber tip, the concrete was vibrated to aid in placement (**Figure 99**). Each bent required approximately 11 cubic yds of concrete; a total of 25 cubic yds of concrete (three trucks) were ordered. The concrete properties were tested again in the middle of the second truck to meet inspection requirements.

Figure 99: Rubber tip vibrator

After filling each bent form, the tops were troweled to a smooth finish (**Figure 100**). By misting the concrete with water, the pier caps are kept moist during hardening. Forms are typically removed after seven (7) days but can be removed after three (3) days if the pier caps are sprayed with a curing compound. Driving operations may not occur within 30 ft of fresh concrete for 72 hours after casting.

Figure 100: Completed pier cap

After performing an as-built survey on bents 3 and 5, it was noted that both of the bents were out of tolerance, as stated in RFC 0001 Rev.1. The error was caused by a surveying inaccuracy when forming the pedestals; the four corners of each pedestal were surveyed to determine the difference in elevations. At bent 5, the difference in elevations vary from 0.8 to 1.8 in. above plan elevations, while at bent 3 the elevations vary from 0.8 to 1.6 in. below plan elevations. The elevations at the four corners of each pedestal are shown in **Table 13**, with the point locations shown in **Figure 101**. At bent 3, a non-shrink grout NC FL GROUT (APL 934-000-019) was used to reach plan elevation, and at bent 5 the pedestals were ground down to reach the plan elevation.

Table 13: Difference in plan and as-built elevations for Bent 3 and 5

End Bent	Beam No.	Location No.	Plan Elev.	As-Built Elev.	Difference (in.)
5	1	1	8.418	8.560	1.7
		2		8.570	1.8
		3		8.540	1.5
		4		8.540	1.5
5	2	1	8.551	8.660	1.3
		2		8.660	1.3
		3		8.660	1.3
		4		8.665	1.2
5	3	1	8.683	8.770	1.0
		2		8.765	1.0
		3		8.770	1.0
		4		8.760	0.9
5	4	1	8.816	8.890	0.9
		2		8.910	1.1
		3		8.885	0.8
		4		8.890	0.9
3	1	1	8.577	8.510	-0.8
		2		8.510	-0.8
		3		8.505	-0.9
		4		8.510	-0.8
3	2	1	8.710	8.605	-1.3
		2		8.590	-1.4
		3		8.605	-1.3
		4		8.610	-1.2
3	3	1	8.843	8.740	-1.2
		2		8.740	-1.2
		3		8.745	-1.2
		4		8.750	-1.1
3	4	1	8.976	8.850	-1.5
		2		8.845	-1.6
		3		8.860	-1.4
		4		8.860	-1.4

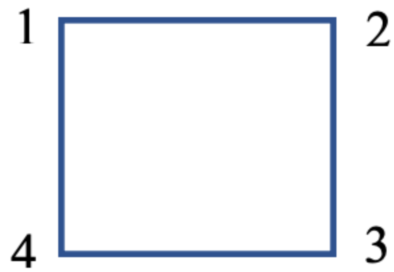

Figure 101: Location of surveyed points

Glass fiber reinforced polymer (GFRP) bars were stored at Astaldi's storage yard located a couple miles west of the bridge site. **Figures 102** and **103** show the stored GFRP bars that were covered in black plastic when not in use. The GFRP pier cap cages were pre-tied by Denson Construction at Astaldi's storage yard. Using a few wooden stands, the GFRP bars were stacked out and hand tied using plastic coated tie wire. Shown in **Figures 104-107** are the wood stands and the tied GFRP cages.

On the week of Jan 7th, 2019, the bottom form for bent four (4) pier cap collapsed resulting in a load and a half of concrete falling into Halls River. The first load of concrete was in the form for over an hour before the second load caused the failure. Damaged occurred to 8A1 GFRP bars located approximately 1ft from the bottom of the pile cap, shown in **Figures 108-110**. Additional cracking occurred in the bottom 6 in. of the cap and spanned the entire width of the pier cap, as shown in Figure 109. To repair the damage to the GFRP bars, holes were drilled to install additional #8 bars 4 ft 6 in. in length next to the damaged 8A1 bars. The bars have an embedment length of 2 ft 1 in. and maintain a minimum spacing of 1 in. from the existing 8A1 bars. The #8 bars were lapped by the 8A2 bars in the construction joint. Additionally, ¼ in. stainless Tapcon screw anchors with an embedment length of 2.1 in. were installed across the repair area. The screws were installed 6 in. on center with a ½-in. concrete cover. An epoxy bonding agent was also applied to the affected area before the concrete pour.

Figure 102: GFRP at storage yard

Figure 103: GFRP at storage yard

Figure 104: Wood stands for GFRP

Figure 105: GFRP pier caps

Figure 106: Tied GFRP cages

Figure 107: Plastic coated tie wires

Figure 108: Damaged 8A1 GFRP bar

Figure 109: Crack along bottom row of GFRP bars

Figure 110: Damaged Pier Cap 4

(Figures 108-110 provided by Astaldi Construction)

4.6 Hybrid Composite Beams

FDOT performed a Quality Assurance (QA) Inspection on the production of HCBs at Kenway Corporation in Augusta, Maine, on February 21-22, 2017. The fabrication operations were suspended because of some quality control issues. The Quality Control (QC) Manual was not being met as follows: there was no quality control manager or quality control activities; there was failure to comply with beam shell width tolerances; the overlapping fabric width was measured 2 in. shorter than required per construction plans; the resin used in the beam shell was expired; and white-out and pencil were used in Glass Lot sheet logs. Other non-compliance issues included:

- failure to supply FDOT with material mill certifications
- no resin calibration records
- no calibrated thermometer used
- witness panels were fabricated in comparison to the sides of the FRP shell instead of the bottom of the shell
- only five Barcol readings were being performed when ASTM standards require ten readings
- failure to perform acetone sensitivity tests
- failure to tag 0.5 in. galvanized strand reel
- failure to use beam mark number
- galvanized strand being damaged when pulled from rack

During Phase II, twenty (20) HCB beams were fabricated at Kenway. On April 12-13, 2017, the research team monitored the HCB fabrication process. The non-compliance issues noted above seemed to have been resolved. Two 33-in.-deep, 70-ft-long HCB shell forms were used for beam fabrication. In order to accommodate the beam depth of 20 $^{13}/_{16}$ in., foam was placed in the form and a fiberglass bottom was constructed. For Phase II, one form was set for 12 HCB shells 36 ft 0 in. in length and one form set for 8 HCB shells 36 ft 2 in. in length. A camber of 2.5 in. was set at the centerline of the forms. Using a reciprocating saw, the fiberglass rolls were cut to the beam width. The following layers were placed in the shell form: blue release film (surface Veil), CFM (continuous flow media), a 24 oz fiberglass layer, and 102 oz fiberglass layers (**Figures 111** and **112**). Two layers of 102 oz fiberglass are used on the bottom of the shell, and one layer is used on the top and sides of the shell. Using a handheld flat paddle, the fiberglass layers were pushed to the edges of the form to ensure proper placement. Workers wear protective eyeglasses, gloves, and shoe covers while working with FRP in the shell forms. If needed 3M Multipurpose 27 spray adhesive was used to adhere the layers together.

Figure 111: Blue release film and fiberglass layers

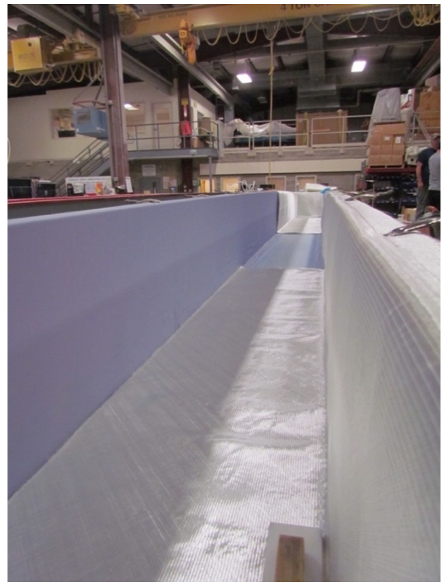

Figure 112: Fiberglass layers

After the fiberglass layers were in place, the 18 ASTM A416 7-wire strands were lifted and placed inside the beam shell. As shown in **Figure 113**, the 7-wire strands are bent at the ends of the beams. In between the middle set of 7-wire strands, four resin flow piping lines were rolled out and cut to length (**Figures** 113 and **114**). Three holes were cut into the fiberglass layers at the beam centerline for resin supply lines; plastic Ts were used to connect the supply lines to the resin flow piping.

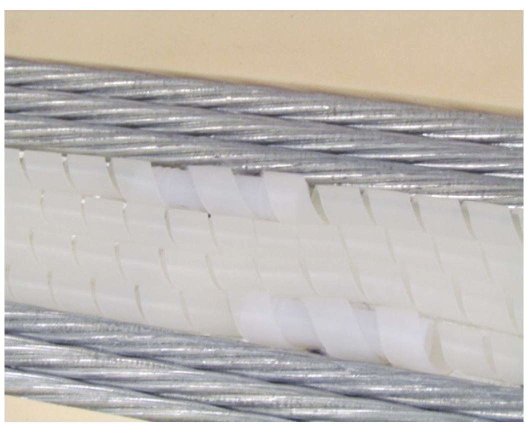

Figure 113: 7-wire strand at beam ends

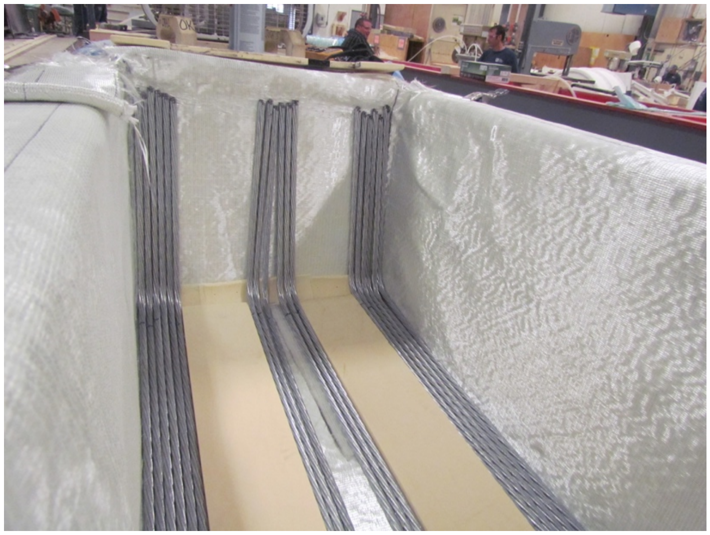

Figure 114: Resin flow piping and 7-wire strands

A layer of black shade cloth, a flow transfer medium, was placed on the bottom and sides of the beam shell with the use of adhesive spray, after which a layer of CFM was placed on the bottom of the shell (**Figure 115**). Vise grips and spacers were used to flatten the fiberglass layers to the shell form.

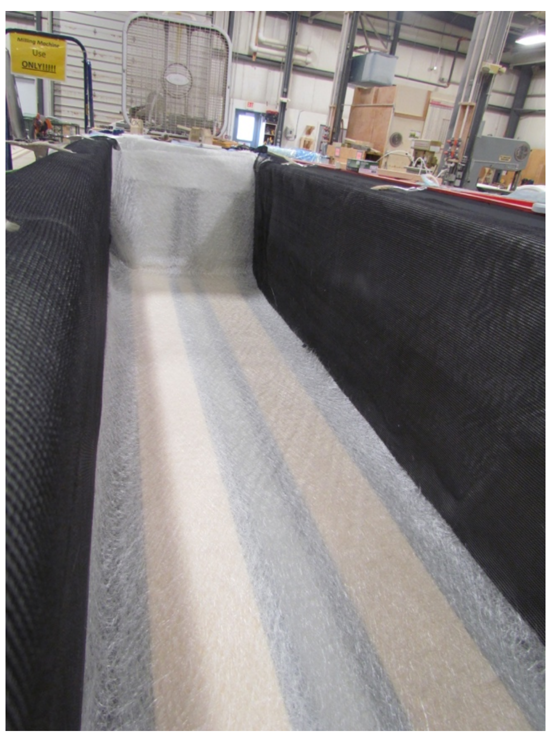

Figure 115: Shade cloth and CFM layers

Pre-cut foam pieces were installed with the use of a sledgehammer to form the concrete arch shape (**Figures 116** and **117**). If the pre-cut pieces were too large, the sides of the foam were shaved down to fit into the form.

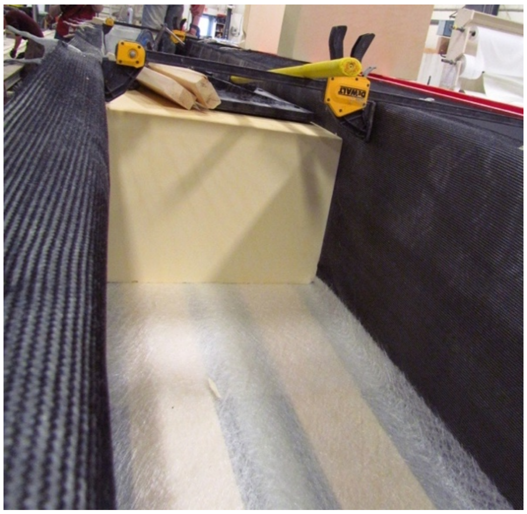

Figure 116: Foam inserts in HCB

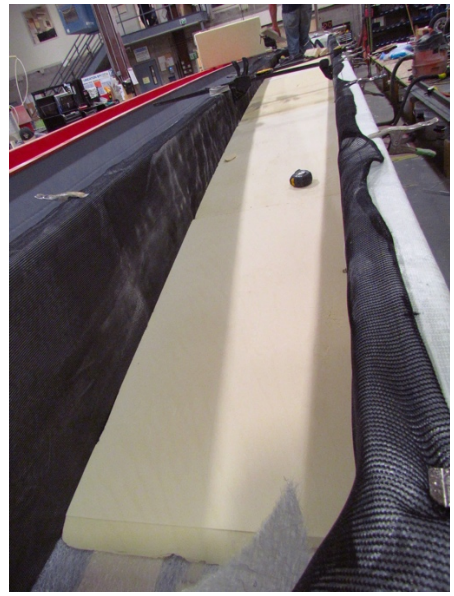

Figure 117: Foam inserts in HCB

An orange plastic inner vacuum bag was placed over the foam arch; the vacuum bag prevents resin from filling the concrete arch. After the foam was in place, a CFM layer and a shade cloth layer were adhered over the center of the foam arch. The fiberglass and flow media layers were folded over the top of the shell and adhered using the spray adhesive. Then a layer of blue release film and black shade cloth were placed over the top of the beam. HDPE plates were used as top forms for shell resin infusion. With the use of butyl putty (green putty) a blue vacuum bag was secured to the top form plates. Holes were then cut into the vacuum bag to allow for the resin supply lines; three supply lines were connected under the center of the beam shell (**Figure 118**).

A shop-vac was initially used to remove air from the vacuum bag, as a leak checker was passed over the beam to ensure proper vacuum bag installation. Once all leaks were sealed, the vacuum was turned on for at least an hour before a drop test was performed. A drop test is the monitoring of the drop-in pressure when the vacuum is turned off. A pressure of approximately 29 Hg is common, and a slight pressure drop is acceptable. The resin was measured out in 5-

gallon buckets with each bucket containing 25 lbs of resin. Approximately 325 lbs of resin were required per beam. Each bucket of resin was mixed with MEKP 925H hardener, at a ratio of 8cc of hardener per pound of resin. The mixed resin was then poured into a plastic trough with three supply lines connecting to the ports under the center of the beam form (**Figure 119**).

Figure 118: HCB resin infusion setup

Figure 119: Mixing of resin with hardener during resin infusion

The resin spread through the beam shell in a closed infusion process. Once the resin started to bubble up the supply lines, the lines were clamped using a vise grip to allow resin flow to another section of the shell. Resin flow in the HCB chimney is shown in **Figure 120**.

Figure 120: Resin flow in HCB chimney

After all supply lines were filled, the vacuum was turned off and the infused shell was allowed to sit overnight. After the resin hardened, the vacuum bag and HDPE plates were removed using the lift and a crowbar. The shell was lifted from the form by placing a 4x4 connected to a tow rope into each shell chimney, located at both ends of the beam. As the beam was lowered to a staging platform, support blocks were placed under the center of the beam. The release film layers were removed from the beam shell and the 4-in. foam channel along the shell centerline was cut out using a reciprocating saw (**Figure 121**).

Figure 121: Cutting 4-in. channel in HCB

The beams were moved to another staging area and carefully flipped over to apply a thick layer of gel coat using a paint roller; the gel coat is primarily for UV radiation protection (**Figure 122**).

Figure 122: Painting on gel coat

After drying, the beams were carefully flipped back over, and resin was applied to areas with imperfections. Four holes were drilled into each corner of the shell for future diaphragm reinforcement, and the top edges of the shell were lightly sanded to a smooth finish (**Figure 123**).

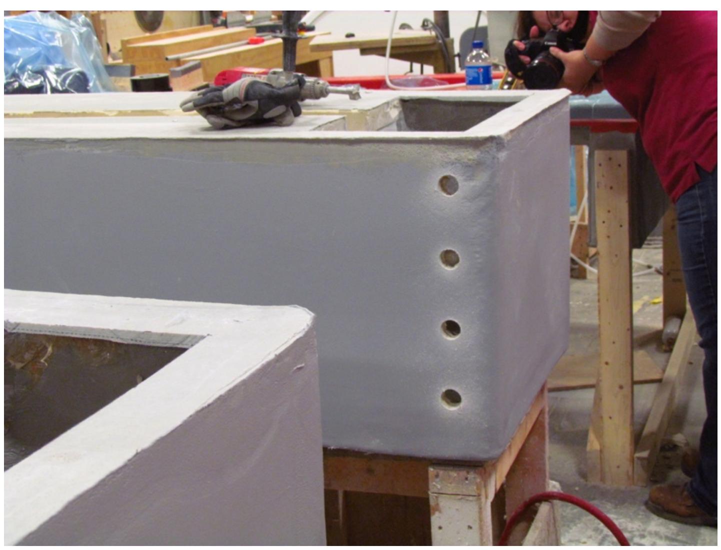

Figure 123: Drilled holes for diaphragm reinforcement

The FRP lids were composed of three layers of 102 oz quadraxial fiberglass and infused with resin in an open infusion process using a 25 to 30 in. mercury vacuum. The lid form was 80 ft long which allowed two lids to be fabricated at once. Using a paddle mixer, a hardener DDM-9 catalyst was mixed with the resin. Approximately 200 lbs of resin were required for the infusion of two fiberglass lids. A 5-gallon bucket with supply lines was connected to a series of resin flow piping to allow for lid infusion (**Figures 124** and **125**).

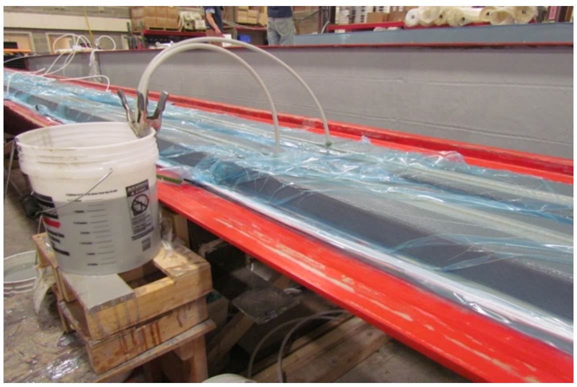

Figure 124: Resin infusion of FRP lids

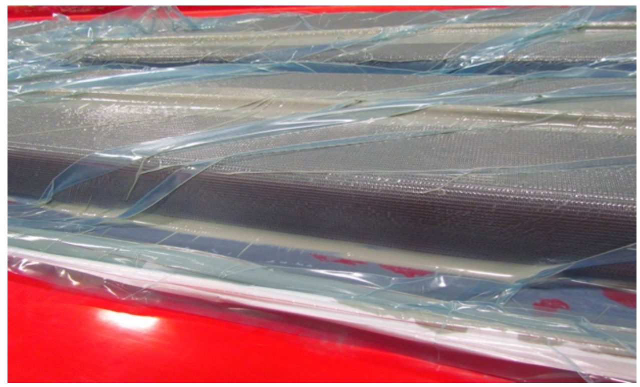

Figure 125: Resin infusion of FRP lids

The resin flow was monitored to ensure proper resin distribution; a vise grip was used to cut off resin flow and aid in distribution. Resin infusion takes place in approximately 20 minutes, but the resin requires at least 6 hours to harden before removing the lids from the form. After removing the vacuum bag and the resin supply lines, the lift was used to remove the flow media from the fiberglass lid. Using a 2x4, prybar, and a hammer, the fiberglass lids were removed from the form. The two lids were separated by cutting through the fiberglass with a jig-saw. The lids were then moved to a cutting area where holes were cut for shear connectors, concrete fill, vibrator holes, and lifting loop holes (**Figure 126**).

Two galvanized strands were placed in the beam shell, after which the lid was lifted and fit over the shell (**Figure 127**). After lifting the lid off the shell, metal ties were placed under the strands and fed through ports along the lid centerline, as the lid was lowered back into place.

Figure 126: Holes drilled into FRP lid

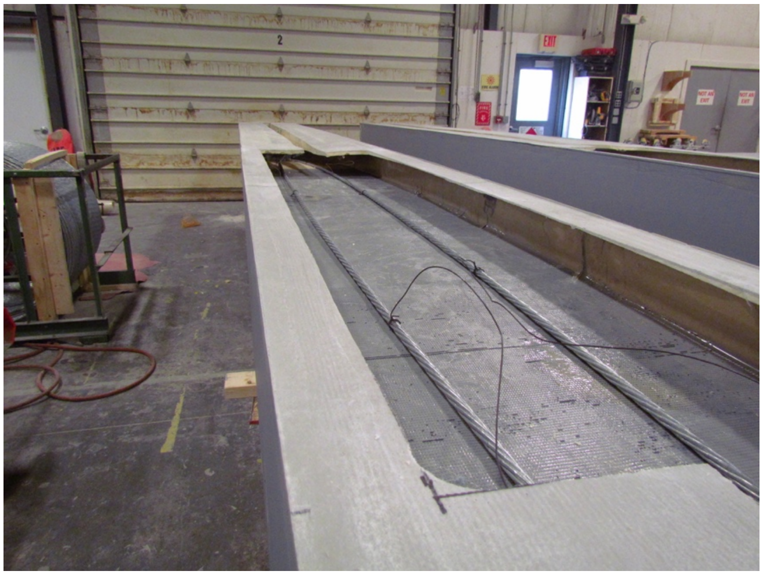

Figure 127: Galvanized strands with metal ties

Using an air hose, excess dust and foam pieces were removed from the beam shell. Along the edges of the shell, a side grip adhesive was dispensed using a two-part epoxy gun. The epoxy was composed of 55 gallons of resin and 5 gallons of hardener. After ensuring a proper bond between the lid and shell using clamps, set screws were screwed into the top of the lid along both sides of the beam. The metal wire ties were pulled up and twisted, and excess wire was cut using wire cutters. The presence of a built-in plastic bar on each end of the beam prevents the galvanized bars from being lifted past a certain point. The complete beams were marked at each foot along the top of the lid, and measurements were taken along the centerline of the beam. Beam sweep and Barcol readings were taken, and results were recorded. Barcol readings are a measurement of the hardness of the fiberglass. Wrapped in plastic, tied, and moved using a lull, the beams were stacked two beams high and stored outside. Ten (10) beams were shipped per truck, but 16 beams could potentially fit on one truck.

On April 15, 2017, the first ten (10) HCB beams were filled with self-consolidating concrete at CDS Manufacturing in Gretna, FL. Shear reinforcements were inserted into the pre-cut holes in the beam lid. The shear connectors were twisted under the two galvanized strands in the shell and lifted where the height from the lid to the top of the shear connector was 5 in. (**Figure 128**).

Figure 128: Placement of shear connectors

After ensuring proper placement, the shear connectors were tied to temporary rebar to stabilize the shear connectors during arch casting. While most of the shear connectors were hooked properly under the galvanized strands, a few shear connectors were not under the strands. Lifting loops were then installed at each end of the beam. Stabilized using temporary rebar, the diaphragm reinforcement was positioned and secured using spray foam (**Figure 129**).

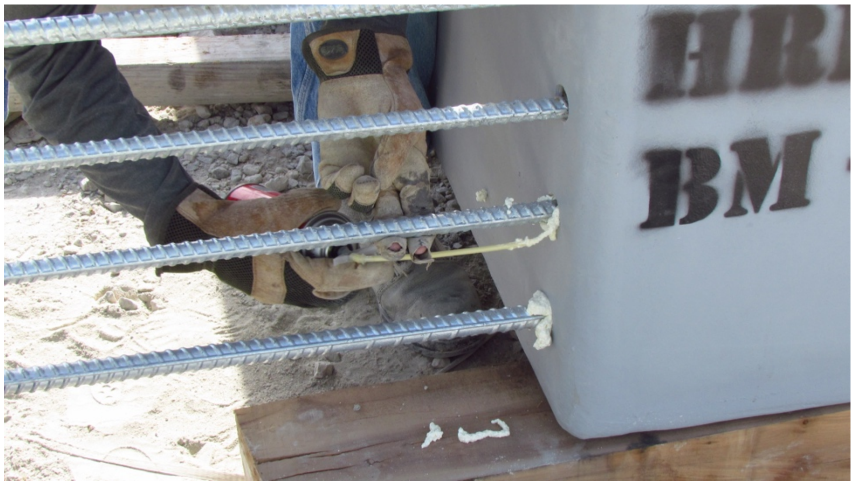

Figure 129: Foaming of diaphragm reinforcement

Plywood squares were screwed into beam lid to cover the self-consolidating concrete (SCC) fill holes and then the beams were shimmed up to provide a level lid surface (**Figure 130**).

Figure 130: Plywood squares to cover SCC fill holes

The self-consolidating concrete was tested for spread, flow, and air content. A J-ring test was performed to determine the spread and flow of the self-consolidating concrete. A spread close to 30 in. was desired for the concrete mix; the first mix was rejected due to a spread of only 26 in. x 26 ½ in. The second mix was accepted with a spread of 28 ½ in. x 29 in. Each concrete truck held 4 cubic yds of concrete and could potentially fill three (3) beams. Starting from the second concrete port from the beam end, a concrete hopper fit into the pre-cut concrete port (**Figure 131**). As the hopper was filled with concrete, the spread of the concrete was monitored from the holes along the beam centerline.

Figure 131: Hopper used to fill HCB

When the beam chimney filled with concrete, the plywood squares were turned to cover the holes and screwed into place to prevent concrete overflow. A mallet was struck against the beam lid to determine hollow areas that required more concrete (**Figure 132**).

Each beam required approximately 8 minutes to fill with concrete. If the concrete was not flowing properly, a rubber tipped vibrator was inserted into pre-cut holes along the lid centerline to aid in concrete flow (**Figure 133**). The vibrator was also inserted into the hopper to aid in concrete flow.

The first two concrete trucks were 4 cubic yds each, and each truck filled three beams, while the last truck was 6 cubic yds and filled the remaining four beams. The centerline channel on the lids was filled with concrete, and excess concrete was removed with a damp sponge (**Figure 134**).

An approximate measurement was taken from the ground to the bottom of the shell at the centerline of the beams before and after filling the beam with concrete, to approximate immediate beam settlement (**Table 14**).

Figure 132: Mallet used to strike FRP lid

Figure 133: Vibrator used to aid in concrete flow

Figure 134: Finishing work on HCB

Table 14: Approximate HCB settlement

Beam No.	Initial Midspan Measurement	Final Midspan Measurement	Settlement
HRB-02, BM1	6 ⅜ in.	6 ⅛ in.	¼ in.
HRB-08, BM-02B	8 ¾ in.	8 ¼ in.	½ in.
HRB-3, BM-04	10 ¾ in.	10 ¼ in.	½ in.
HRB-10, BM-05B	7 ½ in.	7 ⅛ in.	⅜ in.
HRB-07, BM-05	8 ⅞ in.	8 ⅜ in.	½ in.
HRB-5, BM-02	7 ¾ in.	7 ⅜ in.	⅜ in.
HRB-01, BM-04	8 ⅜ in.	7 ⅞ in.	½ in.

As of June 30, 2017, four (4) HCB beams were in place between bent 4 and 5. From north to south, the beams were labeled according to **Table 15** and shown in **Figure 135**.

Table 15: Beam labeling between bent 4 and 5

HRB Number	Beam Number
HRB-01	BM-4
HRB-4	BM-05
HRB-07	BM-05
HRB-10	BM-05B

Figure 135: HCB Beams between Bents 4 and 5

According to RFI 0039, witness panels tested by FDOT Materials Office resulted in a longitudinal tensile strength and longitudinal tensile modulus below the values shown in TSP-T-450. The values obtained from the test results are a longitudinal tensile strength of 33 ksi and a longitudinal tensile modulus of 2,300 ksi.

Structural panels of HCB laminate without the continuous filament mat (CFM) layer were tested by the manufacturer, FDOT, and an independent laboratory. Witness panels were made including the CFM layer and tested by FDOT State Materials Office for beams 1 through 20. The panels were tested based on ASTM D3039 Tensile Properties of Polymer Matrix Composite Materials in the longitudinal (0 degree) direction. **Figure 136**, provided by FDOT State Materials Office, displays both sets of tested panel data. As shown in the figure, there is no significant increase in longitudinal or transverse capacity due to the presence of the CFM layer.

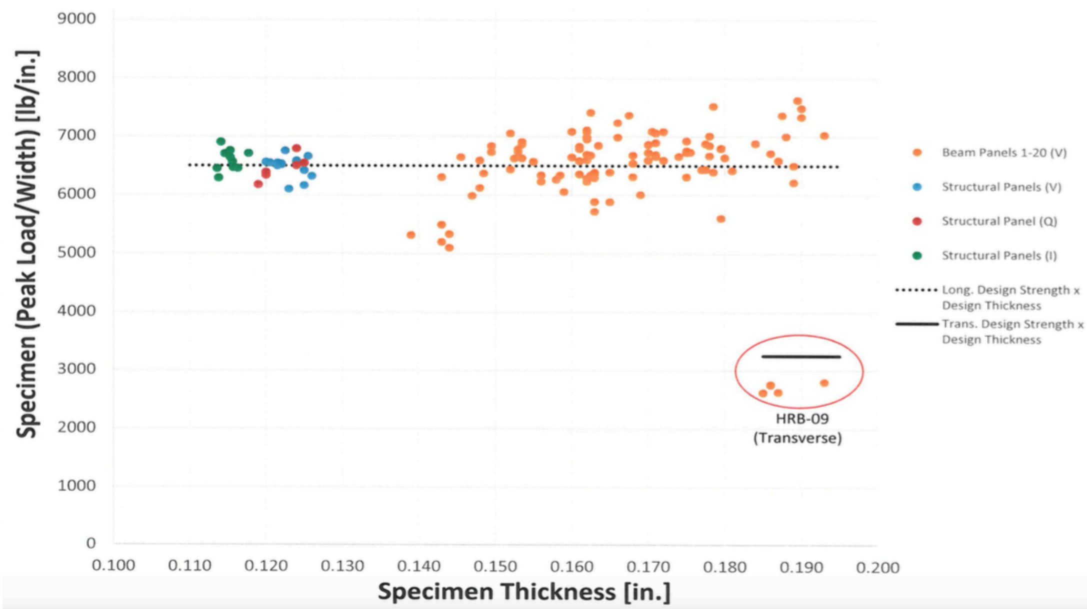

Figure 136: HCB panels without CFM layer

According to RFI 0039, witness panels tested by FDOT Materials Office resulted in a longitudinal tensile strength and longitudinal tensile modulus below the values shown in TSP-T-450. The average values obtained from the test results are a longitudinal tensile strength of 33,000 psi and a longitudinal tensile modulus of 2,300 ksi, which are below the minimum mechanical properties shown in **Table 16**. The ultimate tensile strength of the witness panels should exceed 39,877 psi. FDOT's testing resulted in the average tensile strength of HRB 01, 03, 04, 05, 06, 07, 08, 09, 10, 11, and 15 below the normalized ultimate tensile strength. The minimum required values are based on the design thickness (0.13 in.), minimum tensile strength, and modulus requirements of T450-13.7.3. Five (5) witness panels were tested per beam, excluding Beam 9 where ten (10) witness panels were tested, and the average values are shown in **Table 17**.

Table 16: Minimum mechanical properties (T450-13.7.3)

	Longitudinal	Transverse
Tensile strength (psi)	50,000	25,000
Tensile modulus (ksi)	3,100	2,680

Table 17: Average values of witness panel results for HRB-01 through HRB-20

Beam No.	Thickness (in)	Ultimate (lbf)	Ultimate (psi)	Modulus (Mpsi)	Load/Width (lb/in)	Modulus* Thickness (k/in)
HRB-01	0.165	5,676	36,262	2,526	5,958	416
HRB-02	0.170	6,646	40,729	2,880	6,922	490
HRB-03	0.143	5,102	37,095	3,278	5,288	467
HRB-04	0.168	6,256	39,363	2,928	6,624	493
HRB-05	0.161	6,068	39,464	3,046	6,338	489
HRB-06	0.161	6,126	39,304	2,944	6,319	473
HRB-07	0.189	6,387	34,997	2,546	6,608	481
HRB-08	0.173	6,422	39,107	2,917	6,749	504
HRB-09	0.181	6,748	35,550	2,661	6,418	480
HRB-10	0.176	6,359	37,302	2,786	6,546	489
HRB-11	0.178	6,581	38,879	2,748	6,928	490
HRB-12	0.171	6,604	40,304	2,941	6,874	502
HRB-13	0.162	6,788	43,219	2,993	6,996	485
HRB-14	0.162	6,397	40,954	3,067	6,611	496
HRB-15	0.189	7,251	38,945	2,724	7,361	515
HRB-16	0.161	6,404	40,618	3,123	6,520	502
HRB-17	0.164	7,162	43,643	3,064	7,149	502
HRB-18	0.146	6,304	43,572	3,344	6,378	490
HRB-19	0.152	7,153	45,190	3,148	6,846	477
HRB-20	0.152	6,873	43,063	3,211	6,543	488

Kenway Corporation believed the reduced strength of HRB-03 was due to a missing fiberglass layer. Beam 9 (HRB-09) low values could be due to incorrect labeling of the panel fiber direction thus the results for HRB-09 correspond to transverse tensile properties. FDOT State Materials Office and Kenway Corporation performed burn-off tests to determine the fiberglass structure of the witness panels for beams HRB-03 and HRB-09. Kenway performed the burn-off test by placing the 2 ¾ in. x 3 in. samples into a muffle furnace set to a temperature of 1050°F, removing the samples after four hours, and allowing them to cool overnight, as shown in **Figures 137** and **138**. Kenway reported that the burn-off test of HRB-03 witness panel revealed the 24 oz biaxial E-BX-2400 layer was missing. A summary of the thickness and weights of the specimens tested by Kenway Corporation are shown in **Table 18**.

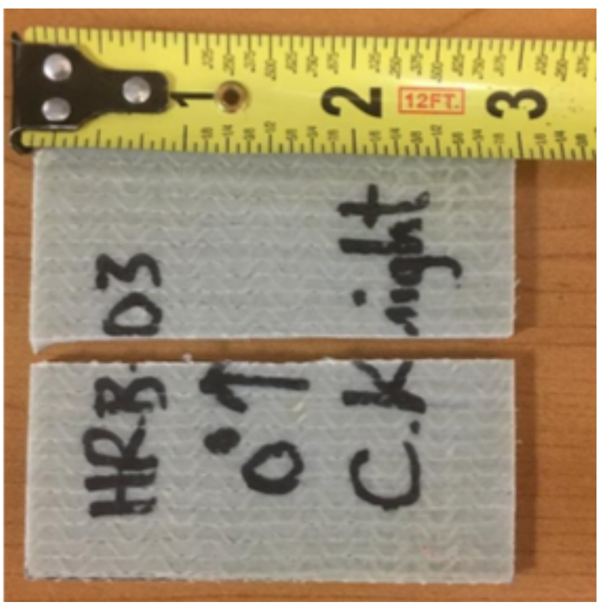

Figure 137: Samples cut for testing (Kenway Corp.)

Figure 138: Samples deconstructed after testing (Kenway Corp.)

Table 18: Summary of samples tested by Kenway Corp.

	Average Thickness (in)	Weight pre burn off (g)	Weight post burn off (g)	Percentage Resin by Weight (%)	Percentage Glass by weight (%)
HRB-03	0.134	15.632	11.455	26.72	73.28
HRB-09	0.172	19.091	12.457	34.75	65.25

FDOT placed samples in a muffle furnace set to 550°C (1022°F) for two hours to burn off the resin. FDOT's results of the burn-off test for beams HRB-03 and HRB-09 were fiberglass layers consistent with the shop drawings (HRB-03 was not missing a 24 oz layer) and confirmation that HRB-09 primary fiber direction was perpendicular to the marked direction, as shown in **Figures 139-145**.

Figure 139: FDOT samples after testing (FDOT)

Figure 140: FDOT samples 1st 102 oz layer removed (FDOT)

Figure 141: FDOT samples 2nd 102 oz layer removed (FDOT)

Figure 142: FDOT samples 3rd layer 102 oz removed (FDOT)

Figure 143: FDOT samples 4th layer 102 oz removed (FDOT)

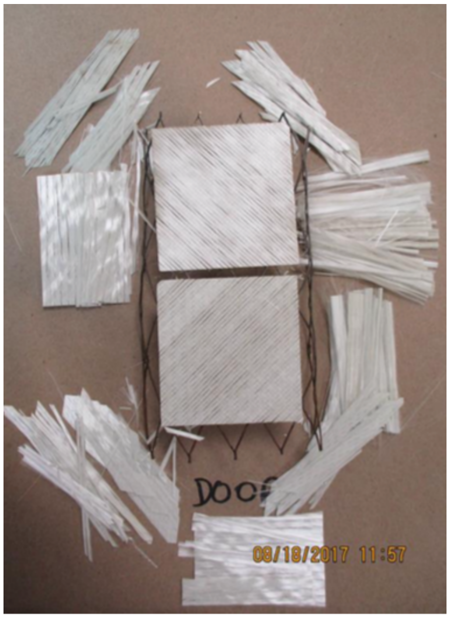

Figure 144: FDOT samples 1st layer 24 oz removed (FDOT)

Figure 145: FDOT samples 2nd layer 24 oz removed (FDOT)

Kenway Corporation submitted two random samples to Intertek testing laboratory. The samples consisted of one (1) layer of 1 oz CFM, one (1) layer of 102 oz quadriaxial, and one (1) layer of 24 oz biaxial fiberglass, for a total sample thickness of approximately 0.163 in. Due to the non-structural properties of the CFM layer, Kenway Corporation neglects the CFM layer when calculating tensile strength, thus the sample thickness would be considered as 0.130 in. FDOT specifications require a minimum tensile strength of 50,000 psi for a 0.130-in. laminate consisting of one (1) layer of 24 oz biaxial and one (1) layer of 102 oz quadriaxial fiberglass. Using the specification provided by FDOT and neglecting the CFM layer, the failure load and normalized minimum tensile strength can be calculated as shown below.

$$Failure\ load\ per\ inch = 50{,}000psi * 0.130in * 1in = 6{,}500\ lb$$

$$Normalized\ minimum\ tensile\ strength = \frac{6{,}500lb}{0.163in * 1in} = 39{,}877\ psi$$

Kenway Corporation is claiming the tensile strength of the fiberglass laminate should meet FDOT requirements if the samples exceed a tensile strength of 39,877 psi. The testing by

Intertek yielded an ultimate tensile strength of 40,300 psi for HRB-03 and a strength of 41,100 psi for HRB-10. Kenway Corporation claims the cutting method and edge preparation of the sample before testing could impact the test results.

As requested by Kenway Corporation, FDOT approved the use of tensile capacity instead of tensile strength for beam acceptance. Beam specimen variations were assessed based on ultimate load per inch (lb/in.). Based on the analysis criteria of design strength and thickness, HRB-01, 03, 05, 06, 09, and 18 did not meet the minimum required capacity of 6,500 lb/in. After confirming HRB-09 was incorrectly labeled, the panel still failed to meet minimum capacity. FDOT has received quality control results for HRB-03 and HRB-10 indicating that HRB-03 does not meet minimum capacity and HRB-10 does meet minimum capacity, as confirmed by FDOT's QA results. HRB-03 will require an EAR to support a claim of meeting adequate structural requirements.

Kenway Corporation contracted Garcia Bridge Engineers P.A. to evaluate the beam data provided by FDOT's Engineer of Record (EOR); the data includes the EOR's calculations and the witness panel laminate tests from FDOT's State Materials Office. When considering the Special Provision Section T450 minimal tensile stress of 50 ksi for the HCB laminate, the beams in question fail based on FDOT's laminate testing. The ultimate tensile stress of the beams is calculated using the longitudinal modulus of elasticity of 3100 ksi and the maximum usable FRP strain of 0.013.

$$Ultimate\ Tensile\ Stress\ (\sigma) = \epsilon_{FRP}E_{11} = 3100ksi\ x\ 0.013 = 40.3ksi$$

Using the thickness of the laminate, 0.130 in., the ultimate tensile stress per unit width is calculated.

$$Ultimate\ Tensile\ Stress\ per\ unit\ width = \sigma\ x\ t = 40.3ksi\ x\ 0.130\ in = 5.239k/in$$

When considering the ultimate tensile stress per unit width, HRB-01 through HRB-20 meet the minimum strength requirements, as shown in **Figure 146**. According to Garcia Bridge

Engineers P.A. the beam laminate exceeds the design requirements and should perform as designed.

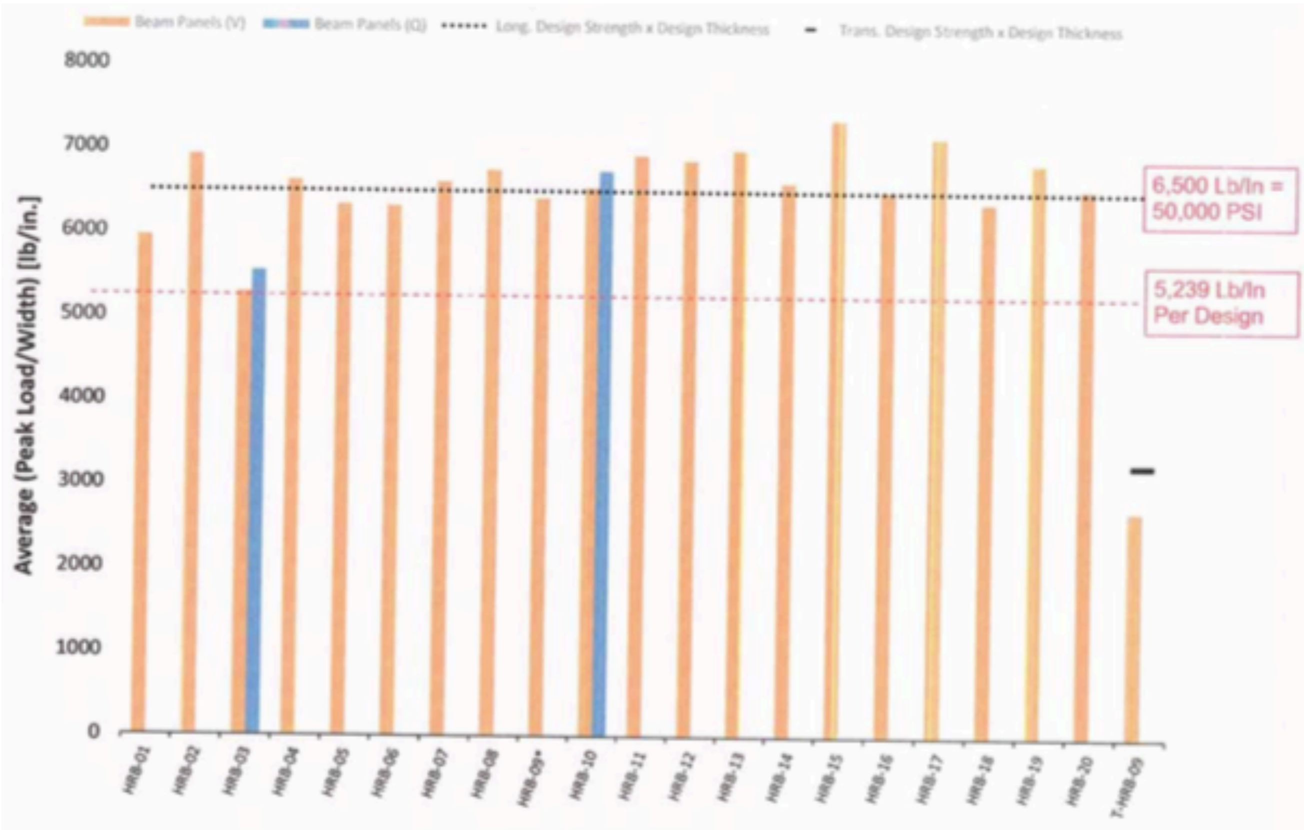

Figure 146: Average load per unit width of HRB-01 through HRB-20

FDOT State Materials Office completed testing of the 45 Hybrid Composite Beams fabricated at Kenway Corporation for the Halls River Bridge Project. **Table 19** shows the testing results of the beam witness panels including the thickness, ultimate load, ultimate strength, modulus, the load per unit width, and the modulus per unit width. The percent failure, in the last column of Table 19, is the percentage that the witness panel tested below the normalized ultimate tensile strength of 39,877 psi. The Hybrid Composite Beams that tested below the normalized tensile strength include beams HRB-01, 03, 05, 06, 09, 18, 21, 22, 25, 27, 28, 29, 30, 33, 34, 35, 36, 38, 40, and 41. FDOT has accepted the beams for construction based on the report and recommendations from Garcia Bridge Engineers P.A.

Table 19: HCB witness panel results (FDOT State Materials Office)

Beam No.	Thickness (in)	Ultimate Load (k)	Ultimate Strength (ksi)	Modulus (ksi)	Load/ inch of Width (kips/in)	Modulus/ inch of Width (kips/in)	Percent Failure
HRB-01	0.165	5.68	36.3	2526	5.958	416	8.34%
HRB-02	0.170	6.65	40.7	2880	6.922	490	
HRB-03	0.143	5.10	37.1	3278	5.288	467	18.64%
HRB-04	0.168	6.26	39.4	2928	6.624	493	
HRB-05	0.161	6.07	39.5	3046	6.338	489	2.50%
HRB-06	0.161	6.13	39.3	2944	6.319	473	2.79%
HRB-07	0.189	6.39	35.0	2546	6.608	481	
HRB-08	0.173	6.42	39.1	2917	6.749	504	
HRB-09	0.187	2.68	14.5	1041	2.712	195	1.26%
HRB-10	0.176	6.36	37.3	2786	6.546	489	
HRB-11	0.178	6.58	38.9	2748	6.928	490	
HRB-12	0.171	6.60	40.3	2941	6.874	502	
HRB-13	0.162	6.79	43.2	2993	6.996	485	
HRB-14	0.162	6.40	41.0	3067	6.611	496	
HRB-15	0.189	7.25	38.9	2724	7.361	515	
HRB-16	0.161	6.40	40.6	3123	6.520	502	
HRB-17	0.164	7.16	43.6	3064	7.149	502	
HRB-18	0.146	6.30	43.6	3344	6.378	490	1.88%
HRB-19	0.152	7.15	45.2	3148	6.846	477	
HRB-20	0.152	6.87	43.1	3211	6.543	488	
HRB-21	0.147	6.5	43	3245	6.303	476	3.04%
HRB-22	0.153	6.6	40.5	3170	6.219	488	4.33%
HRB-23	0.171	7.32	40.9	2944	6.967	502	
HRB-24	0.165	6.96	40.6	2932	6.718	485	
HRB-25	0.152	6.34	40.3	3354	6.105	508	6.07%
HRB-26	0.155	6.94	43.4	3191	6.709	493	
HRB-27	0.138	5.86	41	3281	5.664	451	12.86%
HRB-28	0.146	6.28	41.7	3197	6.1	467	6.15%
HRB-29	0.142	6.71	45.5	3365	6.461	478	0.60%
HRB-30	0.145	6.39	43	3300	6.224	478	4.25%
HRB-31	0.148	7.18	47	3303	6.966	489	
HRB-32	0.159	6.85	41.8	3078	6.638	489	
HRB-33	0.149	6.15	40.7	3220	6.041	478	7.06%
HRB-34	0.149	6.54	43.1	3169	6.413	472	1.34%
HRB-35	0.151	6.24	39.5	3060	5.956	459	8.37%
HRB-36	0.146	6.46	43.2	3182	6.315	465	2.85%
HRB-37	0.151	6.84	43.7	3284	6.603	496	
HRB-38	0.145	6.7	44.3	3348	6.401	484	1.52%
HRB-39	0.149	7.17	45.9	3156	6.84	470	
HRB-40	0.151	5.52	35.7	3186	5.389	481	17.10%
HRB-41	0.141	5.96	40.6	3327	5.727	470	11.90%
HRB-42	0.149	7.11	46.8	3314	6.962	493	
HRB-43	0.147	6.87	45.6	3373	6.674	494	
HRB-44	0.154	7.19	43.7	3140	6.741	485	
HRB-45	0.158	6.88	41.9	3129	6.632	495	

The technical specifications for the HC Beams (T450-13.7.2) shown in **Table 20** allow for a camber variation of ± ⅛ in. per 10 LF tolerance or a maximum of + 1 in. from the design camber. Maximum camber would be at midspan, which is approximately 18 LF from the beam end which allows for a tolerance of less than ¼ in. at midspan.

Table 20: Maximum allowable dimensional tolerances for HCB components (from FDOT)

HCB Component	Inches (except as noted)
Depth, overall (Bottom Shell)	± ¼ in.
Width, overall	± ¼ in.
Length (string line measurement along bottom of beam)	± ¼ in per 25 feet max ± ½ in.
Variation from specified elevation and squareness or skew	± 1/8 in. per 12" depth max ± 3/8 in.
Sweep	± ½" maximum
Camber variation from design camber (Bottom Shell)	-1/8 in.; +1/8 in. per 10 ft, +1 in. Max
Tipping and flushness of beam seat bearing area	± 1/8 in. per 24 inches
Shear reinforcing; longitudinal location	± 1/4 in.
Shear reinforcing; projection from beam surface	+ 1/4 in., - 1/4 in.

The camber measurements taken at CDS Manufacturing Inc. in Gretna, FL, are shown in **Table 21**. Based on the measurements from CDS, some of the beams exceed the maximum allowable camber specified in T450-13.7.2. As stated in Request for Correction (RFC) 0003, Astaldi proposed to adjust the beam build-up to meet the camber values of the contract plans. The corrected build-up of the beams in question are displayed in **Table 22**.

As of December 11, 2017, the beams between bents 2 and 3, 3 and 4, and 4 and 5 were installed. Listed from North to South, the beams installed are labeled in **Table 23**. **Figures 147** and **148** show the installed beams between bents 2 through 5.

Table 21: Camber measurements from CDS Manufacturing Gretna, FL

Beam #	Cast Date	Lot#	End 1		Center		End 2		Camber	Temperature		Ship Date
			feet	inches	feet	inches	feet	inches	(inches)	Beam Exterior	Ambient	
HRB-01 BM-04	4/15/17	1							1 12/16	85 F	90 F	5/2/17
HRB-02 BM-01	4/15/17	1	3	6 7/8	3	6 7/8	3	10 7/8	2	104 F	90 F	
HRB-03 BM-04	4/15/17	1	2	7 1/8	2	8 1/4	3	5/8	1 10/16	106 F	90 F	
HRB-04 BM-05	4/15/17	1							1 10/16	85 F	90 F	5/2/17
HRB-05 BM-02	4/15/17	1	3	7 5/8	3	8 7/8	4	2 1/2	2 3/16	105 F	90 F	
HRB-06 BM-02	4/15/17	1	3	6 1/2	3	6 1/4	3	10 5/8	2 5/16	107 F	90 F	
HRB-07 BM-05	4/15/17	1							2 1/16	85 F	90 F	5/2/17
HRB-08 BM-02B	4/15/17	1	2	10 3/4	2	10 1/2	3	2 3/8	2 1/16	106 F	90 F	
HRB-09 BM-01	4/15/17	1	3	3 5/8	3	5 1/2	3	10 7/8	1 12/16	106 F	90 F	
HRB-10 BM-05B	4/15/17	1							2 1/16	85 F	90 F	5/2/17
HRB-11 BM-02	4/29/17	2	2	6	2	5 1/2	2	8	1 8/16	107 F	90 F	
HRB-12 BM-05	4/29/17	2	2	5 1/8	2	3 7/8	2	6 1/2	1 15/16	107 F	90 F	
HRB-13 BM-02	4/29/17	2	2	8 5/8	2	5 3/4	2	6 1/2	1 13/16	107 F	90 F	
HRB-14 BM-05	4/29/17	2	2	6 3/8	2	4 7/8	2	6 3/4	1 11/16	106 F	90 F	
HRB-15 BM-02B	4/29/17	2	2	9 3/4	2	7 1/4	2	8 1/4	1 12/16	104 F	90 F	
HRB-16 BM-05B	4/29/17	2	2	9	2	6 3/8	2	7 3/4	2	104 F	90 F	
HRB-17 BM-04	4/29/17	2	2	4 5/8	2	4	2	7	1 13/16	102 F	90 F	
HRB-18 BM-05	4/29/17	2	2	3 3/8	2	2 13/16	2	5 7/8	1 13/16	102 F	90 F	
HRB-19 BM-05	4/29/17	2	2	8	2	10	3	3 7/8	1 14/16	105 F	90 F	
HRB-20 BM-05	4/29/17	2	2	9	2	10	3	3 1/4	2 3/16	106 F	90 F	

Table 22: Corrected build-up for hybrid composite beams (Phase II)

BEAM ID	CAMBER (in)	PLAN BUILD UP (in)	SHOP-DRAWING BUILD UP (in)	CORRECTED BUILD UP (in)
HRB-14 BM-05	1.69	0.98	1.13	0.94
HRB-12 BM-05	1.94	0.98	1.13	0.69
HRB-13 BM-02	1.81	0.98	1.13	0.81
HRB-15 BM-02B	1.75	0.98	1.13	0.87
HRB-16 BM-05B	2.00	0.98	1.13	0.62
HRB-17 BM-04	1.84	0.98	1.13	0.78
HRB-18 BM-05	1.81	0.98	1.13	0.81
HRB-20 BM-05	2.16	0.98	1.13	0.47
HRB-19 BM-05	1.91	0.98	1.13	0.72
HRB-11 BM-02	1.53	0.98	1.13	1.09
HRB-4 BM-05	1.61	0.98	1.13	1.01
HRB-9 BM-01	1.75	0.98	1.13	0.87
HRB-6 BM-02	2.31	0.98	1.13	0.31
HRB-02 BM-01	2.00	0.98	1.13	0.63
HRB-3 BM-04	1.63	0.98	1.13	1.00
HRB-08 BM-02B	2.06	0.98	1.13	0.56
HRB-01 BM-04	1.77	0.98	1.13	0.86
HRB-5 BM-02	2.19	0.98	1.13	0.44
HRB-07 BM-05	2.03	0.98	1.13	0.59
HRB-10 BM-05B	2.04	0.98	1.13	0.58

Table 23: Beams installed

Bents		Beam No.
2	3	HRB-17, BM-04
		HRB-16, BM-05B
		HRB-12, BM-05
		HRB-14, BM-05
3	4	HRB-03, BM-04
		HRB-20, BM-05B
		HRB-18, BM-05
		HRB-19, BM-05
4	5	HRB-01, BM-4
		HRB-04, BM-05
		HRB-07, BM-05
		HRB-10, BM-05B

Figure 147: HCBs installed

Figure 148: HCBs tied down

On March 15, 2018, eight (8) HC beams were shipped from CDS Manufacturing in Gretna, FL, to Halls River Bridge in Homosassa Springs, FL. Four (4) beams were shipped on each truck, shown in **Figure 149**, and traffic was stopped to unload the beams. Chains were hooked to the installed lifting loops on each end of the beams and lifted with the crane as shown in **Figure 150**. The first four (4) beams were unloaded on the west side of the bridge, and the second set of beams were loaded onto a trailer to later be unloaded on the east side of the bridge.

Figure 149: Beams arriving at Halls River Bridge

Figure 150: Beams lifted using crane

Upon inspection, one beam (HRB-02, BM-01) displayed surface flaws including an abrasive surface patch, dripping lid sealant, and unpainted beam labels (**Figures 151-153**).

Figure 151: Unpainted beam labels on HRB-02

Figure 152: Abrasive patch on HRB-02

Figure 153: Dripping sealant on HRB-02

After a visual inspection, the beams on the west side of the bridge were lifted into place between bents 1 and 2. The location of the installed beams are listed in **Table 24**. The beams were placed on the bearing pads with 8 in. from the front face of the backwall to the face of the beam and 6 in. from the beam edge to the center of the bearing pad. The placement of the beams is shown in **Figures 154** and **155**, and the installed beams are shown in **Figures 156** and **157**.

Table 24: Beams installed

Bents		Beam No.
1	2	HRB-02, BM-01
		HRB-08, BM-02B
		HRB-06, BM-02
		HRB-05, BM-02
2	3	HRB-17, BM-04
		HRB-16, BM-05B
		HRB-12, BM-05
		HRB-14, BM-05
3	4	HRB-03, BM-04
		HRB-20, BM-05B
		HRB-18, BM-05
		HRB-19, BM-05
4	5	HRB-01, BM-4
		HRB-04, BM-05
		HRB-07, BM-05
		HRB-10, BM-05B

Figure 154: HC Beams being installed

Figure 155: HC Beams being installed

Figure 156: Installed HC Beams

Figure 157: Installed beam on bearing pad

During the installation of a beam between Bents 5 and 6, the FRP beam lid was scraped with the crane. The stay-in-place (SIP) forms were removed to access the damaged area, and as shown in **Figure 158**, there was partial delamination of the FRP. Kenway Corporation provided a piece of Vectorply EQX-10200 for the lid repair, and Astaldi Corporation used Derakane 8084 Epoxy Vinyl Ester Resin. The repair procedures, provided by Kenway Corporation, were as follows: grind the damaged laminate and remove delaminated pieces; use 35 grit discs to roughen the surface; clean the surface with acetone; apply two layers of EXQ10200 with 0-degree ply along longitudinal axis with the fabric extending 4 in. past the damaged area longitudinally and 8 in. toward the center of the beam lid; and keep the laminate clean from dirt until fully cured. The patched beam is shown in **Figure 159**.

Since the repair, the black plastic covering the repair patch has been removed; the patch is shown in **Figures 160** and **161**.

On April 20, 2018, CDS Manufacturing in Gretna, FL, filled 12 HC beams with self-consolidating concrete. HRB beam numbers poured included 30, 32, 36, 37, 38, 39, 40, 41, 42, 43, 44, and 45. The beams were supported on both ends using 4x4 wooden blocks and spaced to allow workers easy access to the concrete fill holes located on the top of the beams, shown in **Figure 162**.

Figure 158: Damaged HCB lid

Figure 159: Patched HCB lid

Figure 160: HC Beam patch

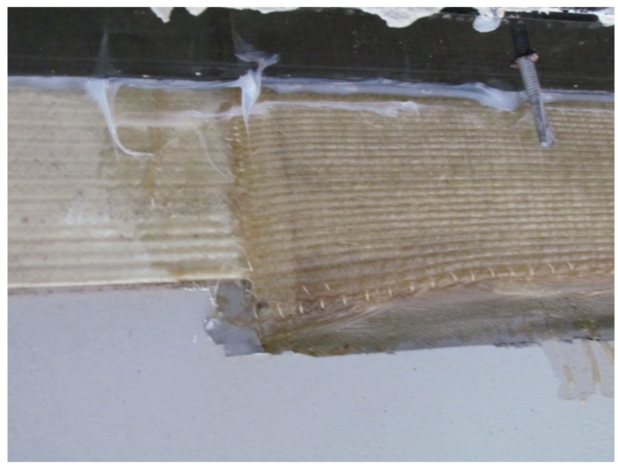

Figure 161: HC Beam patch

Figure 162: HC beams supported by 4x4

Unlike the first HC beam pour where rebar was used to hold the shear connectors at a height of 5 in., wooden blocks were used to position the shear connectors. As shown in **Figure 163**, some of the shear connectors were positioned above the wooden blocks resulting in non-uniform height and position of the shear connectors.

Figure 163: Uneven shear connectors

A spread of at least 29 in. was required for easy flow of the self-consolidating concrete in the HC beam compression arch. The spread, temperature, and air content were tested and recorded for each concrete batch. The spread of the concrete is shown in **Figures 164** and **165**, and the results of the concrete tests are shown in **Table 25**.

Figure 164: Testing for SCC spread

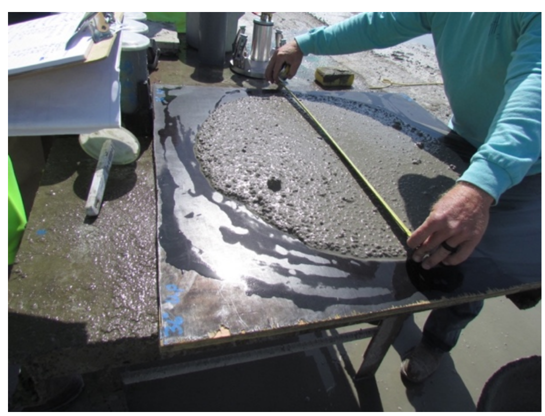

Figure 165: Measuring spread of SCC

Table 25: Concrete data for HCBs

Truck No.	Beams Filled	Unit Weight	Temperature	Spread	Air	W/C	Water Added
1	HRB-40, 42, 37	135.4 pcf	66°F	29 in.	5.2%	0.29	3 gal
2	HRB-39, 41, 43	139.0 pcf	70°F	30.5 in.	3.4%	0.30	5 gal
3	HRB-44, 38, 45	139.8 pcf	74°F	31 in.	2.0%	0.31	5 gal
4	HRB-30, 32, 36	136.6 pcf	76°F	29.5 in.	3.7%	_	5 gal

Similar to the first concrete pour in April 2017, a hopper was inserted into the holes drilled into the fiberglass lid of the HC beams, as shown in **Figure 166**. The spread of the concrete negated the use of a vibrator during filling of the beams. In the first pour, a vibrator was required to help spread the concrete throughout the beam arch. **Figures 167** and **168** show the filling of the beam chimney with self-consolidating concrete.

Figure 166: Hopper used to fill HCBs

Figure 167: Filling of HCB chimney with SCC

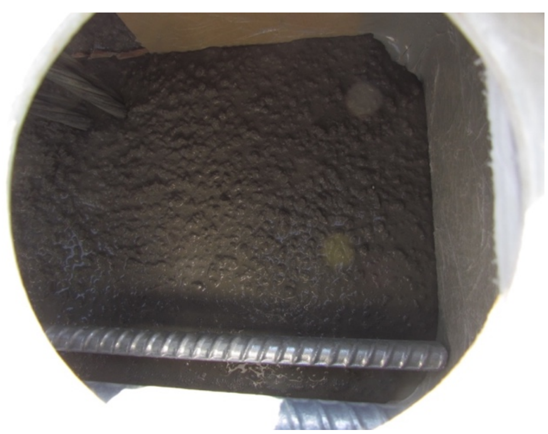

Figure 168: Filling of HCB chimney with SCC

The allowable tolerance of camber is ± ⅛ in. per 10 ft or a camber of ± 0.42 in. for each beam. **Table 26** lists the recorded camber values for Phase III beams. The maximum camber is 2 $^{3}/_{16}$ in. in beam HRB 42 BM-05, which equated to an adjusted build-up of 0.47 in. To compute the adjusted build-up value, the shop drawing build-up is subtracted from the difference between the recorded camber and the computed deflection. The computed deflection in the contract plans is 1.5 in. The equation for the adjusted build-up is shown below:

$$Adjusted\ build-up = Shop\ drawing\ build\ up - [Recorded\ camber - Computed\ deflection]$$

Table 26: Camber Phase III HCBs (Astaldi Construction)

BEAM #	Rod Reading (Ft)			Camber (Inches)	Build up		
	END	MID	END		Plan	Shop Dwg	Adjusted
HRB 37 BM-05	4.39	4.11	4.14	1.86	0.98	1.13	0.77
HRB 42 BM-05	4.40	4.09	4.14	2.16	0.98	1.13	0.47
HRB 40 BM-06	4.37	4.09	4.13	1.92	0.98	1.13	0.71
HRB 43 BM-05	4.34	3.99	3.99	2.10	0.98	1.13	0.53
HRB 41 BM-05	4.34	3.96	3.91	1.98	0.98	1.13	0.65
HRB 39 BM-05	2.82	3.03	3.50	1.63	0.98	1.13	1.00
HRB 45 BM-06	4.40	3.88	3.67	1.86	0.98	1.13	0.77
HRB 38 BM-03	4.46	3.91	3.68	1.92	0.98	1.13	0.71
HRB 33 BM-05	4.37	3.96	3.84	1.74	0.98	1.13	0.89
HRB 29 BM-05	4.29	4.16	4.34	1.86	0.98	1.13	0.77
HRB 34 BM-02	4.30	4.20	4.38	1.68	0.98	1.13	0.95
HRB 31 BM-06	4.82	4.36	4.20	1.80	0.98	1.13	0.83
HRB 44 BM-05	4.62	4.24	4.20	2.04	0.98	1.13	0.59
HRB 36 BM-02	4.54	4.15	4.04	1.68	0.98	1.13	0.95
HRB 32 BM-02	4.61	4.18	4.04	1.74	0.98	1.13	0.89
HRB 30 BM-02	4.61	4.20	4.11	1.92	0.98	1.13	0.71
HRB 21 BM-02	4.51	4.21	4.14	1.38	0.98	1.13	1.25
HRB 24 BM-02	3.98	3.86	3.99	1.50	0.98	1.13	1.13
HRB 23 BM-05	4.16	3.88	3.86	1.56	0.98	1.13	1.07
HRB 35 BM-05	4.80	4.20	3.92	1.92	0.98	1.13	0.71
HRB 28 BM-03	4.80	4.18	3.86	1.80	0.98	1.13	0.83
HRB 25 BM-05	4.70	4.00	3.58	1.68	0.98	1.13	0.95
HRB 27 BM-05	2.91	3.25	3.92	1.94	0.98	1.13	0.69
HRB 22 BM-02	4.52	4.06	3.86	1.56	0.98	1.13	1.07
HRB 26 BM-02	4.40	4.01	3.91	1.74	0.98	1.13	0.89

All the values are in inches.

The hybrid composite beams for Phase III were installed, as shown in **Figure 169**. The beam installation procedure was the same used for Phase II beams. The beams were unloaded and placed on bearing pads using a two-point pick up from the beam ends. The locations of beams as installed for Phase III are shown in **Table 27**. Inspectors noticed superficial cracking of the gel coat on beams 1- 4 from Phase II. Although the gel coat is superficial, it is a moisture barrier between the fiberglass shell and the environment.

Figure 169: Phase III installed HCBs (Figure provided by FDOT)

Table 27: Location of HCBs for Phase III

Span/Location	HCB #
Span 1 Beam 5	HRB - 36
Span 1 Beam 6	HRB - 21
Span 1 Beam 7	HRB - 34
Span 1 Beam 8	HRB - 30
Span 1 Beam 9	HRB - 28
Span 2 Beam 5	HRB - 44
Span 2 Beam 6	HRB - 39
Span 2 Beam 7	HRB - 33
Span 2 Beam 8	HRB - 29
Span 2 Beam 9	HRB - 31
Span 3 Beam 5	HRB - 41
Span 3 Beam 6	HRB - 23
Span 3 Beam 7	HRB - 35
Span 3 Beam 8	HRB - 42
Span 3 Beam 9	HRB - 40
Span 4 Beam 5	HRB - 25
Span 4 Beam 6	HRB - 37
Span 4 Beam 7	HRB - 27
Span 4 Beam 8	HRB - 43
Span 4 Beam 9	HRB - 45
Span 5 Beam 5	HRB - 22
Span 5 Beam 6	HRB - 24
Span 5 Beam 7	HRB - 32
Span 5 Beam 8	HRB - 26
Span 5 Beam 9	HRB - 38

4.7 GFRP Reinforced Diaphragms

As stated in RFM 0010, the 6D1 stirrups required for the end diaphragms were fabricated greater than the contract plans specify. Astaldi proposed skewing the bars to maintain the required concrete cover. By skewing the bars, the back face of the diaphragm will have a bar spacing of 7.75 in. and the front face a 1.75-in. spacing. The contract plans specify a spacing of 6 in. while the design calculations show that a 6.5-in. spacing is acceptable. The reinforced and formed diaphragms are shown in **Figure 170**.

Figure 170: Reinforced and formed diaphragms

When casting the diaphragms, site inspectors mentioned the instability of the 6D1 GFRP stirrups; the stirrups were attempting to float during concrete placement. Upon inspection, some of the GFRP stirrups were too high to provide the minimum required concrete cover of 2.5 in. According to RFM 0013, select stirrups were required to be ground down to maintain a clear cover of 1.5 in., and other stirrups were cut off to maintain a minimum cover of 1.75 in. and a maximum of 2.5 in. The location, elevation, and action required to maintain minimum cover of the stirrups are listed in **Table 28**. **Figures 171** and **172** show the removed GFRP stirrups.

Table 28: Cutting/grinding of 6D1 diaphragm stirrups

Station	Elevation	Below Finished Deck Elevation (in)	Cut or Grind
110+72.91	11.396	1-1/4	Grind
110+72.92	11.331	13/16	Cut
110+72.99	11.156	1-5/16	Grind
110+72.85	11.024	1-1/4	Grind
111+09.71	11.444	1	Cut
111+08.77	11.559	1-1/4	Grind
111+45.87	11.574	1-1/4	Grind
111+84.11	11.221	1-3/16	Grind
111+84.11	11.308	1	Cut
111+84.14	11.365	1	Cut
111+84.16	11.489	1-3/8	Grind
111+83.24	11.596	¾	Cut

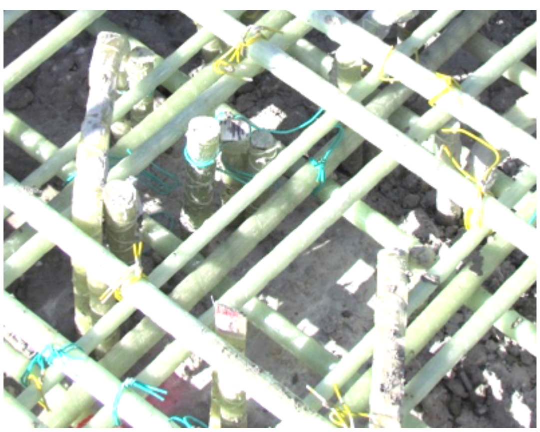

Figure 171: Top of diaphragm reinforcement removed

Figure 172: Removed GFRP

4.8 Casting of RCA and RAP Gravity Walls

On April 16, 2018, the recycled concrete aggregate (RCA) gravity wall located on the east side of Halls River Bridge was cast. The formed gravity wall is shown in **Figure 173** with the reinforced GFRP cage shown in **Figure 174**. The GFRP cage was suspended by wire attached to the top 2x4 form. Two (2) trucks each containing 7.25 cubic yds of concrete were required to fill the RCA gravity wall form. The concrete mix design for the RCA gravity wall is shown in **Table 29**. The mix design includes Type II cement, slag, #57 stone coarse aggregate from Cemex and Independence Recycling of Florida, silica sand fine aggregate, and admixtures.

Figure 173: Formed RCA gravity wall

Figure 174: GFRP of RCA gravity wall

Table 29: Mix design for RCA gravity wall

Product	Quantity
Type II Cement	200 lb
Slag	380 lb
#57 Stone Coarse Aggregate (Cemex)	1280 lb
#57 Stone Coarse Aggregate (Independence Recycling of Florida)	320 lb
Silica Sand Fine Aggregate	1349 lb
Darex AEA Air Ent. Admixture	0.2 oz
WRDA 60 Type D Admixture	17.4 oz
ADVA 140M Type F Admixture	23.2 oz
Recover Type D Admixture	23.2 oz
Water	258 lb
W/CM Ratio	0.45
Unit Weight	140.3 lb/cy

Each concrete truck was tested for water-cement ratio, and test cylinders were made and stored in the on-site cure box. After filling the concrete bucket (**Figure 175**), the bucket was flown to the gravity wall using the excavator. While pouring, a worker used a 2x4 to stabilize the bottom of the GFRP cage, as shown in **Figure 176**. On April 18, 2018, the recycled asphalt pavement (RAP) gravity wall was poured. The mix design for the RAP concrete is shown in **Table 30**; the mix includes Type II cement, slag, #57 stone coarse aggregate, RAP coarse aggregate, silica sand fine aggregate, and admixtures. A figure of the complete RCA/RAP gravity wall is shown in **Figure 177**.

Figure 175: Bucket filling with RCA concrete

Figure 176: Pouring RCA concrete

Table 30: Mix design for RAP gravity wall

Product	Quantity
Type II Cement	200 lb
Slag	380 lb
#57 Stone Coarse Aggregate (Cemex)	1015 lb
RAP Coarse Aggregate (Central Materials Company)	575 lb
Silica Sand Fine Aggregate	1340 lb
Darex AEA Air Ent. Admixture	0.2 oz
WRDA 60 Type D Admixture	17.4 oz
ADVA 140M Type F Admixture	23.2 oz
Recover Type D Admixture	23.2 oz
Water	267 lb
W/CM Ratio	0.46
Unit Weight	139.9 lb/cy

Figure 177: RCA/RAP gravity wall

4.9 GFRP Reinforced Deck

GFRP bars were brought to the bridge site and stored on a trailer during the deck reinforcing stage (**Figure 178**). Unlike traditional rebar, workers were able to lift multiple GFRP bars and position them on the bridge deck (**Figure 179**). Tying the GFRP reinforcement for the bridge deck was subcontracted to Denson Construction. To secure the GFRP bars, green plastic-coated wire was tied and cut using wire cutters, as shown in **Figures 180** and **181**.

Figure 178: GFRP bars stored on trailer

Figure 179: Workers carrying GFRP bars

Figure 180: Tying GFRP deck reinforcement

Figure 181: Tying GFRP deck reinforcement

A few issues were encountered when tying the GFRP reinforcement including the angle and height of the shear connectors, the spacing of the SIP (stay-in-place) deck form anchors, and the height of the GFRP diaphragm stirrups. The angle of some of the shear connectors was causing issues with the spacing of the longitudinal GFRP bars. The contract plans require a 4.5-in. bar spacing, but the location of the shear connectors was allowing for a 3 in. or 6 in. spacing for longitudinal bars. A few of the shear connectors were too high to provide minimum concrete cover; the shear connectors were bent to accommodate bar spacing and height requirements. GFRP bars tied to shear connectors are shown in **Figure 182**.

Figure 182: GFRP bars tied to shear connector

The original plan was to install stay-in-place (SIP) deck form galvanized support bars spaced every 18 in., but after installation the stability of the forms was a concern. To increase the stability of the forms, support bars were installed every 9 in. on the interior beams and every 18 in. on the exterior beams (**Figure 183**). Four Tapcon screws per anchor were used to secure the forms to the HC beams.

Figure 183: SIP deck form support bars

A foam and plywood block-out was used to block-out the area where future traffic barrier will be installed during Phase III. The foam and plywood will eventually be cut out to reveal the GFRP reinforcement and prevent damage to it when installing the traffic barrier; the block-out and plywood walkway are shown in **Figure 184**. The foam and plywood were tied to the beam shear connectors as shown in **Figure 185**.

Figure 184: Plywood walkway and block-out

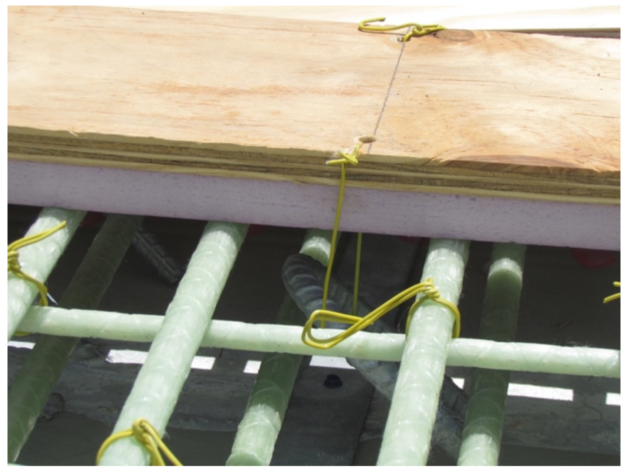

Figure 185: Foam and plywood block-out

To mark the future drilling location for pedestrian handrail installation, wooden dowels were installed, as shown in **Figure 186**. Upon inspection, the wooden dowels were not stable and continued to shift when workers walked across the GFRP reinforcement bars. It was decided to remove the wooden dowels, and the GFRP bars were marked in red to indicate the point of intersection for future handrail installation, as shown in **Figure 187**. During concrete placement, the GFRP bars shifted, resulting in unknown alignment of the GFRP bars in comparison to future drilling locations.

On the morning of the deck pour, inspectors noticed spray foam next to the exterior beams on the deck forms (shown in **Figure 188**); workers were required to remove the foam before the deck pour.

Figure 186: Wooden dowel installation

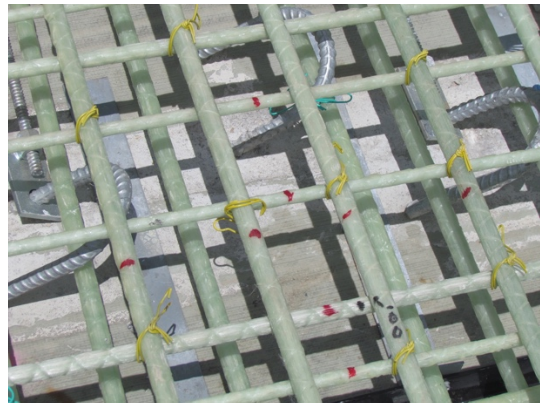

Figure 187: Future handrail drilling location

Figure 188: Spray foam in deck form

On April 28, 2018, a deck pre-pour meeting occurred at the Halls River Bridge Field Office in Homosassa, FL. The meeting covered the planned schedule, key personnel, equipment and materials required, and pouring and curing procedures. The deck was poured from the west to the east side with a concrete pump truck stationed on both sides of the bridge. The pour was scheduled for 2 a.m. on May 02, 2018, but the concrete trucks were delayed until 6:30 a.m. Each truck carried 9 cubic yds of concrete with the entire deck requiring approximately 130 cubic yds. Quality control testing was done every 50 cubic yds of the Class IV 5500 psi concrete; the first truck had a slump of 5 ¼ in. with an air content of 2.6%. The required slump was 7 $\pm$ 1.5 in,. and a minimum pour of 30 cubic yds per hour was required by FDOT standards. The mix design for the deck concrete is shown in **Table 31**.

After filling the pump truck with concrete, one worker manned the hose while several workers used come-alongs to level the concrete in front of the screed (**Figure 189**). The concrete was vibrated with a rubber tipped vibrator, as shown in **Figure 190**.

Table 31: Mix design for deck

Product	Quantity
Type II Cement	350 lb
Slag	350 lb
#57 Stone	1680 lb
Silica Sand	1165 lb
Darex AEA Air Ent. Admixture	0.5 fl oz
WRDA 60 Type D Admixture	21 fl oz
ADVA 140M Type F Admixture	70 fl oz
Recover Type D Admixture	42 fl oz
Water	32 gallons
W/CM Ratio	0.38
Unit Weight	141.1 pcf

Figure 189: Pouring deck

Figure 190: Vibrating concrete

While concrete was being placed, QC checked the depth to the SIP forms and the depth to the top of the GFRP bars, as shown in **Figure 191**. The documented depths are shown in **Table 32**, the depth to the GFRP bars varies from 2 in. to 3 in., and the depth to the SIP forms varies from 8 ⅛ in. to 9 in.

Figure 191: QC checking depth of deck

The screed was tested days prior to pouring, and an extra motor was on-site in case of mechanical issues. After the screed leveled the concrete, a bull float was used to smooth the deck surface (**Figure 192**). Water was applied to dry areas of the deck, and low areas were filled and smoothed (**Figure 193**).

Concrete was poured over the block-out (**Figure 194**), and extra care was taken to vibrate the concrete under the block-out. The block-out was visible even after the concrete was smoothed, as shown in **Figure 195**.

As the concrete was pumped onto the deck reinforcement, the GFRP bars were visibly shifting. Bar movement was especially noticeable when the pump hose was angled, as shown in **Figure 196**. As shown in **Figure 197**, the GFRP bars were tied at multiple locations along the length of the bars. Using a GFRP bar with larger deformations or using a stronger tie wire could reduce bar movement. After screeding the deck, the screed was removed, and the deck end was smoothed using hand trowels (**Figure 198**). The deck was then smoothed with a bull float, and a broom finish was applied, as shown in **Figure 199**. The deck was wet cured for seven (7) days and forms were pulled after approximately three (3) days; no heavy equipment was allowed on the deck for 14 days.

Table 32: QC depth check data

Bent	Depth to GFRP			Depth to SIP Form		
End Bent 1:	$2\frac{7}{8}$	$2\frac{5}{8}$	$2\frac{1}{2}$	$8\frac{5}{8}$	$8\frac{1}{2}$	$8\frac{5}{8}$
	2	$2\frac{1}{4}$	$2\frac{3}{8}$	$8\frac{5}{8}$	$8\frac{1}{2}$	$8\frac{5}{8}$
	$2\frac{1}{8}$	$2\frac{1}{4}$	$2\frac{1}{4}$	$8\frac{1}{2}$	$8\frac{3}{8}$	$8\frac{5}{8}$
	$2\frac{1}{8}$	$2\frac{1}{8}$	$2\frac{3}{8}$	$8\frac{1}{2}$	$8\frac{5}{8}$	$8\frac{7}{8}$
Bent 2:	$2\frac{1}{8}$	$2\frac{3}{8}$	$2\frac{1}{2}$	$8\frac{3}{8}$	$8\frac{3}{4}$	$8\frac{3}{4}$
	$2\frac{5}{8}$	$2\frac{3}{8}$	3	$8\frac{7}{8}$	$8\frac{5}{8}$	9
	$2\frac{1}{8}$	$2\frac{1}{2}$	$2\frac{1}{4}$	$8\frac{1}{8}$	$8\frac{5}{8}$	$8\frac{1}{2}$
	$2\frac{1}{8}$	$2\frac{1}{8}$	$2\frac{5}{8}$	$8\frac{3}{8}$	$8\frac{1}{2}$	9
Bent 3:	$2\frac{1}{8}$	$2\frac{3}{8}$	$2\frac{1}{2}$	$8\frac{1}{8}$	$8\frac{7}{8}$	$8\frac{3}{4}$
	$2\frac{1}{4}$	$2\frac{1}{2}$	$2\frac{1}{4}$	$8\frac{1}{2}$	$8\frac{7}{8}$	$8\frac{5}{8}$
	$2\frac{1}{8}$	$2\frac{1}{8}$	$2\frac{1}{4}$	$8\frac{3}{8}$	$8\frac{1}{2}$	$8\frac{3}{4}$
	$2\frac{1}{8}$	$2\frac{1}{4}$	$2\frac{3}{8}$	$8\frac{1}{2}$	$8\frac{5}{8}$	$8\frac{1}{2}$
	$2\frac{3}{8}$	$2\frac{1}{2}$	$2\frac{1}{2}$	$8\frac{3}{4}$	$8\frac{5}{8}$	$8\frac{3}{8}$
Bent 4:	$2\frac{1}{8}$	$2\frac{1}{4}$	$2\frac{1}{4}$	$8\frac{1}{2}$	$8\frac{1}{2}$	$8\frac{5}{8}$
	$2\frac{3}{8}$	$2\frac{1}{4}$	$2\frac{1}{2}$	$8\frac{1}{2}$	$8\frac{5}{8}$	$8\frac{1}{2}$
	$2\frac{1}{2}$	2	$2\frac{1}{2}$	$8\frac{3}{4}$	$8\frac{1}{2}$	$8\frac{3}{4}$
	$2\frac{1}{8}$	$2\frac{1}{4}$	$2\frac{1}{2}$	$8\frac{3}{4}$	$8\frac{5}{8}$	$8\frac{3}{4}$
	$2\frac{1}{2}$	$2\frac{1}{2}$	$2\frac{1}{2}$	9	$8\frac{7}{8}$	$8\frac{7}{8}$
Bent 5	$2\frac{1}{8}$	$2\frac{1}{2}$	$2\frac{3}{8}$	$8\frac{1}{2}$	$8\frac{5}{8}$	$8\frac{7}{8}$
	$2\frac{1}{8}$	$2\frac{1}{4}$	$2\frac{1}{4}$	$8\frac{1}{2}$	$8\frac{5}{8}$	$8\frac{3}{8}$
	$2\frac{1}{2}$	$2\frac{1}{2}$	$2\frac{5}{8}$	$8\frac{3}{4}$	$8\frac{3}{4}$	$8\frac{7}{8}$
End Bent 6						

Figure 192: Bull float poured concrete

Figure 193: Concrete with smooth finish

Figure 194: Block-out during pour

Figure 195: Block-out after pour

Figure 196: Pumping of deck concrete

Figure 197: Tied GFRP deck reinforcement

Figure 198: Finish screeding deck

Figure 199: Broom finish to deck

After the deck pour in May 2018, construction of the approach slab, roadwork and erosion control activities was completed on the westbound lanes. As shown in **Figure 200**, the eastern side of the westbound lanes required extensive erosion control work due to the steep side slope of the roadway. Two-way traffic was opened on the completed westbound lanes as demolition proceeded on the eastbound lane of the existing Halls River Bridge. Temporary traffic barriers (**Figure 201**) were placed to control traffic during construction on the eastbound lanes.

Figure 200: Erosion control on east side of bridge

Figure 201: Completed westbound lane

Slope pavement was formed and poured on both sides of the westbound lanes, and the forcemain and watermain were installed (**Figure 202**).

Figure 202: Forcemain and watermain

As shown in **Figure 203**, the north side traffic barrier 5D and 5P bars were designed to extend into the deck 6.5 in. Because of the construction phasing, the 5D and 5P bars will be epoxy embedded into the deck. While the design calls for an embedment of 6.5 in., the maximum embedment depth possible is 4.5 in. Astaldi Construction tasked the University of Miami with performing pull out tests of GFRP bars using the sheet pile cutoffs. FDOT Section 416-6, on the field testing of post-installed anchors systems and dowels, will be adhered to during pullout testing. A potential cause for the lack of achievable embedment depth was the failure to block-out or mark the location for future drilling of the post-installed bars during the deck pour. Astaldi Construction attempted to install wooden dowels to mark the drilling location, but installation of the dowels was too time consuming. As a result, the location for future drilling was marked on the deck GFRP bars with the intent to monitor the marks during the deck pour to ensure the bars had not shifted. Unfortunately, the GFRP bars were moving more than anticipated during the pour, especially when the concrete pump was angled at the GFRP bars.

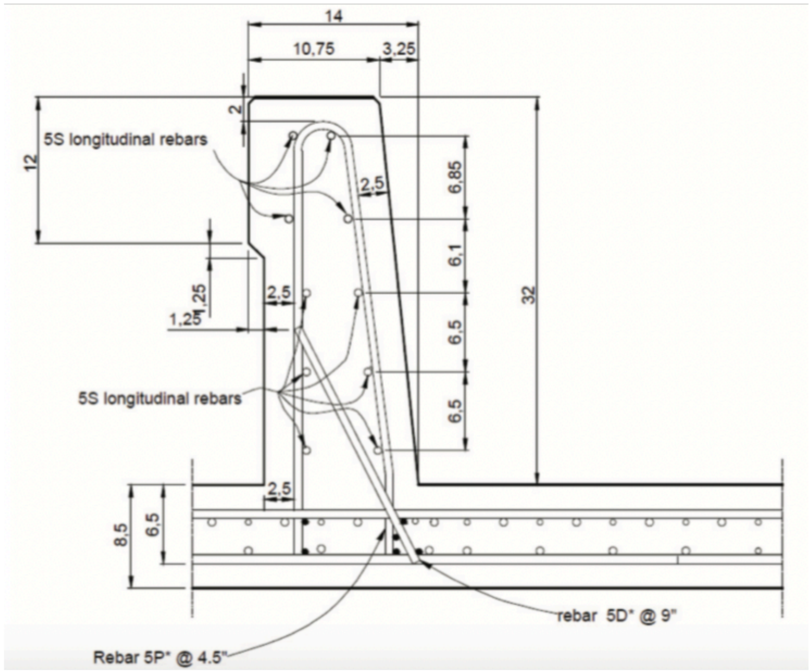

Figure 203: North side traffic barrier (FDOT)

Following the same procedure used in Phase II deck construction, GFRP bars were tied using plastic coated wire. The light weight of GFRP bars allow workers to lift and position multiple bars along the bridge length. **Figure 204** shows the tied GFRP deck reinforcement.

Figure 204: GFRP deck reinforcement for Phase III (Figure provided by FDOT)

Casting of the deck for Phase III occurred on May 23, 2019. The required slump was 7 $\pm$ 1.5 in. with a minimum pour of 30 cubic yds per hour, required by FDOT standards. A broom finish was applied to the smooth deck, as shown in **Figure 205**.

Figure 205: Complete deck for Phase III (Figure provided by FDOT)

4.10 Environmental Concerns and Activities

The micrometer size of the turbidity barrier is small and allows for minimized flow through the barrier, which causes algae buildup. The algae buildup attracts the manatees and causes damage to the turbidity curtain. If manatee are sighted within 50 ft of the bridge, pile driving is halted for 30 minutes from the last manatee sighting.

On March 27, 2017, the crane boom was lowered to begin pile driving operations, and an Osprey nest was discovered on the end of the boom. The crane had been raised over the weekend, allowing time for an Osprey to create a nest (**Figure 206**). Osprey (Pandion haliaetus) are a raptor species that typically nest between December and February. Protected by the U.S. Migratory Bird Treaty Act, Osprey nests cannot be moved without a permit. On April 17, 2017, the Osprey nest was relocated, under the supervision of Florida Fish and Wildlife Conservation Commission, to a post installed on Margarita Grill property. The nest was brought down the crane in a box with a heated compartment for the eggs. After relocation, crows took over the nest, and it is believed that the eggs were eaten by the crows. Strobe lights and flags have been placed on each crane to prevent future bird nesting incidents. Although the eggs were not preserved, the Osprey have been spotted returning to the relocated nest.

Figure 206: Osprey nest on crane

On April 18, 2017, during the pile restrike, the turbidity readings were above the allowable levels due to pumping from the nearby Margarita Grill property. Pumping that caused a change in water color was also noted at noon on April 19, 2017.

On September 11, 2017, Hurricane Irma passed through Citrus County, Florida. Astaldi prepared for Hurricane Irma by securing the jobsite, moving and securing the crane, removing barges from Halls River, and stacking and securing wooden platforms. The HCB beams were secured, as shown in **Figure 207**. The only disturbance to the bridge site was to the storm water control structures, which were easily put back into position. Due to the large number of Florida residents evacuating from Hurricane Irma, FDOT requested Astaldi to stop construction until September 18, 2017.

Figure 207: HCB beams strapped for Hurricane Irma

REFERENCES:

Surarez, C.K., Siddiqui, M., P.E., and Pelham, D. (2016). Halls River Bridge Replacement. Lecture presented at 2016 Design Training Expo in FDOT District 7 Structures Design Office, Tampa, FL.