Abstract—Located southeast of downtown Tampa, Florida, the C. W. "Bill" Young Regional Reservoir is a 15.5 billion gallon earthen embankment reservoir supplying water to the area’s 2.4 million residents. During the wet season, water from surrounding rivers is stored to allow for withdrawals during the dry season—greatly reducing the area’s dependence on Florida’s waning groundwater.

The embankment dam is approximately 5 miles in length with an average height of 50 feet. Tampa Bay Water began operating the reservoir in 2005; however, in 2006 sizeable cracks began to appear in the soil-cement erosion protection layer on the upstream slope. The reservoir was eventually taken offline in 2012 to prepare for the renovation of the upstream slope.

After subsurface investigations and an extensive design process, an upstream solution was constructed. This rehabilitation included removing and replacing the original HDPE liner with a PVC liner and constructing a robust soil-cement erosion protection system. The chosen PVC liner was Sibelon® 2CNT 3300, an 80 mil (2mm) PVC geomembrane with a geotextile layer heat-bonded onto the top and bottom sides of the liner. This geotextile layer was included in order to accommodate the required friction angle of the earthen embankment below and the gravel drainage layer above. Overlying the liner, the soil-cement was designed to provide a minimum of 50 years of erosion protection for the reservoir.

Construction of the upstream renovation began in 2013 and was completed in 2014, with the reservoir being placed back into operation by the end of 2014. Over the last few years the reservoir has been frequently monitored for dam safety. This paper will explore renovation design features and the monitoring findings to date, including the soil-cement erosion protection system and the seepage flow observed surrounding the reservoir.

I. INTRODUCTION

Originally constructed from 2003 to 2005, the 15.5 billion gallon C. W. "Bill" Young Regional Reservoir located just east of Tampa, Florida was built to supply water to Tampa Bay Water’s customers. Water is captured from the surrounding rivers and stored within the reservoir in an effort to reduce the area’s dependence on Florida’s waning groundwater. The reservoir is surrounded by approximately 5 miles of earthen embankment, with a maximum operating water elevation of 136.5 feet National Geodetic Vertical Datum 1929 (NGVD29). The original design included 16-inch thick flat-plate soil-cement erosion protection, underlain by a 60 mil HDPE geomembrane, which was tied into a soil-bentonite foundation cutoff wall at the upstream toe.

First filling of the reservoir was conducted in November 2005. By the end of 2006, the second drawdown cycle revealed unusual cracking in the soil-cement. Tampa Bay Water performed numerous investigations and repairs to the soil-cement with no success. In 2012, Tampa Bay Water took the reservoir offline and began the $129 million rehabilitation construction.

II. RESERVOIR REHABILITATION

The design-build team of Kiewit Infrastructure South, Inc. and Gannett Fleming, Inc. was selected to design and construct a reservoir that could provide the storage of raw water for the region, with minimal maintenance throughout its 50-year design life. Together the team reconstructed the upstream portion of the embankment dam, removing the previous soil-cement and HDPE geomembrane. The rehabilitation design included nominal 24-inch thick stair-stepped soil-cement erosion protection, a 9-inch gravel drainage layer, and PVC geomembrane seepage barrier (see Figure 1).
A. Upstream Design Features

As part of the rehabilitation, the entire upstream slope of the embankment dam was demolished and re-constructed with a robust and resilient soil-cement erosion protection and seepage barrier system.

i. Stair-Stepped Soil-Cement

Chosen as a cost-effective, low maintenance solution, the soil-cement erosion protection system lines the entire 5 miles of the reservoir’s upstream slope. The original 16-inch thick flat-plate soil-cement was replaced with a 24-inch thick stair-stepped design. Several design and construction considerations were evaluated to provide the highest quality produce while minimizing costs.

a) Design Considerations

As the cracking observed in the original soil-cement erosion layer was the impetus for the rehabilitation project, seepage and stability analyses were conducted to assess the new erosion protection system design. Seepage and slope stability cross-sections were developed based on the extensive on-site subsurface investigation and different embankment configurations. The subsurface investigation included numerous field and laboratory testing to characterize the soils and to determine the design parameters.

Since the rehabilitation of the upstream slope did not affect the downstream slope and the overall seepage surrounding the reservoir, the seepage analysis performed for the rehabilitation project was used to determine the phreatic surface behind the new geocomposite and within the gravel drainage layer. This analysis was used to calculate seepage forces behind the geocomposite and to check the uplift resistance of the stair-stepped soil-cement.

Global stability of the upstream slope was evaluated for the end of construction condition, long-term steady-state seepage condition, and the rapid drawdown condition (both operational drawdown and emergency controlled release). In the original design, the upstream slope was construction on a 3H:1V slope, with a geomembrane below the soil-cement, placed on a 2.5H:1V slope. The slope stability analyses determined the final slope of 2.5H:1V was stable and the design could eliminate the need to return the interior of the reservoir to a 3H:1V slope. Additionally, a soil-cement toe buttress was also analyzed and included in the final design to prevent sliding of the soil-cement stair-steps.

The longevity of the soil-cement was also a critical part of the design. The stair-stepped design allowed for easier construction and would prevent the delamination of lifts, thus improving long-term durability. The minimum required 7-day compressive strength was set at 650 psi and a maximum 14% weight loss was required for the wet-dry durability testing. The longevity of the soil-cement would not only be dependent upon the design, but also upon the construction methods. As the soil-cement was to be placed parallel to the slope, the area largely susceptible to weathering would be the stair-steps themselves, most likely due to direct wave impacts and frequent wetting and drying cycles. Achieving compaction along the lift, adjacent to risers of the steps would be difficult and require care quality control during construction.

b) Mix Design

Prior to determining the soil-cement mix, the on-site soils were evaluated to determine if they were suitable for use. The soils predominately found on-site were classified as silty sand (SM) or poorly-graded sand with silt (SP-SM) and were tested for low pH, excessive organics, and excessive clay content. Based on these tests, the soil was found to be acceptable for use in the soil-cement mix design. A test batch program was conducted to determine the appropriate cement content for the mix to achieve the required compressive strength and durability requirements.
c) Construction Methods

A soil-cement batch plant was erected within the interior of the dewatered reservoir (see Figure 2). Stockpiles of the on-site soil were placed adjacent to the batch plant in horizontal lifts and were excavated from the side of the piles to help mix the soil aggregate. The soil was then placed through screens to sift out particles greater than \( \frac{1}{2} \)-inch in diameter and concentrations of clay.

![Figure 2: Soil-cement batch plant and soil stockpiles located on the interior of the reservoir](image)

After batching of the soil-cement, it was hauled by truck to the upstream slope and placed with one of the largest tracked conveyor system ever used on a US dam (see Figure 3). Spreading of the mix was performed by dozers, in compacted lifts between 6 to 12 inches thick. A bonding slurry was used between lifts to ensure proper adhesion of the subsequent lifts.

![Figure 3: Placement of the soil-cement via tracked conveyor system](image)

Compaction was achieved through the use of smooth drum vibratory rollers (see Figure 4). As previously noted, proper compaction of the soil-cement was a main concern during the design and extensive quality control was required during construction. Nuclear density test were performed on the soil-cement insitu and pills were molded and tested for compressive strength. Additionally, great care was taken along the stair-step edges; to ensure the steps were fully compacted, the soil-cement was overbuilt, compacted, and then the overbuild was then removed (see Figure 4).
In all, 525,000 cubic yards of soil-cement were batched, placed, and compacted along the approximately 26,000 linear feet of embankment (see Figure 5).

ii. Crest Wave Barrier

The renovation design also evaluated the wind and wave conditions, particularly tropical storms and hurricanes likely to pass over the region. While stair-stepped soil-cement is typically intended to reduce or eliminate wave run-up during storm events, the stair-stepped design at this reservoir was not. A wave barrier wall was placed on the crest of the embankment (see Figure 6) and was designed to prevent wave-induced overtopping by returning any run-up back to the interior of the reservoir. To be conservative, the design of the wave barrier assumed a flat-plated soil-cement upstream slope (however, as previously mentioned, for ease of construction, the soil-cement was placed in stair-step lifts). As such, the presence of the stair-steps further reduce wave run-up, and provide additional erosion protection, in excess of the flat-plate’s 50-year design life. The wave barrier innovation is seen in only a handful of reservoirs across the United States.
As can be seen in Figure 6, the wave barriers were affixed to the crest with mechanical anchors. The soil-cement was determined to be insufficient to properly anchor the barriers due to the low-strength of the material. As a result, the top three soil-cement stair-steps were replaced with roller compacted concrete.

### iii. Drainage Layer

While the thicker stair-stepped soil-cement provides additional weight against potential uplift forces, a drainage layer was designed and placed as the main line of defense against water pressure build-up below the soil-cement. The drainage layer consists of AASHTO No. 57 aggregate, benched by aggregate-filled gabion baskets (see Figure 7). The drainage layer was designed to have approximately 1,000 times more drainage capacity than the surrounding soil, thus providing a free draining system during filling and drawdown cycles of the reservoir.

Uplift was calculated across the embankment and was determined to be detrimental if there was an approximately a four-foot differential head across the soil-cement. In addition to the design considerations for the drain during reservoir operations, the design also considered the capacity necessary to pass a design storm during construction, when the drainage layer was likely to be exposed to the environment.

### iv. Waterproofing Geocomposite Layer

Carpi USA, Inc. provided the geocomposite designed for the seepage barrier system. Due to the complexity and specific requirements of the project, Carpi created a new product that could meet the required friction angles for both the earthen
embankment below and the gravel drainage layer above, as well as provide cushioning from the placement of the gravel drainage layer. In order to meet these requirements, two geotextiles were heat bonded to the geomembrane during extrusion. The bottom geotextile was a 200 g/m² polyester geotextile, while the top layer was a 500 g/m² polypropylene geotextile (see Figure 8). This new geocomposite was used to cover the more than four million square feet of slope area along the full 5-mile length of embankment dam. This translated into nearly 4,000 rolls of approximately 7-foot wide geocomposite.

![Figure 8: 2 CNT 3300 Geocomposite – a new material designed specifically for installation at the reservoir consisting of polyester geotextile on the bottom and polypropylene geotextile on the top of the PVC geomembrane](image)

**a) Design Considerations**

1) **Veneer Slope Stability**

Stability of the membrane on the prepared embankment upstream surface was evaluated as part of the design. The friction angle between the geocomposite membrane and the underlying embankment (see Figure 9) was identified as the most critical in the suite of interfaces that exist in the geocomposite and surrounding design features. As previously discussed, a soil-cement buttress was constructed at the toe of the upstream slope and provides a great deal of support to the stability of the overall erosion protection system; however this buttress was ignored in the veneer slope stability analysis for conservatism.

To understand the friction angles developed in the various materials in the cross-section shown in Figure 9, laboratory testing was conducted. The veneer slope stability analysis was performed by Dr. Robert Koerner and documented in the report entitled “Slope Analysis of Tampa Bay Reservoir Renovation Project.” Using the interface friction angles taken from laboratory testing, the minimum interface friction angle that could exist in the veneer of materials was 27.9 degrees. The post peak friction angle for the geotextile heat bonded to the PVC membrane falls below this value at 25.5 degrees, but the method by which the geotextile is bonded to the PVC membrane essentially precludes the ability of the geotextile to disbond from the PVC membrane.

Dr. J. P. Giroud reviewed the veneer slope stability analysis of the geocomposite system for Carpi. An infinite slope failure was modeled to conservatively estimate the interface friction between the geomembrane and upstream embankment face. This value was used to establish the criteria for required minimum friction angle and determined to be 29.5 degrees. All of the friction angles were found to be at least 29.5 degrees and found to be stable at the 2.5H:1V upstream slope.

![Figure 9: The geocomposite was underlain by the embankment and overlain with the coarse drain fill layer and soil-cement to buttress.](image)
(2) Interface Adhesion of Geosynthetic Layers

To our knowledge, this was the first time in the world that a double geotextile, coupled to a PVC geomembrane, had been produced in large quantities for a hydraulic structure. After the friction angles were developed for the geocomposite system on the slope and gravel layer, the critical parameter for the geocomposite production was the adhesion between the geotextiles and the PVC geomembrane. A healthy tolerance was placed on the value to ensure the manufactured material had the geotextile stoutly bonded to the PVC geomembrane. This was monitored carefully throughout the production. However, due to the strong adhesion of the top polypropylene geotextile to the PVC geomembrane, the two could not be easily separated in the field for overlapping adjacent rolls around the curves. This challenge eventually led to the team to develop a different approach to avoid stripping the geotextile for welding rolls in the field.

(3) Handling of Curves

During the design, it was apparent that the installation of the PVC geocomposite would be difficult in the curves. In order for the top geotextile to withstand the placement of the soil-cement over the gravel drainage layer, the geotextile had to be resistant to deterioration from cement. Polyester geotextiles deteriorate when they come in contact with cement and thus were not acceptable as the top geotextile of the geocomposite. As such, a polypropylene geotextile was required. However, to install a watertight seepage barrier along the embankment alignment curves, the polypropylene geotextile had to be removed to allow for bonding between two layers of the PVC geomembrane. As such, adjoining rolls were designed with offsetting exposed weld strip areas (see Figure 10).

![Figure 10: The roll on the left was the primary geocomposite, Sibelon CNT3300, a PVC geomembrane with polyester geotextile on the bottom towards the slope and polypropylene geotextile on the top face, with weld strips on opposing longitudinal corners. The right diagram is the geocomposite Sibelon CNT 2800 with PVC geomembrane and polyester geotextile on the bottom face of the slope with an exposed face to allow for overlapping and welding in curves.](image)

There are two types of curves along the embankment: concave and convex. At concave curves, in order for a constant width roll to be deployed, subsequent rolls must start with an overlap of just the width of the weld strips and be overlapped as the roll is deployed down the slope to take up the excess material in the curve. For a convex curve, the opposite is true as the rolls must be overlapped at the crest and then overlap gradually tapered down the slope towards the toe until the overlap is just in the weld strip area at the toe. Thus making the roll geometry is very important.

The veneer slope stability analyses determined the necessary minimum area of geotextile needed to provide overall stability to the embankment. The analysis determined that every third roll could be free of geotextile on the top face, while maintaining proper factors of safety for stability. This resulted in adding two actions to the installation: (1) for each curved area, the crew must be given very precise overlap dimensions at the crest for the rolls to lineup at the toe and (2) after the rolls are welded together, a polypropylene geotextile had to be spot welded to the exposed PVC geomembrane face to protect against the gravel drainage layer above (see Figure 10).

Further analysis was performed to ensure that this alternating roll layout would still meet the required friction angle for the drainage layer above. While some portions of the geotextile was not originally heat bonded to the geomembrane there was a high enough proportion of geotextile along the length of the curve to meet the required friction angle.
b) Installation Methods

Since the geocomposite material was to be covered by the drainage gravel and then the soil-cement, the only required anchorage points were along the crest and the toe. This anchorage was achieved by the use of trenches which were shaped in such a way to provide enough surface friction to hold the material in place, while minimizing trench excavation. Along the crest, the trench was dug following the earthwork preparation of the slope surface. A geocomposite roll was then partially unraveled and placed into the trench. Particular care was taken to ensure the polyester geotextile was in intimate contact with the earthen embankment below. The unraveled length of the geocomposite was held in place with a wood stake (see Figure 12).

In addition to the alignment curves, the layout of the geocomposite rolls had to be adjusted to accommodate interior ramps. At the ramp locations, some of the geocomposite rolls had to be cut and reoriented with the varying geometry of the ramp (see Figure 13). This adjustment primarily occurred at the end of the ramps where the grade changed near the toe. Along some of the flat portions of the ramp the geocomposite was able to accommodate the change in grade and be deployed in one piece all the way to the toe.
To create a continuous seepage barrier, the geocomposite rolls had to be heat welded together. The majority of these welds were completed using a dual track heat weld between overlapping rolls along a weldstrip (see Figure 14). These weldstrips were created by leaving the geotextiles off of the very edge of the roll width where overlapping of adjoining rolls would occur. In some locations with specific details, such as along the bridge columns, these welds were done with a manual heat welder.

Welding was completed starting in the crest anchor trench and progressing down the slope to the toe trench. Every weld required quality control checks performed using nondestructive and destructive methods to ensure a successfully completed weld was achieved. If a location failed any of the quality control checks, the weld would be analyzed in order to find the faulty location. Once found, the weld would be repaired and retested until a passing result was achieved.

The design of the seepage barrier required a watertight transition from the geocomposite to the in situ seepage cutoff wall. To create a watertight seal, only a geomembrane could be used in the trench. A C 2600 geomembrane was welded to the bottom of the geocomposite rolls that had been deployed along the embankment slope. Once all of the geosynthetic material was placed, the trench was backfilled with soil-bentonite (see Figure 15).
Figure 15: C 2600 Geomembrane was installed in the toe trench (left photograph), then backfilled with a soil-bentonite mixture (right photograph).

Watertight seals were also required where concrete structures tied into the soil-cement (see Figure 16). The seal consisted of a stainless steel batten anchored in placed, above the geocomposite using threaded bars and chemical anchors. An EPDM gasket was placed between the batten and the geocomposite. Additionally, between the geocomposite and the concrete, a two-part epoxy resin was used to help to create a smooth surface, sealing the connection through pressure.

Figure 16: Stainless steel submersible seals on concrete structures. The photograph on the left shows the submersible seal going onto the bridge tower. The photograph on the right shows the seal along the foundation of a structure along the reservoir.

c) Installation Challenges

There were two primary site conditions that combined to make the optimization of the installation an iterative process. First, typical summer weather in Florida includes daily afternoon thunderstorms. Secondly, the reservoir embankments were constructed of highly erodible soil (predominately sand). It was difficult to prepare large areas of the slope in an effort to maximize earthwork production. Additionally, the geocomposite absorbs a small amount of water by diffusion; if the geocomposite is allowed to sit and be soaked, it becomes more difficult and time consuming to weld.

As a result, the installation became an iterative process to determine the correct ratio of time required for each operation—earthwork and geocomposite placement. It was important that the production be scheduled to allow all work to be completed before each daily rainstorm or forecasted weather event. Production had to be adjusted each day, but increased once the proper balance was achieved.

B. Ongoing Monitoring and Soil-Cement Observations

The construction of the rehabilitated upstream slope was completed in mid-2014, with full operation of the reservoir commencing by the end of 2014 (see Figure 17).
Figure 17: C. W. “Bill” Young Regional Reservoir near completion of the upstream rehabilitation construction project (Photo taken September 2, 2014)

Gannett Fleming is performing as the Permit Compliance Engineer for five years, post construction with responsibilities including performing monthly and annual dam safety inspections of the reservoir. As part of the dam safety inspections, the upstream soil-cement slope protection is visually inspected, and data is collected for monitoring the permeability of the drainage layer and the seepage through the embankment and foundation soils. To date, the soil-cement erosion protection system, gravel drainage layer, and geomembrane are performing as designed.

Figure 18: Tampa Bay Water is currently performing a drawdown of the reservoir, allowing for increased visual inspection of the soil-cement slope protection
To monitor the seepage forces behind the geocomposite, piezometers were installed under the geocomposite at varying elevations (see Figure 19) around the reservoir. These piezometers below the geocomposite are continuously monitored and, to date, have not exhibited any seepage forces that appear to be detrimental to the upstream erosion protection system.

To assess the permeability of the drainage layer below the soil-cement, several piezometers were also placed above the geocomposite around the reservoir. These piezometers were tipped approximately three feet above the aggregate-filled gabion baskets at the toe of the soil-cement (see Figure 19). As expected, all of these piezometers above the geocomposite have recorded water elevations nearly equivalent to the reservoir’s pool elevation. An example of one the piezometers monitoring the drainage layer permeability is shown in Figure 20. As seen in this graph, the water elevation recorded by the piezometer follows the water elevation recorded within the reservoir.

![Figure 19: Configuration of the piezometers above and below the geocomposite](image1)

As seen by the reservoir stage shown in Figure 20, Tampa Bay Water perform a drawdown of the reservoir from elevation 136.5 feet NGVD29 to approximately elevation 110.0 feet NGVD29, a withdrawal of approximately 10 billion gallons. The drawdown has allowed the dam safety inspections to observe a larger portion of the soil-cement. Only minimal erosion and typical shrinkage cracking has been observed in the soil-cement, with crack widths of no more than a few millimeters.

Through the next two years, Gannett Fleming will continue to monitor the embankment dam and rehabilitated upstream erosion protection system. The successful drawdown and continual monitoring has proved the reliability of the design as a long-term solution. By ensuring adequate water supply and drought resistance, the rehabilitated reservoir minimizes reliance on groundwater for customers today and for the Tampa Bay area’s future generations.
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John Wilkes has 30 years of engineering experience developing specialty installations. He holds degrees from University of Virginia, Duke University, and Johns Hopkins University. In the 1990s, while employed at Ocean engineering building underwater robots, he began working with CARPI, developing underwater installations for geomembranes on dams. In 1997, he joined CARPI, starting the North American Carpi company. For the last 20 years Mr. Wilkes has been developing and executing geomembrane systems on dams, reservoirs, and canals for Carpi, having supervised more than 40 geomembrane installations in North America, South America, Europe, Asia, Australia, and New Zealand.

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