Use of nontraditional precast elements accelerates US331 over the Choctawhatchee Bay in Walton County Florida

Accelerated Bridge Design and Construction
Delivered Using the Design Build Method

Charles Rudie, PE, Parsons Brinckerhoff, Tampa, FL
John Poulson, PE, Parsons Brinckerhoff, Tampa, FL
Christopher Vanek, PE, Parsons Brinckerhoff, Tampa, FL

ABSTRACT

The team of Parsons Brinckerhoff and Skanska were awarded the FDOT design-build project to construct a new two-lane bridge along US 331 over the Choctawhatchee Bay. Not only did the existing two-lane hurricane evacuation route need additional lanes for capacity, but settlement of the existing approaches prompted the need to bypass and strengthen portions of the site's earthen causeways.

Nontraditional precast concrete elements were utilized to accelerate construction and reduce marine operations. Heavily reinforced pile heads, thickened webs at girder ends, and shear keys on pier caps, along with a complex multi-pier ship impact analysis, allowed the design team to reduce the number of piles, minimize the size of pier footings, and eliminate all diaphragms. The smaller footings allowed for the utilization of precast bathtub forms in all locations. Also, for the 200' span at the shipping channel, a high strength post-tensioned bar connection was developed between simple span precast girders in the thickened ends to create continuity over the channel piers and reduce cracking in the cast-in-place deck and closure pour. This procedure eliminated temporary supports and intermediate diaphragms.

Stabilization of the causeways involved an innovative soil mixing method to provide both settlement and storm protection for all approaches.

<u>Deep Foundations:</u>
-

Keywords: Soil Mixing, Deep Ground Improvement, Accelerated Bridge Construction, Precast Concrete, Ship Impact, Post-tensioning, Florida I-Beam (FIB)

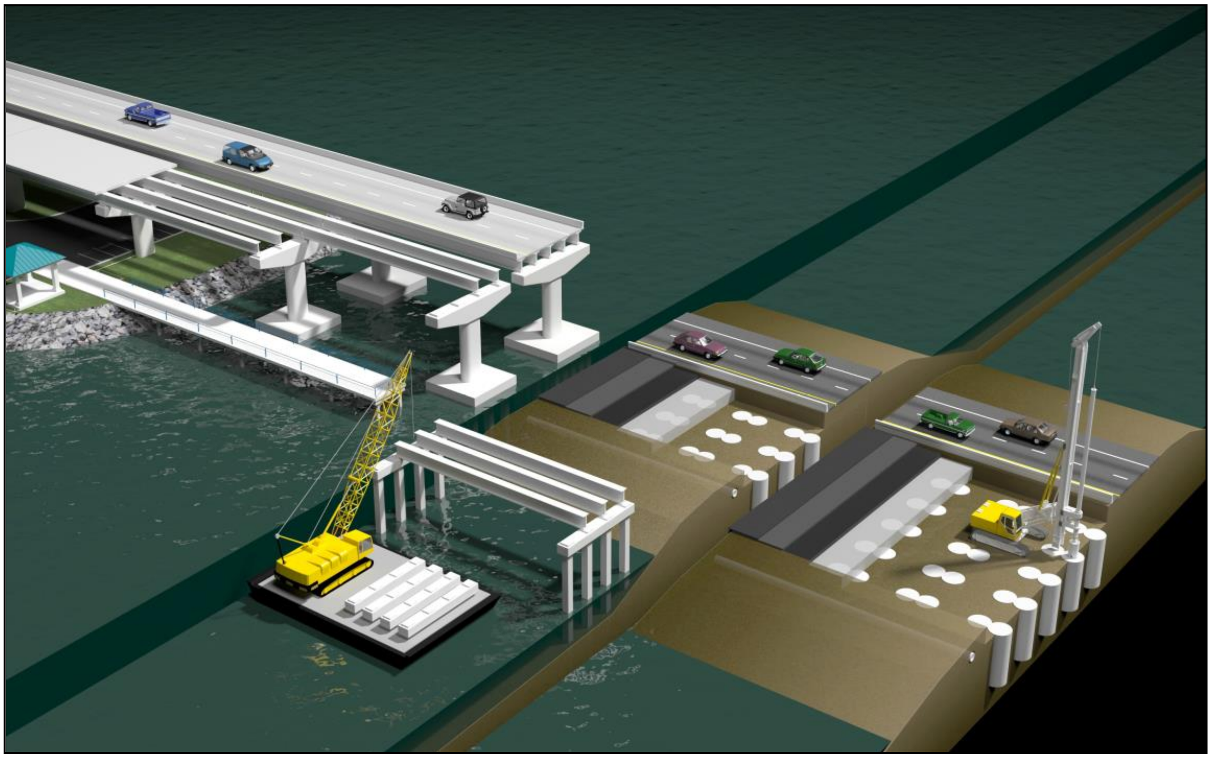

Figure 1 – Project Layout

INTRODUCTION

As the primary north/south roadway in Walton County, SR 83/US 331 provides a crucial link for tourism and commerce between the Walton County beaches and the heavily traveled I-10 corridor. The facility also serves as the primary evacuation route from southern Walton County. This 3.4-mile Project involves expanding the current two-lane crossing over Choctawhatchee Bay to a four-lane facility to increase capacity and dramatically improve mobility within the corridor.

In addition to expanding the capacity of the crossing, this project includes improving the existing causeway approaches which have exhibited chronic settlement problems since first constructed over 70 years ago. The gradual settlement of the causeways are the result of the material selected for fill, erosion, and dynamic loading. As trucks come off of the existing rigid bridge and impact the flexible asphalt pavement, an impact loading is propagated along the approach which causes uneven settlement. This uneven settlement results in an uncomfortable ride to the traveling public and has required milling and resurfacing of the roadway at an unacceptable frequency.

The owner gave flexibility to the proposers on how to mitigate the settlement problem. Options included extending the limits of one or both bridges, using pile supported roadway for approaches, or improving the soil supporting the approaches. Requirements were given for both storm protection and acceptable settlement limits depending on the option proposed. Our team determined the most practical solution was to bypass a portion of the causeway with the new northbound bridge and treat the approaches with a unique application of ground improvement.

The new bridge matches the span arrangement of the existing bridge, but the incorporation of the new Florida I-Beam (FIB) allowed for one piece precast concrete girders to be utilized across the entire bridge. The 200’ channel span beams and their associated 160’ anchor spans will be post-tensioned to improve continuity and more easily satisfy an owner requirement for future utility accommodation. The remaining spans are all simply supported. The alignment also matches the existing bridge to the maximum practical extent. The bridges are separated by 30’, which allows for adequate room to drive the new piles and also allows for materials to be delivered from the existing bridge, including concrete delivery for cast in place elements, as illustrated in Figure 2.

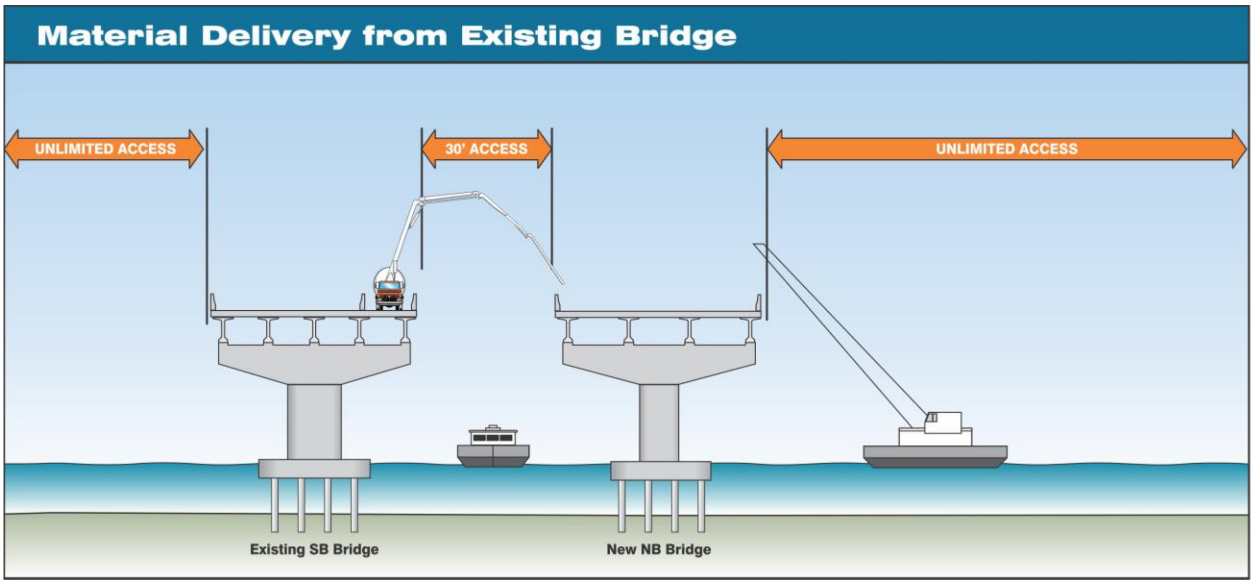

Figure 2 – Material Delivery from Existing Bridge

With the goal of reducing the cost and time associated with marine construction, our team focused on utilizing precast elements to the maximum extent possible. Due to the ship impact loads associated with the project, traditional precast elements had to be modified in order to create adequate load paths between piles, pile caps, and superstructure elements. In addition to these modifications, details were developed through coordination with the contractor and supplier to help further reduce schedule and in some instances eliminate a construction process.

BRIDGE DESIGN

Design of the 12,055 foot long bridge focused not only on efficiency in quantity of material, but also on maximizing the incorporation of precast elements of similar weight so that the same cranes could be used for erection. The majority of Cast-In-Place (CIP) elements consist of the bridge decks and the 21 piers surrounding the channel. With the selected alignment, most of these pours can be accomplished with equipment placed on the existing bridge, or on the existing causeway. Maximizing the use of precast elements and developing an efficient plan for concrete delivery, greatly reduced marine operations and the construction schedule.

In order to begin fabrication on the precast items as soon as possible, an efficient plan for the design schedule was also required. The design was not only broken into component submittals for substructure and superstructure elements, but also broken into two segments. The first 11 spans of the bridge are built over the existing causeway, and therefore consist of a less complex design when compared to the remaining 74 spans which are subjected to ship impact loading. By focusing on the causeway segment first, we were able to have sealed plans months ahead of the duration which would have been required to deliver the whole bridge in one shot.

SHIP IMPACT ANALYSIS

The design of the bridge crossing the Choctawhatchee Bay is subjected to very large marine traffic due to the proximity of local industrial facilities. The owner's RFP prescribed the equivalent static lateral impact forces based on providing a sufficient reliability against collapse for a critical water crossing. Due to the sudden dynamic impact loading from vessel collision, the structure was assumed to undergo significant inelastic behavior and redistribution of force effects. To prevent collapse of the superstructure, sufficient ductility and redundancy of the remaining structure was ensured to prevent catastrophic superstructure collapse.

To evaluate the response of the system under ship impact multiple piers were modeled in a soil-structure interaction program to evaluate both the non-linear response of the structure and soil strata, as shown in Figure 3 below. These models incorporated superstructure restraint to the ship impact forces through equivalent beam elements linked between piers. P-Y curves were developed for each foundation within the crossing and multiple loading cases of both transverse and longitudinal ship impact loading were applied to various piers.

To capture the effects of the highly non-linear dynamic loading, and the anticipation of plastic hinges forming within the pile groups and column, principles typically applied in earthquake engineering were employed to capture the over strength resistance of these sections. Whereby an increased strength of steel and concrete were evaluated based on strain hardening effects, confinement provided in the transverse steel, and increased ultimate compression strain. To implement these effects, constitutive models were developed to generate interaction curves for evaluating the strength of each section. These models were implemented within the soil-structure interaction analysis to capture the softening of the structure during plastic hinge development.

To ensure catastrophic superstructure collapse was not initiated the non-linear response of the piers were verified using push-analysis. Here the model solutions were verified that the structure had sufficient ductility and a redundancy during impact was realized without premature failure and the onset of significant deformation occurred.

By utilizing an iterative process of applying forces to both multi-pier and single pier models, proper load distribution and acceptable performance was achieved in a very efficient system. By taking advantage of strain hardening and developing enhanced pile details acceptable to the contractor and supplier, the individual pile capacities were increased, allowing the team to reduce the overall number of required piles and thereby the construction schedule.

MODELING CHARACTERISTICS - The bridge piers were modeled in FBMulti-Pier in which the lateral soil-pile response was determined in accordance with geotechnical engineering recommendations. Non-linear effects were considered for both the soil-pile and structural elements in the design. To account for the increased strength of the concrete due to the strain rate loading and the presence of increased confinement provided in the structural elements, a stress-strain curve was developed for confined concrete based on Mander (1) for each element. The stress strain curve for steel was also modified due high strains expected in the structure under ship impact. A bilinear stress strain model with strain hardening of 1% was developed to capture this behavior for the overstrength condition. The transfer of forces from the substructure through the superstructure was accomplished using 3-D beam elements with rigid links between the beam seats and 3-D beam element. The beam element properties were determined using an equivalent torsional stiffness method carried out in a FEM Model of each bridge deck typical section. The beam sets were connected to the pier cap by the use of only transverse restraint (Roller). This connection is accomplished with shear keys placed on the pier cap surrounding the beam seats. A number of load cases were run for both transverse and longitudinal loading at both expansion and non-expansion piers to verify the distribution of the ship impact loading were sufficient under all scenarios.

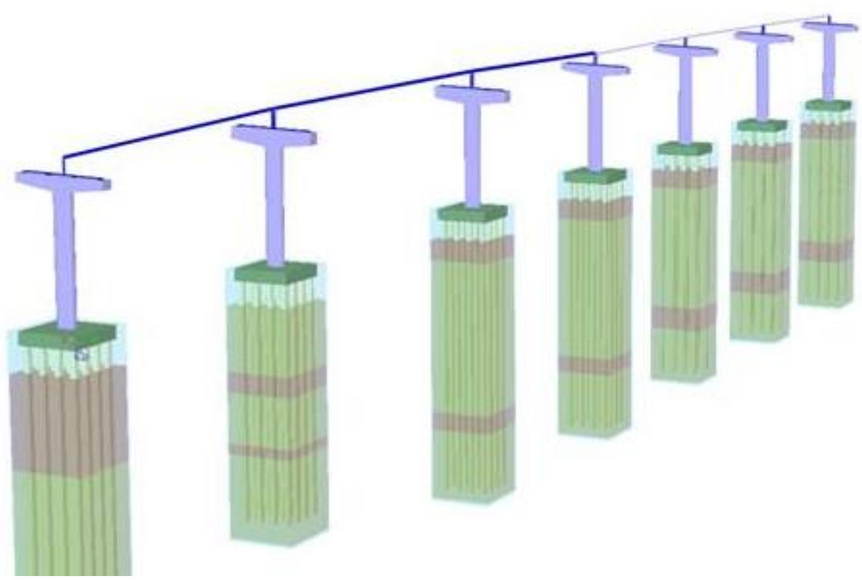

Figure 3 – Seven Pier FEM Model from FB Multi-Pier

SCOUR WITH VESSEL IMPACT - The design of all foundation units required the consideration of both long-term and 100 year scour events. For the design of the piers, the load/scour combination 1 with the RFP required vessel collision loading and ½ Long Term scour was applied in each model. For the pile bents, since the RFP required vessel collision loading is greater than the minimum impact vessel load per AASHTO LRFD 3.14.1 (2) the minimum impact load per the RFP was applied at ½ 100-Year Scour.

DESIGN METHODOLOGY-DAMAGED PERMITTED (SDG 2.11.4 (3) and AASHTO LRFD 3.14.3 AND 3.14.14.1) - Based on these modeling assumptions the modified extreme event II limit state was applied to each group of piers. Whereby extensive damage and local failure of substructure elements including piles is accepted provided that there is sufficient ductility and redundancy of the remaining structure exists to prevent catastrophic superstructure collapse. Therefore, plastic hinges are expected to form in the piles and appropriate detailing was carried out to ensure the capacity of the system was maintained. As a result, the intent of the models are to confirm the overall stability of the system under the design impact force, verify the nominal bearing resistance of the axially loaded piles are not exceeded and determine the design demands on the pier elements under ship impact. To ensure the requirements are met, the non-linear analysis carried out in FBPier was investigated to ensure the mathematical solutions realized are physically reasonable. Here the behavior of the system was compared with different non-linear responses as illustrated in Figure 4. The most desirable response of the structure shown is case 1 whereby the structure displays sufficient ductility upon yielding of the structure at point b and as the structure softens and reaches a maximum load at point c. Other structural responses shown are not as desired such as limited ductility or local premature failure of elements.

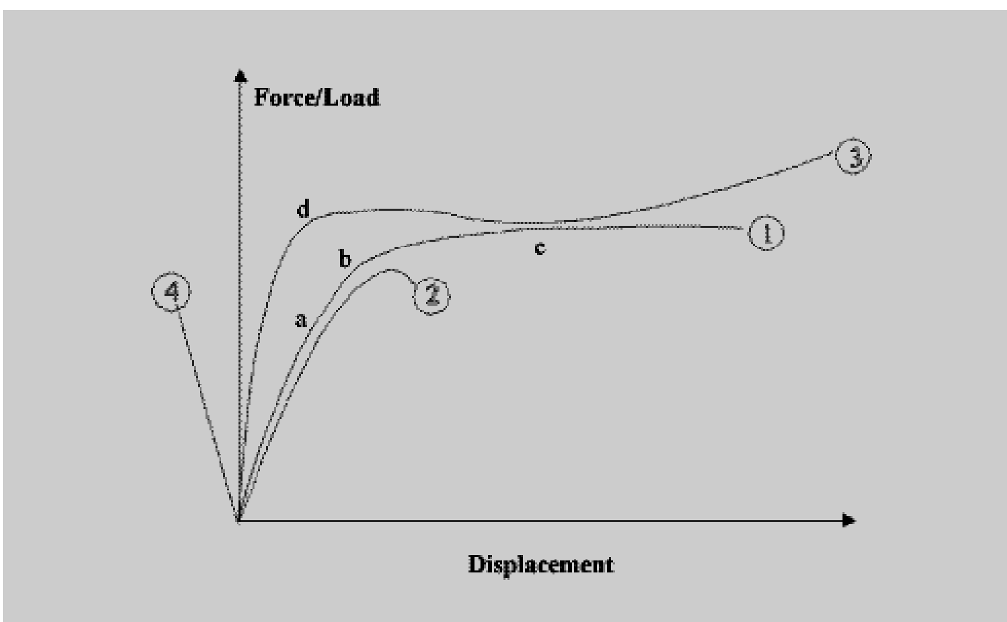

Figure 4 - Different Types of Load Displacement Response

To investigate the structural response a push over analysis was performed on a typical pier foundation. The load-deformation response is shown in figure 5. As can be seen from figure, the response of the structure shown is similar to the graph shown for case 1 in Figure 4. The initial response of the structure is quite linear until the onset of yielding occurs around 2000kips. However, the redundancy in the pier units allow further distribution of loads and enable the structure to continue to deflect and reach higher loads until the maximum load is reached at 3000kip. Afterward, convergence of the structure is not able due to the time and number of iteration required and the trend line represents the anticipated structure response. The behavior of the systems shows the system has sufficient ductility upon yielding and load distribution to enable the maximum RFP required ship impact loading of 2600kips. This conclusion was further supported by investigating the results of individual pier models under the ship impact loading. Some models do display demand to capacity ratios in excess of 1.0. However, not all piles have experienced plastic hinging under the design loads. Therefore, the entire structure is still intact and capable of transmitting more lateral forces until plastic hinging is formed in all piles in which the structure reaches a very large deflection and engages in complete failure. This behavior is shown below in Figure 5 and can be seen in the plastic hinge diagrams under the load cases plotted below.

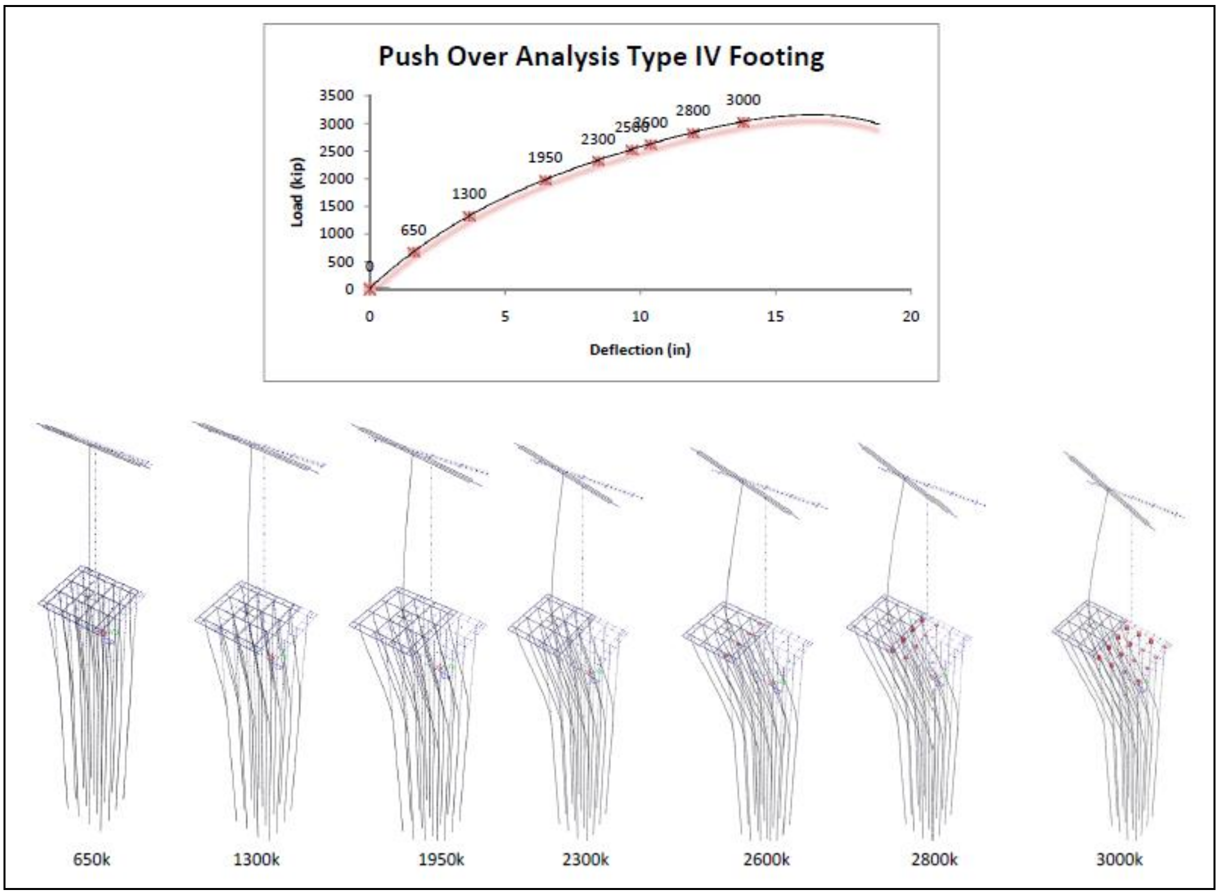

Figure 5 - Load Displacement Response for Pier Type IV

SHIP IMPACT ANALYSIS SUMMARY – By utilizing an iterative process of applying forces to both multi-pier and single pier models, proper load distribution and acceptable performance was achieved in a very efficient system. By taking advantage of strain hardening and developing enhanced pile details acceptable to the contractor and supplier, the individual pile capacities were increased, allowing the team to reduce the overall number of required piles and thereby the construction schedule.

SUPERSTRUCTURE DESIGN

The type of superstructure selected was heavily influenced by the RFP's requirements of a 43'-1" typical section and that the new northbound bridge match the span arrangement of the existing southbound bridge. This span arrangement, and the aggressive environment created by the saltwater in Choctawhatchee Bay, made precast prestressed concrete girders with a cast-in-place (CIP) deck the obvious choice for the superstructure. For the typical span, four 72" Florida I-Beam (FIB72s) with a 12' spacing and a length of 142' was found to be the most economical. The channel span required the use of the deeper FIB96 girder.

As previously discussed, ship impact loads were transferred to the superstructure through shear keys. This transfer typically goes from the shear key to the deck through end diaphragms. Our team wanted to avoid end diaphragms and use the owner's recently implemented thickened end slab not only to save money, but to maintain a RFP mandated future utility corridor. Therefore, in order to transmit the ship impact load from the substructure to the deck, the girders on the spans surrounding the channel featured a thickened end block, as can be seen in Figure 6. The length of the end block was determined by the distribution of the shear force resultants from ship impact models.

CHANNEL SPANS - These end blocks served double duty on the channel spans where a post-tensioned connection was desired for greater continuity and reduced cracking in the deck over the channel piers. The RFP-mandated 200-foot length of the channel span allows for a one-piece FIB96 girder to support the superstructure dead loads simply supported (non-continuous). Therefore, our team developed a cross section with 5 FIB96 girders designed with adequate strength to simply support all loading. However, maintaining compression across this connection provides a superior product which is less susceptible to cracking and corrosion. Therefore, our supplier will cast the FIB96 girders with ducts in the eight-foot-long end blocks to be utilized for an innovative post-tensioning system. After a CIP closure is made between the end blocks of the adjacent girders, high-strength threaded bars will be placed across the continuous end blocks, as shown in Figure 6. These bars will be post-tensioned to support the negative moment associated with the live loading.

Placing the post-tensioning bars in the end block outside the web of a typical FIB96 allows for easy installation and inspection of the anchorage zone. Spanning the entire length with one segment also eliminates the need for intermediate diaphragms. This system offers similar benefits to the spliced girder system utilized in the existing southbound bridge, but eliminates the need for any temporary falsework. The girders will be placed with two barge mounted cranes directly on the finished piers.

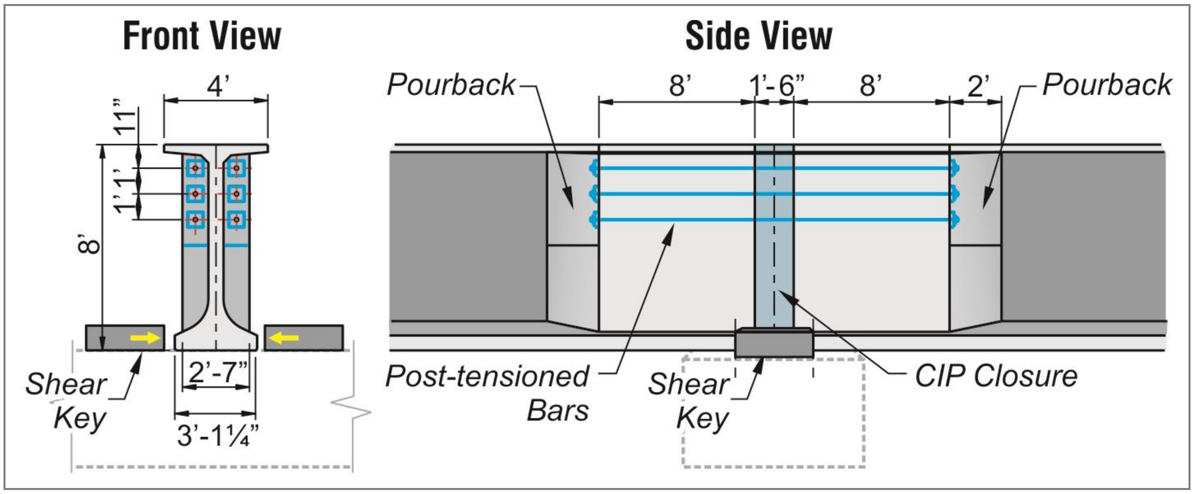

Figure 6 – Post-Tensioned Connection Detail

LOW LEVEL BENTS - Precast bent caps will make up over 70 percent of the pile caps on the project. This system creates more consistent material properties with higher tolerances than a traditional CIP cap. All elements within the splash zone (salt spray) will contain a silica fume ad-mixture to further reduce the permeability of the concrete and increase resistance to corrosion.

The most efficient structure for the low-level portion of the NB bridge is to utilize a four-pile bent with a 30-inch pile directly below every FIB72 beam line. The lateral stiffness of the 30-inch pile eliminates the need for battered piles and speeds construction. The piles are fixed into the cap with the flexibility to incorporate two different connections, as will be discussed below. This fixity provides increased resistance to lateral wind and ship impact loads. Precast pile caps will be incorporated to speed construction and reduce the challenges associated with CIP construction over the water.

The one beam line per pile structure offers simple repetitive construction. This model offers the most expedient construction as one 142-foot span consists of only 10 major elements:

- 4 – 30-inch Square Precast Piles
- 1 – Precast Cap
- 4 – FIB72 Girders
- 1 – CIP Concrete Deck

The design makes efficient use of construction equipment, as the weights of each precast element are similar. This allows the same equipment to be utilized throughout the project, as illustrated in Figure 7.

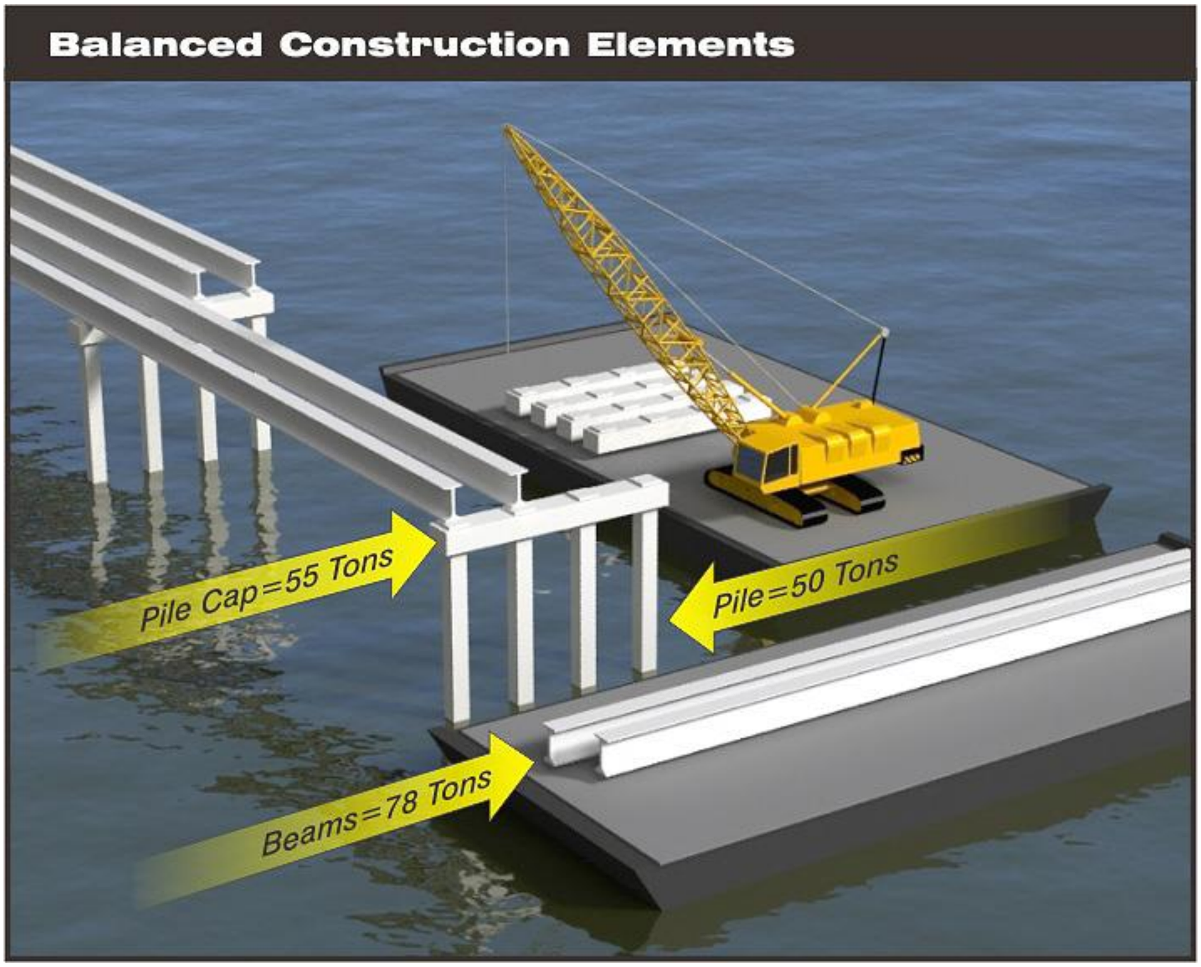

Figure 7 – Precast Element Weights

Two different connection details between the precast cap and pile have been developed so that cut-off can be made below the head or piles can be driven to grade. The first option involves cutting off the head of the pile and placing a reinforcement cage with a plug into the now exposed void of the pile. This reinforcement cage extends into the corrugated void contained in the precast cap. The void is then filled with CIP 8,500 PSI concrete. The second option is more desirable, but requires driving the pile to grade. For this option, the piles are cast with ten corrugated inserts in the head of the pile, which after pile driving is finished, are filled with epoxy and a #11 bar that will extend into the precast cap. Like the first option, the void is then filled with a CIP 8,500 PSI concrete closure. If necessary, this detail also can also be utilized to create a built-up or drivable splice, which makes this system highly adjustable for the variable soil conditions of the Choctawhatchee Bay. Details of these three conditions are shown below in Figure 8.

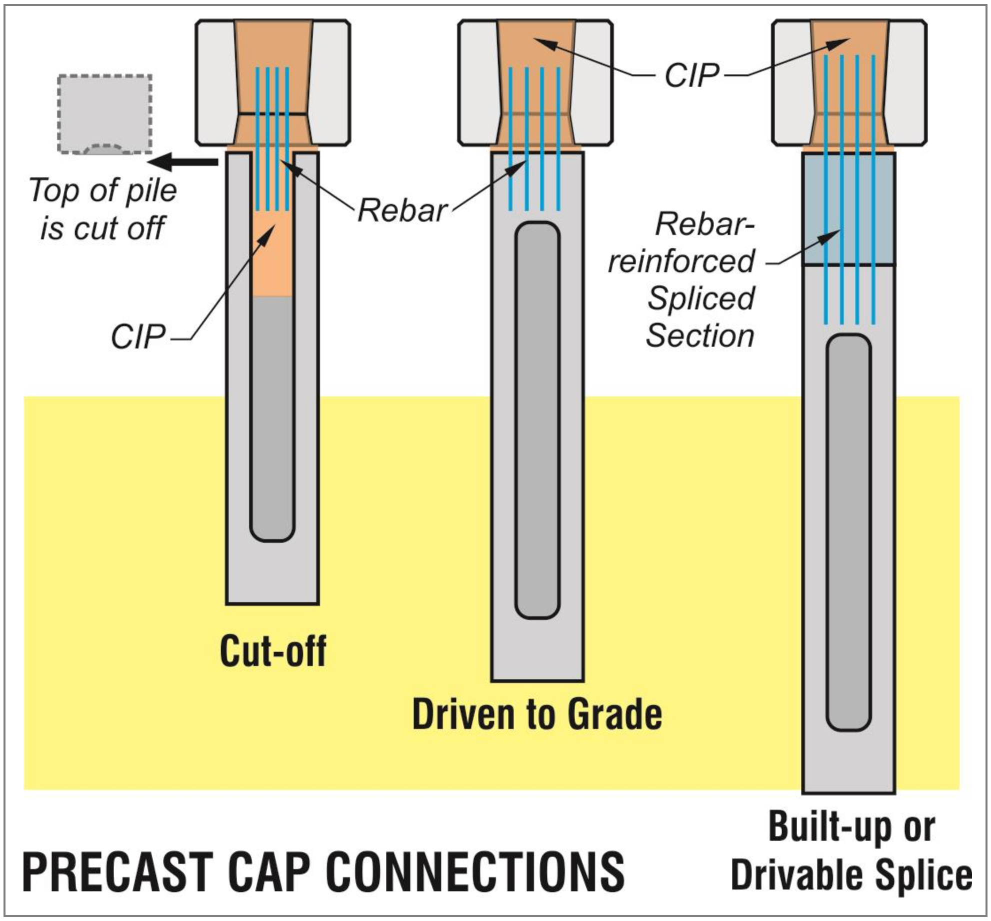

Figure 8 – Bent Cap Connection Options

PIER DESIGN - The owner set the minimum vertical channel clearance at 65’. This necessitated a gradual increase in height from the typical pile bents to the piers surrounding the channel. The shape of all new piers will be similar to those of the existing southbound bridge. The size of the footing and number of piles varies depending on the ship impact loading and height of the pier. Three different high-level pier types with similar aesthetics will be utilized to support the required loads, as illustrated below in Figure 9.

- Type 4 Piers are located within 300 feet of the channel and provide a ship impact resistance of 2,600 Kips.
- Type 3 Piers are located between 300 and 1,000 feet of the channel and provide a ship impact resistance of 2,200 Kips.
- Type 2 Piers are located between 1,000 and 1,600 feet of the channel and provide a ship impact resistance of 1,100 Kips.

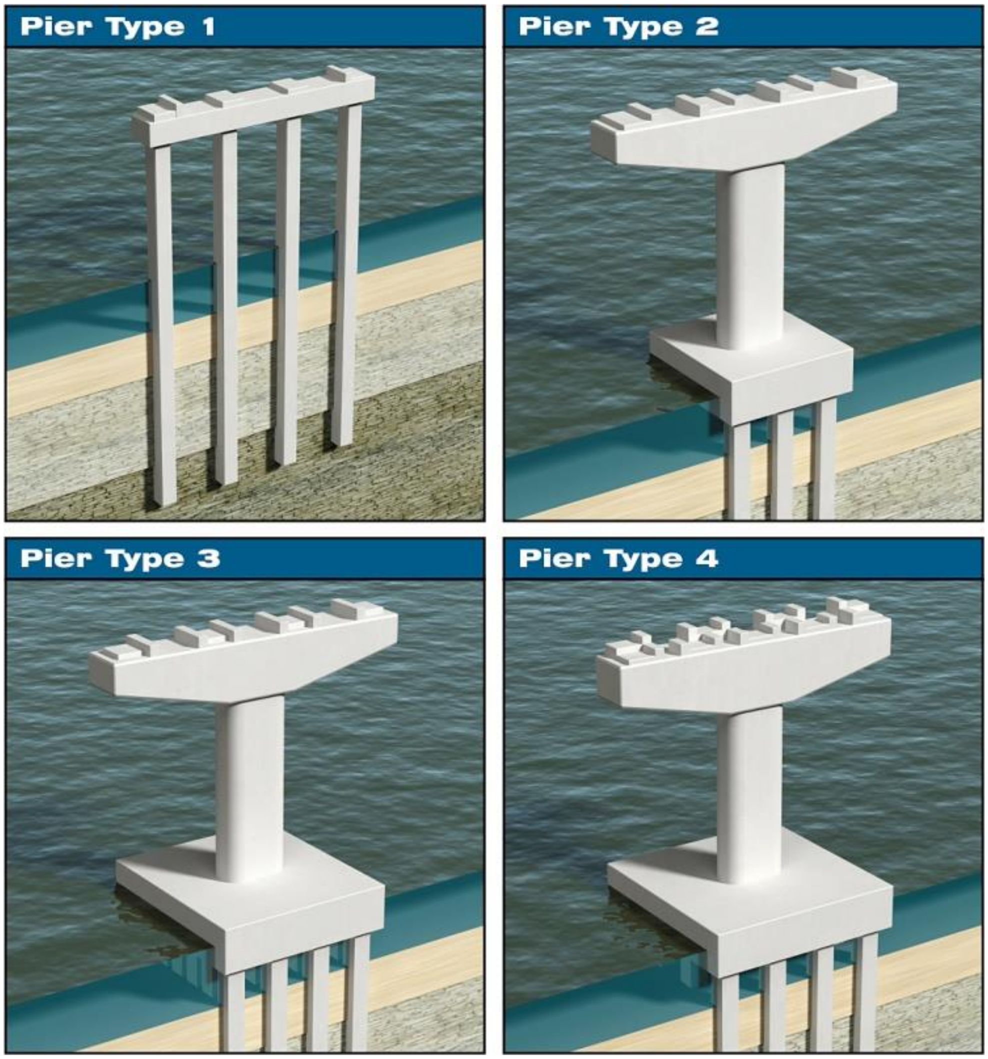

Figure 9 – Four Pier Types

The main difference between these pier types is the number of piles needed and the type of piles utilized, which will be discussed below. All of the piers consist of CIP caps and columns, but thanks in part to the pile design and elaborate ship impact analysis, all footings were kept small enough to take advantage of a precast "bathtub" footing form.

Figure 10 – Precast “Bathtub” Footing Form

This form will be placed and suspended from the pile cluster using threaded rods. This precast piece serves as the bottom form and side forms for the pile cap. After the annular voids around the piles are sealed, the bathtub form is dewatered, the rebar cage is set, and finally the concrete is cast. The threaded rods are encased in PVC pipe and removed after the concrete is set. The sides of these forms are cast with stainless steel bars that extend into the footing’s rebar cage. This is done to ensure that it will remain in place and maintain its aesthetic characteristics. This precast form is not part of the permanent structural design element, but provides additional cover for corrosion protection. This innovative forming method will eliminate time-consuming and environmentally invasive cofferdam construction.

DRIVEN PILES

Driven pile foundations are utilized throughout the Project. Driven piles are well suited for the geotechnical site conditions, provide proven capacity, improve the construction schedule, and eliminate the environmental impacts associated with drilled shaft construction.

The owner has established resistance factors based on testing methods to be applied to factored pile loads when establishing nominal bearing resistance. Based on our knowledge of the highly variable soils in the project area, and the soils tendency to setup after initial drive, our team determined it would be worth the cost and effort of dynamically testing every pile on the job to utilize a higher resistance factor. By performing five static load tests across the job, along with the dynamic testing of every initial pile drive and set-check, we have been able to take advantage of a 0.85 resistance factor.

With the goals of limiting pile lengths and getting an early start on pile driving, a test pile program consisting of one pile at every other pier was established. In order to take precast manufacturing off of the critical path, most of these piles were cast at the contractor's maximum handling capacity of 160' length. This allowed the team to quickly test multiple pile tip elevations, targeting efficient production lengths. As previously discussed, the piles contain a heavily reinforced upper section. Therefore, the test piles contained an especially long reinforced section so that a large portion could be cut-off if necessary without impacting the strength requirements. The test pile program proved out what our soil exploration program suggested. The piles achieved little resistance during initial drive, but substantial capacity on re-strikes. This knowledge not only helped provide better estimates for production pile lengths, but gave the team a better level of comfort for targeting initial drives to grade elevation. Driving the piles to grade eliminates the step of cutting off pile tops and allows for incorporation of the preferred connection detail with inserts in the head of the pile.

PRECAST PILE TYPES - Our design is based on three similar pile types. The standard FDOT 30-inch Square Voided Precast Pile is used at the end bents. A modified version of the standard FDOT 30-inch Square Voided Precast Pile with additional prestress strands and mild steel placed in the top portion of the pile is used for the bents and the piers on the causeway. A solid 30-inch Square Precast Pile with maximum practical prestressing and substantial mild reinforcement in the top portion of the pile is utilized for piers in the water subject to high ship impact loading.

The minimum required length for the additional mild reinforcing in the top of these piles is given in the plans. Then with the help of the geotechnical engineers, the contractor can decide how much should be added to the estimated pile length and equally increase the length of the extra mild reinforcing placed in the top of the pile to compensate for potential cut-off.

Incorporating these modified 30" pile types was the result of an iterative process in which different pile types and the associated number of piles were estimated and weighed against the total estimated pile lengths and the implications to the footing size, alignment, etc. One reason this exercise was done is due to the fact that often piles associated with piers subject to large ship impacts are controlled by structural minimum tips as opposed to axial soil capacity. In the end, it was determined that the savings from reduced pile lengths, reduced material quantities, reduced pile driving duration, and the ability to use precast footing forms more than offset the increased cost associated with these modified piles. Taking advantage of the modified piles reduced the required number of piles by over 20%. With 86 piers across the job a total of just 520 piles were required.

CONSTRUCTION

During the initial phase of construction, priority was set on starting soil mixing. If the team could get soil mixing started early in the schedule, the four phases of soil mixing could be accomplished with one mixing rig and minimal interference with the bridge construction. To achieve this goal, several major utilities needed to be relocated and temporary asphalt needed to be placed so that traffic could be shifted making room for the rig and associated batch plant. Submitting the roadway plans as one of the first component plan sets allowed us to achieve these goals.

Figure 11 – Soil Mixing

After completing the test piles on the landside segment of the bridge, production pile fabrication was begun immediately. Construction of the first bridge segment could then begin as the second segment design was finalized. As the first segment approached completion, marine pile driving on the second segment began.

Construction on the waterside segment of the bridge began with three fronts. As construction of the elements surrounding the channel are the most time consuming, one rig began pile driving piles for the channel piers. A second front began at the south end of the project and moved toward the channel. The final front began at the north end of the bridge to drive pile for the bents and moved south toward the other rigs.

Figure 12 – Pile Driving

Once the driven piles are certified for one of the pile bents, any necessary pile cut-offs are performed. At this point all pile tops are approximately three inches below the bottom of bent cap elevation. Depending on if the piles were driven to grade, or cut-offs were required, one of the pile cap connection operations described above is performed. After the bars are epoxied into the pile, or the reinforced pile plug is cast, the cap is placed on friction collars which double as the form for the bottom three inches of the connection. The CIP connections are then cast, in some cases concurrently with the beam pedestals on top of the bent cap. This process requires very little CIP concrete quantity to complete, making it expedient and reduces the risk of having concrete rejected on site.

Figure 13 – Setting Precast Bent Cap

As pile driving certification is completed on the pier footings, the precast ”bathtub” form is placed and suspended from the pile cluster. The footing rebar cage is tied either on the causeway or on a barge while piles are being completed so that it can be ready to place with the precast form. Once the footing is cast, the remainder of the pier is placed using traditional formwork.

For both the precast pile caps and the precast footing forms, several completed elements are on site ready to go prior to completion of pile driving. These elements are cast with plan dimensions and can be quickly installed if piles are driven within approximately four inches of plan position. However, the team is waiting on production for a portion of these pile caps so that they can be cast after completion of pile driving to better match the as-built conditions if circumstances in the field dictate piles being driven outside of the planned tolerances. The hope is that this flexibility will not only save money by eliminating waste, but that the uniformity in construction process will eliminate confusion in the field and lower the probability for a water breach on the pier footing forms during CIP operations.

Traditional overhang forms are being used for the exterior portions of the deck, but stay-in-place forms are being used for both the deck and thickened end slabs. As the thickened end slabs are a new detail for the area, an iterative process with the contractor, supplier, and engineer was required to develop a system that was easily installable and met the owner’s preferred details for continuous and non-continuous deck joints.

SOIL MIXING

In order to exceed the owner’s requirements for storm protection and settlement of the existing causeways, our team developed a soil stabilization method which included both deep and shallow soil mixing with a wet cement grout. A 10-foot thick rigid platform of Shallow Mixing Method (SMM) will be constructed across the entire typical section from outside shoulder to outside shoulder. This platform will be supported by Deep Mixing Method (DMM) elements that extend down a minimum of 45 feet to the bearing layer, as illustrated in Figure 14. This system is far superior to the traditional geo-textile reinforced earth platform which results in greater potential for differential settlement, especially in the difficult compaction regions associated with the causeway. This system provides a bridge on land across the entire causeway with incredible resistance to storm damage.

Creating a bridge on land required satisfying several key requirements, including compressive stress limits in the deep elements, bending stress in the shallow elements, adequately distributing loads into soil, allowing no traffic on cantilevered shallow elements during traffic shifts, and avoiding existing underground utilities. In order to achieve these goals, engineers had to layout every element across the job and analyze loading from the various traffic patterns. The results were a plan set consisting of an astounding 10,000 precisely located twin column elements that equates to a total soil mixed area of 450,000 square feet, which is roughly equivalent to the square footage of the new northbound bridge.

To accomplish all of this soil mixing requires an enormous amount of cement. The shallow elements, which must have a compressive strength of at least 75 PSI, require 3 tons of cement per element. The deep elements, which must have a compressive strength of at least 150 PSI, require 21 tons of cement. Currently, the soil mixing construction is roughly half complete and approximately 27,000 tons of cement have been used to strengthen the existing causeway.

EXISTING SOILS – Foundation and Geotechnical Engineering (FGE) determined that the in-situ soil supporting the roadway and causeway sections is predominately consolidated sands, silty sands and clayey sands which have been only lightly loaded over their 70+ year service life. The original fill placed in the 1930s was placed on loose material leaving the roadway within the causeway to be supported by approximately 45 feet of loose/weak material. Therefore, all soils beneath the roadway to a minimum depth of 45 feet will be stabilized utilizing a combination of deep and shallow soil mixing.

EROSION RESISTANCE - The rigid transfer platform provides a roadway with incredible resistance to future storms. Using the SMM for the 10-foot layer under the entire typical section, we are essentially installing material commonly used for erosion protection under the entire causeway road system. Our team has designed the ground improvements to withstand the 100-year storm event from shoulder to shoulder.

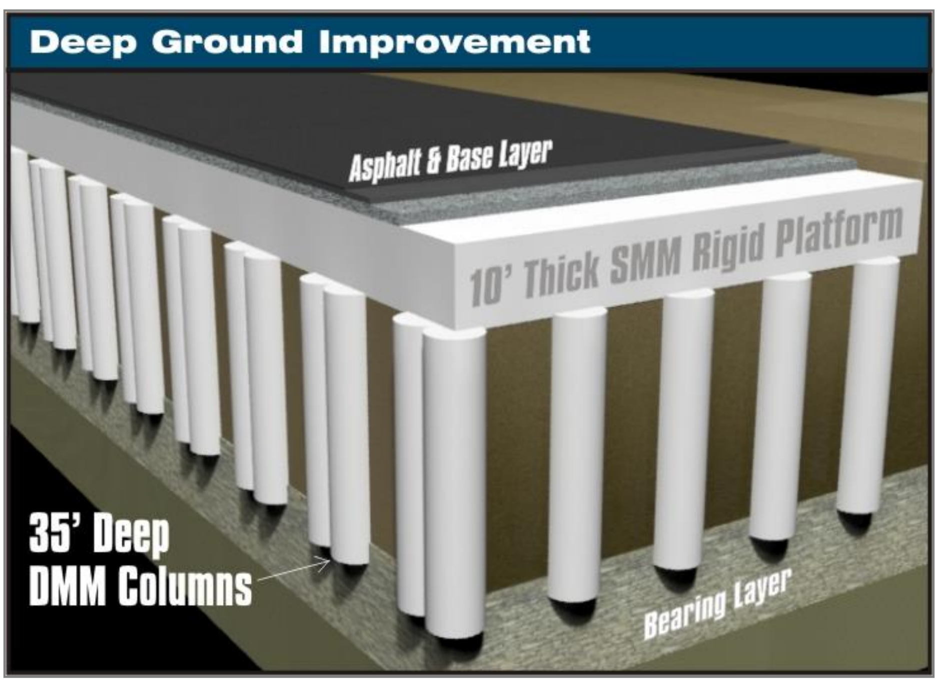

Figure 14 – Model of Deep Ground Improvement

SOIL MIX EQUIPMENT – Soil mixing sub-contractor Treviicos' Trevi Turbo Mixing (TTM) is a recent, yet proven advancement of deep mixing technology which is being utilized on the Choctawhatchee Bay Bridge project. The TTM methodology injects measured quantities of cement grout by high velocity jets simultaneously with mechanical mixing of the in-situ soil, as shown below in Figure 15. A concentrated mixing is achieved through the combined effect of the mechanical energy delivered by the drilling tool and the hydraulic energy provided by injected grout. The ground is cut by the jet action and mixed by blades on which teeth are mounted. Drilling speed and grout pressure is determined by the soil characteristics and design requirements.

Typical equipment used for this work will be SOILMEC SR Series TTM rig with a double shaft configuration that will treat approximately 50 square feet of causeway with each installation. This large area associated with the double shaft configuration equates to a 30 percent gain in efficiency when compared to a single shaft configuration, which helped our team target a schedule 16 months shorter than the owner's requirement. Another key feature of the SOILMEC SR series is the computer automated Drilling Mate System (DMS).

With DMS, the TTM method of soil mixing can install and track the exact cement/binder content and distribution both vertically and spatially. Utilizing GPS, the DMS automatically delivers pre-determined cement slurry volumes. This technology is considered a "wet system" due to the fact that the binder is injected in the form of liquid grout slurry through outlets positioned at the end of the hollow stems of the rig. Typically, the mixing of the soil with the grout takes place both during the down-stroke and up-stroke of the mixing tool, while the high-pressure grout injection occurs only during the down-stroke phase. The additional energy provided by the pressurized grout jets greatly improves the quality of the mixing of the soil with the grout and reduces the time required for the installation of the soil-mix elements.

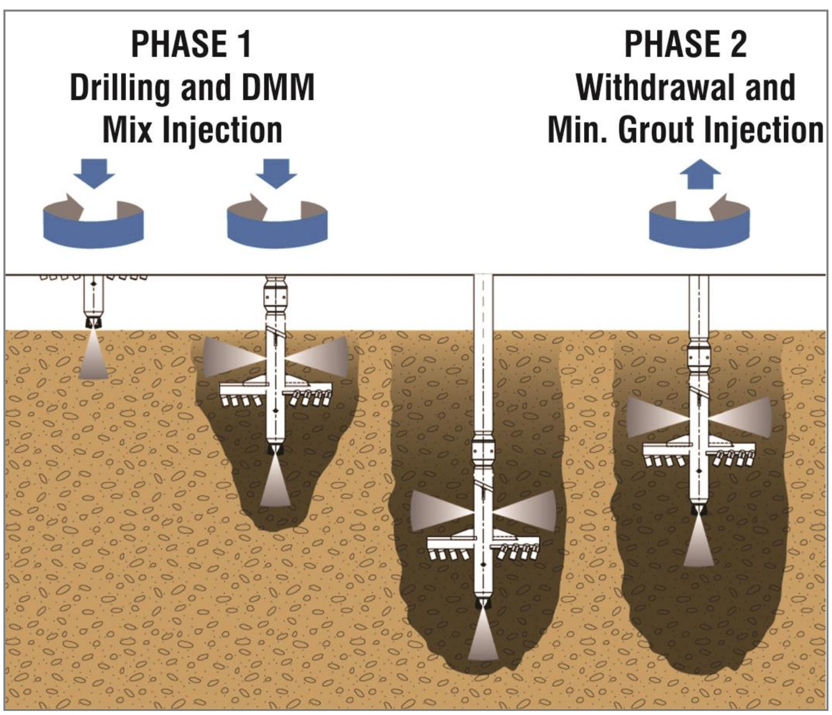

Figure 15 – TTM Soil Mixing Method

BENCH SCALE TEST - Using soil samples from the SPT boring program, bench scale testing was performed in a laboratory to confirm the predicted strength of on-site soil mixed with cement slurry. Once the required cement content was confirmed and a mix design established, a full scale test program was conducted on the southern causeway under a section of the future NB lanes.

EMBANKMENT TEST - The test section consisted of producing a platform based on the soil mix element pattern designed for use under the most heavily loaded section of the MSE wall approach. As shown below in Figure 16, this section was then loaded with 20’ of fill, which is two more feet than what the pattern will see in the actual production location.

The tests included both short-term plate load tests and long-term settlement evaluations. Plate load tests were performed on the shallow soil mixing transfer platform between and directly over deep columns to evaluate the transfer of wheel loads through the SMM layer to the columns.

The test section was surcharged above the highest anticipated embankment height after plate load testing. Both pore pressure and settlement was monitored continuously during embankment placement and daily thereafter. Over three months of data was collected before removing the fill from the test locations prior to continuing production soil mixing. Continued monitoring will be conducted throughout the duration of the project to verify anticipated long-term settlement estimates. The same extensometers used for the test section will be placed every 1,000 feet along all three causeways so that settlement can be continuously monitored during the design-build teams value added seven-year warranty period and beyond.

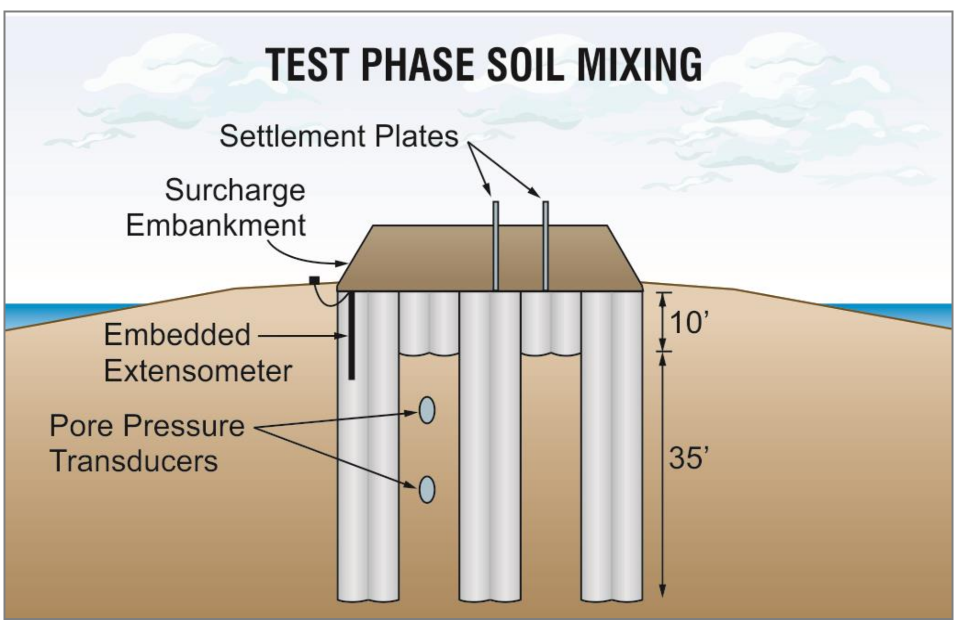

Figure 16 – Embankment Test Instrumentation

INSTRUMENTATION - Direct-push pore pressure transducers were installed between columns and settlement plates were placed both on and between deep columns. Columns under and/or surrounding the plate load test were instrumented with full depth magnetic extensometers to isolate the longitudinal distribution of column compression, allowing us to determine where settlement occurs in the system, as well as the magnitude.

EMBANKEMENT TEST RESULTS – The test section performed incredibly well. While placing 1.5 times the design loads on the static plates, they experienced less than 1/16” of settlement. During the entire surcharge test, the extensometers recorded less than ¼” of total settlement.

CONCLUSION

The decision by the FDOT to utilize a design-build contract for the Choctawhatchee Bay Project encouraged the team of Skanska and Parsons Brinckerhoff to incorporate accelerated construction techniques and innovative precast designs to achieve an efficient system with a greatly reduced schedule. Through close coordination between engineers and contractors, a new application of a proven ground improvement system will create a stable storm resistant foundation for the bridge approaches with very little impact to the heavily travelled roadway. Delivering these strategies for US 331 over the Choctawhatchee Bay will place the travelling public on a low maintenance four lane facility in a reduced amount of time while providing several value added features.

REFERENCES

(1) J.B. Mander, J.N. Priestly and R. Park, Theoretical Stress-Strain Model for Confined Concrete, Journal of Structural Engineering, Vol. 114, No. 8, August 1988

(2) American Association of State Highway and Transportation Officials, AASHTO LRFD Bridge Design Specifications, 7th Edition, Washington D.C., 2014

(3) Florida Department of Transportation, Structures Design Guidelines, FDOT Structures Manual, Volume 1, January 2013