SPLICED GIRDERS FOR THE GARDEN STATE PARKWAY MULLICA RIVER BRIDGE

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ABSTRACT

The New Jersey Turnpike Authority is presently widening a 50-mile section of Garden State Parkway in the southern part of NJ between Mileposts 30 and 80 from two lanes to three lanes with shoulders in each direction. The Mullica River crossing is the largest bridge within this widening program at about 1000 feet between abutments. To accomplish the widening across the Mullica River the construction of a new 1230-foot long parallel bridge to the east of the existing structure is required.

Environmental restrictions related to in-water construction limited the available time to construct pier foundations within the waterway to July 1st through December 31st. This requirement prompted the designers to select a span arrangement that limits the number of piers in the waterway. Eight foot diameter drilled shafts, utilizing Self Consolidating Concrete, were selected for the pier foundations. Prestressed concrete spliced girders were selected over steel girders due to reduced maintenance in the marine environment, and lower cost of the superstructure. The post-tensioning for each spliced girder utilizes four 1230-foot long tendons, which are continuous for the length of the bridge. This paper presents the alternatives considered and the structure that was advanced to final design and is presently under construction.

Keywords: Spliced Girder, Plastic Duct, Self Consolidating Concrete, Sustainability

INTRODUCTION

The Mullica River Bridge is located at Garden State Parkway (GSP) milepost 49.0, between the City of Port Republic, Atlantic County, and Bass River Township, Burlington County (see Figure 1) in New Jersey. The bridge is within the limits of the New Jersey Turnpike Authority's *'Widening of the Garden State Parkway from Interchange 30 to Interchange 80'* project (GSP 30 to 80), Order for Professional Services (OPS) No. 133-572D. The GSP 30 to 80 project scope includes the planning and preliminary design for the mainline widening consisting of the construction of a third travel lane for both the northbound and southbound directions.

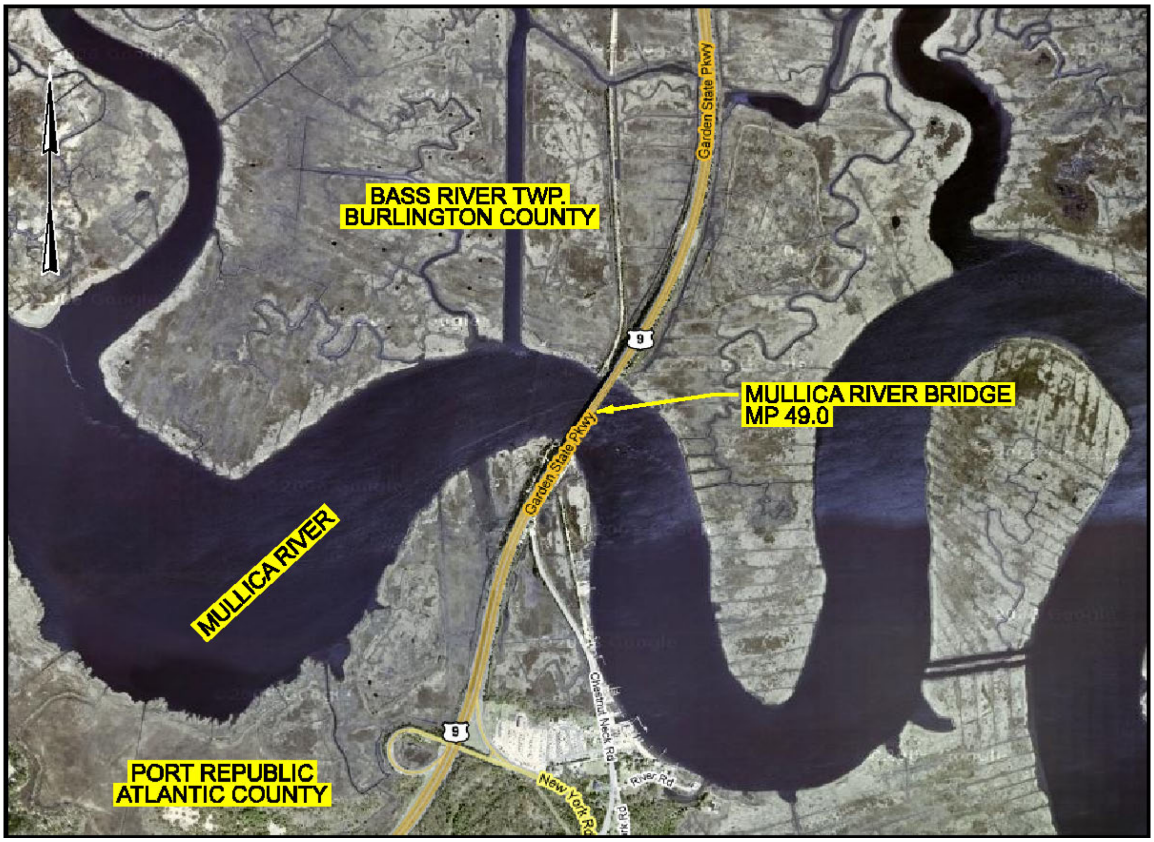

Figure 1 – Project Location

The GSP 30 to 80 Preliminary Bridge Study (February 2000) concluded that a new parallel, independent structure constructed to the east of the existing Mullica River Bridge would be the best feasible option. Two separate construction contracts are planned for the project and designed under *OPS No. P3026, Rehabilitation and Widening of the Mullica River Bridge*. The first Construction Contract, P100.024, which includes construction of the New Mullica River Bridge, is currently under construction and planned to be completed by approximately December 31, 2010. The second Construction Contract, P100.025, which includes deck reconstruction, structural rehabilitation and seismic retrofit of the existing Mullica River Bridge, is planned to be completed by June 30, 2012.

EXISTING STRUCTURE

The existing Mullica River Bridge, which is owned, operated and maintained by the New Jersey Turnpike Authority (NJTA), carries the northbound and southbound roadways of the GSP which is designated as US Route 9 over at this location. Constructed in 1954, it currently carries two 12’-0” lanes in each direction with shoulders varying from 3’-4” to 1’-6”. The Mullica River is a navigable tidal waterway at the GSP crossing. Span 4 is designated as the navigable span with a horizontal clear channel width of 80’-0” at a 10 degree skew to the bridge, a navigation vertical clearance of 30’-0” minimum, and an unobstructed channel depth below Mean Low Water of 12’-0” (see Figure 2).

The existing Mullica River Bridge has eight 107’-6” spans, is on a tangent alignment, and is approximately 67 feet wide overall. The bridge is 877’-6” long between the centerline of abutment bearings, and 978’-6” long out-to-out of the bin type abutments, which support two short spans at each end. The superstructure is a non-composite girder-floorbeam-stringer system. The 7” deck slab is supported by simply supported steel rolled stringers over rolled steel floorbeams. The two riveted built-up deck girders supporting each roadway are spaced at 27’-4” on center and have 100 inch deep web plates. The span lengths and framing of all eight spans of the bridge are identical.

The substructure consists of bin type concrete abutments on cast-in-place piles with parallel wingwalls and an edge beam at the end of the wall enclosures. The top slab over the abutments and wingwalls is supported by reinforced concrete T-beams. Piers are composed of three reinforced concrete columns supported on timber pile foundations.

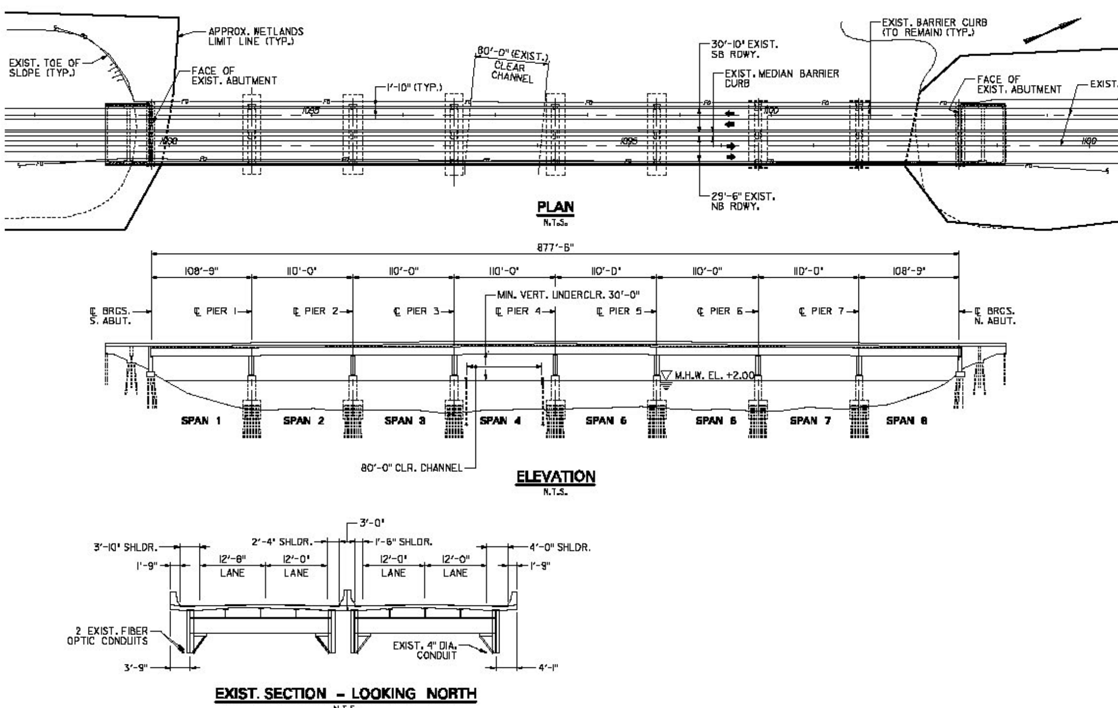

Figure 2 – Existing Garden State Parkway Bridge over the Mullica River

PROJECT DESCRIPTION AND OBJECTIVES

The project includes the design of a new parallel structure and the rehabilitation of the existing structure to provide for a future lane configuration of three 12’-0” lanes with 5’-0” minimum inside shoulders and 12’-0” minimum outside shoulders in each direction. Roadway improvements to the existing GSP mainline will be provided in conjunction with the bridge improvements. The final striping configuration for the structures and the approach roadways will be for two lanes of traffic in each direction at the end of construction of the bridge widening and rehabilitation contracts.

The existing Mullica River Bridge is in need of structural rehabilitation and upgrade in its functionality. Specifically, NJTA has determined that the reinforced concrete deck shall be replaced, including those items incidental to the deck, such as the deck joints, parapets, and median barrier. The rehabilitation will also include the repair of the steel superstructure, strengthening of structural steel members and seismic retrofit of the existing structure. Roadway improvements associated with the bridge widening are also included.

The roadway widening to the north and south of the bridge will not be constructed prior to the completion of the Mullica River Bridge Widening and Rehabilitation Project therefore, the new and existing bridges will be striped to accommodate two lanes in the northbound and southbound directions. In the development of the design, due consideration was given to providing safe and efficient maintenance and protection of traffic throughout the construction. The design width for temporary lanes will be a minimum of 11’-6” with a 1’-0” shoulder provided along all construction barriers. The traffic will be channelized in a temporary roadway configuration that provides for a design speed of 70 MPH.

For the new bridge, possible span configurations were evaluated, types of bridge superstructure and piers that can be successfully constructed within the 15 month construction schedule were identified, giving due consideration to the anticipated environmental timing restrictions that will be imposed. Scour analysis for the new bridge/existing bridge system was performed and a vessel survey was completed to determine the ship impact design criteria for the new fender system and piers.

SUBSURFACE CONDITIONS

The soils in the Mullica River area can be generally categorized as lowland alluvial deposits overlying marine sediments. In portions of the site, the alluvial deposits are covered by tidal marsh soils. The tidal marsh deposits consist of compressible organic silts and peat extending from the depth of approximate mean high water level to a depth of 10 to 30 feet. The underlying alluvial deposits are composed of well defined alternating or intermixed layers of yellow-brown to gray-white quartz sand and gravel. The Cohansey Sand Formation, which is a marine deposit consisting primarily of dense medium to fine silty sands, white to yellow in color, with localized gray to brown clay layers and lenses, underlies the alluvial deposits.

ENVIRONMENTAL CONSTRAINTS

It was determined early in the design phase that there would be restrictions to construction activities within the waterway. This was due to the seasonal presence of anadromous fish and winter flounder, as well as the unique situation of an active oyster industry and the location of oyster beds downstream from the bridge site. The timing restrictions did not permit any construction to take place within the waterway due to the potential for increased turbidity from January 1st to June 30th of each year. This allows six months for in-water construction activity, primarily construction of the piers, and was an important factor in determining the feasible span lengths for the new bridge.

The presence of the oyster beds required the minimization of construction caused siltation and thus a fully contained system was specified for the methods of installing the drilled shafts for the piers. The contract documents include a Shellfish Monitoring Plan. Baseline readings of turbidity were taken before construction started, and will be performed every two weeks during the drilled shaft installation, as well as testing and monitoring oyster samples for stress related disease. Another reading will be performed several months after construction has been completed and compared to the original baseline readings. The Resident Engineer is required to inform the environmental agencies about any spills from construction equipment that may unintentionally occur, including cuttings from the drilled shafts or concrete during pumping into the drilled shafts. The monitoring will track and record storms and tidal data to identify and account for effects on turbidity caused by natural events in the area. Prior to in-water construction activity, meetings were held with the Contractor, the Resident Engineer, the NJTA, and the monitoring Agencies to ensure all project participants are fully aware of and understand the requirements and the need to proceed carefully in this area. The goal of the permit requirements, monitoring plan and project specifications is to minimize the potential for construction-caused turbidity and to determine through the monitoring data that the variations in turbidity recorded during construction are indistinguishable and well within the limits of normal variations which occur from natural causes.

SUSTAINABILITY CONSIDERATIONS

The Mullica River Bridge site is three miles inland as the crow flies from the open waters of the Great Bay, which empties directly into the Atlantic Ocean. Therefore, the durability of the proposed construction materials was considered an important factor in the decision regarding material selection for the superstructure of the new bridge. The durability of the material may significantly influence future maintenance costs. The project’s coastal environment as seen in Figure 3 is not considered suitable for the use of unpainted weathering steel, based on industry guidelines. Therefore, its use was not considered for the bridge superstructure. Painted steel (matching the existing bridge) and concrete were considered for the multi-girder superstructure for the new bridge. Because the periodic future repainting would not be required for concrete girders, concrete was considered the better choice from a future maintenance standpoint. Also, the fact that future repainting is not required for the concrete girders makes the material a good choice for this environmentally sensitive area. High Performance Concrete with epoxy coated reinforcement was specified for the bridge deck slab in accordance with NJTA's standards.

Figure 3 – Aerial View of Existing Mullica River Bridge, Looking Southeast

SPAN LAYOUTS AND SUPERSTRUCTURE TYPES CONSIDERED

Potential span layouts evaluated for this structure were selected considering the construction schedule of the project, cost, superstructure depth, existing bridge pier spacing, future staging requirements for re-decking, hydraulic concerns, and the environmental restrictions related to placement of substructure units within the waterway. A constructability evaluation of the proposed work identified that the environmental restrictions for work in the waterway will dictate the most feasible span arrangement. Fewer piers in the water increases the likelihood that the bridge foundation elements can be installed in a single six (6) month window—July 1 through December 31. Based upon the above considerations and the project requirements, four multi-girder superstructure alternatives were evaluated. All of the alternative span layouts included a bare 9.0 inch thick reinforced concrete High Performance Concrete (HPC) composite deck with concrete parapets.

PIER LOCATION ISSUES

The horizontal alignment of the new bridge was set such that there would be 12'-0" between the completed superstructures. The 12'-0" gap between structures would facilitate access for bridge inspection equipment from the left shoulder/lane of either bridge. However, this close proximity presented a constructability concern related to construction of the pier foundations. For alternatives for which the new piers are in line with the existing piers, the proposed drilled shaft foundations of new piers would be very close to the existing pier foundations (see Figure 5). This proximity led to concerns about the stability of the existing pier foundations during construction of the new bridge for these alternatives. Consideration was given to shifting the alignment of the new bridge away from the existing bridge to alleviate these concerns. However, doing so would have significantly increased the project's impacts to coastal wetlands, which was undesirable from an environmental standpoint.

Scour was another issue to consider when locating the proposed piers. Each of the alternative span arrangements was evaluated for scour. The predicted maximum total scour depths at the new piers varied from 39 feet to 45 feet, depending on the alternative considered. Although all of the alternatives marginally increased the predicted scour at the existing bridge, the scour evaluation determined that the existing bridge would be scour critical, based on current standards, even if no new bridge were to be constructed. Since current methods of scour evaluation are generally accepted to be conservative, a scour monitoring system will be implemented on the existing bridge, in lieu of installing physical scour countermeasures, at the beginning of construction of the new bridge. This system will provide early warnings to the NJTA if excessive scour is experienced at the existing bridge, and may provide data to evaluate the need for additional countermeasures in the future.

SHORT SPAN ARRANGEMENT – CONCRETE

Alternative 1 is a span arrangement that consists of a ten span, 78" deep concrete multi-girder superstructure, with a four-span continuous unit at the south end, a five-span continuous unit at the north end and a 150 foot-long simply supported span over the main navigation channel. Although steel girders could be used for this span arrangement, concrete was determined to be more economical both from a construction cost and life-cycle cost standpoint.

INTERMEDIATE SPAN ARRANGEMENT – CONCRETE

Alternative 2 for the proposed bridge provides a span arrangement of one span @ 175' - four spans @ 220'- one span @ 175' as shown in Figure 4. The proposed abutments have been set back some distance from the existing abutments in order to reduce impacts to the existing bin abutment foundations. The southernmost proposed pier has been positioned to allow reconstruction of the gravel utility access road and bulkhead that wraps around the existing south abutment. Generally, the proposed piers have been set to be approximately midway between the existing piers. This location of the piers avoids the foundation conflicts depicted in Figure 5 and reduces the number of piers in the water to four. Alternative 2 consists of seven AASHTO Type VI Modified (haunched) Post Tensioned Concrete Composite Girders spaced at 8'-6" on centers with drop in segments.

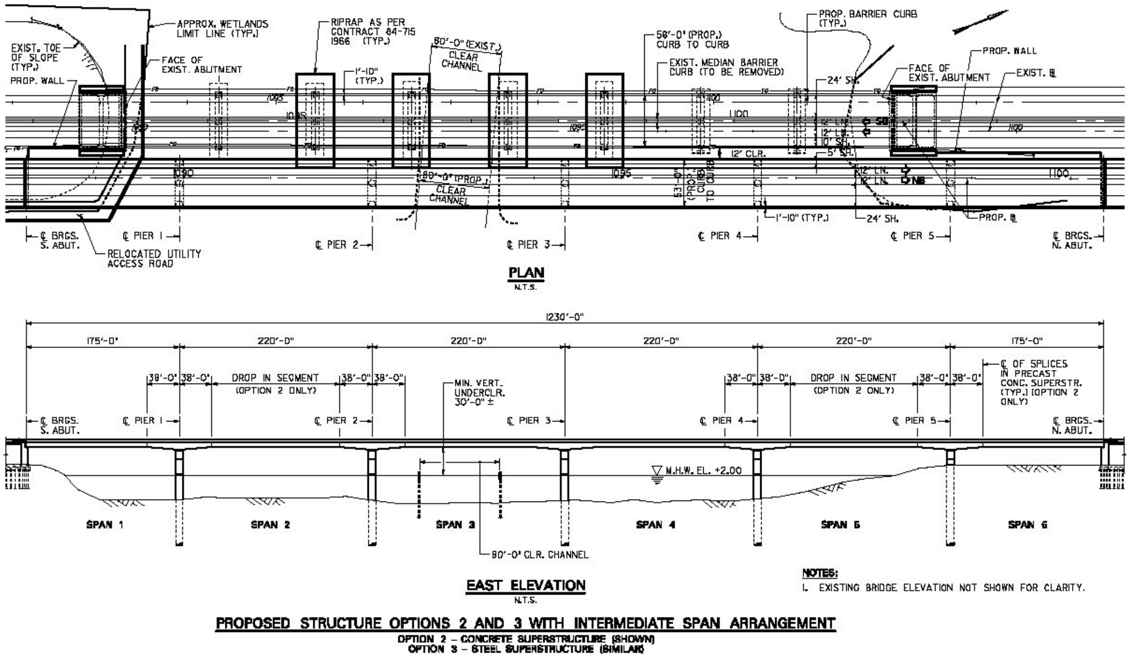

Figure 4 – Recommended Span Arrangement (Alternative 2)

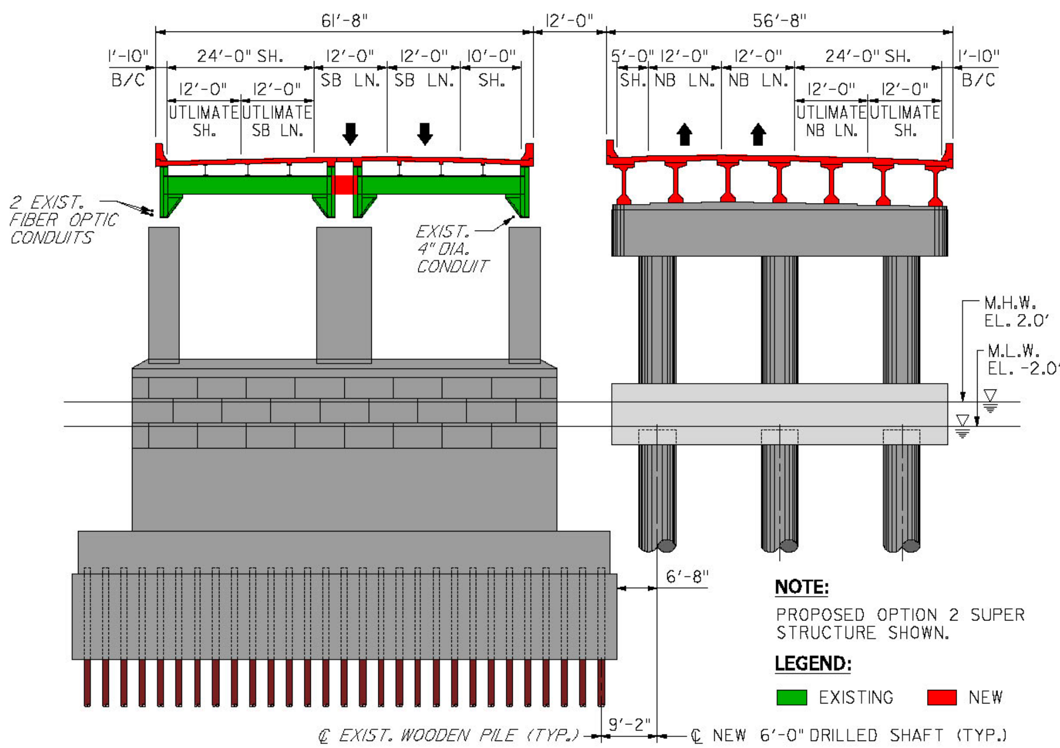

Figure 5 – Typical Bridge Section, Existing and Proposed Bridges

INTERMEDIATE SPAN ARRANGEMENT – STEEL

Alternative 3 provides a span arrangement that is identical to Alternative 2, but contains seven welded steel plate girders spaced at 8’-6” on centers. This superstructure may be fabricated using painted weathering steel and would contain a 6-span continuous unit. This option provides a superstructure depth that closely matches the existing bridge and meets or exceeds the vertical underclearance of the existing bridge.

LONG SPAN ARRANGEMENT – STEEL

Alternative 4 for the proposed bridge consists of a steel multi-girder superstructure with a symmetrical span arrangement of 175’-220’-330’-220’-175’. This layout would only be utilized for a steel alternative and reduces the number of piers in the water to four. This is accomplished by extending the main span over the navigational channel to 330 feet. The alternative features seven steel hybrid plate girders spaced at 8’-6” on center with a web depth of about 100 inches. The superstructure would be continuous from end to end, minimizing the use of expensive deck expansion joints. The center three spans could utilize a haunched fascia girder over the piers adjacent to the 330-foot main span to improve the aesthetic appearance of the bridge while still maintaining the existing minimum vertical underclearance.

COST CONSIDERATIONS

FUTURE MAINTENANCE COST

The low-maintenance concrete girders used for the superstructure in Alternatives 1 and 2 are superior to Alternatives 3 and 4, which utilize painted steel girders and would require expensive periodic repainting in the future.

CONSTRUCTION COST

Cost estimates were prepared for each new bridge option, including approach roadway cost. Alternative 1 was the short span option (150 feet maximum) similar to the existing bridge and its estimated cost was the lowest cost of all options considered at $27.2 million. However, its layout included the largest number of piers in the water (eight), which may have prevented the contractor from meeting the 21-month construction schedule due to the six month restriction on construction within the waterway. It was considered unrealistic to expect the contractor to complete foundations for all eight water piers within a single six month window available for in-water construction. Remobilization would have been required in the spring of 2010 to complete the pier foundations. Delay in the completion of the piers would impact the schedule for the superstructure for these spans. This time extension of the construction schedule was estimated to add $1 to $2 million to the cost of this alternative. Furthermore, Alternative 1 required several of the piers to be constructed in line with and in close proximity to the existing piers, which would risk causing instability to the existing bridge during construction.

Alternative 2 was an intermediate span option (220 feet maximum) with prestressed, post tensioned spliced concrete girders with piers located about midway between existing piers and was about $1.7 million more expensive than Alternative 1 or $28.9 million. This alternative, however, reduced the number of water piers to four which would provide the contractor ample time to construct the pier foundations within the six months available for waterway work. This alternative therefore was expected to be able to be completed by December 2010.

Alternative 3 was an intermediate span option with the same span lengths as Alternative 2, but utilized a steel plate girder superstructure instead of concrete. The cost of this alternative was $34.3 million which was mostly due to the high cost of steel that had been observed in the marketplace during the preliminary design for the bridge in 2007. The estimated cost for Alternative 3 was $5.1 million more than Alternative 2 and $6.8 million more than Alternative 1.

Alternative 4 was a longer span (330 feet maximum) steel plate girder option that attempted to minimize the number of piers in the waterway. However, due to the overall layout of the piers and their proximity to the existing bridge piers, four piers were still within the waterway as occurs in Alternatives 2 and 3. Therefore, there was no real significant benefit for this alternative and the cost was the highest of all alternatives considered at $42.1 million.

Based upon a careful consideration of all the above issues and the confidence that the contractor could install all foundation elements in the waterway within the permissible 6-month in-water construction period, Alternative 2 was the recommended span arrangement for the new bridge.

SUBSTRUCTURE

PIERS

Hammerhead piers and multi column pier bents were considered. In Alternative 1, the piers are generally in line with the existing bridge piers. The proposed foundation pile cap size must be minimized due to the close proximity of the existing and proposed foundations. Furthermore, rip rap previously placed around some of the existing piers will hinder the installation of the new pier foundation elements. With this smaller pile cap, only two columns or a single shaft can be accommodated. For the preferred intermediate span arrangement, a 3-column pier was selected. A three column pier has the added benefit of redundancy during an extreme load event such as a natural disaster, explosion or vessel impact. Each of the three 8’-0” diameter columns for the piers are supported by an 8’-6” diameter drilled shaft. Considering that the existing bridge will remain in service during installation and after construction, any large-size driven pile foundation was determined to be unsuitable due to the possible problems that may develop with respect to vibrations causing damage to the existing bridge during driven pile installation.

The pier reactions for the recommended concrete multi-girder superstructure are large. Each girder is designed to carry a maximum reaction of 691 kips dead load and 209 kips live load + impact at the piers. These heavy bridge loads, combined with the predicted total scour depths in excess of 40 feet, require that the drilled shafts extend down to as low as elevation -230. The 8-foot diameter drilled shaft tips were designed to be embedded into either the very stiff silt/clay layer that occurs at approximate elevation -150 or dense sand layer at approximate elevation -220 to carry the required design load. The depth of scour was considered when determining how much soil can be relied on for both vertical and lateral capacity. The performance of drilled shaft foundations, particularly side resistance, can be greatly affected by the method of construction. A sacrificial demonstration drilled shaft was required to be constructed prior to the production shaft installation to both verify the parameters used in design and the contractor's means and methods. After a successful installation and testing of the demonstration shaft, it has been determined that the drilled shafts may be constructed to the higher tip elevation of approximate elevation -180 (150 feet below the river bed).

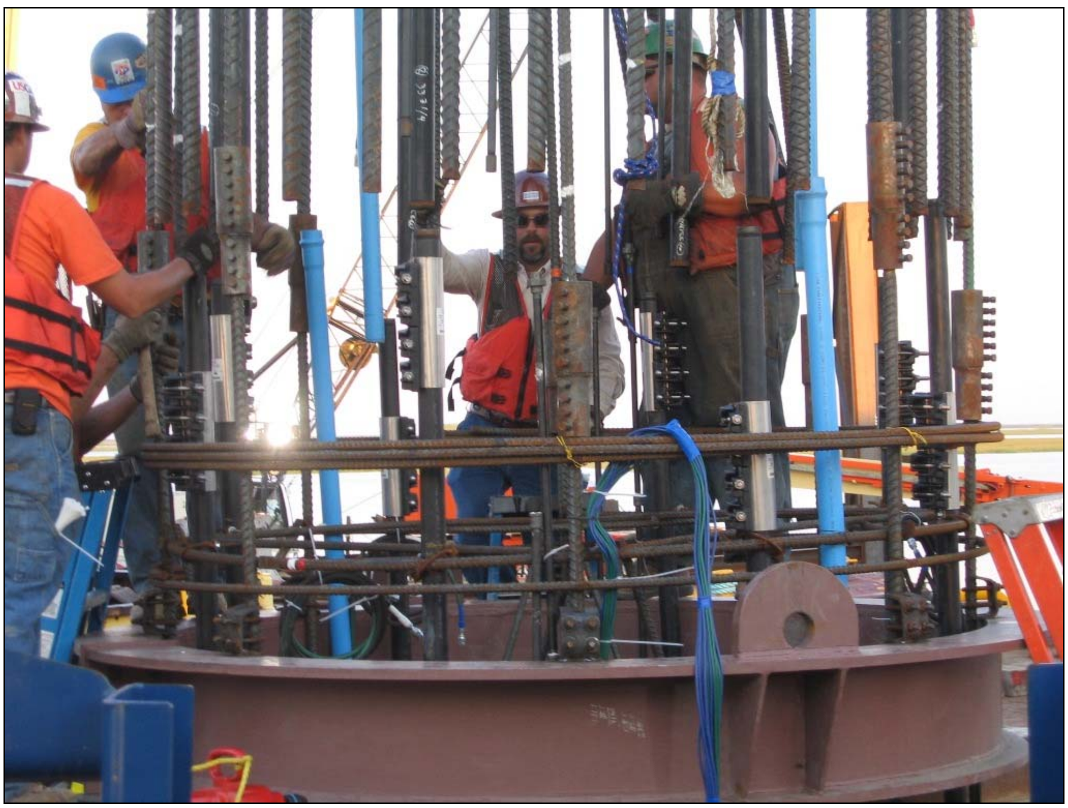

Figure 6 – Construction of the Demonstration Drilled Shaft

Self-consolidating concrete has been specified for the drilled shafts to alleviate concerns regarding possible anomalies in the concrete that could potential occur deep underground during construction of the shafts.

In general, the geotechnical resistance design of axially loaded drilled shafts is comprised of two components, the base resistance and the side resistance. Permanent steel casing will be used to construct the drilled shaft through the water and the portion of soil overburden above the scour line. Since the steel casing may not be in full contact with the soil and within the scourable layer, friction of the steel casing was ignored or discounted for computation of the axial capacity. A demonstration drilled shaft will be installed prior to the production shaft installation to both verify the parameters used in design and evaluate the contractor's means and methods. Osterberg cell load tests will be performed on the demonstration drilled shaft so that a reaction frame will not be required.

The lateral load capacity of a drilled shaft group was determined using the computer program GROUP. The estimated scour depth was considered when determining how much soil can be relied on for the lateral capacity. As a result of the very high design lateral loading and due to the fact that the pier footing will be supported above the mudline, it is recommended that the drilled shafts be designed with a permanent steel casing that has a minimum thickness of a half inch. The permanent steel casing has been considered in the lateral analysis of the drilled shaft with an appropriate allowance due to loss of section from corrosion over the design life of the structure.

ABUTMENTS

The abutments for the new bridge are subject to scour. The use of full height reinforced concrete abutments supported on deep foundations is required to withstand the predicted scour depths. The southeast and northeast wingwalls for the south and north abutments, also subject to scour, require deep foundations as well. The wall lengths, following general guidelines for wingwalls subject to scour, extend a distance of about 4 times the predicted scour depth at the abutments, or about 100 feet. This length is provided to assure that no undermining occurs to the mechanically stabilized earth (MSE) walls that abut the wingwalls. As an additional measure for scour protection, rip rap is specified in front of the MSE walls within the limits of the 100-year floodplain.

The deep foundations for the abutments and wingwalls will consist of piles driven into a very dense sand layer below the soft soils and below the estimated scour line. The most suitable deep foundations for these subsurface conditions and loadings consist of driven square prestressed concrete piles. These displacement concrete piles will provide the required side resistance for the soil conditions. The piles will be located near the existing embankment and new fill will be added over the soft organic clay and peat layers, where large settlement is expected. Downdrag loads from expected settlement are required for the pile design and treated as vertical dead load on the piles. Due to the soft soils behind the abutments and retaining walls, the design employs stone column soil improvement techniques to support the embankments and retaining walls and reduce the anticipated settlements.

SUPERSTRUCTURE – SPLICED GIRDER ISSUES

GIRDER FABRICATION ISSUES

Due to the 220 foot maximum span length, a haunched girder design was necessary. The girder depth varies from 78" deep at midspan to 108" deep at the piers. Prior to selecting the recommended span arrangement, the designers coordinated with local precasters to determine the incremental cost associated with the hanched girder configuration, which requires specially fabricated forms, and to confirm that the large pier segments were practical to deliver to the bridge site by truck. Each pier segments weigh in at 137,500 pounds. Pretensioning was provided in each of the segments to account for anticipated shipping, handling and erections stresses.

CONTINUOUS TENDON LENGTH

With the use of high strength (f'c = 8000 psi) beams and state-of-the-art post tensioning systems, it was possible to provide a 6-span, 1230 ft. long, continuous post tensioned unit, thereby eliminating a deck joint near the mid-length of the bridge. This will be one of the longest continuous post-tensioned units in North America. Corrugated plastic ducts were selected due to their superior durability and protection of the post-tensioning strands. They offer the added benefit of having a lower coefficient of friction compared to that for corrugated metal ducts. Ferrous metal ducts were not permitted. The design specified four tendons each consisting of twelve 0.6" diameter seven wire strands conforming to ASTM A416. Two tendons will be tensioned after the girders are erected to achieve continuity. Then after the concrete deck slab is placed and cured, the remaining two tendons will be tensioned.

The ducts were required to be manufactured from virgin unfilled polypropylene plastic materials using seamless fabrication methods, meeting the requirements of ASTM D4101 "Standard Specification for Polypropylene Plastic Injection and Extrusion Materials" with a cell classification range of PP0340B14541 to PP0340B67884. The duct system components and accessories were designed to meet the requirements of Chapter 4, Articles 4.1 through 4.1.8 of International Federation of Structural Concrete (FIB) Technical Report, Bulletin 7, titled "Corrugated Plastic Duct for Internal Bonded Post-Tensioning". Wear resistance of the duct is designed to meet the 0.08 inch minimum requirement (FIB 4.1.7).

Abrasion resistance of the ducts was of particular concern to the design team due to the need to push the strands over a long distance. During the design process multiple post-tensioning suppliers were contacted and provided input into the constructability of the proposed system. It was concluded that the abrasion resistance of the plastic ducts meets the requirements stated in the paragraph above and will provide a duct material that will perform adequately, if proper installation procedures and precautions are followed.

TEMPORARY GIRDER SUPPORT SCHEME

The use of temporary falsework is the typical method of construction for the erection of spliced girder bridges. However, the limited window for construction in the water due to environmental restrictions made this technique impractical for this structure. As an alternative approach, a scheme was developed to provide a temporary moment connections between pier table girders and the pier cap capable of transferring the unbalanced moment on the pier table girders caused by the placement of the drop in girders. Strongbacks were designed to support the drop-in girders from the ends of the pier table girders at the splice locations. This method required careful consideration of the temporary unbalanced loads on the permanent structure, as well as the vertical and horizontal deflections and loads of both the girders and piers as the various girder segments were added in a time-step analysis. These connections avoid the need for any temporary support towers for the superstructure, resulting in considerable cost savings.

GIRDER ERECTION PROCEDURE

To provide a practical, economical and constructible design, all aspects of the fabrication and erection process details are critical in the design of a spliced girder bridge. As is the case for other types of segmental construction, the erection method assumed during the design process needs to be provided in the Contract Documents so that the contractor may investigate the feasibility of different means and methods and evaluate the implications to the structure. The contractor has elected to follow the erection scheme depicted in the Contract Documents, which is described below and is shown in Figure 7:

1. Construct the piers with the components for the temporary moment connection installed. These consist of temporary post-tensioning bars that are intended to “clamp” the pier table girders to the pier cap (see Figure 8).
2. Erect pier table girders (7 per pier) on top of pier caps. Post-tension each pier table girder to the pier cap to develop a temporary moment connection.
3. Erect middle drop in span girders and block off gap between the pier table girders and the drop in girders (see Figure 9). Erect the two sets of end span girders.
4. Cast the continuity pours at girder splice locations.
5. Perform 1st stage longitudinal post-tensioning to obtain continuity. Release temporary moment connection and seat superstructure on the permanent bearings.
6. Pour and cure deck slab.
7. Perform 2nd phase post-tensioning.

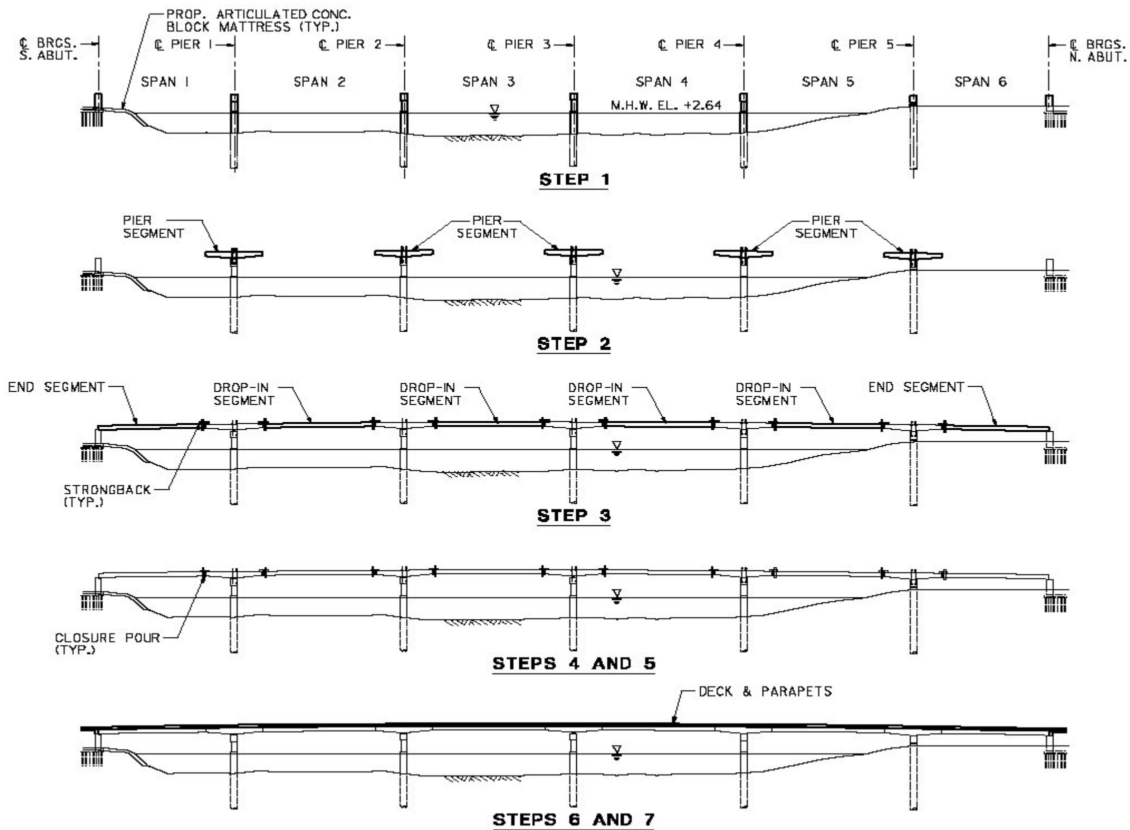

Figure 7 – Girder Erection Scheme

Tie-downs are proposed to create a temporary moment connection between the girders and the pier caps. Details of the temporary tie-downs are shown in Figure 8.

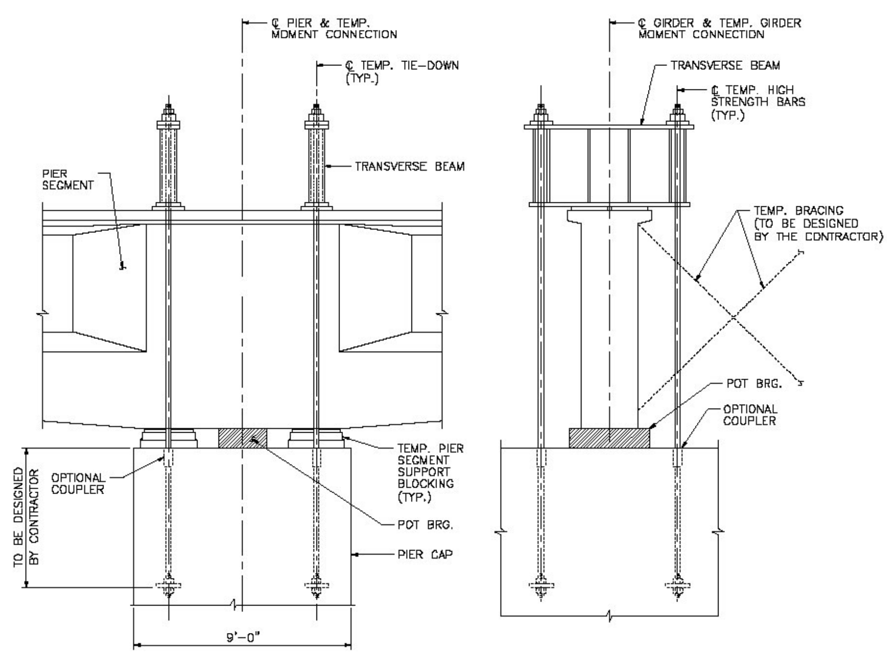

Figure 8 – Temporary Tie-down Details

Once all pier table girders are properly positioned with the temporary moment connection in place, the center drop-in girders are erected one by one. The girders are placed on steel “strong backs” that are attached to the end of the pier table girders. The strong back is illustrated in Figure 9.

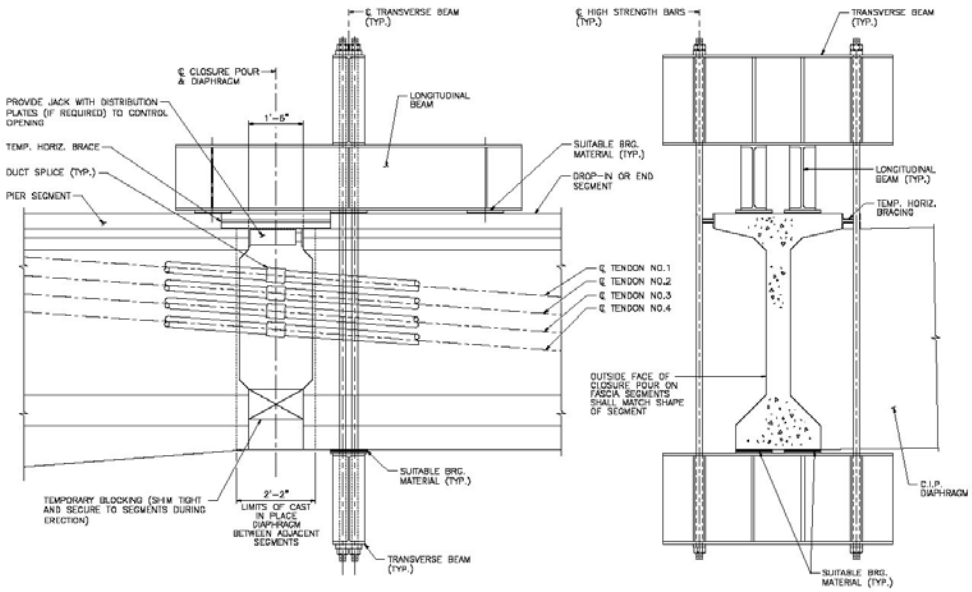

Figure 9 – Strong Back Details

GIRDER ERECTION FORCES

During the time when the center drop-in girders are in place and before the end span girders are erected the piers must resist high unbalanced moments due to the dead load of the girders. The temporary falsework and the girders must be designed to withstand an axial compressive force or a “kick force”, resulting from the tendency for the piers to deflect and rotate inwards towards the drop-in girders. These effects are illustrated in Figure 10.

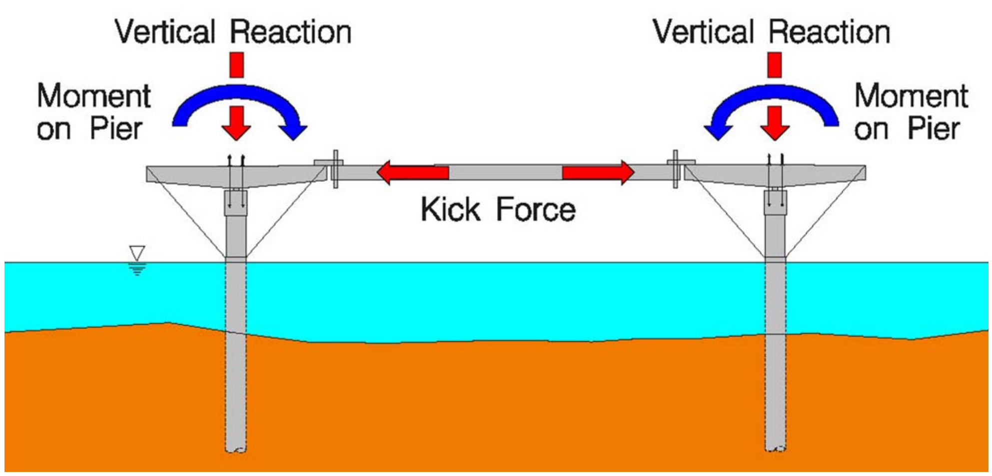

Figure 10 – Unbalanced Moment on Piers and Kick Force on Girders

Once the center span girders are supported on the strong backs, the end span girders are then erected. After all of the girders are in place, the dead load of the end span girders counteracts that of the center span and unbalanced moments at the interior piers are virtually eliminated. At this point, the tops of the interior piers have rotated back to the neutral position and the girders should be properly aligned in their approximate final position. The cast-in-place concrete splices and diaphragms are now formed and poured. Once the splice concrete is cured, the girders are post tensioned longitudinally and become effectively continuous over the six spans.

As with any multi-staged construction, it is imperative to calculate the stresses, deflections and end rotations of the structural components during each of the construction stages. The analysis must take into consideration the sequence in which the elements are placed, and the incremental stresses that each stage imposes on the structural components. For this project, the structural analysis program LARSA was used to do a time-step analysis of the erection loads on the piers and to estimate the forces for the design of the temporary components.

BEARING DESIGN

Generic bearings were specified for the concrete girders, allowing either high-load multi-rotational (HLMR) bearings, including pot or disk type bearings, or reinforced elastomeric bearings. The HLMR bearings were anticipated to be more economical for this range of reactions. Pot bearings have been selected by the contractor. The range of vertical reactions for the girders was specified as follows:

Table 1 – Bearing Design Loads

Loading (Kips)	Fixed Piers	Expansion	
		Piers	Abutments
Maximum Total Dead Load	691	665	202
Maximum Live Load + Impact	209	202	92
Minimum Live Load + Impact	-30	-22	-17
Maximum Vertical Load	900	867	294
Minimum Vertical Load	661	643	185
Maximum Lateral Load (Seismic controls)	240	162	98

CONSTRUCTION ISSUES

The aggressive construction schedule punctuated by the environmental seasonal restrictions for in-water construction defines the biggest challenge for successful construction of the project. The ability to complete all piers in the water during a single July to December period was a critical design challenge as well. Constructibility issues for the pier foundations such as access to the water, production rates, cofferdam and/or steel casings that act as cofferdams for the purpose of reducing disturbance to the water were considered during design in order to optimize the limited time available for in-water construction. The six-span alternative selected for the final design was determined to be the most cost/time effective scheme primarily due to reduction to four pier foundations required in the water.

The contractor has advanced the start of construction in December 2008 to take advantage of the last month before the restriction period, and was able to install a temporary steel sheetpile cofferdam from the southern embankment in front of the proposed abutment out into the river (See Figure 11) By completely enclosing the area where the demonstration drilled shaft and Pier 1 production shafts are located, construction in this enclosed area can proceed unhindered by the in-water work restriction. Once the demonstration shaft has been successfully installed and tested, the productions shafts for Pier 1 can continue inside this cofferdam without a separate mobilization.

Figure 11 – Cofferdam for Demonstration Drilled Shaft and Pier 1

Construction of the Pier 1 shafts is scheduled to be completed as the window opens for work in the waterway. The contractor can then proceed directly to construct the shafts for Pier 2, 3 and 4. Pier 5 is on land, near the northerly end of the bridge, so it is not subject to the in-water work restrictions.

In addition, it is anticipated that the concrete construction of pier columns/stems and pier caps will require at least two crews to meet bridge target dates. Bridge construction crews must work through the winter to complete the pier construction and erect the girders. For the selected concrete spliced girder design, girders will straddle each pier and cantilever outward by about 75 feet. To accommodate erection of the drop in girder components, the contractor must develop a method for temporarily supporting the cantilevered girders so that splice connections can be made. Since girder erection will likely fall in the first 6 months of 2010, the option to provide a support system driven into the river bottom is not available, unless it is installed prior to the restriction on in-water construction. Therefore, the contractor has elected to temporarily support the girder system directly from the piers using the scheme shown on the plans to provide a temporary moment connection between the pier segment girders and the pier caps. Concrete deck construction must start in the spring in order to complete the bridge by the required completion date of December 31, 2010.

CONCLUSIONS

MINIMIZING IN-WATER CONSTRUCTION REDUCES PROJECT DURATION AND COST

A key element to this project was the early awareness of the design team to the six month window of time available for the contractor to construct piers in the waterway. The design team was therefore able to examine several span arrangements, from 110 feet to 220 feet using steel or concrete to determine the best arrangement and material to meet the overall project goals. The new bridge with interim approaches meeting a 70 mph design speed will carry four lanes of interim traffic during the rehabilitation and deck replacement of the existing structure. The use of prestressed concrete spliced girders was a critical element of the project that permitted span lengths suitable to reduce the piers in the water and with economy of material providing competitive bids to come in below the Engineer's Estimate.

CONTINUOUS DESIGN AVOIDS THE NEED FOR INTERMEDIATE DECK JOINTS

The use of long continuous post tensioned concrete girders for the full length of the bridge avoids the need for intermediate deck joints. Deck joints are prone to leaking, which causes premature deterioration of the components underneath. Because the girders are continuous from end to end, the deck slab can also be continuous, allowing the salt-laden runoff to be contained within the drainage structures. Runoff can be directed into the roadway drainage system without leaking onto the bridge seats and bearings.

SELF-CONSOLIDATING CONCRETE (SCC) WILL HELP ENSURE HIGH-QUALITY FOUNDATIONS

The use of SCC in the drilled shafts will aid significantly in producing high quality foundations that do not require rework or repairs to these major elements of this project. The ability to place this concrete to depths of 200 feet below the mudline without the need to vibrate it, and have confidence that the finished product will meet the project requirements limits the risk to the contractor and the cost to the owner. This is truly an example of using the right material for the right application.

CONCRETE SUPERSTRUCTURE A SUSTAINABLE CHOICE FOR COASTAL ENVIRONMENT

Various aspects of concrete, its components and manufacturing process make it a sustainable material. The fact that it does not require periodic future repainting makes concrete an economical and environmentally sensitive choice for the main girders. Concrete uses recycled materials, abundant materials, and environmentally conscious manufacturing processes, making it a sustainable material choice.