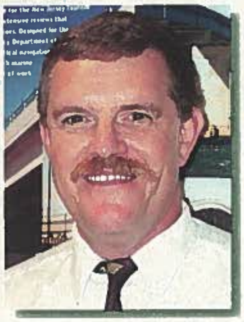
Ocean City-Longport Replacement Bridge Requires Precast Concrete Durability for Harsh Marine Conditions



Joseph Mumber, P.E.
Assistant Vice President
Chief Structural Engineer
Parsons Brinckerhoff
Princeton, New Jersey

Dale M. Foster, P.E.
Cape May County Engineer
Cape May Court House, New Jersey

E. David Lambert III, P.E.
Project Manager
New Jersey Department of Transportation
Trenton, New Jersey

Chad A. Saunders
Chief Engineer
Bayshore Concrete Products Corporation
Cape Charles, Virginia

On the North Atlantic Seaboard, a 75-year-old weather-beaten steel bascule bridge spanned the Great Egg Harbor from the New Jersey mainland to the barrier island resort community of Ocean City. Here, coastal exposures entail corrosive salt-water spray, strong currents, extreme tides, extensive scour, potential vessel impact, ocean storms, and potential hurricanes. As the last remaining evacuation route to the mainland during severe weather, the owners demanded a replacement bridge that embodied durability with minimal maintenance in this hostile marine environment. The resulting solution was a 26-span precast/prestressed concrete bridge using continuous spliced girders, cylinder piles, pile caps, and half-deck panels. Reducing the number of piers and lengthening spans over deep water were major design accomplishments. Two significant construction techniques developed were temporary tie-downs for spliced girder erection and precast concrete cofferdam tubs to form pier footings at water level.

In waters of the North Atlantic on the Eastern Seaboard, bridges must withstand severe marine conditions – including corrosive elements, underwater scour, ship collision and hurricane-force winds. At the mouth of Great Egg Harbor, a 75-year-old steel bridge stood in water depths up to 60 ft (18 m) spanning tidal seas from the New Jersey mainland to a populated barrier island. Built in 1927, the

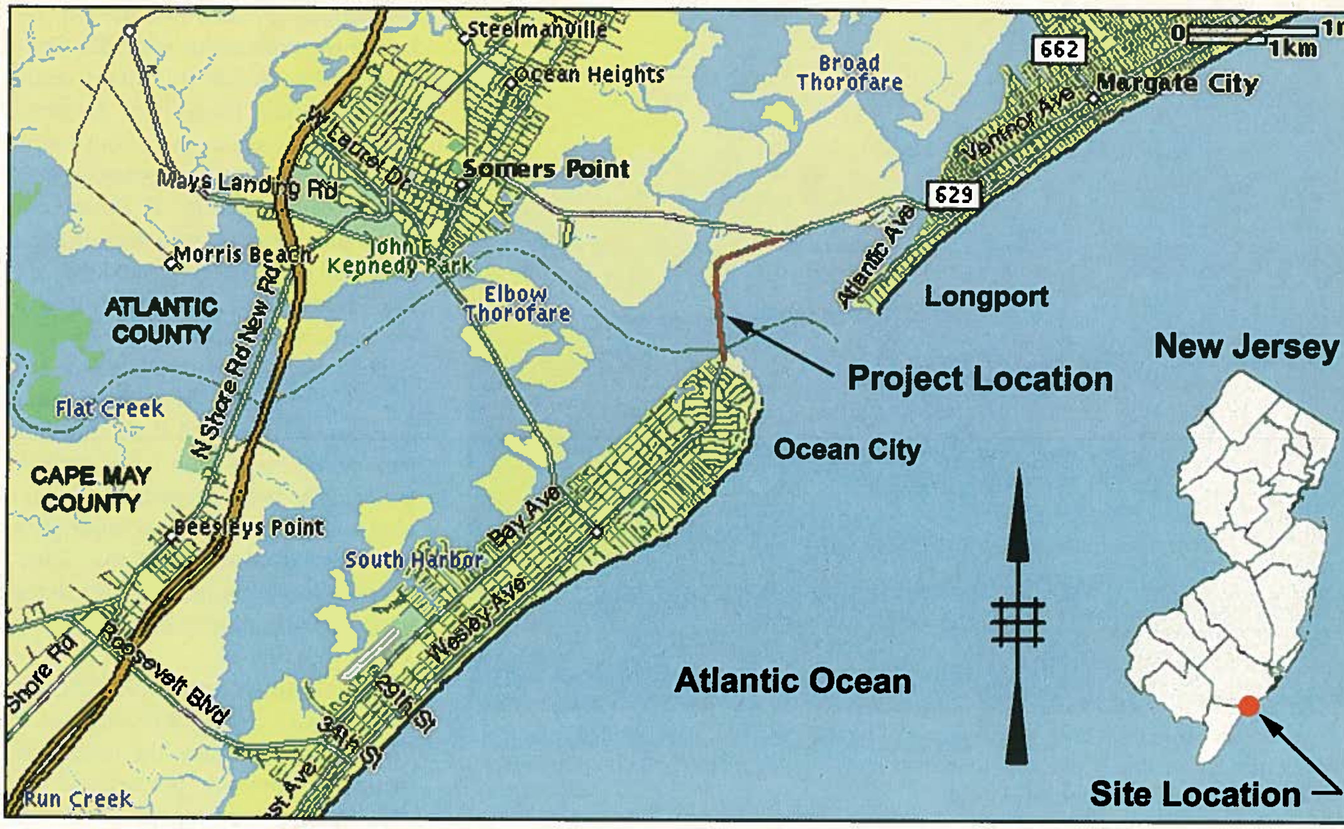


Fig. 1. Location map: Ocean City is located on a barrier island on New Jersey's North Atlantic Seaboard.

bridge served the community as a storm evacuation route for the coastal resort cities of Ocean City and Longport (see Fig. 1).

Located off New Jersey's Atlantic Shore, Ocean City and Longport are very popular seaside vacation destinations. With 15,000 year-round island residents, the population of Ocean City swells ten times – to 150,000 – during the peak summer tourism season from May through September. Ocean City is connected to the mainland by three bridges, the northernmost of which is the Ocean City-Longport Bridge.

The old steel bridge was the last of the three vehicular routes to the mainland to remain passable in stormy conditions, as the other two bridges flooded and became impassable during very minor storms. Over time, severe marine exposures had taken their toll on the low-level bridge (see Fig. 2), and, with its advanced state of deterioration, would endure only a few more years of service.

As the last remaining passage out of Ocean City to the mainland during high tide conditions and impending storms, a reliable vehicular bridge is critically important to the local population for emergency evacuation to the mainland. Dangerous storm condi-

Fig. 2. The 75-year-old low-level steel bascule bridge was in imminent need of replacement.

Fig. 3. A close-up of the historical restoration of a section of the old bridge as a fishing pier for local residents and tourists reveals an elegant precast concrete design. Photo courtesy: Gregg Kohl, AC Photo.

tions and extreme high tides occur frequently in this coastal location. Since the island's peak population during the warm summer months coincides with the North Atlantic hurricane season, a reliable evacuation route is also a vital concern to New Jersey's tourism industry.

Structural evaluations revealed that the bridge was in very poor condition, with less than five years of remaining service life. A replacement for the dilapidated bridge became imperative. With completion of the Ocean City-Longport Replacement Bridge, an essential coastal evacuation route has been preserved. Demonstration funds from the Federal Highway Administration (FHWA) were utilized for the design and construction of this important bridge.

In this paper, the authors present the innovative plans and novel erection sequences developed to build an economical and durable replacement bridge, as well as the precast concrete designs that met the owner's need for minimal maintenance. The design for the erection of a spliced precast concrete girder bridge without falsework towers solved some difficult problems and required careful consideration of the erection loads on the temporary and permanent structure.

OLD AND NEW BRIDGES

The original Ocean City-Longport Bridge was a steel trestle-bent design with a low-level, double-leaf bascule (hinged) span. The steel bridge was 3450 ft (1052 m) long, with a 122 ft (37.2 m) movable span, and incorporated a 35.0 ft (10.7 m) long trestle bent design for most of its length. Built in 1927 as an evacuation route to the New Jersey mainland, the bridge was also part of a regional scenic drive project.

The structure was one of six movable bridges along the Scenic Ocean Drive, a coastal route that winds through Cape May County along the Atlantic Seaboard. Employing a novel spring-latch mechanism for locking the bascule span, the bridge was deemed eligible for the National Register of Historic Places.

Over the years, several rehabilitation projects were undertaken to replace the deck slab, paint the steel superstructure, restore pier bents, and provide rip-rap scour protection for the inlet bottom. Recent structural evaluations revealed that the old steel bridge required major rehabilitation or replacement. A 490 ft (150 m) section of the original bridge was preserved and rehabilitated as a fishing pier as part of the regulatory agency's permit requirements (see Fig. 3). Precast concrete pier caps on existing concrete piles and double tees created this popular community resource requiring minimal maintenance.

The new parallel structure is a 3450 ft (1052 m) long, high-level fixed-span bridge with a 65 ft (20 m) clearance above mean high water for navigation. The precast concrete bridge is composed of longitudinal multi-girder prestressed concrete spans, with a post-tensioned spliced girder design over approximately half the bridge length (see Fig. 4).

Completed in September 2002 at a total project cost of $52 million, the 26-span bridge spans the ocean inlet utilizing a precast, prestressed concrete multi-girder superstructure with several different span lengths and girder sizes (see Tables 1 and 2). The superstructure supports a reinforced CIP concrete deck slab constructed with half-depth precast, prestressed concrete deck panels. Selection of span length and girder size depended on the height of the structure over the inlet and the water depth at the piers.

DESIGN AND ANALYSIS

In the late 1980s, the bridge owner, the Cape May County Bridge Commission, hired Parsons Brinckerhoff (PB) to conduct an assessment of conceptual design alternatives for replacement of the existing steel bridge. The study investigated the relative cost of construction of alternative bridge types, financing options, toll rate schedules, effects on navigational traffic during and after construction, maintenance of vehicular traffic during construction, aesthetics, serviceability, geologic and soils conditions, and foundation requirements.

PB engineers originally considered four types of high-level fixed-span bridges in the conceptual studies: a cable-stayed bridge, segmental concrete spans, multi-girder steel superstructures, and multi-girder pre-

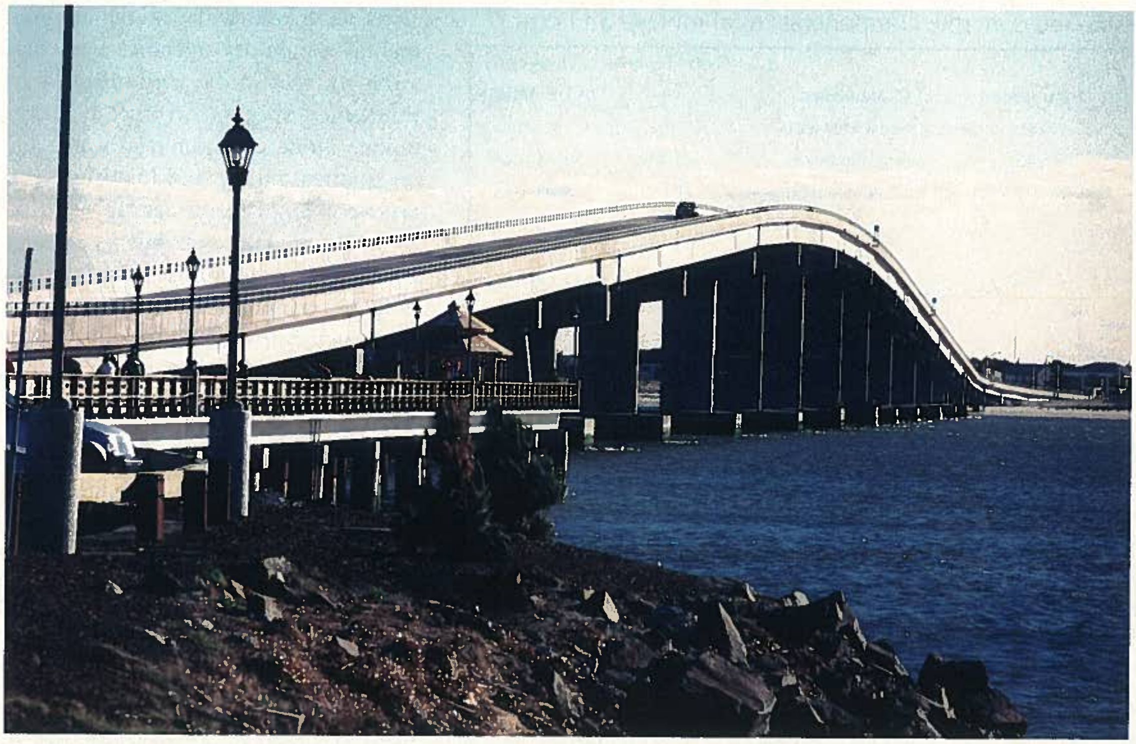


Fig. 4. A west view of the completed replacement bridge reveals the aesthetically pleasing design and graceful line of the precast concrete spliced girder bridge. Note the harmonious blending of the structure with its environment. Photo courtesy: Gregg Kohl, AC Photo.

stressed concrete bridges with drop-in spans to achieve longer spans. As the existing bridge was eligible for inclusion in the National Register of Historic Places, engineers also investigated a low-level movable double-leaf bascule bridge similar to the existing steel bridge, rehabilitation, and a no-build alternative.

With a structure continually exposed to rough weather and corrosion, the owner requested that the new structure be made as durable and maintenance-free as possible. For minimal maintenance in these exposures, concrete structures were given preference over those of steel, leading to the selection of the precast, prestressed concrete spliced girder bridge design.

After reviewing structural alternatives, the owner expressed a preference for a concrete superstructure over that of steel – mainly due to potential costs of future bridge maintenance. Precast systems were preferred for its ease of construction, reduced construction time, and lower life-cycle cost. A precast, prestressed concrete multi-girder superstructure with a post-tensioned spliced girder system was finally selected as the most economical and durable choice.

The main emphasis of design developments addressed the severe environment at the site. Rough seas in this North Atlantic Harbor precluded common bridge building practices more typically executed in the water. The approaches were redesigned to meet the existing arterials, while elevating the north approach roadway above the 100-year flood elevation ensured that the evacuation route was maintained. To minimize adverse effects on existing wetlands, the reconstruction of the north approach roadway was held entirely within the existing 80 ft (24 m) wide right-of-way.

Inlet waters stretch virtually from abutment to abutment, except for about 330 ft (100 m) of beach on the south (Ocean City) side of the bridge. Waters beneath the bridge become very deep close to the beach on the south side, with an abrupt drop-off to 50 ft (15 m) within 165 ft (50 m) of the shoreline. Beyond the drop-off, depths remain at approximately 33 to

Table 1. Project timeline, including precast manufacturing and erection.

Begin final design	February 1998
Complete final design	March 1999
Advertise	June 1999
Notice to proceed	September 24, 1999
Order, fabricate, and deliver cylinder piles	January 2000 to March 2001
Install cylinder piles	April 2000 to September 2001
Order, fabricate, and deliver cofferdams	August 2000 to June 2001
Erect cofferdams	March 2001 to September 2001
Fabricate, deliver superstructure members	June 2000 to February 2002
Erect superstructure members	July 2000 to February 2002
Fabricate and deliver precast deck panels	April 2000 to September 2001
Erect precast deck panels	October 2000 to March 2002
Bridge open to traffic	July 19, 2002
Fabricate, deliver precast fishing pier bent caps	September 2001 to April 2002
Erect precast fishing pier bent caps	February 2002 to June 2002
Fabricate, deliver precast double tees	March 2002 to July 2002
Erect double tees	June 2002 to August 2002
Open fishing pier	September 2002

Table 2. Precast/prestressed concrete components: total footage and cost.

Description	Number	Dimensions	Total length, ft	Total cost in US dollars
Substructure precast elements				
Hollow cylinder piles	145	54 in. diameter	15,555	$6,485,200
Square piles	100	14 in. square	4978	$331,900
Cofferdam open-top boxes 8 in. thick walls 10 in. bottom slab	8	21.75 x 44.25 x 9.0 ft deep		$1,650,000
Cofferdam open-top boxes 8 in. thick walls 10 in. bottom slab	3	21.75 x 33.0 x 9.0 ft deep		
Precast girder elements				
AASHTO Type IV girders			5425	$987,000
2 spans, 6 girders each	12	87.5 ft long		
10 spans, 5 girders each	50	87.5 ft long		
AASHTO Type VI girders				$439,800
4 spans, 5 girders each	20	121.0 ft long	2420	
Modified AASHTO Type VI (90 in. deep) girders				$2,438,475
5 Modified AASHTO Type VI girders consisting of the following:			8860	
Pier table girders	30	75.83 ft long		
Middle span drop-in girders	15	146.33 ft long		
End span drop-in girders	30	146.33 ft long		
Simple span of 5 girders	1	144.0 ft long	720	
Other precast components				
Half-depth deck panels		$3^1/_2$ in. thick, 83,803 sq ft		$1,363,425
Double tees for fishing pier superstructure replacement	14 spans of 2 DTs	35.0 ft long	980	$698,000
Fishing pier bent caps	14 piers	22.0 ft long	308	$205,000
Bridge		3450 ft long, 40.5 ft wide		
Deck		140,000 sq ft		
Span description of 26 total spans				
AASHTO Type IV girders	12 spans	87.5 ft	1050	
Modified AASHTO Type VI girders	1 span	144.0 ft	144	
Three-span continuous (Modified AASHTO Type VI girders)	3	590.66 ft each	1772	
Four-span continuous (AASHTO Type VI girders)	1	121.0 ft	484	
Post-tensioning				$1,000,000
Total precast concrete system cost				**$15,598,800**
Total bridge construction cost				**$46,000,000**

Note: 1 in. = 25.4 mm; 1 ft = 0.3048 m; 1 sq ft = 0.0929 m^2.

60 ft (10 to 18 m) for nearly 1310 ft (400 m).

Proceeding north from this area, the seabed rises to present a constant depth of about 10 ft (3 m), continuing all the way to the north bridge abutment. A 150 ft (46 m) clear navigation channel was required by the U.S. Coast Guard in addition to the 65 ft (20 m) clearance above mean high water to accommodate vessel passage.

Daily currents at the site are extremely swift, with a large drainage area consisting of numerous back bays shoreward of the bridge. This contributes to the large volume of water that passes through the Great Egg Inlet with the daily tides. High winds and constant sea breezes add to the difficulties of site construction.

Geological investigations revealed that the seabed consists of a loose sand down to an elevation of about –60 ft (–18 m), below which is located a homogeneous, very dense sand identified as the Cohansey Layer. Discussions were held with various environmental regulatory agencies on issues such as wetlands, endangered or threatened species, wildlife habitats, unique biotic communities, water and air quality, noise, and historical and archaeological resources. In addition to environmental assessments, a comprehensive community involvement program was initiated to obtain public input on the proposed project.

Superstructure

Spliced post-tensioned concrete I-girder construction is a concept that has become increasingly popular with bridge designers to achieve medium length spans with a low-maintenance material like concrete. In this type of structure, longer spans than those possible with simple spans can be achieved by post-tensioning together individual girders longitudinally to achieve continuity.

When designing a spliced girder bridge, it is crucial to consider all aspects of the fabrication and erection processes. Attention must be paid to all details in order to ensure a practical, economical, and constructible design. As is the case for other types of segmental construction, the erection method assumed during the design process is provided in the contract documents so that the contractor knows what loads the permanent structure is designed for and to give the contractor the opportunity to consider the implications to the structure if another erection method is chosen.

Extensive falsework at a location such as this typically requires the design of temporary tower bents to withstand very large construction loads, scour and wind loads, and possible vessel impact forces. At this location, deep water and other site constraints led to the creation of an innovative erection sequence for the spliced girder system that eliminated the need for tall falsework towers to support the drop-in spans.

Where falsework towers were not used to support the various girder segments before post-tensioning to make them continuous, deflections and rotations of the piers and girders needed to be carefully evaluated as a system to

Fig. 5. Underneath view of the precast concrete piers for the shallow water foundations. A precast concrete system allowed for versatility and economy in the design of deep and shallow foundations. Photo courtesy: Gregg Kohl, AC Photo.

ensure that adequate gaps were maintained at the girder splices when the piers deflected and rotated due to placement of the drop-in spans. The analysis considered the sequence in which the girders were to be placed one at a time, the incremental stresses that each successive girder imposed on the piers, and the structural design of the girders themselves.

Using the finite structural analysis program LARSA, a time-step analysis was performed for the assumed erection loads on the piers. Because of limited duration of exposure, wind forces during construction are not normally a significant loading for bridges. However, before a spliced girder bridge is made continuous, the superstructure is in a particularly vulnerable state. Furthermore, as the deck slab is not yet constructed, girders leeward of the fascia girder are partially exposed to wind. Bridge construction at the doorstep of the Atlantic Ocean meant the possibility of a significant storm occurring during construction.

The AASHTO specifications do not fully address this loading situation. Therefore, procedures adapted from ASCE-7-98 and the Ontario Bridge Design Code were used to develop the appropriate wind loading. The contractor designed a system with wire ropes connected to the top flange of the girders to provide cross bracing in the plane of the girders that would resist this load and prevent a catastrophic failure in the event of a significant storm.

Use of temporary tie-downs made the piers integral with the pier-table girders during erection. Unbalanced moments due to the erection loads could then be transferred directly into the piers.

Approach spans range in length from 87 to 122 ft (27 to 37 m). The first six approach spans on both ends of the bridge span over the beach and shallow water and consist of 87 ft (27 m) long AASHTO Type IV prestressed concrete girders made continuous for live load. The next four spans on the north side are located in relatively shallow 10 ft (3.0 m) deep water, but as the bridge climbs to provide the required vertical clearance, the design was changed to utilize longer spans to offset the increased cost of taller piers.

In the deeper waters of the harbor, the number of foundations was minimized by utilizing a post-tensioned concrete spliced girder design to achieve longer spans. The deep-water spans near the center, including the new navigation span, consist of three separate three-span continuous units composed of 90 in. (2290 mm) deep spliced post-tensioned concrete Modified AASHTO Type VI I-girders. This continuous span was designed with a drop-in construction method to achieve a maximum span length of 222.2 ft (87.7 m). The end spans for the three-span continuous units were 184.25 ft (56.2 m).

In a three-span continuous structure of this type, a girder segment (pier table girder) is placed over both piers, extending out from the pier as much as 38 ft (12 m) on both sides. A drop-in span girder is placed between the pier table girder ends to form the middle span and between the end of the three-span unit and the pier table girder to form the end span.

Metal stay-in-place forms were subject to corrosion and removable forms would be very costly for such a tall structure. Consequently, half-depth precast concrete deck panels were designed for the deck slab construction, with precast deck panels serving as both formwork for the top half of the deck and as the structural bottom half of the deck.

A reinforced concrete deck slab

Fig. 6. View of a completed pile cap and hammerhead pier shaft with the temporary forms still in place.

Fig. 7. Precast concrete cofferdam during construction of footing.

supported on the longitudinal precast girders was designed and constructed using half-depth precast concrete deck panels as the bottom half of the deck. This design eliminated the need for deck formwork, with a CIP reinforced topping made composite with the panels.

Foundation

Several design alternatives were considered for the foundations of the deep-water piers: drilled shafts, large deep-water caisson foundations utilizing full-depth cofferdams, and precast concrete cylinder pile foundations with a pile cap at the water line.

A cost comparison was performed for drilled shafts versus cylinder piles, and a clear advantage was determined for the cylinder pile deep foundations. Factored into the consideration were the scour depths that would result from the various foundation types. Local scour at this site was estimated to be up to 40 ft (12 m) for the cylinder piles and 30 ft (9.1 m) for the drilled shaft alternatives, due to the extremely swift currents at the site.

The scour depth for the cylinder piles was deeper because of the additional turbulence caused by the use of a larger number of, and more closely spaced, piles than would be used for the drilled shaft alternative. Therefore, the piles or drilled shafts would need to be extremely long, extending through a water depth of 60 ft (18 m), 40 ft (12 m) of expected scour, and then developed below that point to support the structure. In the deepest water, the piles needed to be seated at an elevation as low as –135 ft (–41.1 m).

Two different substructure designs were used, one for shallow water and another for deep water. The main spans in the deep water are supported by hammerhead piers constructed on concrete pile caps near the water level. Piers are founded on 54 in. (1372 mm) diameter precast, prestressed concrete cylinder piles. The substructure for the approach spans in the shallow water and on the beach consists of a series of pier bents formed by 54 in. (1372 mm) diameter precast concrete cylinder piles with a CIP pier cap (see Fig. 5).

Water depths and longer spans, as well as concerns for vessel impact to foundations near the navigation channel, precluded consideration of bent-type foundations using cylinder piles. Viable substructure designs called for either drilled shafts or prestressed concrete cylinder piles utilizing a pile cap at water level, and hammerhead piers extending up to the superstructure. Precast cylinder pile deep foundations offered a clear cost advantage when compared with the other alternatives (see Fig. 6).

For the substructure, large caisson foundations with full-depth cofferdams were quickly ruled out because of the cost of cofferdam construction at such frequent intervals. Consequently, drilled shaft foundations were compared for constructibility and cost with prestressed concrete pile foundations.

Construction of the pier footings using conventional construction techniques would have required that temporary steel sheeting be driven into the seabed to form a temporary cofferdam. This cofferdam could then be sealed using tremie concrete and dewatered so that the footing concrete could be cast in-the-dry.

Deep water and swift tides at this ocean inlet would have made this conventional cofferdam technique physically impractical and very expensive. An alternative to traditional methods was a novel construction scheme developed using precast concrete cofferdams to create footings near the waterline without the use of temporary sheeting.

In the design phase, the precast concrete cofferdams were analyzed for the erection loads during lifting and placing, and for the loads that they would experience during the construction sequence proposed, including the buoyancy effect after the openings around the piles were sealed, the weight of the poured concrete footing inside the cofferdams, the hydrostatic pressure on the walls of the cofferdams, and allowance for dynamic effects of wave action. It was also required that the contractor provide additional shielding around the cofferdam to prevent wave action or very high tides from fouling the CIP concrete inside the cofferdam during curing.

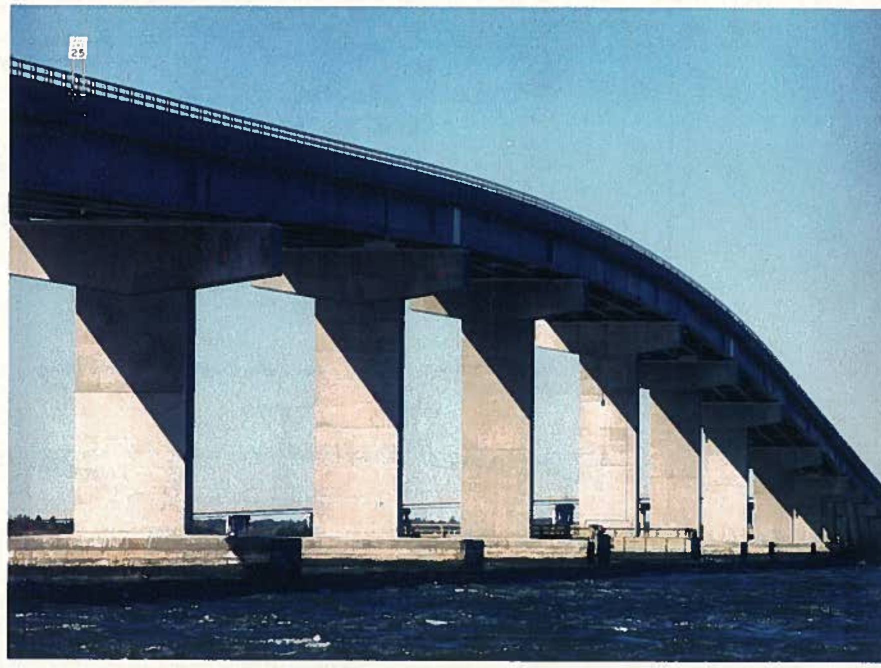


Fig. 8. Hammerhead piers founded on precast concrete cofferdam pile caps in rough waters. Photo courtesy: Gregg Kohl, AC Photo.

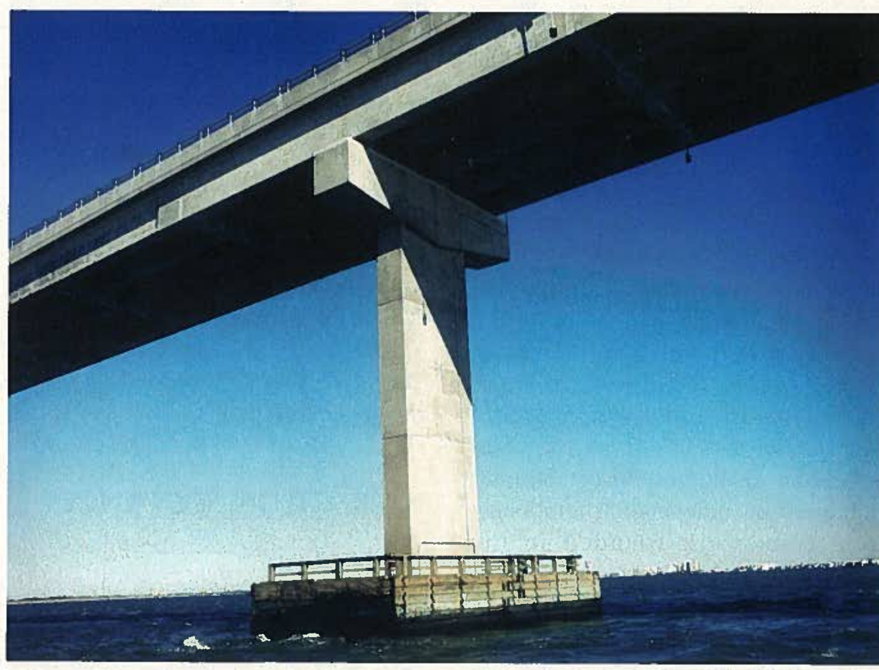


Fig. 9. Close-up of hammerhead pier. Photo courtesy: Gregg Kohl, AC photo.

The AASHTO Guide Specifications for Vessel Collision were utilized for the design of the water piers, but separate assumptions had to be made and approved by the client, the New Jersey Department of Transportation (NJDOT) and the FHWA, for simultaneous occurrences of extreme events. For instance, the Guide Specifications do not contain design provisions for simultaneous extreme events such as vessel collision in combination with scour events.

The design team developed guidelines for the simultaneous events based upon available research and procured approval from all parties for the following three load cases:

1. A fully loaded barge striking a pier at the same time as a full short-term scour event – on the order of 40 ft (12 m) for this project – need not be considered because of the low probability of simultaneous occurrence.

2. A fully loaded barge striking a pier should be considered with long-term scour [5 ft (1.5 m) for this project] occurring simultaneously.

3. An empty barge drifting with the current striking the pier should be considered simultaneously with short-term scour.

A fender system was used on the main channel piers to dissipate energy for the design of the piers, allowing the main piers to be designed for less than the full vessel impact force while still satisfying the overall vessel impact criteria for the bridge.

The bridge owner required that the foundation design for in-the-dry pile cap construction should follow the requirements of NJDOT. The DOT specifications for a foundation at the water level would have required the use of either granite stone protection for the concrete pile cap, or stainless steel facing to combat the harsh marine environment and potential concrete erosion from the tides and currents at the site.

These alternatives would have significantly increased the ultimate pile cap construction cost. In lieu of standard NJDOT protection for the pile cap, the contract drawings were developed utilizing a 5000 psi (35 MPa) strength precast concrete cofferdam for footing construction. The precast concrete cofferdam offered the following benefits:

• Permitted construction of the pile cap in-the-dry.

• Eliminated the need for costly pile cap formwork.

• Provided a method of protection for the structural pile cap concrete, as

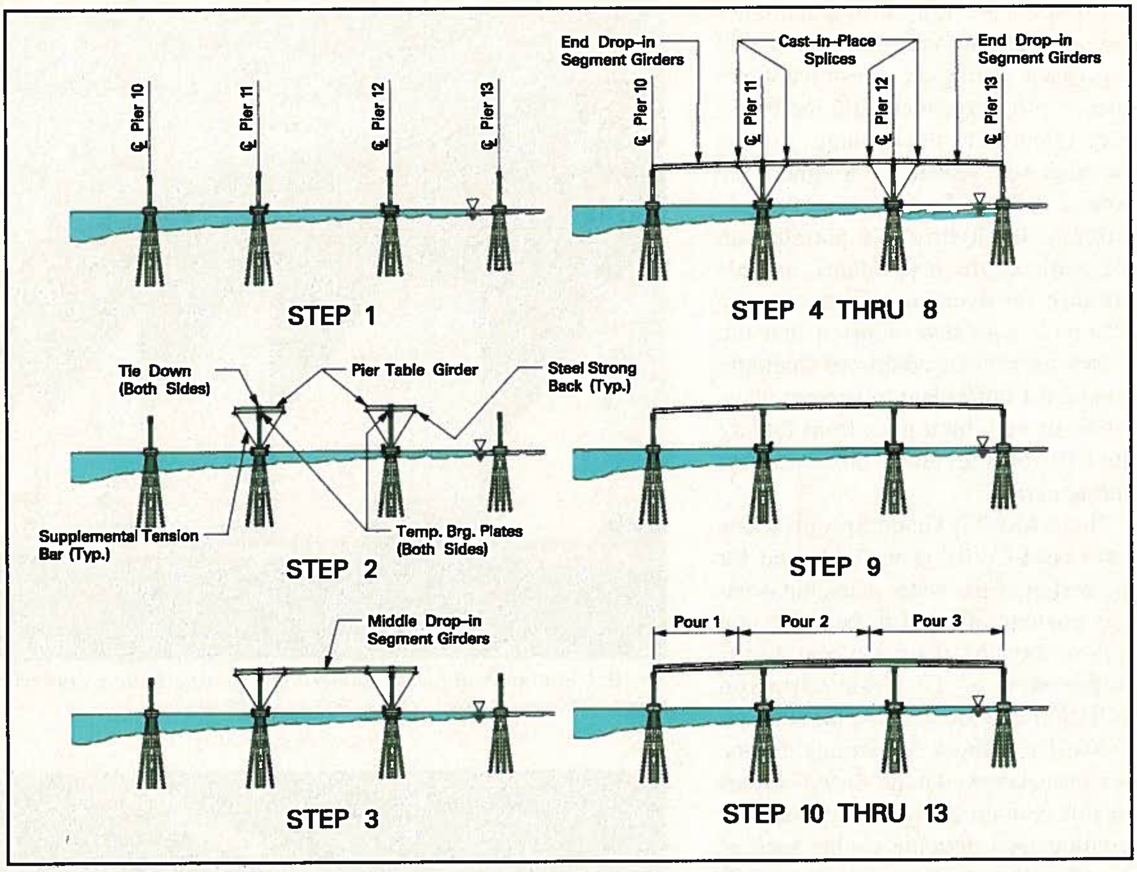


Fig. 10. Girder erection sequence.

the precast cofferdam was left in place.

• Allowed for quick erection once the piles were completely installed at the pier.

After all of the piles in one group were installed, the precast concrete cofferdams (or tubs) were lowered over the battered pile groups through holes cast in the bottom of slab and supported on independent temporary steel framework founded in the seabed.

The pile cap at the water level was designed so that even in extreme high or low tides, the bottom of the cap would always be below water to prevent the deterioration of the concrete from continual wetting and drying; at extreme high tide, the cap top would always be visible so as not to represent a navigation hazard to vessels and the numerous recreational boaters.

Precast cylinder pile foundations were designed in groups of either six or eight piles per foundation, depending on the depth of water at the pier, and the relationship to the navigation channel for vessel collision considerations.

Piles were designed to be spaced at a minimum of 11.25 ft (3.4 m) on center; therefore, the overall out-to-out dimensions of the precast concrete cofferdams were set at 21.75 x 33.0 ft (6.6 x 10.0 m) for the smaller cofferdams and 21.75 x 44.25 ft (6.6 x 13.5 m) for the larger cofferdams.

ERECTION SEQUENCES

Foundation

Prestressed concrete cylinder piles were installed in groups of six or eight piles per pier (two rows of either three or four piles), and were designed for scour depths of up to 40 ft (12.1 m).

Precast cofferdam — Since deep harbor water ruled out costly full-depth cofferdams, the use of a temporary precast concrete cofferdam permitted the use of 54 in. (1.3 m) precast concrete cylinder piles driven to capacity to be an economical foundation system. The contractor was thus able to use a one-piece cofferdam designed to be lowered over the battered piles with a 6 in. (150 mm) minimum tolerance.

Precast cofferdams were fabricated off-site with holes cast in the bottoms to accommodate the different pile batters at a given pier location. Dimensions of the cofferdams are given in Table 2. The design plans were developed using divers to assemble steel collars around the tops of the piles, forming a steel framework for cofferdams support on temporary steel supports near the surface.

The contractor opted to revise this detail by installing large independent temporary support piles and frames for each of the piers. The framework was disassembled after placement of the pile cap and reused on other piers. The collars and interconnecting framework were disassembled after placement of the pile cap and reused at other piers (see Figs. 7, 8, and 9). When the concrete achieved sufficient strength, the ballast holes were plugged and the water pumped out for work in-the-dry.

The construction sequence the contractor used for the construction of the pier caps is as follows:

1. Drive steel pipe piles at the four corners of the new pile cap.

2. Assemble temporary steel frame cofferdam support.

3. Lower precast concrete cofferdam over concrete cylinder piles and secure to steel frame.

4. Seal ballast holes and openings between piling and cofferdam bottom.

5. Place a 1 ft (0.3 m) thick tremie concrete seal inside cofferdam.

6. Dewater cofferdam.

7. Place the first pour of the footing in-the-dry and cure to specified strength.

8. Remove temporary steel frame supports for reuse at next location.

9. Place second pour of the CIP footing.

Footings — The footing was poured in two lifts because of the volume of concrete required in the cofferdam and the heat of hydration that could develop with such a large pour. Once the first lift had achieved sufficient strength, the footing was firmly bonded to the piles to resist dead load and buoyancy forces, and the temporary framework then removed. Finally, the second lift was reinforced and poured. By moving the temporary framework to the next pier location, two sets of temporary framework steel were used to enable the construction of the pile cap footings to leapfrog and continue in an efficient manner.

Superstructure

Temporary falsework is typical in construction of spliced girder bridges. Site conditions in the harbor, particularly the strong currents and potential vessel impact, made use of traditional falsework prohibitively expensive. As an alternative, temporary moment connections were designed between pier table girders and pier caps that transferred any unbalanced moment caused by erection of drop-in girders on the pier table girders.

Girder erection sequence — The suggested erection sequence for the precast girders is shown in Fig. 10:

1. Construct hammerhead piers with temporary post-tensioning strands embedded within the pier cap for the tie-down of the pier table girders.

2. Erect pier table girders (five per pier) on top of the two interior hammerhead pier caps for each three-span

Fig. 11. Erection of precast concrete pier table girder with strongbacks already installed on the ends of the pier table girders.


Fig. 12. Erection of precast concrete drop-in span girder; temporary blocking and bracing were left to the contractor.

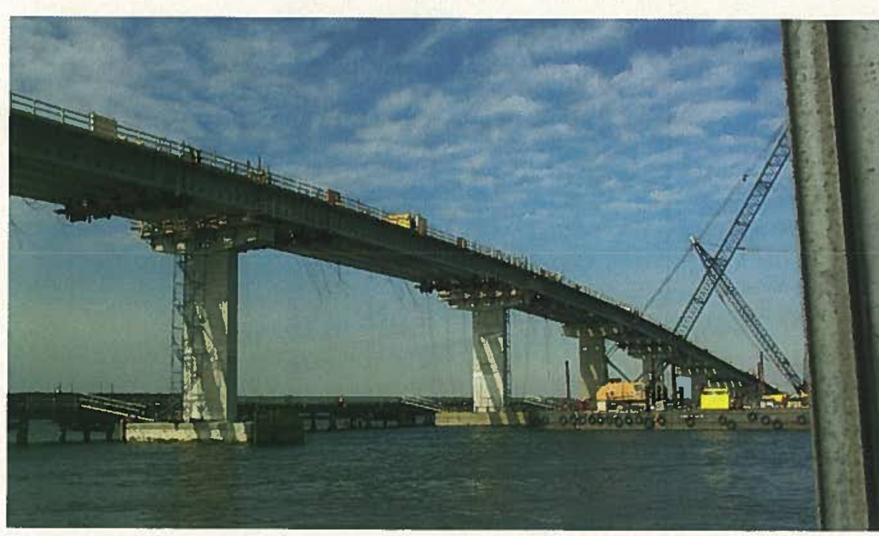


Fig. 13. Partially completed structure: barge-mounted cranes at the replacement bridge during construction of the second three-span unit.

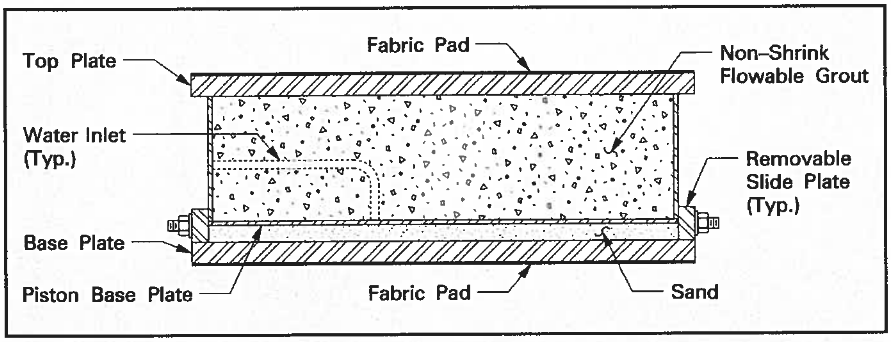


Fig. 14. Sand jack detail devised by the contractor. A steel box filled with sand is topped by a piston-like weldment.

continuous unit with strongbacks already installed on the ends of the pier table girders to support the drop-in span girders one at a time (see Fig. 11).

3. Check for girder alignment and post-tension each pier table girder down to the pier cap to develop a temporary moment connection.

4. Erect middle drop-in span girders one at a time on the strongbacks and block off gap between the pier table girders and the drop-in girders (see Figs. 12 and 13).

5. Erect the two sets of end-span girders.

6. Cast the continuity pours at girder splice locations.

7. Perform first-stage longitudinal post-tensioning to obtain continuity.

8. Release temporary post-tensioning on the pier table girders and lower superstructure unit on to permanent bearings.

9. Pour deck slab.

10. Perform second phase post-tensioning.

Temporary tie-downs — Originally, the temporary tie-down design called for temporary blocking under the pier table girder on either side of a jack placed at the location of the permanent bearing. The girder was to be set on blocking without engaging the middle jack while the girder was post-tensioned to the pier cap through the blocking.

After all the drop-in girders were post-tensioned to make them continuous, the intent was to slightly jack the structure using the middle jack, only enough to remove the blocking. At that point, a pair of flat jacks would be placed at the previous location of the blocking. The flat jacks would be engaged and the middle jack removed. Permanent bearings would be placed and the structure would be dropped down onto the permanent bearings.

Sand jacks — While the contract documents provided basics for the post-tensioning tie-downs to the pier cap, the means and methods of the temporary blocking and any necessary temporary bracing were left to the contractor. In this case, the contractor simplified the original blocking design with a novel technique using "sand jacks" instead of the more commonly used flat jacks to raise and lower the girders. A sand jack is a sand-filled steel box topped by a piston-like cover (see Figs. 14 and 15).

After the hammerhead pier caps were erected and ready to receive the

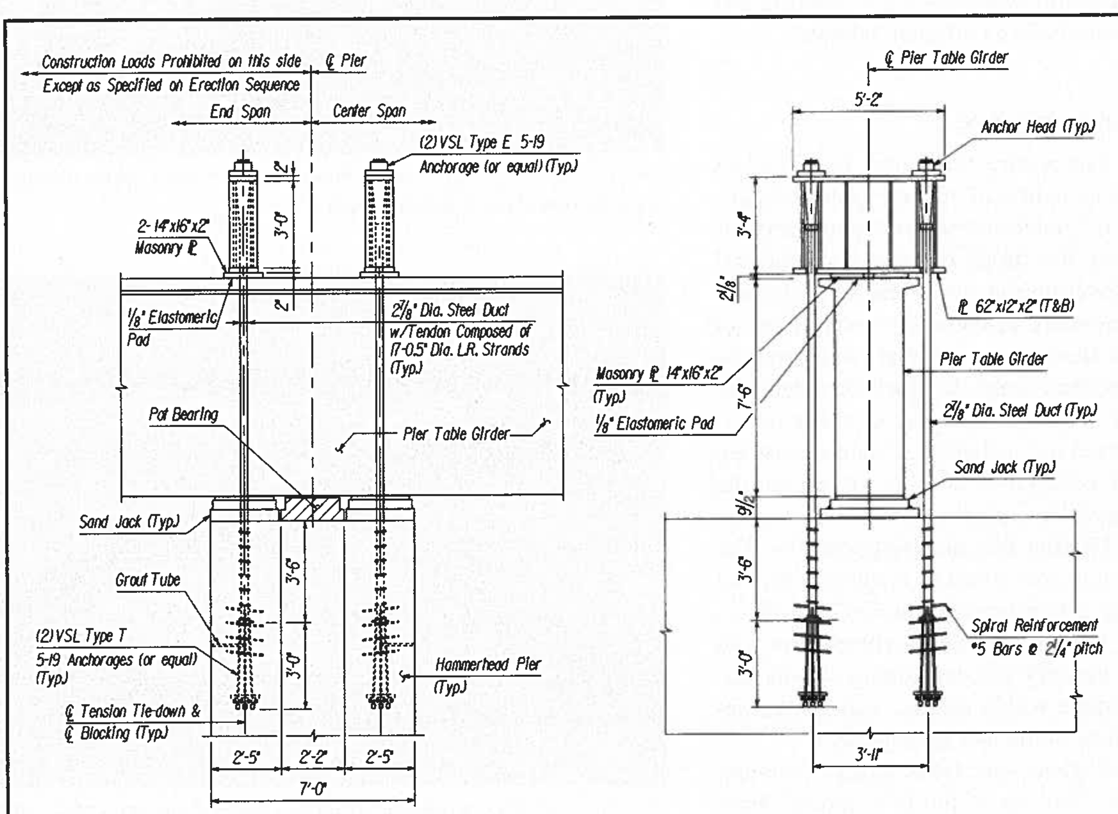


Fig. 15. Temporary post-tensioning tie-downs with location of sand jack and pot bearing.

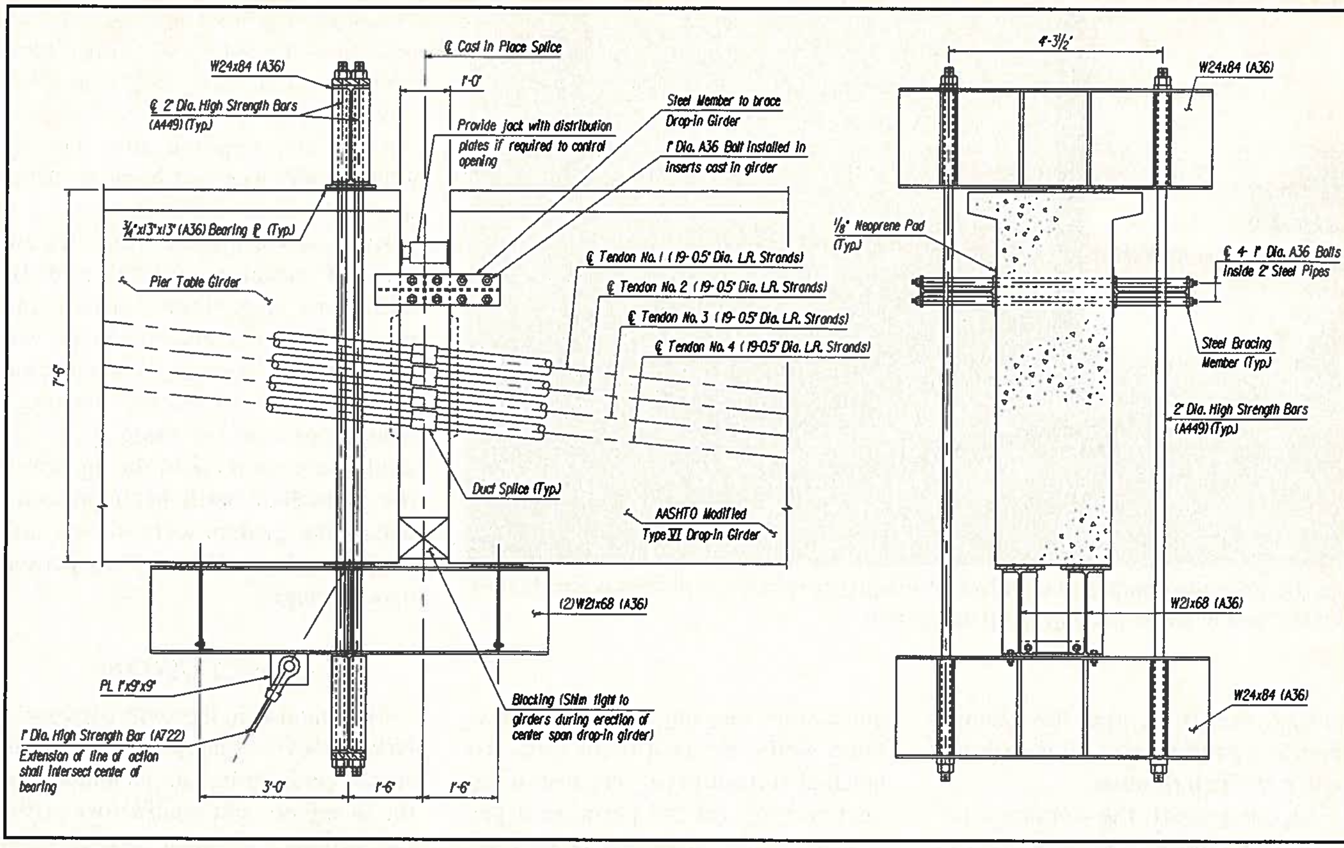


Fig. 16. Strongback detail. Center drop-in girders were erected one by one and placed on steel strongbacks attached to the ends of the pier table girders.

girders, the contractor placed the permanent bearing on top of the piers in its correct location in the middle of the pier width, using one sand jack on each side of the bearing of each of the interior piers of a three-span unit. In this way, the sand jacks were used under girders at tiedown locations as a temporary support for the pier table girder. The confined sand was able to transfer the large compressive forces of girder post-tensioning to the pier.

Once the girders were positioned on the sand jacks and the temporary moment connections were in place, the center drop-in girders were erected. The center girders were placed on steel strongbacks attached to the ends of the pier table girders (see Fig. 16). Significant unbalanced moments occur after the center drop-in girders are placed and before erection of end span girders. To maintain the required gap at the splice location due to the tendency of the piers to rotate and deflect, blocking was required to be placed at the splice. A "kick force" develops through the blocking resulting from the tendency of the piers to deflect and rotate in toward the drop-in girders (see Fig. 17).

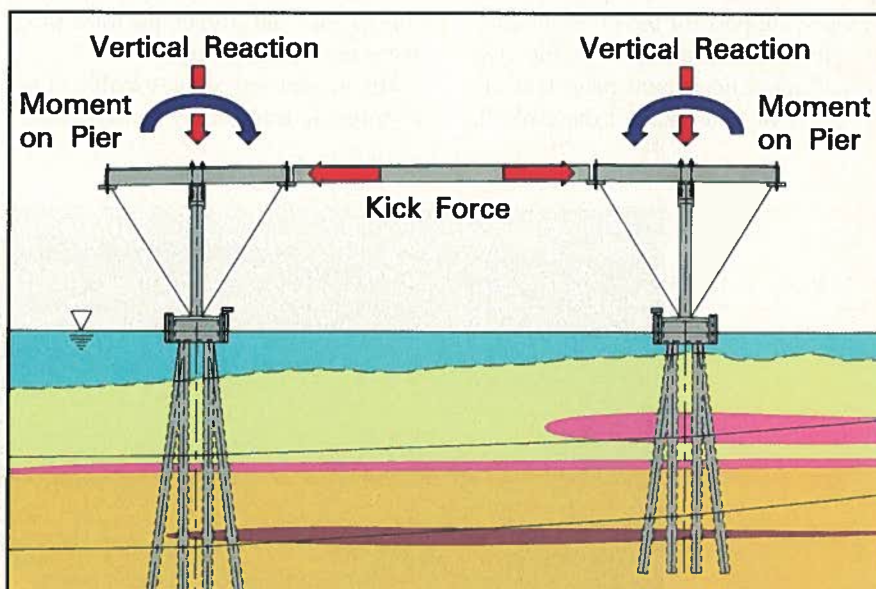


Fig. 17. "Kick force" from drop-in span illustrates moment forces generated by deflection and rotation of piers inward toward the drop-in girders.

Once the center span girders were supported by the strongbacks and blocked, the end span girders were erected. After all of the girders were in place, the dead load of the end span girders counteracted that of the center span and unbalanced moments at the interior piers were virtually eliminated. At this point, the tops of the interior piers have rotated back to the

Fig. 18. Restrainer plate and sand jack. A temporary horizontal plate was attached to the permanent girder bearing to aid in erection.

neutral position, and the girders should be properly aligned in their approximate final position.

For this project, the contractor developed a temporary positive connection attached to the girder bearing at the pier table girders that greatly facilitated the girder erection process, and provided support for the imposed construction loads. As shown in Fig. 18, an additional horizontal plate was attached to the sole plate of the girder's permanent bearing. The horizontal plate would be used to fit into two notched vertical plates attached to the anchor bolts for the permanent pot bearing as the girder was being lowered into position on top of the pier cap. The top of the notch was tapered so that the girder could be quickly guided into the proper position prior to the tie-down operation.

The guided restrainer, capable of resisting all temporary construction loads, greatly facilitated the erection process to the point where the girders could be set in a couple of hours. Additional wire rope tension cross bracing was also required after the pier table girders were set because of the large wind forces at the site.

After all the girders were erected, the CIP concrete splices and diaphragms were then formed and placed. Once the splice concrete was cured, the girders were post-tensioned longitudinally and became effectively continuous over the three spans. As sand was washed out of the interior of the sand jack with high-pressure water, the girders were slowly and uniformly lowered onto their permanent bearings.

CONCLUSION

Construction in the swift currents of New Jersey's Atlantic Seaboard presented challenging site conditions for the designers and contractors of the Ocean City-Longport Replacement Bridge. The owner's demand for durability and low maintenance led to the selection of a precast concrete spliced girder bridge system. A very durable structure was the end result, a graceful bridge that meets the critical transportation goals of Cape May County

Fig. 19. Aerial photo of completed precast bridge with vehicular traffic viewed from the south Ocean City shore. Photo courtesy: Gregg Kohl, AC photo.

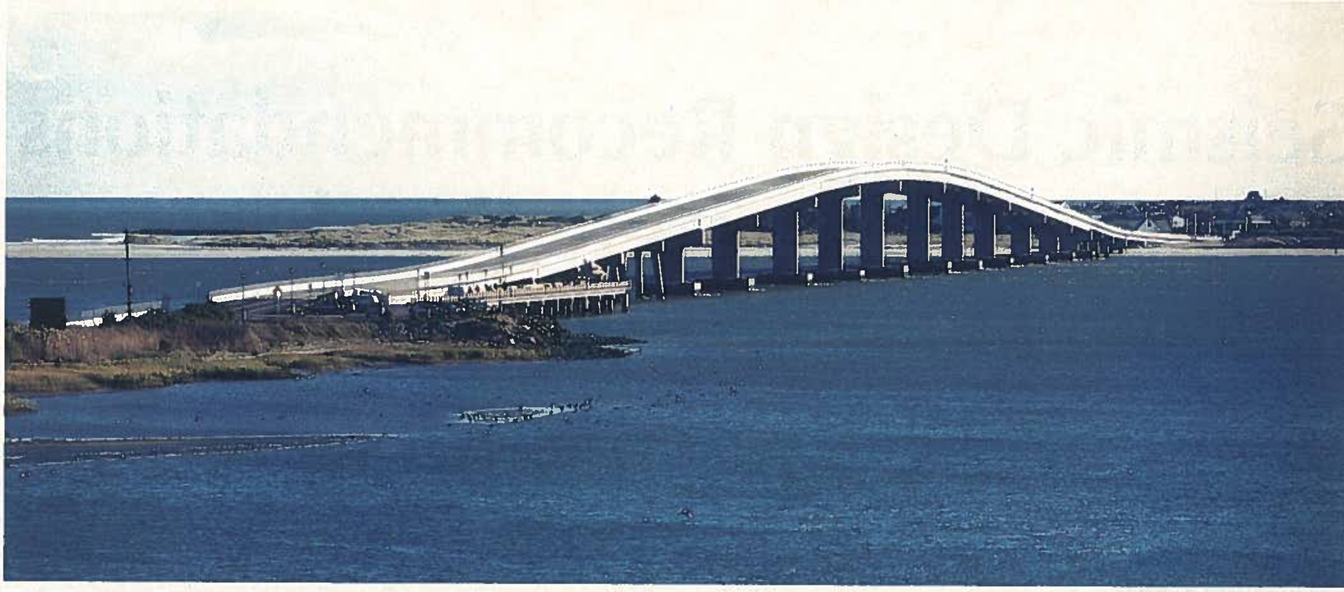


Fig. 20. Long-distance view of west side of bridge, showing fishing pier at the northwest corner of the bridge. Photo courtesy: Gregg Kohl, AC Photo.

Bridge Commission, NJDOT and the community (see Figs. 19 and 20).

The most notable project efficiencies were achieved by minimizing the need for construction in deep and rough waters. Contract documents showed the suggested innovative construction techniques, allowing the contractor to both take advantage of these techniques in the bidding process and to make adjustments to the suggested erection methods. Significant erection innovations were the use of temporary moment connections for spliced girder construction – without the need for expensive falsework towers – and precast cofferdams as a viable alternative to temporary steel cofferdams for placing pile caps.

Before his recent and untimely death, Gerard Desiderio, chairman of the Cape May County Bridge Commission, expressed the owner's pleasure with the project cost: "Precast prestressed concrete was an excellent choice for the Ocean City-Longport Bridge due to its low maintenance, economy, and aesthetic appeal, especially in light of the harsh environment at this site.

The Ocean City-Longport Replacement Bridge was the winner in two categories in the 2003 PCI Design Awards Program – the Harry H. Edwards Industry Advancement Award and the Best Bridge with Spans Greater than 135 ft (41 m). The following are the jury comments:

Harry H. Edwards Jury: "This project found engineered solutions using precast/prestressed concrete in a very difficult situation. The team's approach shows that close coordination of all aspects of the job opens up unlimited possibilities."

Bridge Jury: "The seven different precast/prestressed concrete components represented on this bridge provided simplicity of construction and recognized the material's excellent durability in an aggressive environment."

ACKNOWLEDGMENTS

The success of this project was the result of the close cooperation throughout the design and construction process of the designer, Parsons Brinckerhoff; the owner, the Cape May County Bridge Commission and, in particular, the late Commission Chairman Gerard Desiderio; the Cape May County Engineer, Dale Foster; the New Jersey Department of Transportation (NJDOT), which oversaw the design and served as construction manager; the Federal Highway Administration (FHWA); and the contractor, Kiewit/Tidewater, a Joint Venture.

CREDITS

Owner: Cape May County Bridge Commission

Client: County of Cape May

County Engineer: Dale M. Foster, P.E.

Engineer of Record: Joseph Mumber, P.E., Project Manager, Parsons Brinckerhoff

Contractor: Kiewit Construction Company and Tidewater Skanska, Inc., a Joint Venture

Project Manager: Greg Hill, P.E., Kiewit Construction Company

Precast Concrete Fabricator: Bayshore Concrete Products Corporation

Chief Engineer for Precaster: Chad Saunders, P.E., Bayshore Concrete Products Corporation

Design Oversight and Review: New Jersey Department of Transportation

Project Manager for NJDOT: E. David Lambert III, P.E.

Construction Supervision and Inspection: New Jersey Department of Transportation

Resident Engineer, NJDOT: Jeff Palmer, P.E.