
GC CONSTRUCTORS

# Reconnected

*Hurricane Katrina knocked parts of the bridge carrying U.S. Route 90 over Biloxi Bay, in Mississippi right off their piers and into the rapidly moving water below. When the storm retreated, engineers were left to design and construct a replacement crossing that had to have one lane in each direction operational in just 18 months and three lanes in each direction operational in less than two years. The design/build project delivery method made it possible.*

**By Patrick Cassity, P.E.**

The alignment of the 72-span bridge was curved at both ends to avoid the wreckage of the original crossing, some parts of which will remain in the bay for use as reefs. The loss of the bridge created a significant hardship for the surrounding communities, adding roughly 30 minutes to what had been a 2-minute trip.




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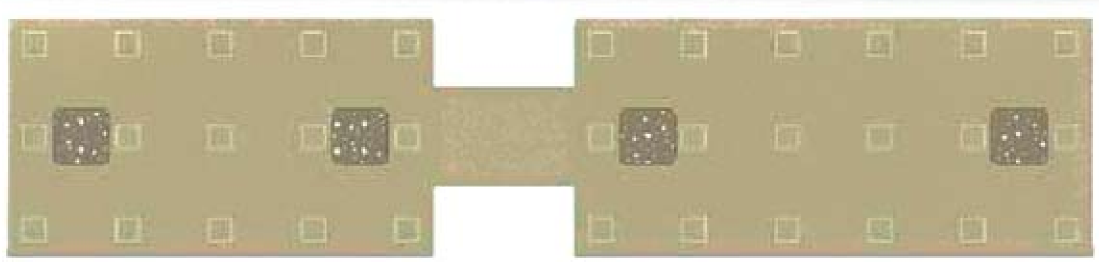
When Hurricane Katrina made landfall, on August 29, 2005, devastating the Gulf Coast, the bridge in Mississippi carrying U.S. Route 90 over Biloxi Bay between Biloxi and Ocean Springs was heavily damaged, one of many major highway and railroad bridges knocked out of service by the storm. In the months following this ruinous natural disaster, the Mississippi Department of Transportation and the Federal Highway Administration elected to replace the damaged bridge with a high-level crossing using the design/build method of project delivery.

This fast-tracked, $339-million design/build contract was awarded to GC Constructors, a joint venture of Massman Construction Company, of Kansas City, Missouri; Traylor Brothers, Inc., of Evansville, Indiana; Kiewit Southern Company, a subsidiary of Kiewit Corporation that has its headquarters in Peachtree City, Georgia; and Parsons Corporation, which is headquartered in Pasadena, California. Massman Construction served as the managing partner while Parsons Corporation served as the principal design firm. The team was required to have one lane open to traffic in each direction by November 2007, a mere 17 months after the issuance of the notice to proceed, and the entire project completed by April 2008, just 22 months from the notice. The project was to have a total length of 2.4 mi (3.9 km) and include a 1.6 mi (2.6 km) bridge over Biloxi Bay and an 800 ft (244 m) long bridge over a nearby CSX rail line. The bay bridge was to comprise two parallel structures.

The original, 1959 bridge consisted of low-level approach spans and a bascule span for ship navigation. The approach spans comprised simply supported prestressed, precast girders cast integrally with the deck; each span was 52 ft (16 m) long. The 33 ft (10 m) wide superstructure units weighed 340 kips (154,224 kg) each and were supported on pile bents.

The eye of the storm passed 60 mi (97 km) west of Biloxi. The bay saw peak wind gusts of 100 mph (160 km/h), a peak storm surge of 22 ft (7 m), and waves of up to 8 ft (2.4 m). In general, the spans in which the lower chord was at an elevation of 23 ft (7 m) or less were badly damaged while those with lower chords above that level remained relatively intact. Many of the low-level superstructure units were displaced from their pile caps and tossed into the water, some of them even being flipped upside down. A hydraulic study of the bridge concluded that because it was damaged above the elevation of the average water level, the wave-induced loads imposed by wave crests hitting the decks were the primary damaging agent (*Wave Forces on Bridge Decks,* by S. Douglass, Q. Chen, J. Olsen, B. Edge, and D. Brown [Washington, D.C.: Federal Highway Administration, 2006]). These waves produced both a horizontal and an uplift force large enough to overcome the bearing connection capacity. Buoyancy loads from trapped air under the deck as the waves crested and the surge rose were probably only of secondary importance.



PARSONS

**Two-column piers with waterline footings support each of the parallel structures. To strengthen the columns of the piers that are 55 ft (17 m) and taller, a cross-strut was added at a height of 40 ft (12 m) from the top of the footings. Keeping the cross-strut at the same elevation regardless of pier height maintained consistency in the rebar cages, formwork, and concrete placements.**



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MISSISSIPPI DEPARTMENT OF TRANSPORTATION

**The community's desire for an attractive structure led the designers to choose a blue-green color for the fascia and ivory for the superstructure and substructures. Overlooks spaced along a path shared by the parallel structures feature benches and ornamental aluminum railings. At night the bridge is illuminated with a string of ornamental necklace lights.**

Approximately 35,000 cars per day crossed the four-lane, low-level bridge before the hurricane, and there were no other crossings of the bay. The loss of the bridge created a significant hardship for the surrounding communities; the detour route on Interstate 10 added roughly 30 minutes to what had been a 2-minute trip.

The roadway alignment of the new bridge is curved at each end to shift the new bridge's alignment 150 ft (46 m) south of the original crossing. No portions of the original bridge were salvageable for reuse in the new structure, but some parts are being recycled into reefs in the bay.

The new bridge, as mentioned above, comprises two parallel structures, each carrying three lanes of traffic. The eastbound bridge also has a 12 ft (3.6 m) path that will be shared. The out-to-out width is 129 ft (39 m). Since the surrounding communities wanted an attractive structure, aesthetic considerations loomed large in developing the design. As a result, a concrete coating was used to color the fascia girders blue-green, and the formed concrete surfaces of the superstructures and substructures were colored antique ivory. The pedestrian railing along the shared path is an ornamental aluminum picket railing. There are also three overlooks spaced along the path, each with a bench. The outside traffic barriers feature an open concrete design rather than the traditional solid configuration of Jersey barriers. At night the bridge is illuminated with a string of ornamental necklace lights attached to the fascia girders, and there are edge accent lights on the piers.

The bridge comprises 72 spans numbered from west to east. The curved portion of the low-level approach on the west side has 11 spans, each 85.75 ft (26 m) long, with 54 in. (1,372 mm) deep girders. The next, or tangent, portion includes 30 spans 120 ft (36.6 m) in length with 72 in. (1,828 mm) deep girders. The curved portion of the low-level approach on the east side has 6 spans 85.5 ft (36 m) in length with 54 in. (1,372 mm) deep girders. Its tangent section has three 120 ft (36.6 m) spans with 72 in. (1,828 mm) deep girders. Altogether there are 9 spans in the high-level approaches on the two sides of the main channel portion of the bridge, which provides clearance for navigation beneath. Each approach span here is 150 ft (46 m) long and has 78 in. (1,981 mm) deep girders. The majority of the approach spans consists of three-span units. The three-span channel unit itself has a 250 ft (76 m) main span and two 200 ft (61 m) long side spans. It features 12 ft (3.66 m) deep haunched segments above the piers and modified 78 in. (1,981 mm) deep girders for the drop-in and end segments, which are part of respectively the main span and the side spans. The 54 in. (1,372 mm) girders were transported to the site via truck, and the rest of the girders were transported via barge.

**It was important that a well-engineered and well-planned geotechnical program be developed during the bidding phase and then be rapidly implemented once the winning contractor was named.**

The western and eastern abutments, referred to respectively as spans 72 and 1, are supported on 18 in. (457 mm) square precast, prestressed pilings. All of the piers are supported by waterline footings supported by precast, prestressed piles. Piers 2 through 40 and 65 through 71 use 24 in. (610 mm) square piles and piers 41 to 64 use 30 in. (762 mm) square piles. The piles were transported to the site via barge.

The massive reconstruction efforts that took place along the Gulf Coast after Katrina placed extreme demands on the precast-concrete industry, and this posed risks in the areas of supply and scheduling for this project. The design was therefore developed in such a way as to diversify the required precast piling and girders and to minimize the number of specialty products, which are produced by only a few precasters. This approach gave GC Constructors wide latitude during construction in the event that a particular precaster was unable to deliver products within the required schedule. This is the main reason that 72 in. (1,828 mm) girders were used extensively, rather than the more typical (and more economical) 78 in. (1,981 mm) girders. Moreover, 36 in. (914 mm) square piles were avoided outright because of the limited number of producers, even though the significant vessel collision loads that this bridge may experience are more effectively resisted by such large piles.

The vertical profile of the bridge at each end slopes up sharply until the elevation of the lower chord is above the critical wave height of the predicted 100-year storm, which would be equivalent to Katrina. This is the most effective and reliable method of guarding against future damage to the bridge from such a hurricane. What is more, 18 in. (457 mm) tall concrete shear keys were used on those portions of the superstructure below the critical elevation to help keep the superstructure from being knocked off its piers during a storm.

There is no more important early critical path activity on design/build projects than geotechnical investigation and engineering. The input from this activity is necessary for completing the designs of the foundations and embankments, the first elements that a contractor must attend to in the field. Increases in pile quantities and schedule changes were two of the greatest risks that the design/build team faced on this project. Therefore, it was important that a well-engineered and well-planned geotechnical program be developed during the bidding phase and then be rapidly implemented once the winning contractor was named. Eustis Engineering Company, Inc., of Metairie, Louisiana, and Dan Brown & Associates, LLC, of Sequatchie, Tennessee, served as the geotechnical engineers.

The subsurface conditions over most of the site feature clays ranging from very soft at the mud line to stiff at a depth of 80 to 100 ft (24 to 30 m). Extremely dense sand layers beneath the soft clay layers at the far eastern end of the bridge required 50 to 90 blows per foot to pierce in a standard penetration test. Since the vast majority of the piles are friction piles established in clay, it was critical to determine the setup curves—that is, the curves that define the increase in pile capacity after the pore water pressures from pile driving dissipate—and to validate the required pile lengths using an indicator pile program. The target ultimate capacities were 200, 450, and 550 tons (181, 408, and 499 metric tons) for respectively the 18, 24, and 30 in. (457, 610, and 762 mm) piles. A factor of safety of 2.0 was used because both Statnamic load tests—developed by Bermingham Foundation Solutions, of Hamilton, Ontario—and static load tests were performed.

All told, 19 indicator piles were driven away from the bridge alignment and monitored with dynamic pile testing equipment. The Case Pile Wave Analysis Program (CAPWAP), developed by Soil Dynamics, Inc., of Cleveland, was used to perform a soil analysis. Pile restrikes were carried out at various intervals of time up to 28 days after construction to determine the pile setup curves. Enough data were gathered to reliably predict the final pile capacities without performing restrikes during production pile installation. The elimination of restrikes during production piling allowed team members to remain focused on building the bridge within the accelerated schedule rather than on testing piles.

**The entire project includes not only the 1.6 mi (2.6 km) bridge over Biloxi Bay but also an 800 ft (244 m) long bridge over a nearby CSX rail line. The bridges are designed to withstand a hurricane of the same strength as Katrina.**

KEN MURPHY

PARSONS

**The new bridge carries three lanes in each direction in addition to a path that the two parallel structures share. The out-to-out width is 129 ft (39 m).**

Static and Statnamic axial load tests were performed on separate 24 in. (610 mm) piles, and two Statnamic tests were performed on 30 in. (762 mm) piles. The static axial test was used to validate the results of the Statnamic test. The cost and time associated with Statnamic testing are significantly less than for static tests for large piles. In addition to the axial tests, a Statnamic lateral load test was performed for each pile size to provide validation of the soil-structure interaction modeling, and these data were used in a vessel collision analysis.

One of the most challenging aspects of the design was developing an economical approach to pier and foundation construction that addressed the wide range of geometric and loading conditions. The piers vary in height from 6 ft (2 m) to 90 ft (27 m). The maximum total vertical reactions range from 1,000 kips (4,450 kN) to 7,000 kips (31,000 kN), and the vessel collision loads range from 200 kips (890 kN) to 2,850 kips (12,680 kN) at the main channel spans. The goal was to develop a design that could be built using an assembly-line type of approach that would minimize variability in the areas of formwork and equipment.

The solution was to use two-column piers with waterline footings supporting each of the parallel bridges. Pile bent and single-column hammerhead piers were considered but were determined to be less economical than two-column piers. The pile bents could not be economically configured to meet the owner's vessel collision requirements, and the hammerhead piers required much more material than did the two-column piers. To strengthen the columns for the taller piers, a cross-strut was added at a height of 40 ft (12 m) from the top of the footings in the piers that were 55 ft (17 m) or taller. Keeping the cross-strut at the same elevation regardless of pier height maintained consistency in the rebar cages, formwork, and concrete placements. The pier columns in the three-span channel unit are 6 ft (1,829 mm) square while the remaining columns are 5.5 ft (1,676 mm) square.

All piers located in a water depth of 2 ft (0.6 m) or more were designed for a 200 kip (890 kN) impact from a drifting barge. The vessel loading requirements for an aberrant barge tow are shown in the accompanying table.

### VESSEL COLLISION LOADS

| PIER TYPE | DISTANCE FROM CHANNEL CENTERLINE | LOAD |
|---|---|---|
| Channel pier | 125 ft (38 m) | 2,850 kips (12,680 kN) |
| Channel approach pier | 275–450 ft (84–137 m) | 2,650 kips (11,790 kN) |
| Approach pier | 451–850 ft (137.5–259 m) | 2,500 kips (11,120 kN) |
| Approach pier | 851–1,100 ft (259.5–335 m) | 1,350 kips (6,005 kN) |
| Approach pier | 1,101–1,500 ft (335.7–457 m) | 800 kips (3,559 kN) |
| Approach pier | 1,501–1,900 ft (457.6–579 m) | 300 kips (1,334 kN) |

The local scour depths from the 100-year design storm ranged from 11 ft (3.4 m) to 26 ft (7.9 m) in the main channel. The maximum water velocities and scour occur when the storm subsides and the surge flows back to the ocean.

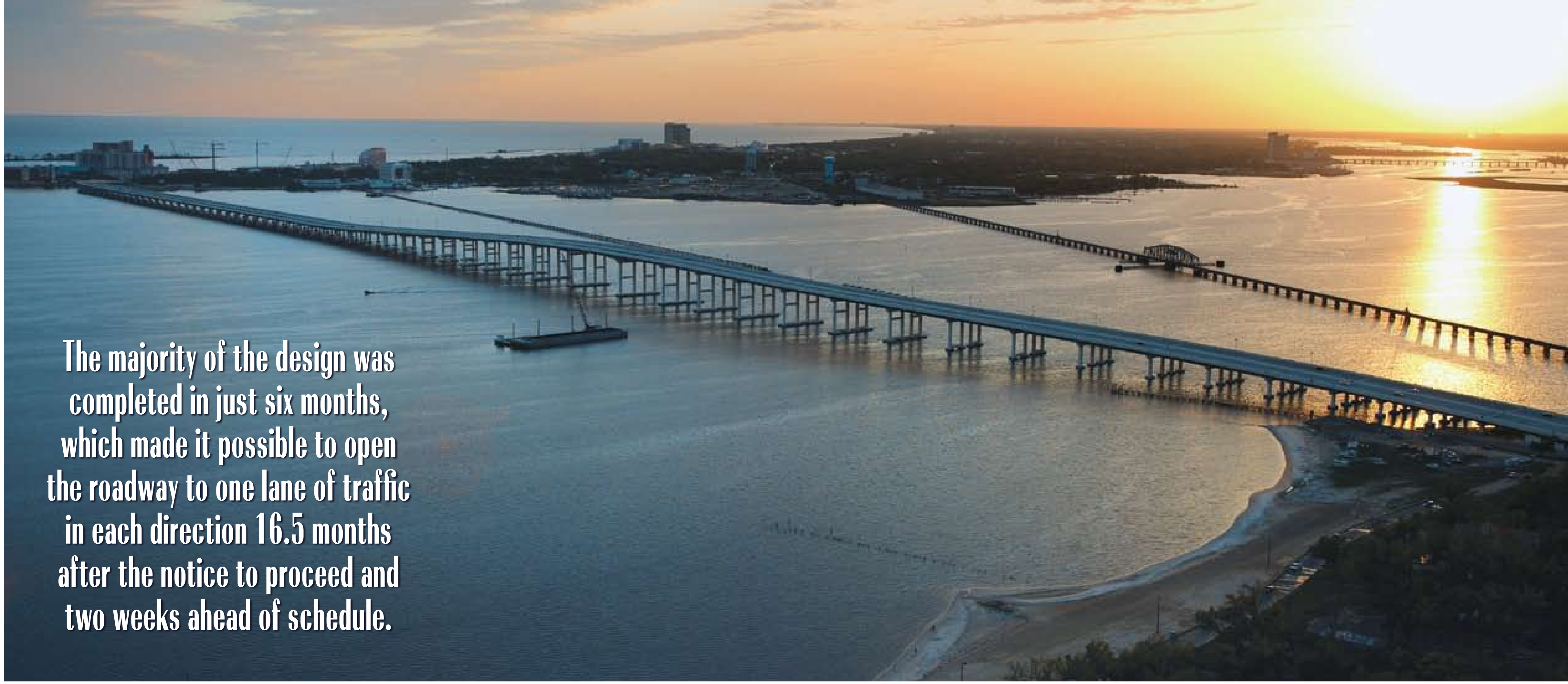
The foundations that had designs governed by vessel collision loads of 200 kips (890 kN), namely, those for piers 4 to 40 and 64 to 70, consist of four 24 in. (610 mm) precast piles under each column. The compressive strength of the piles is 6,000 psi (41 MPa). The 300 kip (1,334 kN) foundations, which are beneath piers 41 to 43 and 62 to 64, maintain the same footing size but use four 30 in. (762 mm) piles with a compressive strength of 8,500 psi (59 MPa). The 800 kip (3,559 kN) foundations, which are beneath piers 44, 45, 60, and 61, utilize the same footing and pile configuration but include a strut at the top of the footings to distribute the collision load between column foundations.

The next group of foundations all fit within a 25 by 116 ft (7.6 by 35 m) area but utilize different footing sizes and pile configurations within that area to resist the required collision load. The 1,350 kip (6,005 kN) foundations, which are beneath piers 46, 47, 58, and 59, use six 30 in. (762 mm) piles under each column, and the two columns are connected by a strut at the top of the footing. The 2,650 kip (11,120 kN) foundations for piers 48 to 50 and 55 to 57 are supported by combined footings under each bridge pier with a waterline strut connecting the footings. The total number of 30 in. (762 mm) piles in this foundation is 26. The 2,650 kip (11,120 kN) foundations for piers 51 and 54 have the same footing configuration but here the total number of 30 in. (762 mm) piles is 33. Finally, the 2,850 kip (12,680 kN) channel foundations, which are for piers 52 and 53, use a solid footing of 25 by 116 ft (7.6 by 35 m) atop a total of 36 of the 30 in. (762 mm) piles.

The vessel collision analysis was performed using two different models and software packages. The first model involved a nonlinear, three-dimensional analysis of the pier, foundation, and soil layers. The superstructure was modeled using equivalent spring supports. This model made it possible to derive the foundation stiffness matrix, which was entered into the second model. This global model was a three-dimensional frame model of the piers and superstructure. The unit of the pier being impacted, as well as adjacent units, was represented in the model. The superstructure was modeled as a grid, and the foundation stiffness matrix at the base of the piers was entered. The superstructure essentially acts as a rigid diaphragm to help distribute the collision forces to adjacent piers that are not impacted. The distribution of the collision load is a function of the foundation stiffness of the impacted pier, the stiffness of the impacted pier above the foundation, and the stiffness of the adjacent piers.

The analysis was iterative in nature because of the nonlinear response of the soil and the resistance to the collision load by multiple piers. The design criteria for the project required the structure to remain elastic under 90 percent of the collision load. The vessel load was applied in the global model, and the displacements at each of the piers were compared with the results of the local models of the corresponding pier types. If the displacements differed significantly, the new foundation load was entered into the local model and the analysis was rerun. This was repeated until convergence was achieved.

The required vertical and horizontal navigational clearances are respectively 95 and 150 ft (29 and 46 m). To accommodate the clearance, the navigation unit, as mentioned above, consists of a three-span unit with a main span of 250 ft (76 m) and side spans of 200 ft (61 m). The girders consist of 78 in. (1,981 mm) bulb Ts for the drop-in and end-span segments with haunched segments that vary from 6.5 to 12 ft (2 to 3.7 m) over the piers. The end segments are 141 ft (43 m) long and weigh 96 tons (87 metric tons); the drop-in segments are 132 ft (40 m) long and weigh 87 tons (79 metric tons); and the haunched segments are 115 ft (35 m) long and weigh 104 tons (94 metric tons). All of the segments were delivered by barge from the precast yard. Each line of girders comprises five segments, and these were spliced together using four 17-strand posttensioning tendons, each 0.6 in. (15 mm) in diameter, that were anchored in the end segments. Both the rebar and the ducts were spliced within the 18 in. (457 mm) wide cast-in-place closures.




JOHN THOMAS

A web thickness of 9 in. (229 mm) was used instead of the 7 in. (178 mm) thickness for the typical 78 in. (1,981 mm) bulb T girder. This allowed adequate clearance for the posttensioning ducts. The typical 78 in. (1,981 mm) girder forms were used to cast the drop-in and end segments by just spreading the forms apart and letting the top flange, web, and bottom flange increase by 2 in. (51 mm).

**The significant reconstruction effort that occurred within the Gulf Coast region in the aftermath of Hurricane Katrina created tremendous demand for precast-concrete products. For this reason, the designers chose to diversify the types and sizes of precast pilings and girders they used and to minimize the number of specialty products.**

Cast-in-place diaphragms are located at each closure concrete placement and at the piers. The pier diaphragms were posttensioned with three five-strand tendons measuring 0.6 in. (15 mm) in diameter. The channel unit is supported by fixed pot bearings at each of the piers for the main span and by sliding pot bearings at the piers for the end spans. The height of the channel piers is such that creep, shrinkage, thermal movements, and elastic shortening during tendon stressing would not generate significant loads in the piers and foundations. A time-dependent analysis accounting for the erection sequencing was performed to accurately account for all locked-in forces and force redistributions caused by creep and shrinkage.

The navigation unit girders were erected using steel falsework bents on each side of the piers for the main span to support the haunched segments. The falsework was supported by the permanent waterline footings. Strongback hanger beams were attached to the drop-in and end segments to support them from the haunched segment until continuity was achieved.

The majority of the design was completed in just six months, which made it possible to open the roadway to one lane of traffic in each direction 16.5 months after the notice to proceed and two weeks ahead of schedule. With this innovative and streamlined approach to management, coordination, and design, the entire project was completed in just 22 months. This project is an excellent example of how design/build delivery, precast construction, and a great deal of hard work and cooperation on the part of designers, constructors, and owners can result in the successful fast-track delivery of major transportation projects. **CE**

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