DESIGN/BUILD OF US-90 REPLACEMENT BRIDGE OVER ST. LOUIS BAY

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ABSTRACT

With the loss of the US 90 bridge spanning St. Louis Bay, the Mississippi DOT awarded a design/build contract five months after Hurricane Katrina for a 2.1 mile long replacement structure. The deadline for opening the first two lanes was May 16, 2007 followed by November 30, 2007 for the entire bridge. To meet the rapid construction requirements, the design relies upon precast and prestressed elements for both the substructure and superstructure. Numerous precasting facilities were utilized to meet production requirements.

Precast, prestressed piles, ranging in length from 50 ft to 164 ft, form the bridge foundations with both pile bents (24 in. and 36 in. piles) and waterline footings (30 in. piles) being utilized. Additional pre-cast substructure elements include waterline footing soffit slabs and struts and two piece precast cap beams for the pile bents.

Typical superstructure elements consist of 59 spans of BT-78 girders (maximum span = 154 ft) and 14 spans of AASHTO Type IV girders (maximum span = 112 ft). To provide 85 ft of vertical and 150 ft of horizontal clearance at the navigation channel, a three span, post-tensioned, spliced haunch girder system was utilized and consists of a 250 ft main span and 200 ft end spans, respectively.

This paper discusses the advantages of utilizing precast, prestressed elements to speed the construction of this critical and time sensitive project.

Keywords: Design Build, Rapid Construction, Piles, Precast, Substructures, Precast, Contractor Alternate, Post-tensioning, Spliced Girders

INTRODUCTION

When Hurricane Katrina hit the Gulf Coast of Mississippi on August 29, 2005, the devastation included the loss of a vital link between the communities of Pass Christian, MS and Bay St. Louis, MS, located approximately 1 1/2 hours east of New Orleans, LA. The storm surge of Hurricane Katrina dealt a lethal blow to both the superstructure and substructure of the Bay St. Louis Bridge, making replacement the only reasonable solution to reconnect historic US Highway 90 and the communities which it ties.

The destroyed bridge was constructed in 1953-4 and consisted of 241, 41 ft-long reinforced concrete spans with a three span steel navigational unit made up of a 124 ft bascule span flanked by 75 ft - 8 in back spans. A majority of the typical spans rested upon pile bents supported on 24 in. precast concrete piles. Poor soil conditions over a section of the crossing required that 25 bents be supported by concrete jacketed steel H-pile that had maximum tip elevations of 192 ft below mean sea level. A major contributing factor in the destruction of the bridge was the close proximity of the deck to the water which ranged from approximately 13 ft near the bridge ends to a maximum of 23 ft at the bascule. A typical view of the decimated bridge is presented in Fig. 1.

Fig. 1 Looking west from the destroyed Bay St. Louis Bridge bascule spans

BASIC REQUIREMENTS

After the damage from Katrina was assessed, replacing the obliterated bridge as quickly as possible was deemed a key component to speeding the recovery of the region. The need for a rapid reconstruction led the Mississippi Department of Transportation

(MDOT) to issue the first major design/build contract in Mississippi as a means to obtain a replacement bridge in the shortest possible time. With the support of the Federal Highway Administration (FHWA), a Request for Proposals (RFP) for a replacement structure was released on November 3, 2005, just slightly more than two months after Hurricane Katrina made landfall. Governing design requirements of the replacement structure as expressed in the RFP included:

- Design to AASHTO Standard Bridge Design Specifications[1]
- Navigational span clearances
    - 85 ft, vertical
    - 150 ft, horizontal
- Low chord elevation of remaining bridge = 37.00
- Four 12 ft lanes with respective shoulders and breakdown lanes
- 12 ft shared use path
- Vessel impact per AASHTO Guide Specification and Commentary for Vessel Collision Design of Highway Bridges[2]

Three teams submitted bid packages and after the technical scores, bid prices, and days to completion were tallied, the successful team was Granite-Archer Western (GAW), a joint venture, with HNTB Corporation acting as the contractor's designer. The total project cost was $267 million, with FHWA funding the project and MDOT assuming ownership. The contract was divided into two milestones, the first requiried two lanes of unrestricted traffic crossing the bridge by May 16, 2007 which has been met. The second milestone requires completion of all contract work by January 5, 2008.

During the proposal phase, segmental and steel superstructure types were briefly considered, but ruled out due to the extremely aggressive construction schedule. To be successful in winning the job and to provide the bridge on time, GAW's construction approach was to use as few pieces as possible and to utilize precast elements where feasible. The final design included the application of pile bents, waterline footing bents, AASHTO Type IV girders, BT-78 girders, modified BT-78 haunch girders, and a cast-in-place (CIP) deck. Design and construction efforts were focused on opening the two eastbound lanes allowing for temporary two way traffic to meet the first milestone. The completion of the westbound lanes will coincide with the completion of the second milestone.

WINNING DESIGN

The project bids were opened on January 25, 2006 and the design team commenced work on January 26, 2006 and the Notice to Proceed was given on February 20, 2007. Demolition commenced shortly thereafter and permanent construction at the site began on approximately April 30, 2006, well ahead of completing a final set of plans. By July 1, 2006, with construction well under way, the design was approximately 90 percent complete, with only miscellaneous detail drawings such as expansion joints and top of slab elevations left to submit. To execute such an enormous design effort in the compressed time frame, the design of the bridge was conducted in the Kansas City office of HNTB, with several designers from other offices temporarily relocating. Additionally, the design of the substructure and superstructure for approach Spans 63-76 were subcontracted to the Jacksonville office of Reynolds, Smith and Hills, Inc., which played a significant role in meeting the design deadlines. To achieve the first contract milestone, the design was focused to match the order of construction with select groups of pile bents, waterline footings, and beams being designed first to provide adequate lead time for the precast suppliers.

PRECAST/PRESTRESSED SUBSTRUCTURE COMPONENTS

Facing a compressed schedule, the joint venture team looked to utilize precast and prestressed elements within the substructure to speed the construction. Four different precast components were used within the substructure including precast/prestressed (P/S) piles, precast pile bent caps, precast soffit slabs for the waterline footings, and precast struts to join the waterline footings. Each of these components will be described in greater detail in the following sections.

PRECAST/PRESTRESSED PILES

The replacement bridge is supported by 77 bents, all of which rely upon driven precast/prestressed (P/S) piles. To provide GAW more flexibility with precast suppliers, various pile sizes were used throughout the job. The waterline footings utilized 30 in. P/S piles (f'_c = 6000 psi), the pile bents on the water utilized 36 in. P/S piles (f'_c = 6000 psi), and 24 in. piles (f'_c = 6000 psi) were used for the end bents and pile bents on the land. The foundation design was particularly challenging, as there were variable layers of loose sand, silty clay, and dense sand resulting in vastly different pile behavior from one location to another. Some piles relied almost entirely on end bearing, while others relied almost entirely on skin friction with most piles requiring a combination to meet the design capacities.

After evaluating different options, the joint venture team adopted a primary strategy of minimizing the number of piles per bent. This resulted in a significant increase in the loads acting on a given pile within a bent. For a comparison, typical design bearing in the State of Florida for a 30 in. pile is limited to 1200 kips[3]; where as for the Bay St. Louis design, the design bearing was increased to a maximum of 1570 kips, a 31 percent increase. Similar increases in design bearing were carried to the 36 in. piles with a maximum resistance of 1650 kips required in some locations.

Due to the compressed schedule, providing the pile lengths for the precaster was necessary prior to completing a full geotechnical investigation. Production pile lengths and initial driving criteria were developed based on the observed behavior of 26 out-of-position indicator piles which were scattered along the length of the bridge; all driving was monitored with the Pile Driving Analyzer[4] (PDA) system and the data processed via GRLWEAP[5] analysis. Since the sample size of indicator piles was relatively small, approximately three bents per indicator pile, and the variability of the underlying soil great, the first production pile driven at each bent was also monitored with the PDA equipment to record the driving response.

Initial hammer sizes were based upon preliminary geotechnical boring information. Upon starting the Indicator Pile Program, the required driving energy to obtain bearing was such that larger hammers were needed. The same larger hammers were used to finish the Indicator Piles, drive the PDA pile at each bent, and drive all production pile. Data of interest included the tension and compression stresses in the pile during driving and the estimated bearing at the end of the initial drive. The information gained from the installation of the first production pile was then compared to the applicable indicator pile; the driving criteria for the respective bent were then updated if significant differences in behavior were present.

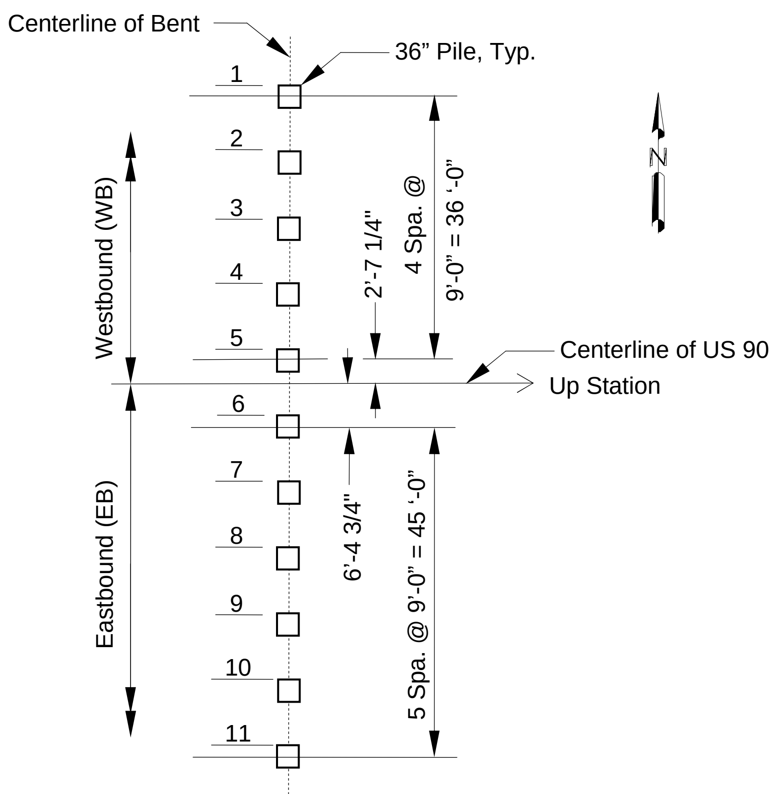
Pile Bents

Spans 1-26 and 51-76 are supported by pile bents which consist of vertical pile joined at the top with a continuous concrete cap beam. Pile bents were selected due to the economy of limiting the construction steps, and implementing precast cap beams for the 36 in. pile bents further sped the construction. The geometry and a typical completed 11 pile bent are shown in Fig. 2.

36 in. Pile Bents

All the pile bents which utilized the 36 in. piles are located in the waters of St. Louis Bay. This includes Bents 2-26 and 51-63 which contain 11 piles per bent supporting nine lines of BT-78 girders. Bents 64-68 consist of nine piles per bent supporting 11 lines of AASHTO Type IV girders. The 36 in. piles, constructed with a 22 1/2 in. diameter void, were selected for the described bents due to the ability to resist both significant axial loads (1650 kips max) as well as resist the 200 kip vessel impact loading required per the RFP (for bent locations greater than 1900 ft from the navigational channel centerline).

With lengths ranging from 130 ft to 164 ft and weighing approximately 936 lbs/ft, lofting and installing the piles required the use of particularly heavy equipment and rigging. In the preliminary design phase, the piles were designed with 28 - 0.5 in. diameter 270 ksi low relaxation strand. The strand was initially tensioned to 75 percent of the ultimate capacity resulting in a compression stress of 1.36 ksi due to prestress prior to losses. This limited the two point pick length of the pile to 156 ft before the tension stresses resulting from flexure exceeded $5\sqrt{f'_c}$ per AASHTO design. By increasing the 28 strands to 0.6 in. diameter and maintaining the same tensioning stress, the two point pick length was increased to 176 ft due to the 41 percent increase in prestress. The increased cost for the larger strand was determined by the contractor to be offset by the time savings resulting from the reduced rigging necessary to three point pick and loft the piles. Table 1 provides a comparison in the maximum pick lengths given the 0.5 in. diameter vs. 0.6 in. diameter strand. Fig. 3 shows a typical two point lofting operation of a 133 ft, 36 in. precast pile with 0.6 in. diameter strand.



a. Geometric configuration of a typical pile bent

b. Completed bent with precast cap beam (looking west, back station)

Fig. 2 Typical 11 pile bent

Table 1 Maximum pile pick length in feet given strand and rigging configuration

Pile Size (in.)	0.5 in. diameter 270 ksi strand			0.6 in. diameter 270 ksi strand		
	Single Pt.	Two Pt.	Three Pt.	Single Pt.	Two Pt.	Three Pt.
30	94	135	189	106	152	213
36	109	156	218	123	176	246

Fig. 3 Two point lofting of 133 ft, 36 in. pile at Bent 57

The piles were driven to capacity in a closed-end configuration using a D-100 diesel hammer. Major design requirements included proper pile alignment, axial capacity and minimum embedment to resist lateral loading. Once the design requirements within a bent were satisfied, the piles were cut off to the design elevation exposing the circular void. The void was used in the connection of the pile to the cap beam as described in the Precast Cap Beam section.

As a result of unpredictable soil conditions, 48 of the 463 piles did not meet axial design capacity prior to reaching the design cut off elevation. These piles were then driven a maximum of an additional 25 ft and built up with a solid reinforced concrete section. Continuity between the pile and the buildup was developed through the installation of doweled reinforcing bars: details of the typical pile buildup are shown in Fig. 4.

24 in. Pile Bents
All substructure units located on land utilized 24 in. piles including Bents 69-76 which contain 11 piles per bent and supported 11 lines of AASHTO Type IV girders. Bents 1 and 77 and the associated seawall and wing walls also utilize 24 in. piles. The piles ranged in length from 50 ft to 90 ft and resist a maximum design bearing of 910 kips; a Pileco D-62 diesel hammer was used to install the piles. Originally designed to have a 10 1/2 in. diameter void, the contractor opted to use solid piles for reasons of economy. The prestressing was held constant at 16 - 0.5 in. diameter 270 ksi low relaxation strands initially stressed to 75 percent of ultimate. Due to the common size and location on land, the installation of these pile bents followed standard practices with minimal difficulty.

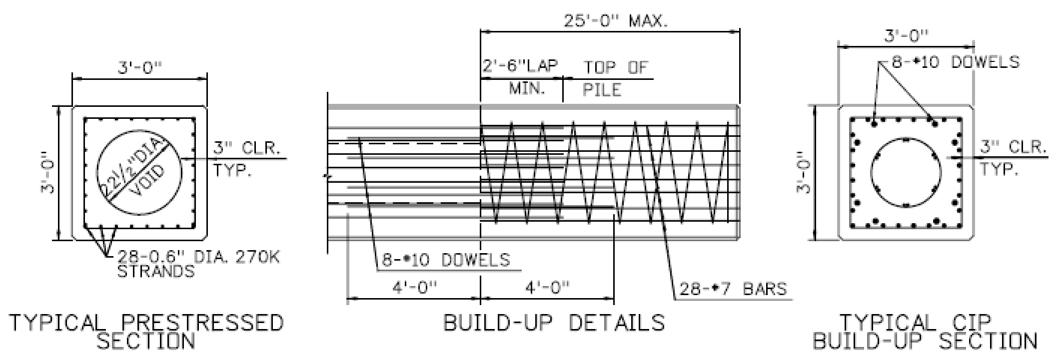


Fig. 4 Typical details for the 36 in. pile build up.

Waterline Footings

From Bents 27-50, the substructure units are waterline footings with 30 in. precast piles below the CIP footing, column and hammerhead cap above; the completed waterline footings at the navigation channel are shown in Fig. 5. The switch in substructure types over the middle 1/3 of the bridge was a result of the increased design loads for vessel impact. The RFP prescribed transverse vessel impact loadings ranged from 800 kips to a maximum of 2200 kips at the main channel. The switch in substructure type also accommodated the increasing superstructure height which provided a vertical clearance of 85 ft at the navigational channel between Bents 38 and 39. As stipulated in the RFP the vessel impact design requirements followed the AASHTO Guide Specification and Commentary for Vessel Collision Design of Highway Bridges[2].

The 436 - 30 in. piles supporting the waterline footings were driven using Pileco D-80 diesel hammers. For most bents, the piles were driven to meet the design requirements and then cut off; however, for a limited number of bents, the full length of the piles were used with the piles being driven to grade. The connection between the piles and the CIP waterline footing was made by embedding the piles 3 ft into the footing.

Fig. 5 Completed waterline footings, Bents 37-40

Bents 27-34 and 43-50 consisted of two individual waterline footing structures per bent, one supporting the eastbound roadway and the other the westbound. The remaining waterline footings were joined to gain the combined lateral resistance of all the piles located at the given bent. For all the waterline footings, a rectangular pile configuration was utilized with the number of piles ranging from eight to 28 per footing. Pile lengths ranged from 104 ft to a maximum of 153 ft and were designed to carry maximum design loads in excess of 1200 kips as previously described. Due to the variability of the soil, the footings were designed so additional piles could be added if the initial design bearing was not achieved. An example of the rectangular pile geometry is shown for Bent 36 in Fig. 6.

A symmetric 20 strand pattern was utilized for the 30 in. piles which were cast closed ended with a 16 1/2 in. diameter internal void. Similar to the 36 in. piles, the design was changed from 0.5 in. to 0.6 in. diameter strand to increase the pile pick lengths. A secondary result of the increased strand size was an increased lateral capacity of the piles for vessel impact loads. Reference Table 1 for the difference in lofting configurations given the two strand sizes pulled to the same pretensioning stress.

Stability of the bridge structure subjected to a design vessel impact loading was analyzed using FB Multipier[6] software. The design criteria allowed for an inelastic pile response in the region just below the footing; but the piles were to remain elastic below the mud line. Thus, the 30 in. piles were designed to develop plastic hinges in the region of the pile/footing interface as a means to dissipate the impact energies while maintaining resistance. To facilitate the development of the plastic hinge, the concrete core of the pile

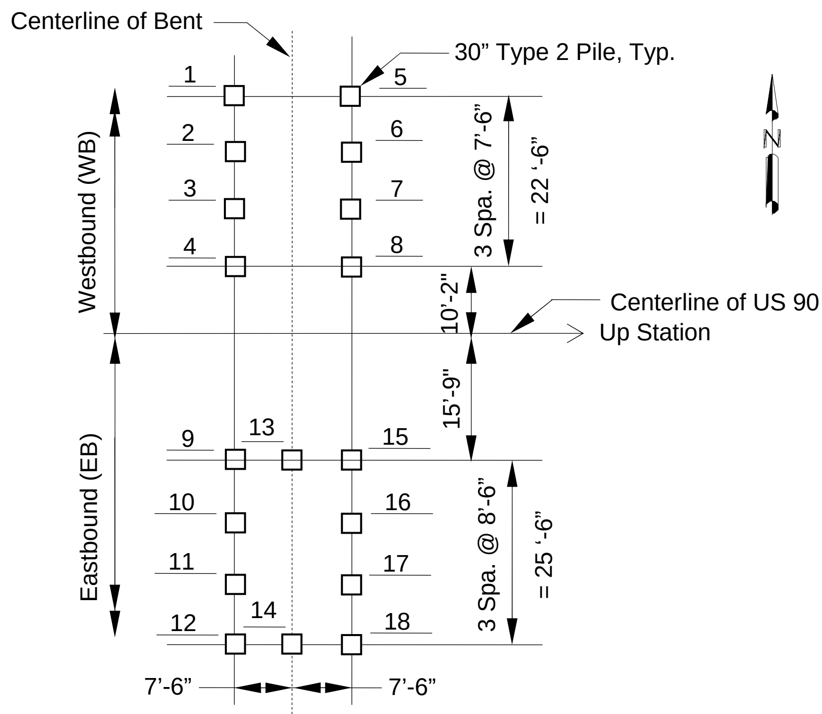


Fig. 6 Geometric layout for Bent 36 waterline footing

was confined by reducing the pitch of the W11 wire spiral to 2 in. over the top 30 ft of the pile. Since the pile tip elevations varied, a larger length of increased spiral reinforcement was detailed to ensure the presence of confining reinforcement in the plastic hinge zone.

PRECAST CAP BEAMS FOR PILE BENTS

One of the most significant items on the critical path for this project was the completion of 43 - 36 in. pile bents, all of which were located on the water. The initial design concept was to utilize precast elements for these cap beams. The cap beams were thus designed to be fabricated offsite by a precast supplier and then installed by the contractor. Due to equipment limitations, the 4 ft - 6 in. deep x 5 ft wide cap beam consisted of two pieces, weighing approximately 158 kips and 135 kips respectively. The individual pieces were barged to the bent location, placed atop the cut off 36 in. piles and joined with a closure pour. The closure joint was located between the sixth and seventh pile for the 11 pile bents and between the fifth and sixth pile for the nine pile bents. The joint corresponded to a location that required minimal flexural resistance, limiting the required connection reinforcement. The cap beam components were designed to withstand the loss of any pile within the bent for the vessel impact requirements.

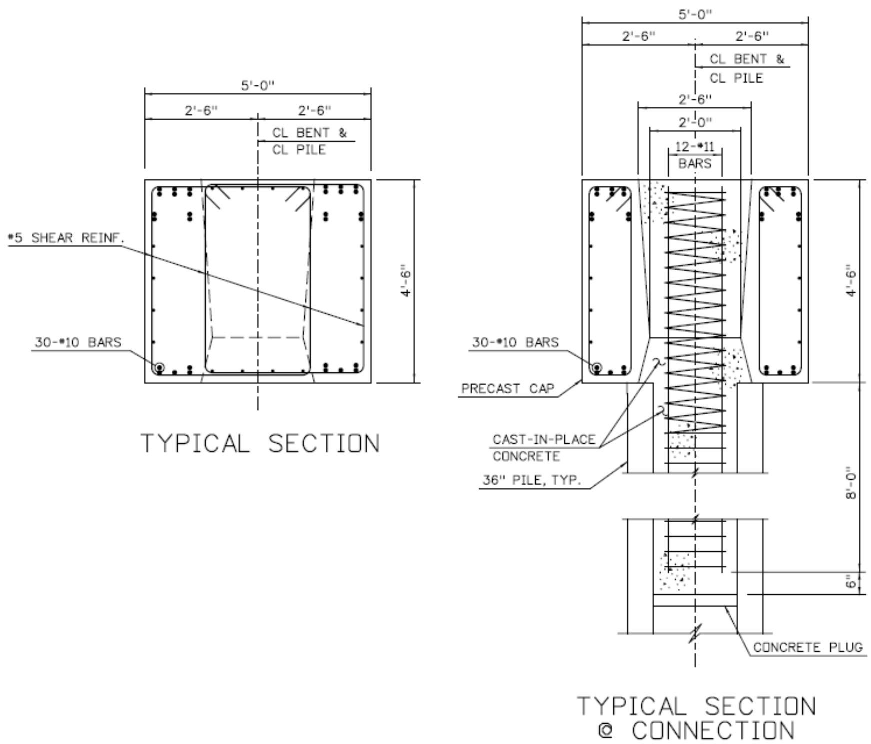
Over each pile location, a trapezoidal void was cast into the cap beam to allow for a connection to the piles. This connection was formed by placing a 12 ft - 7 in. long reinforcing cage (consisting of 12 - #11 bars confined by a #4 spiral set at a 4 in. pitch) into the voids of the cap beam and the matching pile voids. After a plug was placed inside the pile void, concrete was placed into the voids creating a moment bearing connection between the piles and the cap beam. The pile/cap connection details for a typical 11 pile bent are shown in Fig. 7.

The voids in the cap beam were designed to accommodate piles deviating +/- 6 in. from the design location and the cap beam reinforcement was also designed based on this tolerance. However, the long 36 in. piles proved challenging to drive within the given tolerances and the cap beam design was revisited. With piles located up to 10 in. from design, torsional effects on the cap beam resulting from the eccentric piles required both additional longitudinal and shear reinforcement. Due to the equipment constraints, the cross sectional area of the cap beam could not be increased, thus limiting the geometry of the precast cap beam. With the owner's approval, alternative caps were designed to provide additional torsion capacity, allowing for the continued use of precast bent caps. A comparison of the reinforcement between the regular cap and the cap reinforced for torsion is presented in Fig. 7.

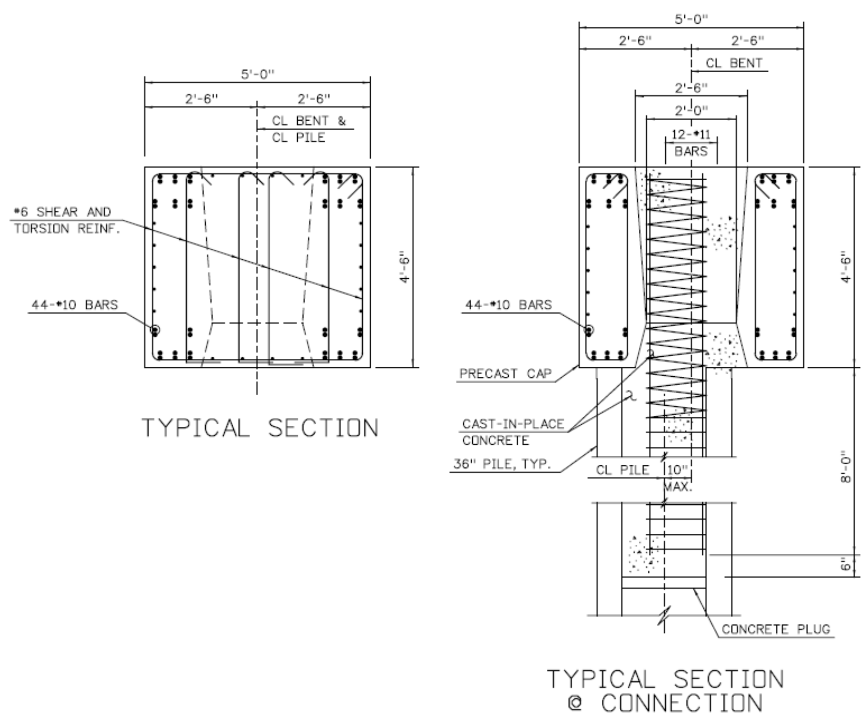
Of the 43 potential precast cap beams, eight were required to be CIP due to issues relating to the piles. At Bents 2 and 56, an interior pile was determined to be broken during driving and did not meet the design requirements; however, the adjacent piles possessed excess capacity. In this case the reinforcement within the cap beam was increased to bridge the broken pile requiring a CIP solution. A similar solution was applied at Bent 19 where an exterior pile did not meet the required bearing. The remaining five CIP bents were required due to the piles being out of tolerance in excess of 10 in. resulting in the cap geometry and reinforcing being increased. The design/build team worked to minimize the number of CIP caps since a precast cap could be installed in three days while the construction of a CIP cap required ten days, requiring a significant difference in onsite resources.

PRECAST WATERLINE FOOTING SOFFIT SLABS

As described, the waterline footings consist of 30 in. pile groups with a 6 ft thick CIP concrete footing. To construct the waterline footing elements, the contractor chose to utilize 9 in. thick precast soffit slabs with the same dimensions as the waterline footing. Block outs were placed at the pile locations resulting in oversized holes so the slab could be lowered over the respective pile group. Designed by the contractor's subconsultant, the slabs were treated as two-way slabs with additional reinforcing around the pile openings. The slabs were then supported from the piles by threaded rods and channel walers which rested atop the piles. Both the soffit slab and the supporting hardware were sacrificial and were considered to provide no structural benefit to the footing. Fabrication of a typical precast soffit slab is shown in Fig. 8.



a. Typical cap beam



b. Enhanced torsional resisting cap beam

Fig. 7 Connection and reinforcing details for the precast cap beams.

Fig. 8 Fabrication of a typical precast soffit slab.

The soffit slabs were fabricated by the contractor on land at the construction site and then barged to their respective location on the water. Prior to setting the soffit slab, the entire formwork system for the waterline footing was assembled, which included the slab, the supporting hardware, steel side forms and bracing. Once prepared, this footing form was lowered over the piles into the water and suspended at the design elevation. The holes around the piles were then sealed with concrete, the salt water pumped out and reinforcing set (see Fig. 9). After placement, the footing concrete cured for 30 days prior to the removal of the steel side forms as per the owner's requirements.

Fig. 9 Waterline footing at Bent 42 being prepared for concrete placement.

PRECAST WATERLINE FOOTING STRUTS

For Bents 35-37 and 40-42, the vessel impact loads in the transverse direction of the bridge are of such magnitude that the engagement of all the piles within the bent is necessary to maintain structural integrity. Preliminary design called for the footing at these bents to be continuous between the columns. However, the pile layout left a gap in excess of 22 ft between the eastbound and westbound pile groups. This posed a constructability issue in supporting the formwork for casting the monolithic footing. At the contractor's request, the designer reviewed the feasibility of replacing the middle portion of the footing that bridged the eastbound and westbound pile groups with two struts to transfer the impact force to all piles within the bent. Additional analysis confirmed the ability of the struts to transfer the transverse impact forces to all piles. Per the vessel impact design criteria, the longitudinal impact force is half that of the transverse and for these bents, each individual pile group satisfied the longitudinal design requirements. As a means to speed construction, precast struts were designed and detailed. The struts, 3 ft x 3 ft x 22 ft 3 in. for Bents 35, 36, 41, and 42 and 4 ft x 3 ft x 21 ft - 6 in. for Bents 37 and 40, were fabricated off site, barged to the respective bent, and then set into the preformed block outs in the footings. To ensure a complete load transfer between the footings, the struts were pinned into place with #6 dowels and the block outs grouted. Fig. 10 shows the installed struts joining the eastbound and westbound pile groups at Bent 37.

Fig. 10 Precast struts joining the individual footings at Bent 37

PRECAST/PRESTRESSED SUPERSTRUCTURE

The superstructure is 101 ft - 4 in. out-to-out and is typically supported by nine girders spaced at 11 ft - 5 in. Beam design was facilitated through the application of the CONSPAN[7] analysis and design software. The majority of the girders are BT-78 beams ($f'_c = 8500$ psi) and the typical span length is 154 ft. The units are either three or four spans made continuous for live load. On the east end of the bridge where the new alignment curves to match the existing US-90 alignment the typical span length is 110 ft and AASHTO Type IV girders are used.

BT-78 GIRDERS

To provide for the most efficient and timely construction of the bridge superstructure, BT-78 girders were utilized extensively. The typical span lengths for Spans 5-36 and Spans 40-62 are approximately 154 ft and 151 ft, respectively. The span lengths were maximized so as to minimize the number of substructure elements. Spans 1-4 also utilized the BT-78 girders, but the lengths varied from 132 ft to 145 ft to avoid the existing foundations since the current alignment ties back into the previous bridge alignment. Due to the size of the girders, installation required the adaptation of specialized spreader beams and rigging. Fig. 11 provides a typical view of the rigging and spreader beam used to lift a 154 ft girder into position.

Based on feedback from the precast suppliers, the BT-78 girders were detailed with straight strands only, eliminating the costs associated with harping strands. To minimize the number of strands, a more rigorous analysis was conducted to take advantage of AASHTO code provisions allowing for a reduced live load distribution factor. Consequently a 3D linear finite element model was developed with results supporting the reduction of the live load distribution factor from S/11 to S/13.

Since the beams were also designed and detailed to be continuous for live load, the continuity moment induced by the long-term creep of the structure was taken into account. To determine the amount of additional positive moment to included in the design, a time dependent creep analysis was conducted using HNTB's T-187 program. The results were influenced by the age of the beams at the point of establishing continuity. Based on discussions with GAW, a minimum time of 60 days from casting was determined as the time continuity would be established. The resulting moment due to creep was superimposed with the CONSPAN output to determine the final design stresses. For a majority of the BT-78 beams, 60 - 0.6 in. diameter, 270 ksi low relaxation strands, which coincided with the maximum strand capacity of the precasting beds, were required to resist the aggregate actions of the dead load, live load and creep.

Fig. 11 Rigging configuration of typical 154 ft BT-78

The combined effects of the high concrete strengths, long spans and large amount of prestressing resulted in the need to monitor the cambers of the beams up to the time of erection. A majority of the beams were within tolerance for the design camber. In some instances, however, the haunch tolerances were exceeded, resulting in the adjustment of the pedestal elevations.

Due to the large number of beams required over the short time frame, multiple precast suppliers were used. The BT-78 girders were fabricated in Tampa Bay, FL, Savannah, GA, Birmingham, AL, and Memphis, TN and then barged to the site. This required additional coordination and effort by the design/build team to maintain quality control and remain on schedule.

AASHTO TYPE IV GIRDERS

At the east end of the bridge, where the typical span lengths were approximately 110 ft, AASHTO Type IV girders (f'_c = 8500 psi) were the most economical section. These girders were fabricated in Hattiesburg, MS and trucked to the site. Unlike the BT-78s the strands for the Type IV girder were harped to gain the necessary structural resistance in the 100 ft plus spans. One interesting design issue was the skew utilized at the crossing of 3rd Avenue at the east end of the bridge. In order to keep the girder design standardized, one span was used to transition from a normal bent to a 35 degree skewed bent. This resulted in the girders within Span 74 varying from 45 ft to 105 ft. Since the girders were considerably shorter in Span 74, the number of strands was consequently reduced from a maximum of 36 - 0.6 in. diameter, 270 ksi strand to only 14.

NAVIGATIONAL SPAN

To provide the required horizontal and vertical clearance at the navigation channel, a three span continuous haunch girder unit was utilized. This main unit consists of five spliced girders segments per girder line which are post tensioned together with four, 19 strand tendons. Each girder line is made up of two 116 ft - 6 in. haunch segments, two 141 ft - 9 in. end segments and a 133 ft - 6 in. drop-in segment. The end segments and drop-in segments are modified BT-78 girders (the web is thickened by two inches to provide clearance for the post-tensioning ducts) while the haunch segments have a varying section depth from 6 ft - 6 in. at the ends, linearly increasing to 12 ft at the support. When complete, the navigational unit consists of two 200 ft end spans with a 250 ft main span. The profile of navigational span is shown in Fig. 12.

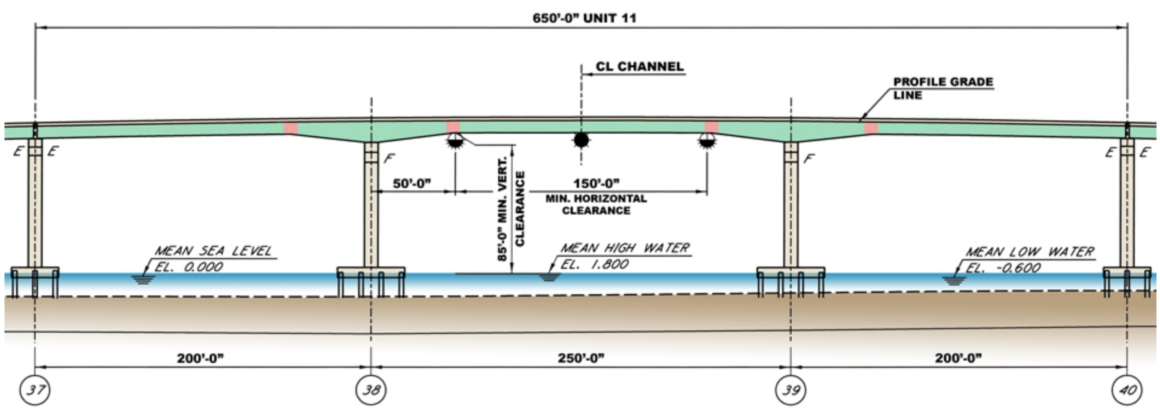


Fig. 12 Navigational unit profile

The staged erection of the spliced segments was completed in the following sequence: The haunch segments were placed atop the respective bent and anchored using a temporary shoring tower that was supported on the footing as shown in Fig. 13. The shoring towers were positioned away from the navigational span so as to support the end segments as well. Once secured, a diaphragm was placed at the bent and the haunch segments post tensioned together in the transverse direction. The end segments were then set into position, being supported by steel strong backs which are clamped to the end of the segment at the closure joint and the shoring towers. The end segments acted as a counter weight to the drop-in sections which were placed last and are also supported by strong backs.

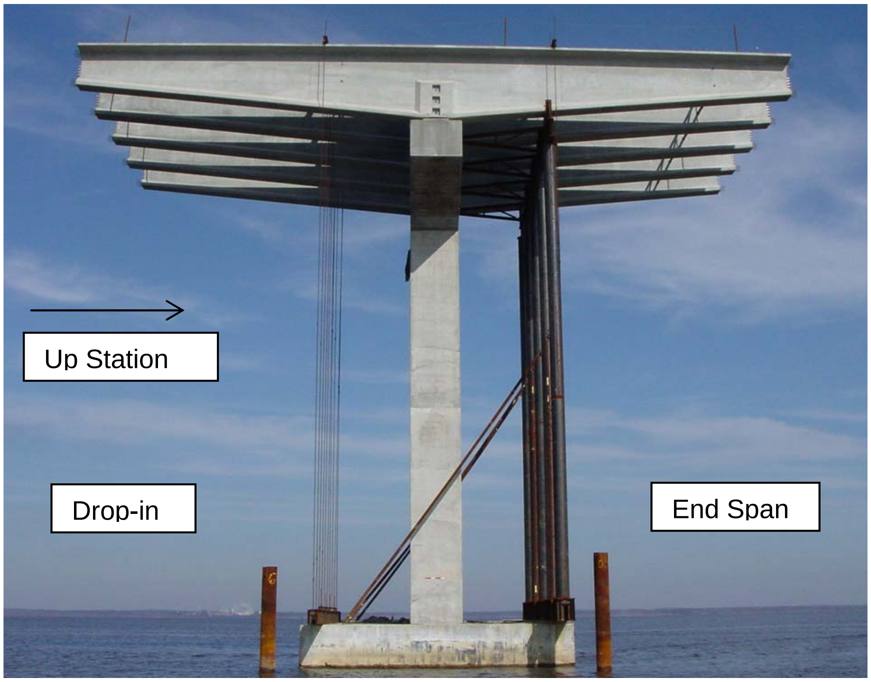


Fig. 13 Haunch girder segments supported by temporary shoring towers

After the end diaphragms and closure pours were made, two of the four tendons that join the segmental girder were fully stressed and grouted. The shoring towers were removed and the deck cast, creating a continuous, composite section. After the deck reached the design strength, the remaining two tendons were stressed and grouted completing the navigational span.

SUMMARY

In short, this project would not have been feasible in the given time frame and budget without the full exploitation of precast and prestressed technologies. This became evident to the design/build team when reviewing alternative sections and materials that standard P/S beams which could be manufactured at various locations would be the only reasonable option. Steel and segmental designs either required too much time to fabricate and construct or the materials were cost prohibitive. Likewise, local precast suppliers were already tooled up to produce these common beam elements including the haunch girder segments.
Due to the prevailing soil conditions, P/S piles were the most reasonable solution to provide a firm foundation in the soft sedimentary deposits taking advantage of both side friction and end bearing. Completing the substructure as quickly as possible was crucial to the success of the project as the critical path followed the pile bents. As noted, the use of precast cap beam elements played an important role in reducing the amount of time spent in the field completing these units. Likewise, the design/build team implemented

precast technology to save both time and money with the development of the struts to transfer vessel impact and to form the waterline footings.

By utilizing precast and prestressed components, GAW took a project that would normally span three to four years and compressed the time frame to only 18 months. The result of this effort was the replacement of a critical link on the Mississippi Gulf Coast, providing a much needed point of progress for the rebuilding communities.

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The opinions, findings, and conclusions expressed in this publication are those of the authors and not necessarily those of the Mississippi Department of Transportation, the joint venture partners, Granite Construction Corporation and Archer Western Construction, the designer HNTB Corporation, or any sub-consultants or subcontractors.

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