HURRICANE DECK BRIDGE REPLACEMENT OVER THE LAKE OF THE OZARKS

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BIOGRAPHY

Martin Furrer is a Senior Project Manager with Parsons in the Chicago office and he is the Lead Bridge Design Engineer for the Hurricane Deck Bridge Project. He has 18 years of experience in the management, design, analysis, load rating and construction of complex bridges including fifteen bridges over major waterways. Mr. Furrer led the design of the of the North America's longest freestanding arch bridge in Hastings, Minnesota and the 1000 ft long Christopher S. Bond Cable-Stayed Bridge on Kansas City. Mr. Furrer is registered PE and SE and has a M.S. in Structural Eng. from the Federal Institute of Tech., Zurich Switzerland.

Pamela Yuen is a Senior Bridge Engineer with Parsons in the Chicago office and was heavily involved in the design of the Hurricane Deck Bridge Project. She has over 9 years of experience working on complex bridge projects. Ms. Yuen graduated from University of Illinois at Urbana-Champaign and is a registered PE.

Scott Gammon is Vice President of American Bridge Company headquartered in Pittsburgh, Penn. His extensive bridge construction resume includes numerous major river and lake crossings and many different structure types. Mr. Gammon holds B.S. and M.S. degrees in Civil Eng. from the University of Missouri-Columbia. He is a registered PE in multiple states and holds certification as a designated design-build professional by the DBIA.

Dennis Heckman is a State Bridge Engineer for the MoDOT since 2007. Mr. Heckman holds his B.S. degree in Civil Engineering from the University of Missouri – Columbia. He is a registered PE and is also a member of the AASHTO Subcommittee on Bridges and Structures. He serves on two technical committees; T-1 Security and T-3 Seismic Design (vice chair).

SUMMARY

The Hurricane Deck Bridge carries Missouri Route 5 over the Osage Arm of the Lake of the Ozarks in Camden County, MO. Built in 1936, the existing bridge is 2,200-foot-long steel deck truss structure with 463-ft spans and is supported on dredged caissons in up to 85 ft. of water. This bridge was selected for replacement by the Missouri DOT due to its functional obsolescence and structural deficiencies. Parsons was selected to develop a baseline design that consists of building a new steel delta frame structure immediately adjacent to the existing bridge and supported on temporary piers at locations directly adjacent to the existing piers. The baseline concept envisioned that traffic is maintained on the existing bridge while the new bridge is built on an offset alignment on temporary footings, to switch traffic to the new bridge while the existing truss and pier caps are demolished and slide the new superstructure onto the rehabilitated permanent piers using a weekend closure.

To encourage contractor innovation, MoDOT elected to employ an Alternate Technical Concept (ATC) procurement process allowing contractors to develop alternate approaches to the baseline design with the intent of reducing costs, provided all the projects objectives were satisfied. American Bridge Company proposed the low bid ATC that consists of an easterly offset alignment through the project limits where a new conventional parallel flange plate girder bridge was constructed. Once the new structure was complete, roadway connections were constructed, traffic moved to the new alignment and the old structure demolished.

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Introduction

For the Hurricane Deck Bridge replacement the owner elected to employ an Alternate Technical Concept (ATC) procurement process to replace the existing 2,200-foot-long steel deck truss with 463-ft spans over the Osage Arm of the Lake of the Ozarks in Camden County, MO. This paper discusses the development of the delta frame baseline and the winning ATC design, the unique contracting approach and the construction of this bridge with piers in up to 85 feet of water.

Project Constraints

The existing Hurricane Deck Bridge, built in 1936, is a 2,200-foot-long steel deck truss structure with 463-foot spans supported on dredged caissons in up to 85-feet of water. The bridge carries MO Route 5 over the Osage arm of the Lake of the Ozarks in Camden County near the town of Sunrise Beach. It is a critical link in a region which is one of Missouri's most significant recreation and tourism destinations.

The project site presented several challenges, including significant Right of Way (ROW) restrictions, rock bluff constraints in the approach roadway, and environmental concerns which included nearby Native American burial grounds. Closure of the bridge for the duration of the reconstruction was also deemed unacceptable by the local stakeholders due to the 42 mile detour. All of these constraints were considered during the preliminary design phase in order to develop a solution which minimized environmental impacts.

Project Contracting Approach

Parsons was selected to provide the preliminary and final design services for the baseline design concept for the Hurricane Deck Bridge replacement. To encourage contractor innovation, the Missouri Department of Transportation (MoDOT) elected to employ an Alternate Technical Concept (ATC) procurement method for the project. Under the ATC process, contractors were invited to develop alternatives to, or modifications of, the baseline design with the intent of reducing costs without sacrificing MoDOT's project defined objectives.

MoDOT ATC's typically involve relatively minor

Figure 1 – Project Location

modifications to the baseline concept. In these cases, an approved ATC is taken to final design and incorporated into a bid package that is prepared exclusively for the contractor that has submitted the ATC. For the Hurricane Deck Bridge project, two contractors submitted ATC's that represented significant departures from the baseline concept. The extensive nature of these two ATC's rendered design cost and the required resources prohibitive to performing a complete final design prior to the bid opening. In consideration of this limitation, these two contractors collaborated with MoDOT and Parsons to develop a conceptual design focused on defining the variables most crucial to the development of a detailed cost estimate and bid price for the project. Ultimately, these two ATC designs were advanced to only 30% completion prior to bid submittal, and pre-bid engineering deliverables were minimized.

Figure 2 – Existing Hurricane Deck Bridge

In a nine month period, Parsons mobilized four teams of designers, including one to perform preliminary design and prepare bid documents for the baseline approach and three design teams to prepare bid packages for the confidential ATCs proposed by the contractors. These design teams were staffed from different offices and administrative firewalls were designed to ensure complete confidentiality throughout the bid document preparation process.

Five contractor bids were received on this project, two contractors bid the baseline design with no modifications, one elected to bid the baseline design with minor ATCs proposed, and two proposed major ATCs to the baseline design. American Bridge Company's (AB) proposed major ATC was the lowest bid at $32,303,295 followed by the contractor that bid the baseline design with a minor ATC at $45,765 (0.1%) higher.

While the project delivery method was conventional Design-Bid-Build, the conceptual design process utilized for bidding the condensed construction schedule resulted in an execution strategy resembling a fully integrated Design-Build project delivery. Upon contract award, the project team collaborated to parse the project into individual design packages, allowing construction and design to advance concurrently. Six individual design packages were defined including: drilled shafts, substructure, steel plate girders, superstructure, approach spans, and roadway approaches. To facilitate communication and collaboration between all parties, weekly design meetings were conducted and an over-the-shoulder review process was employed. Formal reviews were completed by MoDOT at 50% and 100% progress sets. Upon resolution of comments for each package, a Released for Construction (RFC) package was produced, allowing construction to begin on the work contained in the package. Parsons released the drilled shafts package within 2 months from notice to proceed to allow AB to meet their aggressive construction schedule. Final plans for AB's full ATC were complete in just six months. As a further testimony to the success of the collaboration between MoDOT, Parsons, and AB, final construction quantities were ultimately within 2% of the bid quantities developed from the 30% preliminary design, and the net cost to AB attributable to design growth was less than $40,000.

Baseline Design

With considerations of environmental constraints, limited ROW, and long detour route, an innovative proposal was developed as the baseline concept for the Hurricane Deck Bridge Replacement. This involved the construction of a new steel delta frame plate girder structure. The initial planning for the baseline design does not considered the offset alignment similar to the AB ATC due to the proximity of the existing bridge, only two feet of clear distance between the existing and proposed structure. In addition, such new alignment would require new foundations.

The new steel rigid delta frame structure proposed in the baseline design was to be built immediately adjacent to the existing bridge and was supported on temporary piers at the same locations as the existing piers. The new bridge was intended to be used on these temporary foundations as a detour route. Traffic was to be maintained on the existing bridge while the new bridge was built on the offset alignment on temporary footings. When construction of the new superstructure was complete, traffic would have then switched to the new bridge on the temporary alignment. The existing bridge superstructure and pier caps were then to be demolished. Once the new pier caps were complete, the new superstructure would be moved laterally onto the rehabilitated permanent piers during a weekend closure. Once the new superstructure was moved, all of the temporary elements would then be demolished.

With the baseline design's planned reuse of the existing dredged caisson piers, a thorough underwater inspection, coring program and extensive material testing were carried out. This investigation detected little deterioration and determined the substructures of the existing bridge would be suitable for reuse.

The triangular shaped delta frame, extending from the pier cap up to the bridge girders, was proposed for this project due to its ability to support long spans at a significant height with few piers. With the use of the delta frame, original bearing elevations were maintained thus minimizing any necessary retrofit to the existing substructure. The design team designed this element using a mix of modeling and intricate calculations. Special attention was paid to detailing at the load transfer points or knuckles. At the top knuckles, i.e. the intersection of the inclined and horizontal girders, the inclined legs were bolted to the bottom of the plate girders by an end plate and inclined stiffeners to transfer the loads. For the bottom knuckles, the legs converged on bearings embedded in the substructure and an extensive array of stiffeners was used to transfer loads to both the final bearings as well as to the temporary jacking assembly that was required to move the delta frame from the temporary substructure onto the existing piers. The delta frames were detailed to allow assembly on a staging area at the shore of the lake, floating them on a barge onto the lake and setting them down on the temporary substructure next to the existing bridge piers.

Figure 3 shows the span layout with a typical span of 462'-10" matching the existing deck truss spans. The delta legs connected to the girder at 115-feet from the centerline of the pier. Three delta frame girders spaced at 13'-2" were used to support the 40'-8" roadway cross section which consists of one 12-foot lane in each direction with 5' shoulders and MoDOT Type B Safety Barrier Curbs supported on a 9 ½" concrete deck cast on stay-in-place steel forms (Figure 4).

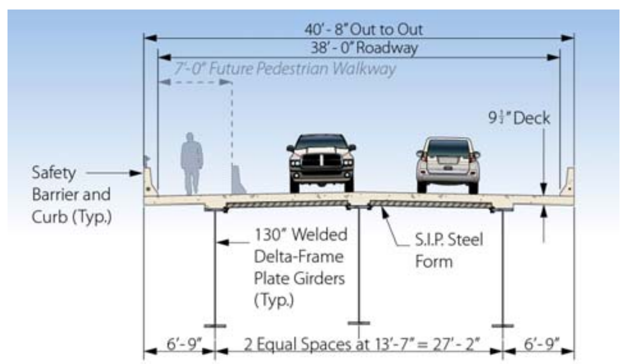

Figure 4 – Cross Section Baseline Hurricane Deck Bridge

The delta frame welded plate girders were 130 inches deep and the frame legs had a typical depth

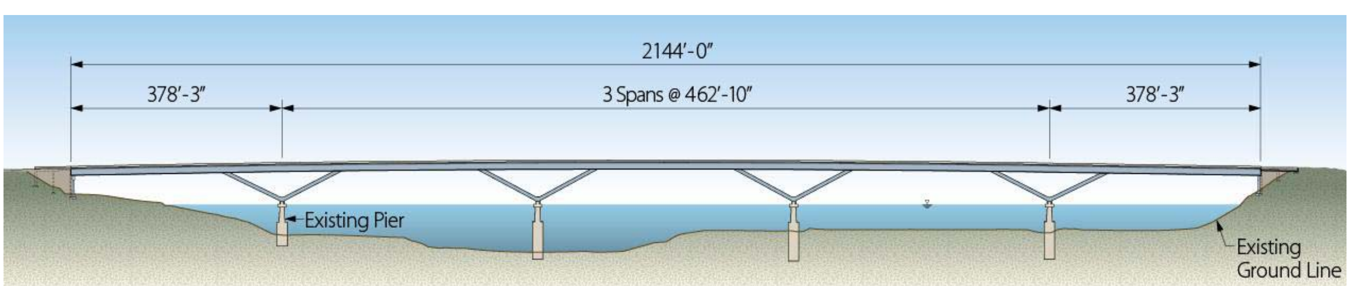

Figure 3 – Baseline Design Delta Frame Elevation

of 48 inches. All structural steel was designed to be unpainted ATSM A709 Grade 50W weathering steel. Grade 70W steel was not anticipated to be more economical because the design is controlled by deflection.

While the existing piers were found to be in good condition, structural analysis indicated that they would not provide sufficient lateral capacity in the longitudinal bridge direction to handle the relatively large kicking forces generated by the much higher flexibility of the delta frame when compared to the existing deck truss. It was necessary to release the longitudinal restraint at the top of each pier and detail the delta frame superstructure to substructure connections with guided bearings to allow free longitudinal movement. The longitudinal loads were distributed back to the south (right) abutment where the superstructure was tied into surface bedrock via a series of rock anchors.

Figure 5 – Preliminary Rendering of Baseline Hurricane Deck Bridge

Extensive temporary works and detailing were anticipated on this project because the new delta frame structure was supported on temporary piers at directly adjacent to the existing piers. In order to reduce contractor procurement duration, Parsons fully developed the temporary work details, including temporary substructure and temporary bracing that tied the temporary piers to existing piers.

American Bridge Company (AB) – Alternate Technical Concept (ATC) Design

The American Bridge Company ATC involved a total redesign of the baseline concept with a new permanent structure on a new parallel alignment just east of the existing deck truss. The total steel weight for the AB ATC design is 4,207,290 pounds when compared to 8,588,360 pounds of steel for the base design. The proposed ATC alignment provided just two feet of clear distance between the existing and proposed structure when measured within the deck truss unit of the existing bridge. The new structure is comprised of two plate girder units with six typical spans of 265 feet, 210-foot end spans and an in-span hinge connecting the two units. The steel superstructure is founded on twin drilled shafts. A two span precast girder unit at the north and a single precast girder unit at the south end complete the crossing. The precast girder land structures are founded on a combination of driven piling and spread footing foundations (Figure 6).

The bridge cross-section for the AB ATC is the same as the typical section developed in the baseline design. Instead of proposing a 3-girder bridge as in the baseline design, a 4-girder bridge was used to reduce the deck thickness (8 ½”), reinforcement, forming, and future replacement (Figure 7).

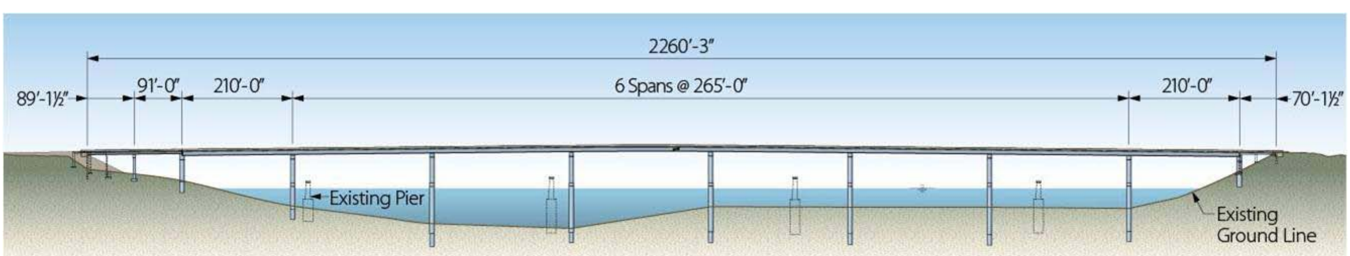

Figure 6 –AB ATC Hurricane Deck Bridge Elevation

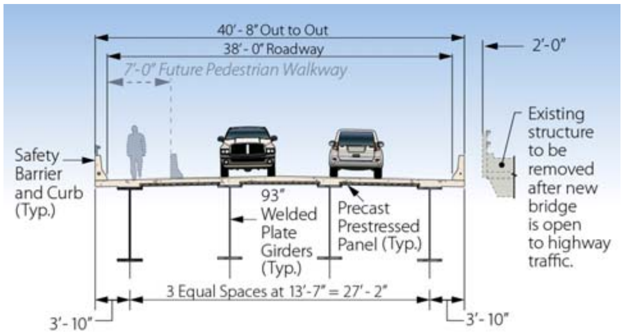

Figure 7 – AB ATC Hurricane Deck Cross Section

Substructure Design

Several unique features on the project posed design and detailing challenges, particularly within the steel units. The steel units were founded on drilled shafts where the diameter used for the intermediate piers was 8.5-feet, with lengths of up to 120-feet. These intermediate piers also contained an 8-foot diameter barbell strut that is 8-foot deep and it sits between the drilled shaft and the 8-foot diameter column. Also included was a 6-foot deep pier cap (Figure 8).

In order to reduce the longitudinal flexural demand on the piers, a unique structural approach was developed whereby the steel structure was divided into two structural units with an in-span hinge. With the use of this hinge detail, the longitudinal loads were significantly lowered and an optimal strategy for pier fixity and thermal expansion joint locations could be developed. It was determined that only one expansion joint in the entire steel span would be required. The expansion joint was located away from the pier location, which would simplify the pier details and durability. In addition, a pinned connection between the superstructure and the pier was developed such that all of the intermediate piers share similar details and design. Longitudinal forces were distributed back to the first land pier location on each side of the lake where it was more efficiently resolved. Due to the fact that the land piers were shorter than the intermediate piers, they were much stiffer and were treated as the anchor piers. The land piers were designed to brace the other intermediate piers for stability where unit 2 is tied to the north end pier (Pier 3) and unit 3 to the south end pier (Pier 11) as illustrated in Figure 9. Most of the longitudinal loads were resisted in bending of the anchor piers which requiring a 9 foot diameter drilled shafts.

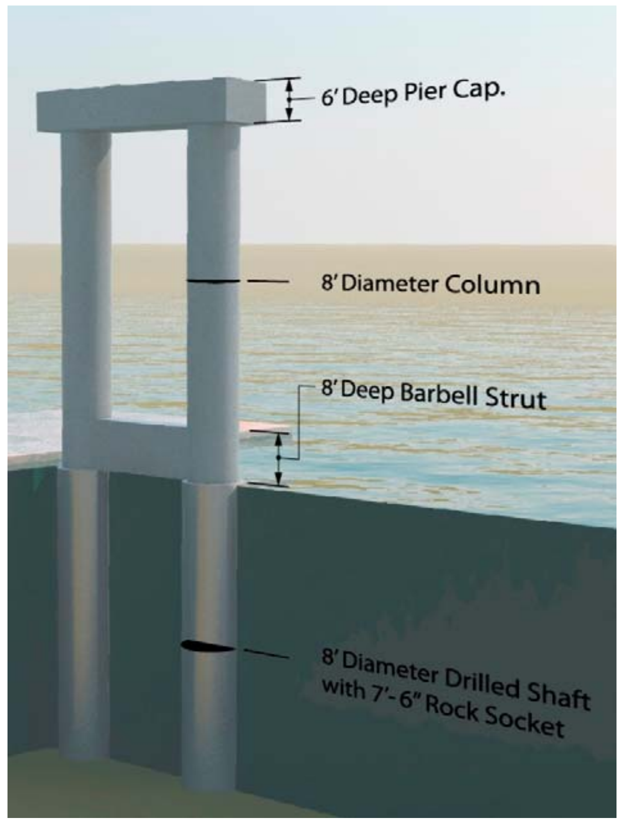

Figure 8 – AB ATC Hurricane Deck Pier

AASHTO (1) design code permits slenderness effects to be ignored when the slenderness ratio is less than 22 for members not braced against

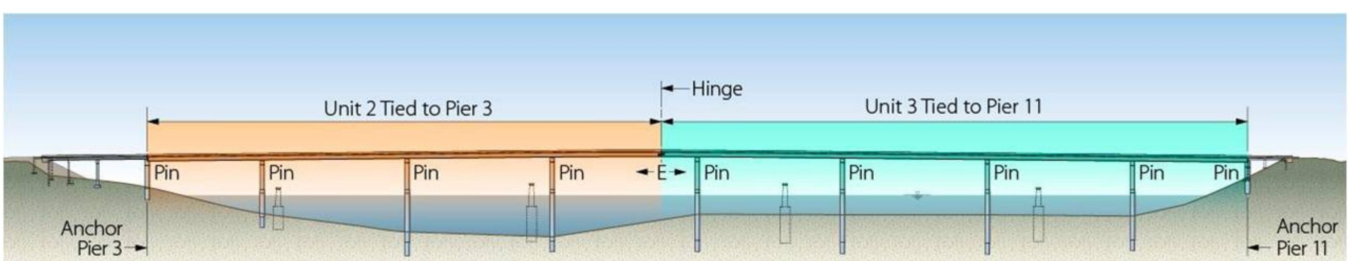

Figure 9 – AB ATC Hurricane Deck Bridge Steel Units Fixity Concept

sideway. Due to the geometry and section of the piers, slenderness effects needed to be considered in the design. The intermediate piers when acting as unit sway globally but each individual pier does not sway locally. For the design, both global and local effects were investigated in the longitudinal direction. In computing the amplification factor that was applied to the moment, which was the end result of the slenderness effect, the following equation (AASHTO Eqn. 4.5.3.2.2b) was used:

$$M_c = \delta_b M_{2b} + \delta_s M_{2s}$$

in which:

$$\delta_b = \frac{C_m}{1 - \frac{P_u}{\Phi_K P_e}} \geq 1.0 \qquad \delta_s = \frac{1}{1 - \underbrace{\frac{\Sigma P_u}{\Phi_K \Sigma P_e}}_{1/\alpha}}$$

Due to the height of the piers, the piers were also designed to accommodate a construction tolerance for the overall plumbness of one percent, which was set in concurrence with AB. In addition to the consideration mentioned above, due to the fact that the anchor piers served as the bracing support for all other intermediate piers and to ensure global stability of the structure, the anchor piers were required to take addition longitudinal force generated in the superstructure from the secondary bracing effects.

In order to obtain the longitudinal global stability factor (α), a detailed three-dimensional finite element model (FEM) was constructed using the LARSA 4D program to investigate global buckling effect on the structure. In the model, linear elastic line elements were created for the entire steel units in the FEM model. The model also included deck and girders modeled as single beam elements. Loads from the superstructure were then distributed through a rigid transverse beam to the pier cap. Short beam elements were modeled to connect the gap between the rigid transverse beam and the top of the pier caps. These beam elements also simulated the fixed bearings. An expansion joint was included in the model as a hinge connection and all piers were fixed at the bottom at a depth recommended by the geotechnical engineer. The model contained 708 members and 721 joints. Using the same model, vertical, lateral, and longitudinal (including temperature, braking, and wind) forces were applied and results were obtained to perform the substructure design.

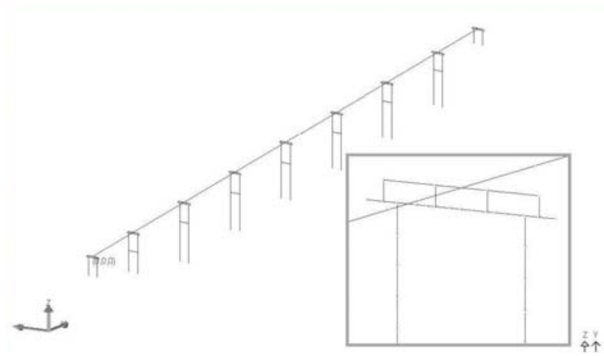

Figure 10 – 3D FEM Model on Hurricane Deck Bridge Steel Units

A non-linear buckling analysis was performed using the FEM model to develop P-Delta curve (Figure 11). This analysis utilized a step-by-step load increment approach where uniform vertical loads applied at the top of each pier were incrementally increased to capture the maximum load capacity the structure can sustained. The pier stiffness was reduced to 0.40EI to simulate cracked concrete in the model.

From the P-Delta curve, the bifurcation point was at around 50,000-kips and conservatively, the design used the buckling load of 40,000-kips as the overall critical vertical load for stability. The global stability factor was then obtained by dividing the buckling load by the total factored axial load on the piers.

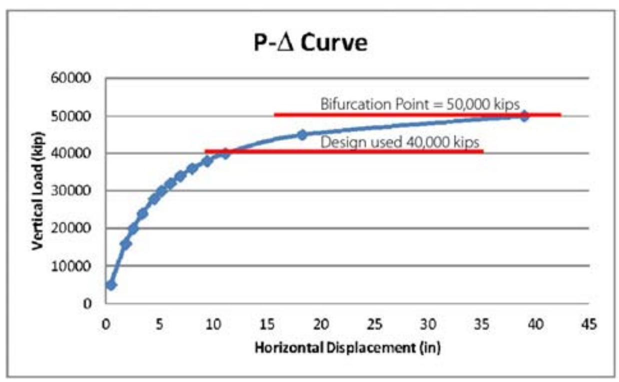

Figure 11 – P-Delta (Δ) Curve from 3-D FEM Model

For local stability in the longitudinal direction, the anchor pier works as a bracing support to the intermediate piers in each of the steel units; therefore, the anchor pier is governed by global stability. Local stability of the anchor piers was not a concern due to its short length. For all other piers, the local stability factor was calculated as Pe/Pu, where Pe was the Euler buckling load and Pu was the ultimate factor load of the pier. An effective length factor, K = 0.80 was used. The moment magnification factors on global and local in the longitudinal direction were then compared and it was determined that global governs the design.

In the transverse direction, lateral stability of the piers was highly dependent on the flexural stiffness of the struts and/or pier caps. Since the effective length factor, K, is a function of the total flexural restraint provided by either the strut or pier cap at the ends of the column, when considering the flexural stiffness of the system, the stiffness of the cap or strut and the column were considered. Figure C4.6.2.5-2 in AASHTO graphically presents the relationship among the effective length factor (K) and the ratio of stiffness of column to stiffness of members resisting column bending at two ends of the column under consideration (Ga and Gb) (Figure 12). By obtaining the effective length factor, stability factor, Pe/Pu can be calculated and moment magnification factor in the transverse direction were then obtained and used for the design.

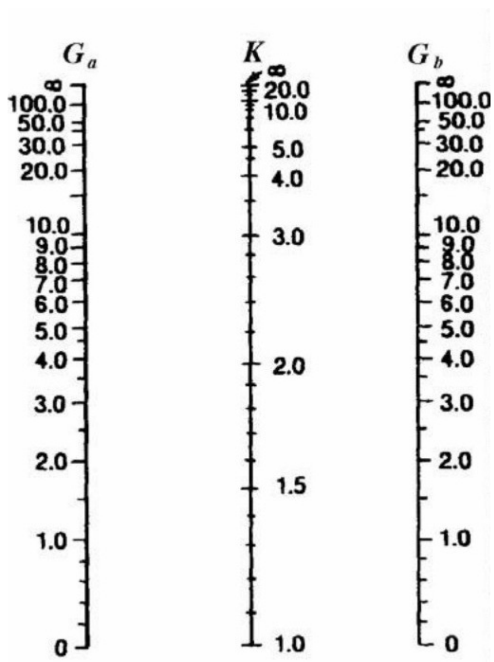

Figure 12 – Alignment Chart for K value (AASHTO, Figure C4.6.2.5-2)

Load combinations of the piers were performed based on AASHTO LRFD. The bracing force on the anchor piers were factored using the factor for dead loads. The moment magnification factors were applied to all factored load effects except thermal force, where the forces counteracted each other in the frame system and did not cause stability concern.

Superstructure Design

Parallel flange plate girders with a 93-inch web depth were utilized as they were found to optimize the proposed span arrangement. Parsons utilized the Merlin Dash steel girder design program together with a series of in-house proprietary design tools to optimize the steel girder design. The unique ATC contracting arrangement allowed for flange plate sizes to be optimized directly with AB's selected steel fabricator. Unpainted ASTM A709 Grade 50W weathering steel was used throughout the project with localized painting at the joints between the structure units.

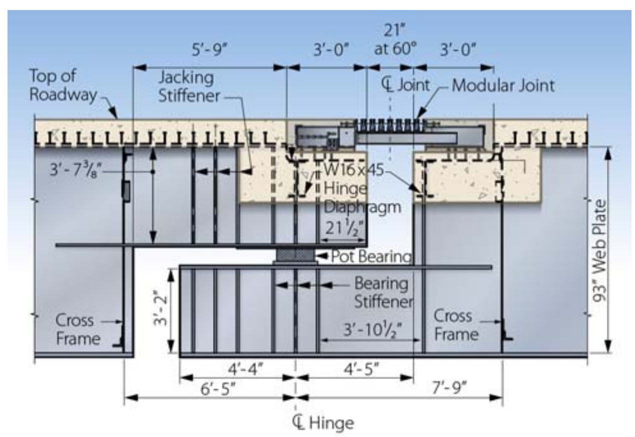

Figure 13 – In-Span Hinge

The slender substructure design concept and the desire to minimize the number of piers necessitated the use of an in-span hinge. Modifications were made to MoDOT standard in-span hinge details for this large scale hinge to optimize load paths and provide for improved fatigue detailing. Sufficient support lengths were developed to accommodate the Seismic Category A movement. Special attention was given to accommodate the selected modular expansion joint to maximize joint durability and that the shear capacity was sufficient with the reduced web depth.

In addition to the typical cross-frame and intermediate diaphragm needed, temporary lower lateral bracing was required to be installed between the exterior and first interior girder during construction (outer bay) prior to development of design strength in the cast-in-place concrete bridge deck. The lateral bracing was designed to resist lateral force mainly due to wind load. Connecting the lateral bracing directly to the flange was preferred as it eliminated the need for connecting elements to the girder web that can be sensitive to fatigue issues. In conjunction with AB, a removable and reusable bracing system was developed using post-tensioning rods attached to steel brackets mounted directly to the bottom of the bottom flange ensuring a direct load path and stability of the bridge system during construction.

Figure 14 – Lower Lateral Bracing

Construction

As a result of AB's major ATC, final design was performed after project award. One of the MoDOT constraints on ATC approval was that the project had to be open to traffic not later than the date established in the base design, thus the project team had to compress both design and construction of the ATC into the same schedule allotted by MoDOT just for construction of the baseline design. This required an accelerated start to the project. The project was awarded to AB on January 4th, 2012. A design kickoff meeting was conducted just two days later on January 6th, and by January 9th AB had mobilized marine equipment to the site to support the Parsons geotechnical exploration program along the proposed ATC alignment. Furthermore, individual package productions were scheduled carefully so that the RFC drawings were available in time to begin each successive work activity. This design-build style integrated project management approach facilitated successful design and construction in less than the time provided just for construction of the baseline bid.

Drilled shaft installation commenced just a few months after award in May 2012. Drilled shaft construction on the project was complicated by a number of factors, including the large diameters (up to 9feet), the long lengths (in excess of 120-feet), and lake depths of up to 85-feet. Further complicating the shaft construction was the presence of very thin soil overburden which in some locations was less than 10-feet. Conventionally, drilling templates are constructed and pinned in the overburden with spud piles. In the case of the Hurricane Deck project, the lake depths combined with the thin overburden necessitated the design and construction of a unique "floating" template system. The template system was constructed from standard AB pontoon sections and falsework cages (Figure 15). The template could be floated from position to position, where it was temporarily spudded for rough position and then anchored to the lake bottom using Danforth style marine anchors.

Figure 15 – Floating Drilled Shaft Template Being Moved

After installation of the permanent casing, drilling of the overburden materials was accomplished with a crane-mounted drill rig and auger. Once the overburden materials had been excavated, rock sockets were drilled using a Worth Model PBA 933 reverse-circulation drill mounted to the permanent casing (Figure 16). Due to the size of the shafts, installation of the reinforcing cages also proved to be a challenge. "Righting" reinforcing cages of this size from horizontal to vertical can prove to be difficult and dangerous, particularly when accomplished on a floating platform. In order to eliminate the risks associated with the more conventional installation method, AB devised a system by which the individual longitudinal bars were suspended from a frame over the hole. The cage was then lowered into the hole as the stirrups were installed.

During the process, an unexpected issue arose during installation of the rock sockets at three shaft locations. Solution features were discovered in the dolomite, some of which were several feet beyond the rock surface through competent dolomite. The features consisted of clay-filled voids within the dolomite. A remedial action plan was developed by the drilled shaft team whereby the voids were progressively backfilled with concrete fill and then re-drilled through the concrete once strength was attained. At one location, Bent 8, this remedial action proved ineffective and one shaft had to be redesigned for a secondary casing to isolate the shaft from the voided zone in the rock. The secondary casing method worked as planned, and the shaft was completed without further complications.

Figure 16 - Worth RCD Drill in Operation

Upon completion of the two drilled shafts at each bent, substructure was constructed including a barbell strut, two round columns and a conventional beam cap. The substructure consists of conventional cast-in-place concrete construction and was constructed using ganged steel form systems. Certain elements met mass concrete criteria, and required specialized procedures to prevent thermal damage during the curing period. By December of 2012 all drilled shafts and substructure had been completed.

Upon completion of the substructure, work commenced on the erection of the two units of steel plate girders spanning the lake begins. As with any long span plate girder project, the key to safe steel erection was performance of a detailed erection analysis with particular focus on temporary stability of the structure during intermediate stages of erection. AB performed a staged 3-dimensional FEM analysis of the proposed erection sequence. The lake depths and geotechnical conditions provided challenging conditions for employing conventional methods of erecting falsework to provide temporary support of the girders during erection. AB engineers overcame this challenge by devising an erection scheme that did not require conventional falsework in the lake. AB's system utilized pier brackets at the permanent bent locations combined with selective erection of girders in pairs to maintain stability (Figure 17). The scheme worked as planned and erection was completed in March 2013.

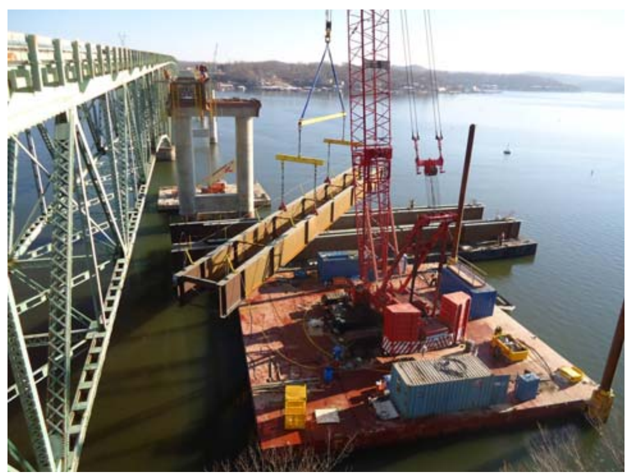

Figure 17 - Girder Erection in Pairs

With erection complete, work began on the concrete deck. The deck of the bridge consisted of 3-inch thick precast prestressed concrete deck panels with a 5.5-inch reinforced cast in place concrete topping. AB and Parsons worked to optimize the deck placement sequence to minimize the downtime between deck placements, ultimately developing a scheme that employed 15 individual placements. Upon completion, standard 32-inch jersey style parapets were casted using slipform methods. After roadway connections were completed, the bridge was opened to traffic on September 9, 2013, three months ahead of schedule.

Figure 18 – Concrete Deck Placement

Acknowledgements

Missouri Department of Transportation

American Bridge Company

Case Foundation Company

AFCO Steel

Dan Brown and Associates

References

(1) *AASHTO LRFD Bridge Design Specifications*, 5th Edition, 2010. American Association of State Highway and Transportation Officials, Washington, DC.