

INNOVATIVE STEEL ARCHES OVER THE MISSISSIPPI UTILIZING ABC TECHNOLOGY



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BIOGRAPHY

Mr. Vincent Gastoni is a Principal Bridge Engineer and Senior Project Manager with Parsons in their Minneapolis office. He is one of Parsons leading Design-Build and complex bridge Project Managers with over 22 years of structural experience in the design and management of major transportation projects. Mr. Gastoni's signature projects include the New Carquinez Suspension Bridge (Vallejo, CA), I-64 Design Build (St. Louis, MO), the Lafayette Bridge Replacement over the Mississippi River (St. Paul, MN), and the Hastings Bridge over the Mississippi River (Hastings MN). He is currently serving as the CEI Manager to the Minnesota DOT on the new extradosed bridge over the St. Croix River between Minnesota and Wisconsin.

Mr. Mark Buckholdt is a construction engineer with Lunda Construction Company. After graduating from Marquette University with a degree in civil engineering, Mr. Buckholdt began working for Lunda in February of 1999, where he has been employed ever since. His major bridge project experiences include the Sixth Street Viaduct (Milwaukee, WI), the Marquette Interchange reconstruction project (Milwaukee, WI), the Wakota Bridge (Newport, MN), the Cass St. Bridge (LaCrosse, WI) and the Liberty Memorial Bridge (Bismarck, ND).

SUMMARY

The new 545 ft long freestanding tied arch bridge over the Mississippi River in Hastings, MN utilizes an innovative cost effective approach to steel design, fabrication and erection and was selected by the Minnesota DOT through a competitive best value process. The design build team of Lunda Construction, Ames Construction and Parsons chose the unbraced steel box rib, full depth steel floor beam and stringer system combined with a post-tensioned concrete tie to provide a straightforward cost effective fabrication of the independent steel components.

The use of steel allowed for innovative accelerated bridge construction techniques. The elimination of upper bracing and use of a concrete tie allowed for the rib and floor system to be fabricated separately and then erected off site utilizing temporary steel bracing and ties for transport. Once erected and hangers installed, the arch span was transported by Self Propelled Modular Transporters (SPMT's) onto barges, floated downstream, skid jacked into place and lifted into its final location using strand jacks to lift the 3,300 ton superstructure.

Contributing to the success of the project was also the sustainability of steel, including the efficient and cost effective ability of the steel temporary works to be reused and repurposed over the course of the work and the salvage value of the existing bridge.

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Overview/Introduction

One of the newest crossings over the Mississippi River in Hastings, MN is the result of a cost effective use of steel and synergistic collaboration of design and construction engineering. The record breaking 545 foot long free standing tied arch bridge integrated the design, fabrication and erection approach through a competitive best value process to be selected by the Minnesota Department of Transportation (MnDOT). The new bridge replaces the existing two lane structurally deficient continuous steel arch truss built in 1950, and was accelerated through a design-build delivery method following the tragic collapse of the I-35W Bridge in Minneapolis, MN.

The design and construction team collaborated early in the process to develop and evaluate alternatives to MnDOT's cable-stayed and basket handle tied arch baseline structures. The team focused on an unbraced tied arch design utilizing tapered trapezoidal box steel arch ribs and a redundant steel floor beam/stinger system to simplify fabrication through fully independent ribs and floor systems tied together with a cast-in-place post-tensioned concrete tie. The design-build team also integrated the steel fabricator into the project design process to improve quality, expedite shop drawings and reduce project schedule risks.

Taking advantage of the cost effective and lightweight properties of steel, the team used an innovative erection approach to accelerate construction by building the main span off site on land and then moving the arch into its final position using Self Propelled Modular Transporters (SPMT's), barges and skid jacks. The contractor then lifted the arch via strand jacks all within a 36 hour move-in period. A key feature of this move-in was the use of skid jacks to reposition the bridge between the main piers with only inches of tolerance of the 545 ft long span. Steel was utilized to facilitate all stages of erection with some key elements remaining in the structure after the tie and deck concrete was placed.

The overall project cost of \$120 million dollars resulted in an average bridge cost of \$520/SF. The project design began in June 2010 and was completed in May 2011. Project Construction began in August 2010 and was substantially complete in May 2013 with the bridge commemorated in November 2013. Final project closeout will occur in June 2014.

Project Team

The Lunda/Ames design-build team with designer Parsons, combined the expertise of two major contractors and the leading designer located in the Minnesota region: Lunda Construction Company



Figure 1: View of the completed free standing tied arch structure

of Black River Falls, Wisconsin and Ames Construction of Burnsville, Minnesota. Lunda is a well known regional bridge heavy contractor with multiple Mississippi River crossings to their credit. Ames is a national contractor with numerous large design-build project experience. Parsons is an international design firm with extensive experience in both design-build and long span bridges accented with heavy regional experience on major MnDOT bridge projects. This design-build team combined their unique skill sets and expertise to evaluate and develop the winning innovative free-standing arch through the Alternative Technical Concept (ATC) process.

Project Constraints

Located at a tight bend on the Upper Mississippi River, the project site presented many physical constraints as well as stringent performance requirements for the design team to address regarding redundancy and durability.

Site: The single most significant physical challenge at the site was the Mississippi River itself. The bridge is located just over a mile below Mississippi Lock and Dam No. 2 and about three miles above the confluence of the Mississippi and St. Croix rivers with the bridge passing directly through the historic town of Hastings. During the river navigation seasons operations needed to be continuously coordinated with the U.S Coast Guard and lock and dam operations. Freeze up of the river in winter followed by the annual spring

flood season all presented constant reminders of the risks and challenges facing the team. In evaluating alternatives these constraints influenced the team's decision to use a one lift main span erection scheme when compared to a higher risk multi-year continuous lift operation required for the cable stayed alternative. MnDOT had also made a lay down area available for the successful contractor to use adjacent to the site which was used to erect the main span arch and load it onto barges for the eventual float in operation.

Performance: The final design had to address stringent redundancy and durability requirements exceeding normal AASHTO and FHWA requirements, including criteria for a 100 year service life design. MnDOT supplemented the AASHTO LRFD specifications with additional live load cases, load combinations, redundancy live load cases for strength and service design, and site specific wind load analysis. These additional requirements included full operational capability in the event of a floor beam loss and failure for both service and strength limit states.

Redundancy load cases were very specialized and included fracture load cases for the floor system, hangers, and tie girder post-tensioning under strength and service limit states including fracture dynamic force effects to ensure load path redundancy. Basic requirements included: no fracture critical members (load path redundant); check failure of any one hanger; and check fracture of any one floor beam. Strength and



Figure 2: Site layout of new and existing structure.

Service load redundancy combinations included:

Strength

At time of fracture:

$$1.25DC + 1.35(LL + IM) + 1.1FDF$$

(FDF = fracture dynamic force)

After fracture:

$$1.25DC + 1.35(LL + IM)$$

Capacity-to-demand ratios ≥ 1.0

Service

At time of fracture:

$$1.0DC + 1.0(LL+IM) + 1.0FDF$$

After fracture:

$$1.0DC + 1.3(LL+IM)$$

Competitive Alternatives

Best Value Approach: Through the MnDOT design-build process teams were able to submit Alternative Technical Concepts (ATC) to the baseline concepts provided by MnDOT which included a single diamond tower cable stayed structure and a basket handle network tied arch structure. The Lunda/Ames/Parsons team thoroughly evaluated these base designs via preliminary engineering and cost estimating

exercises and ultimately produced a comprehensive evaluation matrix tool which accounted for material costs, risks, and developed an overall best value technical approach.

In conjunction with the baseline design evaluations, Parsons developed alternative designs based on the Request for Proposal (RFP) performance criteria and goals. The design team was successful in having two bridge type ATC's accepted by MnDOT; a single tower dual-leg vertical cable-stayed structure and the free-standing tied arch structure. These alternatives were added to the team's evaluation matrix and the free-standing arch was selected by the team as the best alternative.

Component Selection: The RFP requirements, best value scoring, and construction costs/risks ultimately drove the final selection of bridge components. The steel free-standing arch ribs provided an easily maintained, aesthetically pleasing element. They also simplified fabrication since each arch can be assembled independently. MnDOT specified additional criteria for this alternative including a minimum Factor of Safety against arch buckling of 2.0 at the Strength Limit state, and a limiting lateral deflection of R/300 (where R is the rise of the arch rib at the service limit state under the Service I load combination).

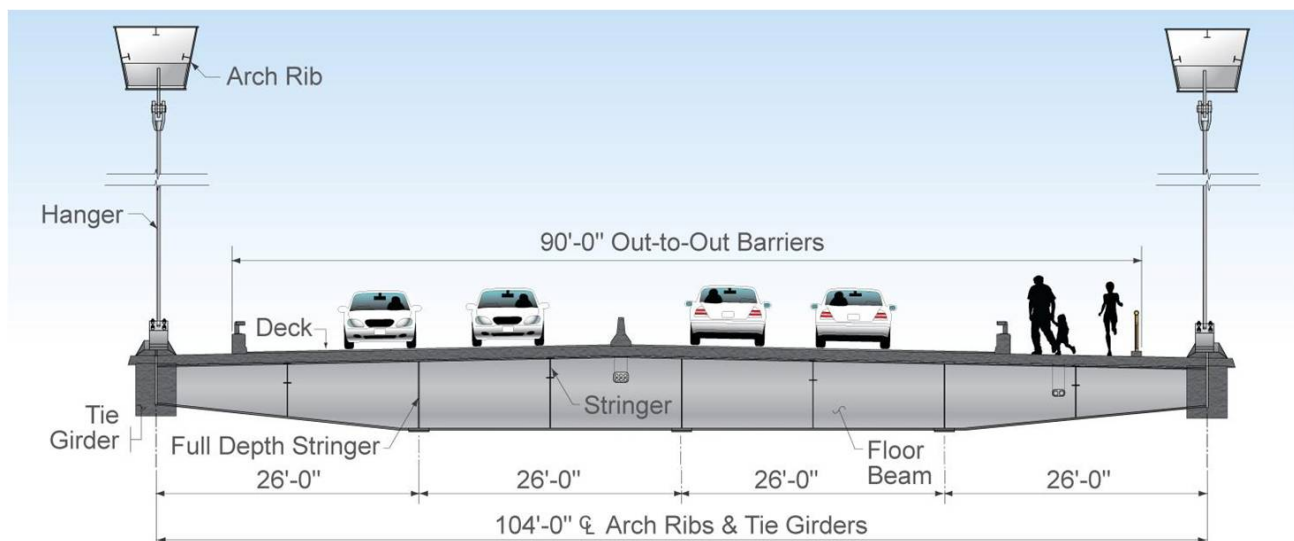


Figure 3: Cross section of new structure.

To meet the floor system redundancy requirements a steel grillage system was designed with full depth longitudinal stringers framing into the floor beams. All floor beams and full depth stringers had to meet full service and strength criteria under the fracture cases. Additionally, the criteria required that the deck must be able to be fully replaceable one-half at a time. This drove the team to use tapered floor beams and full depth stringers with the maximum use of bolted connections and redundancy details.

The post-tensioned concrete tie girder and framed knuckle was selected in order to meet the rigorous redundancy and load requirements of the tie system. This alternative also improved the inspection/maintenance of the system, eliminated any shop fit-up assembly issues with the rib, and provided complete active-tension control of the system during erection. Combined with the greater durability of the concrete tie over a steel tie, this approach provided a best value win-win solution.

The foundations also presented significant challenges with bedrock near the surface at the south river pier and 190 feet of soft clays and mud over bedrock at the north river pier. Drilled shafts

were evaluated for the north pier but represented a higher cost and higher risk from settlement to the existing bridge pier due to potential caving. Based on the recommendations of the team's geotechnical engineer Dan Brown & Associates, unfilled 42-inch driven steel pipe piles driven to rock were ultimately selected for the north river pier. At the south pier the final rock elevations were deeper than originally expected and short caissons were drilled into the rock to support the pier.

Design Approach to Redundancy & Slenderness

Redundancy: All structural tension members are load path redundant for fracture at any point in a single member or connection subject to tension; consequently non-redundant fracture critical bridge elements on the structure were eliminated. The capacity-to-demand ratios for all members under the fracture load case strength limit state are designed to be greater than or equal to 1.0. Consideration of multiple fractures occurring simultaneously was not required. Redundancy of the arch rib was not necessary since it is a compression element.



Figure 4: Independent steel grillage floor system at off-site erection yard.

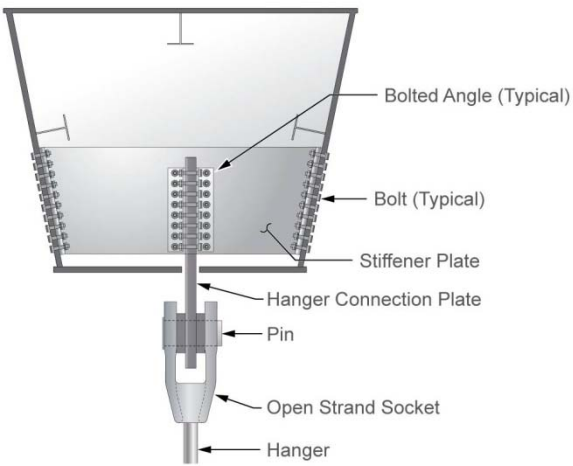


Figure 5: Box Rib Section

The redundant load paths are designed to meet strength and serviceability criteria under an element loss. The alternate load paths and member designs include both steady-state and transient force effects which only occur at the time of fracture. The dynamic effect was studied by performing time history analysis including multimodal structural interaction with appropriate distributed mass and damping distribution. This was correlated back to an equivalent single Fracture Dynamic Force modifier to use in design.

The transverse steel floor beams (spaced at 27'-6") and full depth longitudinal steel stringers form a redundant grid floor system, which allows load transferring in both the longitudinal and transverse direction with the primary load path through the transverse floor beams. Each hanger anchors to an individual steel anchor plate that is bolted to the end plate on the steel floor beam at the lower connection. The top hanger connections are bolted to the steel arch rib. Bolted connections are used to prevent propagation of cracks to the arch rib and are independent of other hanger connections.

The solid cast-in-place tie girders are post-tensioned with grouted multi-strand post-tensioning tendons providing a redundant tie girder system. The tie girder is designed to be in net compression under service level loads and also

allows for up to 25% of the tendons to fracture with the remaining tendons designed to resist all loads and limit the tensile stress to 0.0 psi.

Slenderness: Lateral stability of the free-standing arch rib is provided through the large rib box section. The 1:3.33 web slope provides a larger transverse section modulus to resist out-of-plane forces and creates a unique and pleasant visual element. In addition to a strength approach to stability, the hanger and deck system also provide a significant self restoring force to out-of-plane deflections and contribute to the stability of the free-standing arch rib.

A detailed three-dimensional finite element model (FEM) was used to study the nonlinear stability behavior of the rib utilizing a step-by-step load increment approach. The approach first performed a second order incremental buckling analysis to obtain the moment magnification factor for factored axial loads. External loads are then applied to the model to generate the first order modes in accordance with the AASHTO load combinations. The applied loads are then increased incrementally until the arch rib reaches the stability limit. A factor of Safety of 3.25 was achieved for the rib in its final condition with a value of 1.90 in the unbraced temporary erection configuration.

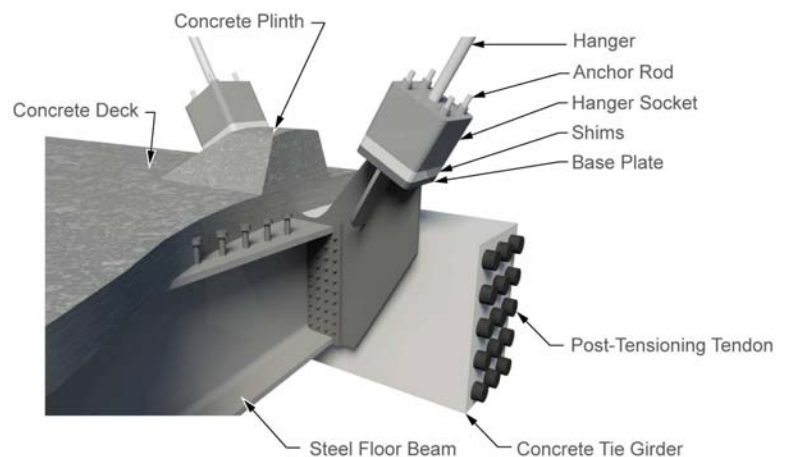


Figure 6: Floor beam, hanger connection, and tie.

Steel plate slenderness in the context of the arch rib flanges and webs is not fully addressed in the current AASHTO provisions. The design team evaluated current and previous AASHTO specifications and historical methodology (Nettleton) to develop a project specific approach to the cross sectional plate design. Based on this approach the main span arch rib flanges were treated as unstiffened plates. The general case of a stiffened plate under a uniform and linear gradient of stress was considered applicable to the design of arch rib web plates.



Figure 7: Off-site erection of main span.

Design Approach to Erection

With a focus on accelerated bridge construction techniques, the construction/design team determined very early that the traditional methods of “stick building” the arch in place did not meet the project schedule and presented significant schedule risks due to the river constraints. To accelerate the schedule it was decided to erect the arch offsite while the substructures were being constructed and then move the main span into position. When considering arch move-in alternatives the method of building the arch high on temporary towers and floating it in over the piers was deemed too risky due to the high center of gravity and variable river elevation. Therefore, the team elected to erect the arch on land, move it

onto barges, float it down river and then lift it into place.

This erection concept presented many challenges to the design of the structure to accommodate the multiple number of load configurations encountered during construction. The primary focus was to minimize the weight of the structure for the move and lift operations while optimizing the post lift construction schedule. To achieve this, the steel ribs, hangers and floor system were erected in the staging area with temporary steel tension ties and knuckles to stabilize the rib for the move and lift operations.

Steel tie: Each of temporary steel ties comprised of two W36x194 girders to stabilize each arch rib reaction during erection and placement of the concrete tie girders. The temporary tie was also used to stabilize the floor system through temporary bolts at floor beam intersections. To accelerate schedule operations after erection, the concrete tie girder forms and reinforcement were attached to the tie girder prior to erection. After installation and post-tensioning of the concrete tie, the temporary steel tie was removed prior to deck placement.

Steel knuckle: The connection between the permanent arch rib and temporary tie was critical to both the erection and final design and was designed to be integrated into the final works. The



Figure 8: Temporary steel knuckle and tie.

base of the arch rib was specially designed to accommodate the connection of the temporary steel knuckle. The knuckle itself primarily consisted of two 1-3/4 inch thick plates in a canted “K” shape approximately 12 feet high and 19 ft long. The arch rib connected at the top leg of the “K” with the temporary tie girder connection at the bottom leg of the “K”. The plates were braced with internal diagonals to form a very rigid “knuckle box”. A specialized 2-inch thick reinforced lifting plate was designed to sit in the knuckle box and serve as both the lifting connection for the strand jacks and the bearing support after erection. The entire temporary steel knuckle was integrated into the final concrete knuckle with a positive load transfer made through post-tensioning of the concrete tie system.



Figure 9: Support beams were inserted into the temporary knuckles after erection

Arch support beams: With the arch erected between the piers, the team had to design a temporary support to hold the arch in place after the lift. This temporary support would also be permanently left in place and cast into the concrete knuckle and tie beam. The principle challenge was to develop a system that could be installed within hours during the lifting operation to accept the 1,800 kip reaction load transfer from the strand jacks to the support beam. Further complicating matters was the need for the support to be compact enough to support the temporary knuckle and be included in the permanent works. The design team once again turned to steel to develop a stiffened

box like support beam that could be jacked into the back of the temporary steel knuckle at the end of the lifting operation. The arch reaction force was applied to the beam through the same 2-inch thick reinforced lifting plate on the temporary knuckle with support stabilized by four 2 ½ inch diameter post-tensioning bars cast into the top of the pier. The system worked as planned and also served as the jacking platform to jack the piers apart to account for the post-tensioning long term creep and shrinkage effects of the concrete tie girder.



Figure 10: Temporary bracing was used during erection to stabilize the arch ribs.

Arch braces/K-frame knuckle braces: Until the concrete knuckle, tie-girder and deck were in place the arch ribs required temporary bracing to assure stability and alignment. As part of the construction engineering, Lunda Construction designed the temporary bracing to accept the erection loads per the design team’s analysis. The bracing consisted primarily of 24-inch pipe struts with single diagonals located near the 1/3 points of the ribs. The upper connection of the bracing was designed to allow for the vertical deflections that would occur between the deck and the rib over the course of lifting, concrete placement and post-tensioning. Additional horizontal “K” bracing was provided at each end of the arch to stabilize the temporary knuckles and provide additional rigidity to the system.



Figure 11: Center arch segment placement and fixed center tower support

Bridge Erection

The bridge erection sequence was driven by a combination of strict erection tolerances and managing the risk of assembling steel that was fabricated in three different shops. To meet the project schedule, the fabricator chose to build the floor beams, stringers and lower lateral bracing in one shop, the arch steel and temporary knuckle in the second shop and the temporary tie girder in a third shop. Further complicating the matter, the contractor self performed the fabrication of the arch bracing, temporary tie girder diaphragms and temporary tie support columns, which connected all three of the fabricator's elements. To mitigate fit up risks between the temporary knuckle and the temporary tie girder, Lunda chose to overextend the temporary tie girder and field fit and drill the temporary tie girder ends. Lunda also chose to use a top-down approach for the arch rib erection since it was too risky to start the arches at the temporary knuckles and assemble towards center of span. The center segments were carefully surveyed into position to best align the center of the arch with the center of the floor system. To prevent the floor system from "walking" out of alignment due to temperature changes, the center of floor system was fixed longitudinally while the remaining 30

support locations were longitudinally unrestrained on elastomeric bearings. The arch members were similarly fixed at the center of span by use of braced towers, while the remaining 12 towers allowed for expansion by allowing rotation in the tall, unbraced towers. After the arch members were connected to the temporary tie girders, hanger cables and floor system, misalignment was no longer of concern.

Single towers directly under the arch steel would interfere with the diagonally placed hanger cable installation and eventual tower removal so two row of heavy duty steel shore towers were used to support each arch segment. A dual tower configuration provided advantages over a single tower configuration given the single arrangement could not support the weight of the arch and floor system or accommodate the first stage of hanger tensioning.

Bridge Transport

Tower Lift Off: The temporary shoring was designed to support the arch and allow access directly under the knuckles for the Self Propelled Mobile Transporters (SPMT's). After load was transferred from the shoring towers to the SPMT's, it was undesirable to set the structure back on the towers due to the redistribution of loads and the tie geometrics. As such, the vertical "lift-off" activity required careful planning and coordination.



Figure 12: Arch supported by SPMT's ready for transport to barges.



Figure 13: Arch is loaded on to ballasted barges.

During lift-off, 4.3” of elongation was expected to occur as the structure changed from a continuous beam supported on shoring to a fully functional tied arch supported on the SPMT’s. The elongation was accommodated by temporarily rotating the axles of the SPMT’s on one end of bridge and allowing them to freely roll during lift-off. After the structure was fully supported by the SPMT’s, all temporary support structures were cleared to allow for horizontal transport.

Land to Barge Move: Due to the poor soil conditions, contaminated subsoils and proximity to the navigation channel it was determined that constructing two trestle bridges was the best means of transporting the structure to the barges. Through coordination with US Coast Guard and Army Corps of Engineers the majority of the

temporary trestles were constructed without impacting river traffic. The remaining portion of the bridge move was fully coordinated with the Coast Guard with Lunda Construction providing continuous marine support.

The trestles were designed with a profile to equally rotate the bridge transversely out of plumb by a maximum of 3.43% and to carry the 3565 ton load down the 9 ft elevation drop from the staging yard surface to the barges. The SPMT’s allowed further rotational adjustment to maintain a 2% maximum slope for the tied arch. The entire 405 ft. move took about 8 hours from beginning of the lateral move to set down on the barge grillage. The SPMT move was monitored to keep within the 6” racking tolerance permitted by the engineer. The barges were preballasted and sequentially

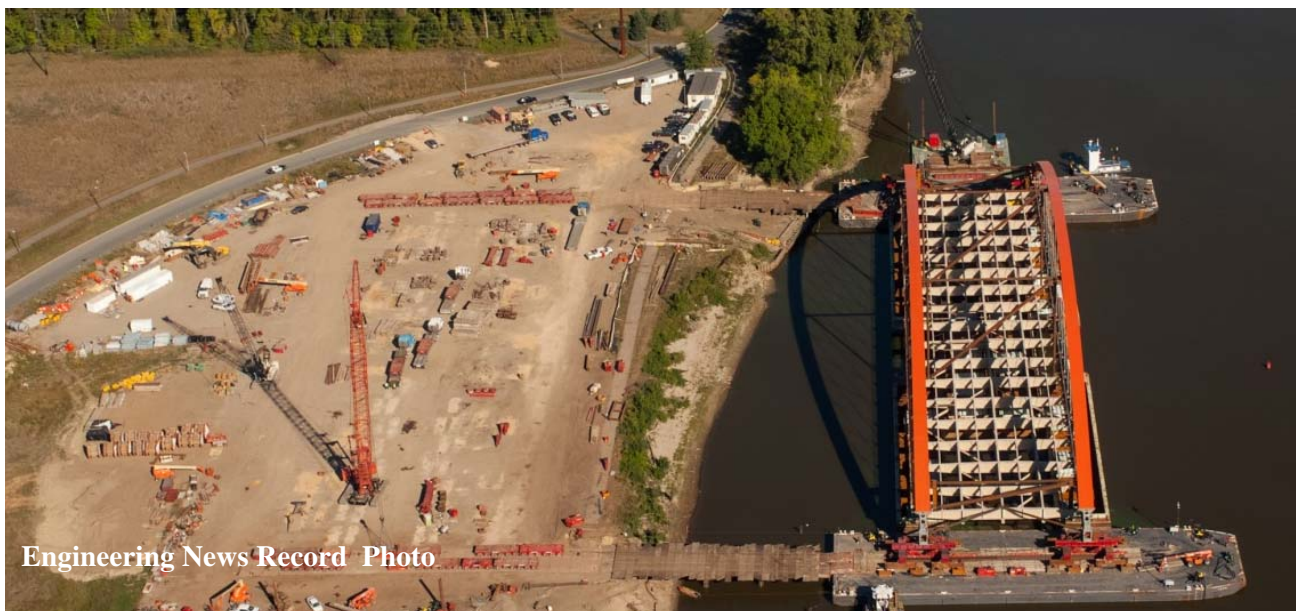


Figure 14: Aerial view of the erection staging area and arch span on barges.

unballasted while the SPMT's moved the arch onto the barges.

Barge Float In: Once transport to the barges was complete, 16 hours was required to stabilize the system for float out. The structure was then



Figure 15: Barge move out to mooring area

moved out and moored at a temporary location outside of the navigation channel until a window became available for the final move. For added safety, a tugboat was secured to one barge and occupied with a pilot around the clock. During an approved shutdown coordinated with the US Coast Guard and the MnDOT, river navigation and Highway 61 bridge traffic was closed and the operation to move the arch downstream to the pier locations could begin. It took 3 tugboats to move the barges ½ mile to the bridge site in 1 hour. An extra tugboat was brought in to assist due to the prediction of high winds on move day.



Figure 16: South skid support

Barge to Skid Bent Move: Due to the shallow water and exposed shoreline at the south pier it was not possible to directly float the structure between the supporting piers. A system of falsework supports was required to move the structure south and then east to align it within the piers prior to the vertical lift. To meet this challenge, the construction team developed a creative step-wise solution:

- Position barge supported arch upstream and adjacent to piers;
- Transfer south arch support to skids;
- Skid arch south onto shore using skid jacks to align between piers;
- Transfer south arch support to new skids aligned east-west.
- Laterally move arch east into position using skid jacks at the south support and simultaneously from the north barge support using a combination of skid jacks and SMPT's

The south skid move was completed in 10 hours as ballast water was removed from the south barge



Figure 17: South skid operation

and the north barge was pulled south. The eastward movement from north barge support to final pre-lift position was completed in 12 hours.

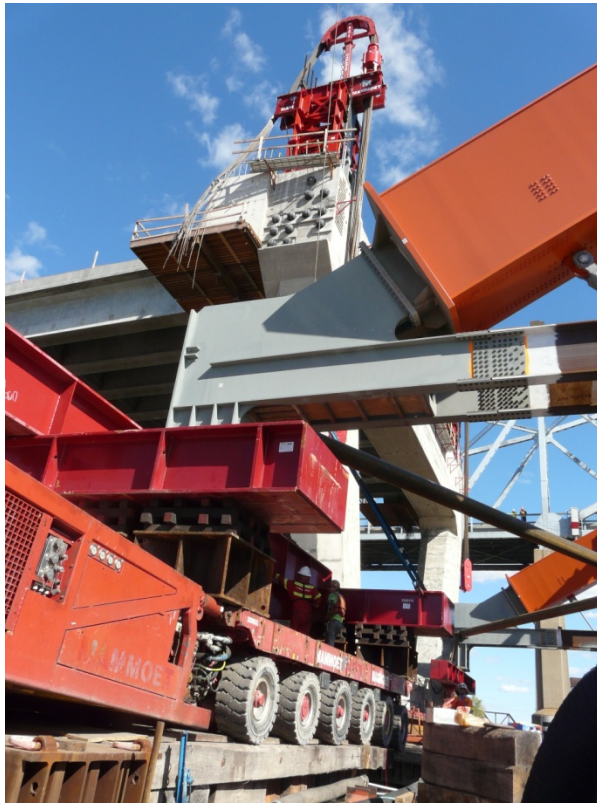


Figure 18: North support east skid operation

Bridge Lift: The supporting piers were designed to withstand the effects of the cantilevered load imparted from a custom lifting frame. The lifting frames supported Mammoet’s high capacity strand jack system that was in-place prior to the barge move. A pin of Mammoet’s custom strand jack anchor block was shipped to the steel fabricator to guarantee proper fit up to the temporary knuckle connection. The strand jacks

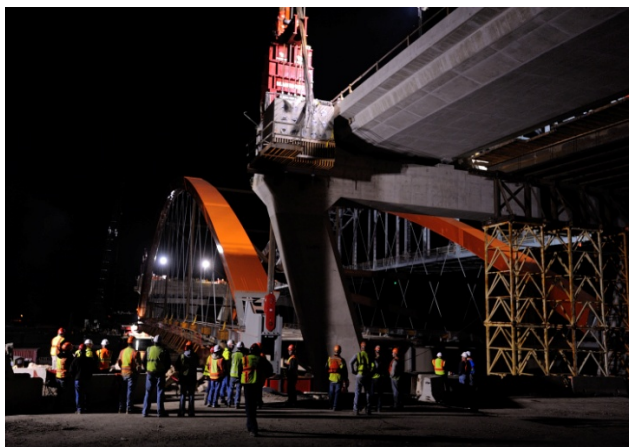


Figure 19: Lift operation begins

were attached to the arch and the lift was completed in a three hour window. River and Highway traffic were then re-opened while the arch was jacked, shimmed and permanently secured to the piers over the next 10 hours.



Figure 20: Lift operation is completed

Conclusion

The Hastings Bridge design and construction is a unique solution that efficiently addressed the project challenges through the optimum use of steel in the final works and during the erection operations. The synergy of the design-build team allowed for the development and execution of a unique alternative approach that combined the best of design and construction engineering. The team’s solution was only feasible due to the efficiency and high strength properties of steel and was the winning solution in a competitive bid best value selection due to the cost effectiveness of steel.

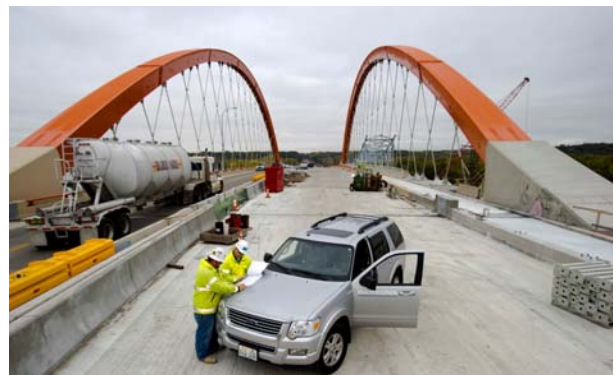


Figure 21: Completed bridge open to traffic