DESIGN AND CONSTRUCTION OF A LONG SPAN STEEL PLATE I-GIRDER BRIDGE: I-270 BRIDGE OVER CHAIN OF ROCKS CANAL

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BIOGRAPHY

Brandon Chavel is a Senior Bridge Engineer and Professional Associate for HDR Engineering in Cleveland, OH. In his 12 years of experience he has been the lead structural engineer for several complex bridge designs, construction engineering, and load rating projects. His highway and railroad bridge experience includes completing bridge type studies, curved girder bridge design, complex load rating, seismic design, erection engineering, rehabilitation design, and bridge research studies. Dr. Chavel received is Ph.D. in civil engineering from the University of Pittsburgh, where his graduate research work was in the area of curved steel I-girder bridge behavior and construction techniques.

Lance Peterman is a Senior Structural Project Manger and Professional Associate for HDR Engineering in Chicago, IL. Mr. Peterman has been responsible for directing the design of several complex bridge and transportation projects. He has more than 23 years of experience in the planning, design, and construction of highway and railroad bridges, rapid transit structures, and retaining walls, as well as design-build projects. Mr. Peterman is a registered Structural Engineer in Illinois, and received both his Bachelor's and Master's Degree in Civil Engineering from the University of Illinois Urbana-Champaign.

SUMMARY

Located in southwest Illinois, the new steel plate I-girder Chain of Rocks Bridge is designed to carry six total lanes of the I-270 corridor over the main navigation channel for Mississippi River barge and vessel traffic. The new Chain of Rocks Bridge is a five span continuous haunched steel plate girder bridge with spans of 250′, 440′, 490′, 440′, and 350′, and is one of the longest steel plate girder bridges in the State of Illinois. The new structure replaces twin steel truss bridges that are now functionally obsolete.

Several constraints associated with the Chain of Rocks navigation channel and adjacent levees, and roadway geometry resulted in the span arrangement and choice of a steel girder superstructure. All superstructure steel is unpainted weathering steel, and an efficient combination of Grade 50W and HPS70W is utilized for the various girder sections. The bridge cross section consists of 10 equally spaced steel plate girders, which support the 94'-2" wide cast-in-place concrete deck. The steel plate girders are haunched to maximize structural efficiency in the negative moment regions.

The paper will discuss the challenges faced during the design and construction of the long span I-270 Chain of Rocks Bridge. The design and construction of a long span I-girder bridges can be complex, however when all of the issues are addressed, the end result is a successful and efficient steel girder bridge.

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Introduction

I-270 is a four-lane interstate expressway that serves as a north bypass to the City of St. Louis. I-270 carries a significant number of daily commuters between Illinois and Missouri with an average annual daily traffic count of 54,700 vehicles and 20% truck traffic. This project includes a complete reconstruction of a pair of truss bridges over the Chain of Rocks Canal with a single bridge on a new alignment north of and adjacent to the existing bridges.

The Chain of Rocks Canal provides a bypass for all Mississippi River barge traffic in the region that is necessary due to the rock outcrop in the portion of the river in the vicinity of I-270. The canal is open to the main channel of the Mississippi River on the upstream end, and it is controlled on the downstream end by Lock and Dam 27. The St. Louis District of the U.S. Army Corps of Engineers owns and operates the Chain of Rocks Lock #27. On average over 70 million tons of cargo pass through Lock #27 a year making it the busiest navigation structure on the Mississippi River.

Parallel to the canal on the west is a saddle dam and to the east is a major flood protection levee. The Chain of Rocks east levee is a critical link in the overall Mississippi River levee system, providing 500 year flood protection to Illinois residents in an 85,000 acre area. The United States Army Corps of Engineers (USACE) prohibits any construction activity that would impact the integrity or function of the east levee as the flood protection function cannot be comprised.

The existing I-270 bridges are over 50 years old and consist of two identical 12-span bridges (see Figure 1). Each bridge has four approach spans at the west end and five approach spans at the east end. The approach spans are constructed of continuous steel plate multi-girders. The main spans over the canal consist of three-span cantilevered through trusses with a suspended span over the canal. The main span over the canal is 480′-0″ and the total length of each of the structures is just less than 2000 feet. Each existing bridge accommodates two 12′-0″ wide lanes and 3′-0″ wide shoulders. The existing bridges are structurally deficient and functionally obsolete, two-lane structures that have insufficient width shoulders. The structures were deteriorating at an accelerated rate and this lead to an increase in the frequency of bridge repairs. The floorbeams and the deck were in poor condition and required recurring repairs. The maintenance activities caused significant disruption to traffic since a lane had to be taken out of service during these operations. The gusset plates throughout the truss were evaluated given the recent collapse of I-35 in Minnesota and found to be satisfactory. Additionally, the existing bridges are located in a Seismic Performance Zone 2 and did not satisfy current seismic ductility demands.

Figure 1: Existing I-270 Bridges Over the Chain of Rocks Canal

The existing bridges are currently being replaced with a haunched steel plate girder bridge with a total length of 1970′. The proposed structure consists of 5 continuous spans: 250′, 440′, 490′, 440′, and 350′, as shown in Figure 2. The span arrangement was dictated by the need to span the canal and adjacent east flood protection levee. The bridge is 94’-2” wide and can accommodate a future lane arrangement of 6 total lanes. The bridge consists of 10 variable depth steel plate I-girders. Scheduled to be complete in 2014, the new I-270 bridge represents the largest steel plate I-girder bridge in Illinois.

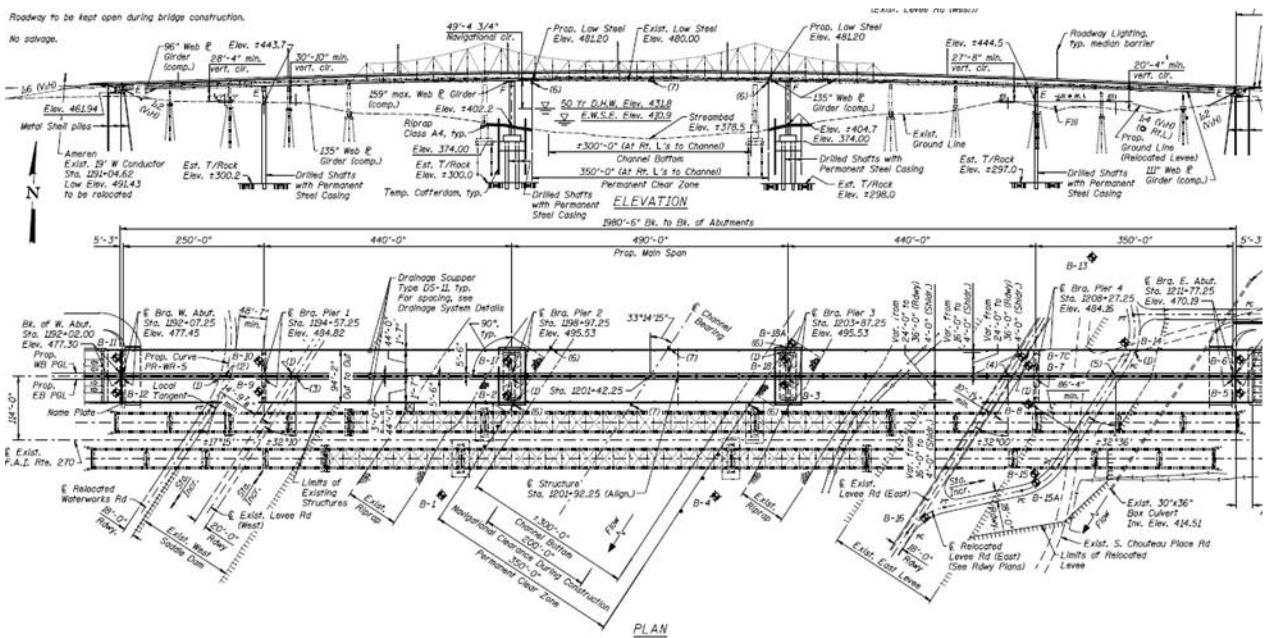

Figure 2: Plan and Elevation Drawing of the new I-270 Bridge over the Chain of Rocks Canal

Bridge Type Study

The design and construction of the new I-270 bridge had to consider the navigation requirements of this canal. The United States Coast Guard (USCG) required the proposed bridge to provide a 350 foot horizontal navigational clearance in line with the existing bridge opening and that the vertical clearance match that provided by the existing I-270 bridges. The vertical clearance from low steel to the 50 year high water level is less than 50 feet. Barge traffic in the canal operates 24 hours a day, 7 days a week, and year-round. A reduction in the 350 foot horizontal clearance was permitted by USCG during construction. The minimum temporary horizontal clearance was 200 feet. This temporary horizontal clearance accommodated one shipping vessel passing under the structure as well as the pier cofferdams and 35 foot wide working barges. These temporary horizontal clearance requirements were considered when determining the main span length of the new bridge.

The spans adjacent to the main span had to consider the structural efficiency of the continuous structure as well as the west saddle dam and east flood protection levee. Bridge replacement options had to either span the existing levee or include provisions to relocate the levee to avoid conflicts between proposed piers and the east levee. An access road located on the levee had to remain open at all times for USACE maintenance and emergency response. The centerline of the flood protection levee is approximately 400 feet east of the canal. The cross section of the levee is a 20 foot wide crown with 1:4 side slopes. In order to open up the bridge type study to multiple alternatives it was proposed that the existing levee be left intact and that a new parallel levee embankment be constructed. This new levee arrangement allowed proposed bridge piers to penetrate the existing levee but not the footprint of the proposed parallel levee.

The vertical and horizontal alignments of the proposed bridge are interrelated to the IL Route 3 interchange located approximately three-fourths of a mile to the east of the Chain of Rocks Canal. Accommodating the westbound entrance ramp of this interchange with the proposed westbound alignment of I-270 was a significant design constraint. The new bridge had to accommodate two lanes of traffic in each direction and be compatible with a future I-270 corridor widening project.

The bridge type study commenced with a design charrette meeting. Key decision makers from the Illinois Department of Transportation (IDOT) and the Federal Highway Administration (FHWA) met with the HDR consultant team to review and discuss detailed information pertaining to the existing structures and design alternates to help expedite a consensus on the alignment and bridge types to carry forward for detailed study. This process identified the key design constraints and ranked the importance of each. Following the charrette meeting, the studied options with main spans in the 400 to 700 foot span range consisted of: Multi-Span Plate Girder (up to 500′ maximum span); 3-Span Tied Arch (600′ / 800′ / 600′); 2-Span Cable Stayed (1300′ / 700′). The bridge types were evaluated for cost, levee impacts, adjacent interchange geometrics impacts, right-of-way impacts, constructability, structural redundancy, seismic performance, and maintenance and inspection. Truss bridge alternatives were not evaluated due to high construction cost and redundancy concerns in light of the recent I-35 collapse.

The continuous steel plate girder bridge was evaluated as the least cost and highest ranked in the structural redundancy, seismic performance, and maintenance and inspection categories. A key for this evaluation of the plate girder alternative was the construction of the relocated levee. This enabled a feasible and efficient span arrangement. The steel tied arches alternative was evaluated as second in cost. The arches provided the ability to span the canal and levee but the erection of the main span tied arch would require that temporary support structures be placed within the canal. Fabrication and erection costs for tied arches were considered higher than for deep plate girders, with less competition among fabricators. Seismic resistance was considered to be more challenging than the plate girder alternative since the substructure units for the tied arch alternative would have a larger seismic base shear due to larger dead load reactions, resulting in larger substructure units. Maintenance and inspection was also considered inherently more difficult with access to internally redundant tie girders and hangers associated with this complex bridge type. Cable stayed bridge options were considered the most expensive option but offered the greatest flexibility in terms of span lengths. With the option of relocating the east flood protection levee the benefit of span length flexibility was not cost justified. A five span continuous plate girder bridge was selected. The superstructure was supported with solid wall piers on drilled shafts on either side of the canal, with multi-column pier bents on drilled shafts supporting adjacent interior spans, and with open abutments on both ends supported on metal shell piles. Although skewed piers would have reduced the main center span length, they were not used due to significant structural complications, including differential deflections, highly loaded cross frames, and constructability concerns. Furthermore skewed piers would have altered the structure lateral stiffness and shorten the structure period leading to higher seismic forces to be resisted. The bridge is designed in accordance with the AASHTO LRFD Bridge Design Specification (1) and the IDOT Bridge Design Manual (2). The project was ultimately bid on by four contractors and was awarded to Walsh Construction Company.

Substructure Design

The geology of the site consists of the alluvial plain of the Mississippi River. This area is characterized by 100 to 150 ft. of sand sediments overlying bedrock. Directly above the bedrock layer there are intermittent mixtures of gravel and cobbles. The bedrock within the project area consists of a high quality limestone with an unconfined compressive strength of 10,000 psi.

The project site is approximately 140 miles form the New Madrid fault zone and in a Seismic Performance Zone 2. The Design Spectral Acceleration at 1.0 sec is 0.23g and the Design Spectral Acceleration at 0.2 sec. is 0.50g. The multimode elastic method of analysis was required for this bridge due to the span ratios and being classified as an essential bridge. The seismic resistance strategy was based on an elastic response to a relatively long structure period greater than 3 seconds. A linear dynamic analysis was conducted using a 3 dimensional LARSA model of the complete bridge. Cracked section moduli of the substructure were used in determining the structure period. Member forces and displacements were computed by CQC combination method.

Open type abutments on metal shell piles were used. The abutment piles were designed to resist the vertical loads in friction and were approximately 50 feet long. Piles were selected over drilled shafts at the abutments because of economy and the reaction at the abutments was substantially less than the piers.

Piers 1 and 4 each consist of 5 columns supported individually by 6 foot diameter drilled shafts. See Figure 3 for Pier 1. The pier cap was 7 feet deep, the columns were 23 feet tall and the drilled shafts were 140 foot long and socketed into the underlying bedrock 15 feet. Permanent steel casing was used to drill through the sand. Piers 1 and 4 were expansion piers and designed to resist longitudinal lateral loads that were transmitted through friction in the guided pot bearings. Transverse lateral loads from wind and seismic were resisted by the frame action of the pier and the group effect of the single row of shafts. The group effect of concern resulted from the shafts behind the leading shaft exhibiting less lateral resistance because of interference of the leading shaft. The shadowing effects associated with the shaft and soil interaction were considered by reducing the soil resistance, p, from a single shaft p–y (lateral deflection of the shaft) curve using a constant reduction factor or p multiplier.

Figure 3: Pier 1

The main Piers 2 and 3 were both fixed against translation and consisted of solid 8 foot thick wall piers supported by 2 rows of 5, 6 foot diameter drilled shafts. Figure 4 shows the construction of the drilled shafts for Pier 3. The main piers were located in the outer limits of the canal and as such required a 17 foot thick seal coat to construct a 10 foot deep footing over the drilled shafts. These dimensions required the contractor to utilize several methods to keep the mass concrete from overheating during curing. The precise location of these piers was determined to minimize the main span length considering not only the permanent condition navigational requirements but also the temporary horizontal clearance requirements considering the footprint of the cofferdams and working barges. A compact footing design was required which lead to the efficient use of the group of drilled shafts. These piers were designed to resist both vessel collision forces and seismic forces which proved to be essentially equally onerous. Vessel collision was evaluated per AASHTO section 3.14 as an Extreme Event load case. The barge velocity, energy and force were evaluated and the main piers were designed to absorb the vessel collision impact elastically. The impact forces were applied to a LARSA model as a stability check. The flexure caused by lateral seismic and vessel collision loads at the main piers is resolved into combined bending and axial shaft loads. A single row of drilled shafts was considered but resistance by the lateral bending capacity of individual drilled shafts would have resulted in much larger diameter drilled shafts. The tall main unit canal piers were inherently flexible and changes to the overall period of the structure were not significantly impacted in comparing the single row of shafts to the two rows of shafts.

Figure 4: Pier 3 Drilled Shaft Construction

Superstructure Design

The span lengths of the bridge exceed the limits of applicability for the use of approximate live load distribution factors provided in AASHTO LRFD Section 4. Additionally, steel erection, initial deck placement, wide permit loading, and future part-width deck replacement had to be considered during the design of the steel superstructure. For these reasons, a 3D finite element model is employed for the final design of the steel superstructure. LARSA 4D finite element modeling software is utilized for the analysis of the superstructure. A model is used for the design loads (dead, live, wind, thermal) and a separate but similar model is used for the seismic analysis. The discussion herein focuses on the design model.

Analysis Model

The 3D finite element model for the I-270 bridge considered all of the main structural components. Girder flanges are modeled with beam elements and shell elements are used to model the girder webs. Cross frame members and top flange lateral bracing members are modeled with truss elements, and the deck is modeled with shell elements. Figure 5 shows a screen capture of a portion of the model in the non-composite condition. Boundary conditions are developed for the appropriate fixities at each substructure unit. Where applicable, substructure support stiffnesses are developed and modeled with spring elements in the superstructure design model (seismic analysis model included substructure elements as appropriate).

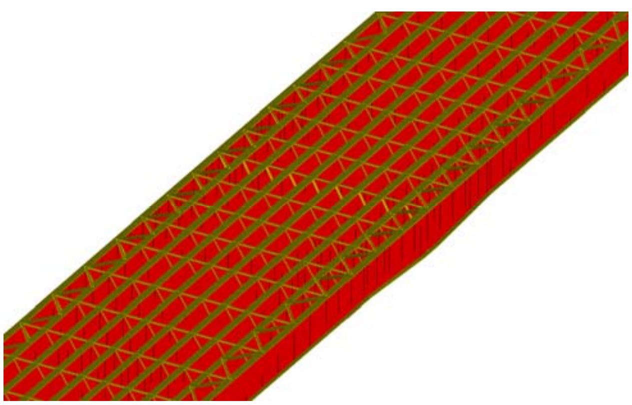

Figure 5: Portion of the 3D Model in the Non-composite Condition

The design model loading consisted of typical dead, wind, thermal, and live loads. An influence surface is employed for the live load analysis. With an influence surface, ordinates are determined within the finite element program by applying a vertical unit load at longitudinal and transverse positions at defined increments on the concrete deck. Truck and lane live loads are then placed on the influence surface at critical locations to determine the maximum/minimum structural component force effect. Live load force demands are considered in the design of the girders, cross frames, and lateral bracing.

In addition to the HL-93 live load demands, permit loads are considered in the design of the I-270 bridge as well. Permit loads, provided by IDOT, included:

- 6-axle, 132,000 lb truck with a 6 ft wheel spacing,
- 9-axle, 188,000 lb truck with a 6 ft wheel spacing
- 12-axle, 438,000 lb truck with a maximum width of 14.5 ft, with 4 wheels across the width.

The 6- and 9-axle permit trucks only occupy a single lane, therefore the analysis for these permit trucks considered HL-93 live load in all other lanes. The 12-axle permit truck occupies multiple lanes and requires traffic restrictions in the direction of travel. Therefore, the analysis of the 12-axle permit truck considered the permit truck only in one direction, with HL-93 live load in the opposite direction. Due to the reduced travel speed of the permit trucks, reduced impact factors are considered in the analysis; 1.20 for the 6- and 9-axle permit trucks, and 1.15 for the 12-axle permit truck.

Girder Design

The I-270 bridge employs the use of variable depth girders, as well as hybrid girder sections at the interior support locations. Given the amount of steel required for the 1970 ft long, 10-girder cross section, the design strived to achieve economy with regard to material, fabrication, and construction.

Flange plate thicknesses are repeated throughout the structure as much as possible, in an effort to reduce the number of plate sizes required to be procured by the fabricator. For the 18 girder field pieces along each girder line, only six different Grade 50W flange plate thicknesses are used, and only for different HPS70W flange plate thicknesses are used. Flange plate transitions were limited to field splices only, except for a flange plate transition on each side of each interior pier. A thicker web is used at the support locations in order limit the number of stiffeners required.

Variable depth girders are employed to reduce the amount of web material, but also to provide appropriate girder depth for the required demands. Table 1 shows the girder depths utilized in each span and at interior support locations. Gradual straight-line bottom flange transitions are used in Span 1 just before Pier 1, on each side of Piers 2 and 3, and just past Pier 4 into Span 5. Figure 6 shows the web depth transition at Pier 1. Straight-line depth transitions are used to simplify the girder fabrication and reduce fabrication costs.

Table 1: Girder Depths Along Each Girder Line

Location	Girder Depth
Span 1 (250′)	8′-0″
Pier 1	11′-3″
Span 2 (440′)	11′-3″
Pier 2	13′-3″
Span 3 (490′)	11′-3″
Pier 3	13′-3″
Span 4 (440′)	11′-3″
Pier 4	11′-3″
Span 5 (350′)	9′-3″

Figure 6: Girder Web Depth Transition at Pier 1, Looking Back Towards Abutment 1

HPS70W is used for the top and bottom flange plates of the interior support girder sections only. As compared to Grade 50W steel, the higher strength steel allows the use of a smaller flange plate to resist the larger interior support bending moments. Since the use of HPS70W reduces the bottom flange width, it also helps to reduce the width of the bearing assemblies at the interior support locations.

For plate girder bridges with long spans, it is important to balance the strength and service design with the need to meet prescribed live load deflection requirements. The girders for this bridge are designed to meet a prescribed live load deflection limit of L/800. The maximum design live load defection in span 3 (490 ft span) is 5.9 inches, which is less than the permissible maximum live load deflection of 7.4 inches. The tighter girder spacing (girders are spaced at 9′-7″), in addition to the deep girders within the span and at the piers as well, provides the stiffness needed to meet the prescribed live load deflection criteria.

The bridge is fixed for longitudinal movement at Piers 2 and 3, which are the wall piers on each side of the canal. As a result, the center of thermal movement is within the middle portion of span 3. The bearing fixity arrangement causes long expansion lengths, and thus large longitudinal movements due to thermal loading at the both abutments and Piers 1 and 4. When the thermal movement is combined with wind on structure and live load movements, the total longitudinal movement is nearly 9 inches at the abutments, and 6.5 inches at Piers 1 and 4. Large longitudinal movements like these cause the bearing stiffener to move significantly away from the centerline of the pot bearings used at each of these supports. As such, auxiliary bearing stiffeners are included in the girder design at both abutments and Piers 1 and 4, and are placed on each side of the bearing stiffener (see Figure 7). The auxiliary bearing stiffeners provide a means for transferring load into the girder when the bearing stiffener itself is offset from the centerline of bearing, or in the case of the abutments when the bearing stiffener moves completely off of the bearing due to longitudinal movements.

Future jacking stiffeners are provided on the girders at each support location as well. At the abutments, jacking stiffeners are provided just “in front” of the bearing, while they are provided on each side of the bearing at each pier (see Figure 7). The jacking stiffeners provide a means to raise the bridge in the future, and allow for removal of the bearings should they need to be replaced. The jacking stiffeners are located to allow jacking assemblies to be placed beneath the steel girder, and bear on the concrete substructure.

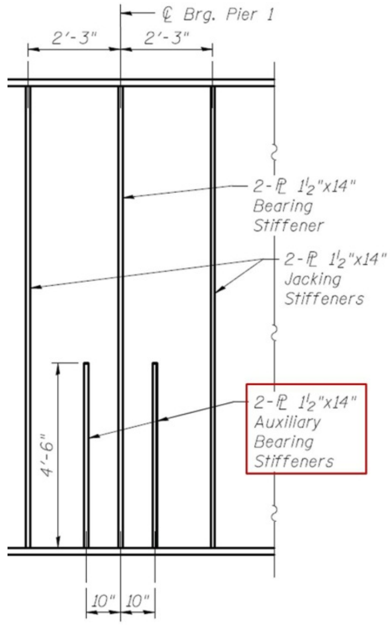

Figure 7: Auxiliary Bearing Stiffeners and Future Jacking Stiffeners at Pier 1 (shown) and Pier 4 (similar**)**

Detail Designs

Intermediate and Pier cross-frames are an X-type shape, due to girder spacing and girder depth which provide a mostly square shape for the cross-frame. WT sections are used for all cross frame members. Based on the analysis, the cross frames are subjected to dead, wind, thermal, live load, and seismic demands. Additionally, the future part-width deck replacement is considered in the design of the cross-frame members. All intermediate cross frames utilize the same WT sections, and girder connection details throughout. Typical Intermediate cross-frames are shown in Figure 8. Larger WT sections are required for the cross-frames located at the piers due to the fact that these cross frames transmit lateral loads to the bearings and substructure.

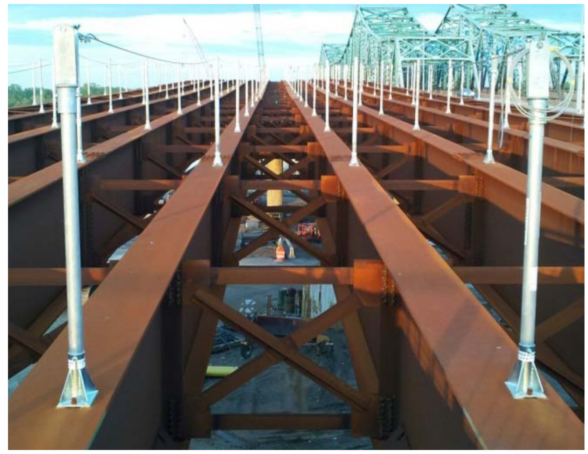

Figure 8: Typical Intermediate Cross-frames

Top flange lateral bracing is utilized in the exterior girder bays along the entire length of the bridge, as shown in Figure 9. The top flange lateral bracing, consisting of WT members, is required while the bridge is being constructed. The top flange lateral bracing prevents excessive lateral movement due to wind at intermediate stages of steel erection, and when the bridge is in the non-composite condition, prior to and during placement of the concrete deck. Additionally, in each span, the steel erection begins with a twin girder system. The top flange lateral bracing adds torsional stiffness and increases global buckling strength of the initial twin girder systems during steel erection. The issue of global lateral stability in multi-girder systems has been reported on by Yura et al. (3). Including the lateral bracing during the design phase should be considered in long span plate girder design projects.

Figure 9: Top Flange Lateral Bracing

Lateral bracing is placed at the top flange level in order to reduce the amount of live load that it is subjected too in the final constructed condition. The lateral bracing participates in the system that resists applied load, and the farther away from the neutral axis the more force it will carry. This is observed in the 3D analysis model of the structure. In the final condition, with the deck in place, the neutral axis is closer to the top flange, and as such, the top flange level lateral bracing carries far less live load than it would had it been placed at the bottom flange level. Additionally, placing the bracing in the top flange level provides more of an increase in the global buckling strength, than if it were located at the bottom flange level, for twin girder systems during steel erection.

The top flange lateral bracing is connected directly to the girder top flanges. The wide girder flanges required by design allow for this connection. The holes caused by connecting directly to the flange were considered in the design of the flange. Connecting directly to the flange eliminates the need for gusset plates, and eliminates additional connection eccentricity at the ends of the bracing members. The same end connection for the WT member to the top flange was used throughout the bridge, providing for efficient fabrication and construction. To compensate for the elevation difference between adjacent girders due to the deck cross-slope, the contract plans showed a member bend detail. In lieu of this detail, the fabricator, Stupp Brothers, Inc., decided to use a thin beveled fill plate at each connection, proving to be less costly than bending the WT as well as less costly than using gussets to attach the bracing to the girder flanges.

The length of the bridge, and the field piece lengths required for shipping, required the use of 17 field splice locations. In an effort to achieve economy in design, only three different plate thicknesses were used for the top and bottom flange splice plates, and only two different web splice plate thicknesses. As much as possible, similar bolt patterns were used at the splices, especially for the web given the large web splices where splices are in higher than normal moment areas. Again, the use of repetitive details can make for more efficient fabrication and construction.

Deck Placement

The placement of the concrete deck is considered in the design of the girders for both checking the girder constructability limit state per AASHTO LRFD, but also for the girder camber. The girder camber is dependent on the sequence of the deck placement. For this structure, the dead load deflection due to concrete varies significantly between the deflection assuming a single monolithic deck pour, and the accumulated deflection due to the deck placement sequence. The maximum difference in camber due to concrete dead load at the midspan locations between a single pour and using the deck pour sequence is nearly 1.0 inch. Therefore, the girder camber due to concrete dead load is based on the deck placement sequence.

As shown in Figure 10, the deck placement sequence consisted of 17 different deck pours. Based on prior experience and conversation with multiple contractors, the deck pour sequence assumed that the maximum amount of concrete that could typically be placed in one day is 450 cubic yards. The width of the deck is 93′-10″, and in combination with the volume of concrete limitation, resulted in a maximum single pour length of 150 ft.

Additionally, to reduce the potential for cracking of the concrete deck during deck placement, a sequence was developed in which the maximum tensile stress in the deck after each deck placement pour was less than 0.90 times the modulus of rupture of concrete, assuming a minimum f'_c of 3500 psi before the next pour could commence. Therefore a deck tensile stress limit of 0.40 ksi was employed for each stage of the deck placement. Using the 3D analysis model, the stress in the concrete deck is tracked for each deck pour. The results are shown in Figure 11, which shows that the limit of 0.9 times the modulus of rupture is not exceeded at any time during the deck placement sequence.

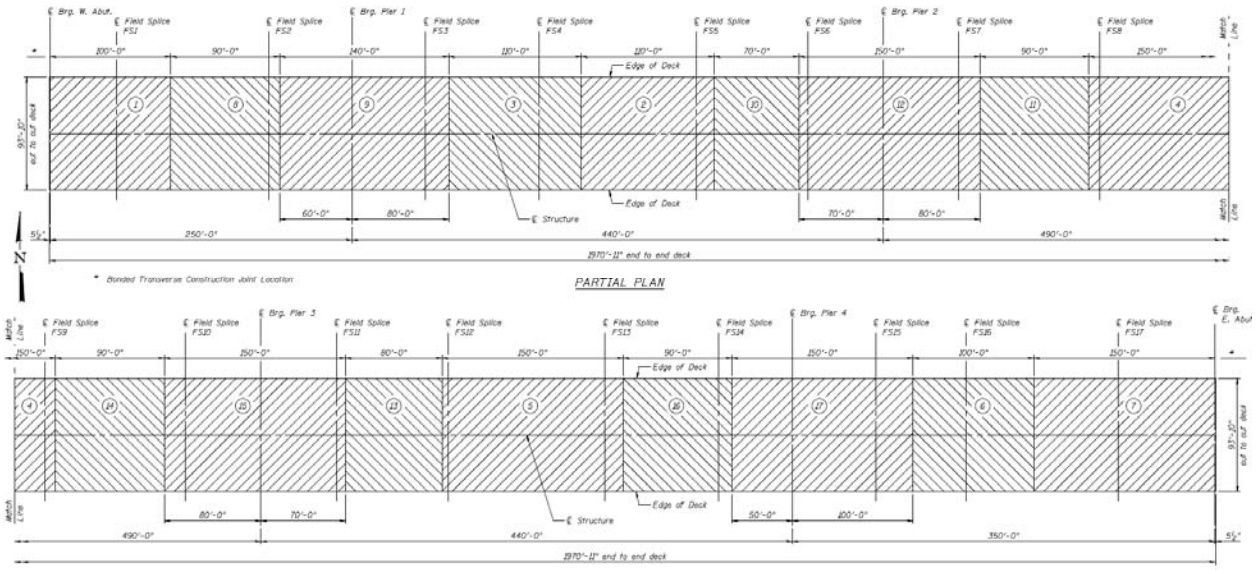

Figure 10: Plan View of Deck Placement Sequence

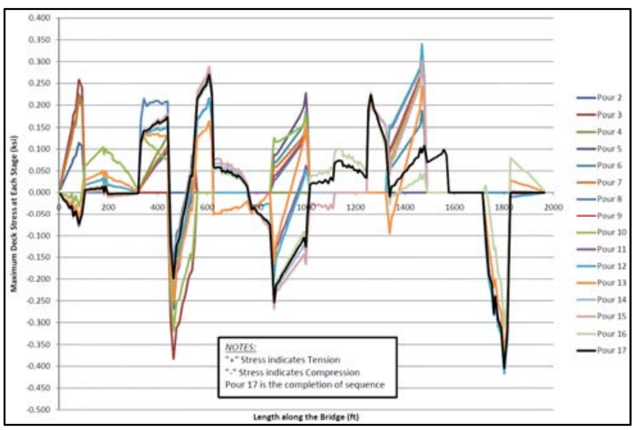

Figure 11: Maximum Predicted Deck Stress for Each Stage of the Deck Placement Sequence

Consideration of Steel Erection During Design

The design of the I-270 bridge considered a conceptual erection sequence during the design phase of the bridge to ensure that the bridge could be erected. The conceptual erection sequence developed during design was provided in the contract plans. The actual erection sequence used in the field by Walsh Construction was fairly similar to the one shown in the contract plans. The placement of temporary support structures had been restricted due to the presence of the levee and saddle dam on the sides of the canal, and the fact that the canal could only have limited shutdowns to construct the bridge. Any temporary supports located in the limits of levee had to be coordinated with the USACE.

No temporary support structures are allowed in the canal due to the navigational clearance required during construction. Additionally, the canal could only be closed for no more than a 24 hour period. These limitations resulted in three assembled center span field pieces (segments H, I, and J) being lifted from a barge by strand jacks, as sketched in Figure 12. The pick length of the three assembled girder pieces is 364 ft.

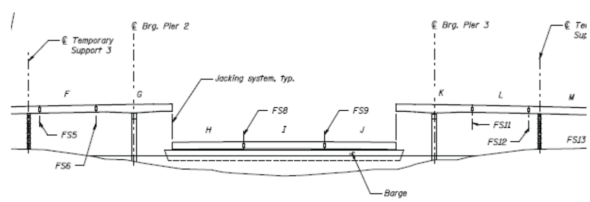

Figure 12: Center Span Lift of Girder Segments

The conceptual erection sequence in the contract plans showed that steel erection could to start at each abutment, and work its way towards the canal from both sides, leaving the center span as the last erected element. Then girder segments H, I, and J could be to be assembled on barge and all 10 girder lines be connected together with cross-frames and lifted with strand jacks as one unit. The contractor chose to break this strand jack lift into 3 pieces, erecting girder lines 1 thru 4 in one lift, then girder lines 5 and 6 in a second lift, and then girder lines 7 thru 10 in the third and final lift.

The second lift included girder lines 5 and 6 being lifted from the barge. Girder lines 1 thru 4 had already been lifted and connected to the rest of the steel superstructure. It should be noted that the contractor had to provide additional temporary bracing at the ends of the two girder lift, due to the potential for global stability issues.

The design considered the conceptual erection sequence shown on the contract plans, to verify that the structure was stable, and that the girders and other structural components such as the cross-frames and lateral bracing were not overstressed. Each stage of the conceptual steel erection was considered in the design. The area of most concern was the center span lift. For this stage, the girder cantilevers were checked not only for strength, but for stability and deflection as well. If the girder tips deflected significantly, the strand jack lift could have been compromised. The analysis showed the girders were sufficient for the applied loads and the defections would not cause issues during the lifting of the center span segments. Figure 13 shows a screen capture of the design model at this particular stage, with point loads located near the cantilever tips, representing the anticipated strand jacking operation.

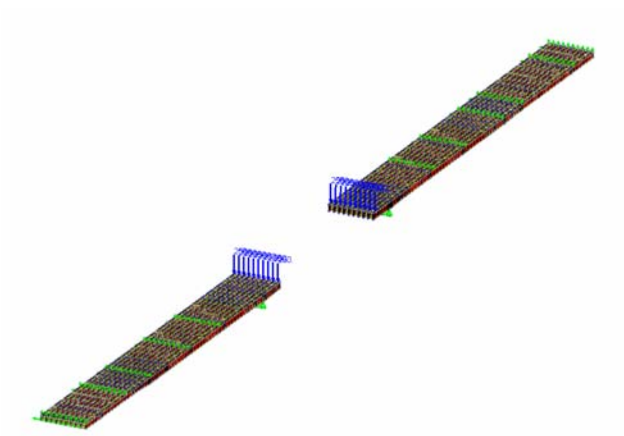

Figure 13: 3D Analysis Model for Conceptual Erection Sequence, Center Span Segment Strand Jack Lift

Construction

The project was awarded on July 19, 2011, and the construction start date was October 31, 2011. By July 2012, the majority of the concrete had been placed for the west abutment, and construction of the Pier 1 had begun with the excavation for the drilled shafts. Cofferdam construction for Piers 2 and 3, on each side of the canal had also commenced. At this time the delivery of the steel girders had also begun, and they were being stored at a local storage yard very near the project site, as shown in Figure 14.

Figure 14: Storage of New Bridge Girders in Local Storage Yard

At the end of September 2012, the Pier 3 cofferdam construction was complete and the Pier 2 cofferdam construction was still in progress. Drilled shafts for Pier 1 had been completed, and the excavation for the Pier 4 drilled shafts was on-going. Structural steel was still being delivered to the local storage yard at this time.

Steel erection began on the west side of the bridge, before the completion of Piers 3 and 4 on the east side of the canal. The first girders of were set on November 15, 2012, with the span 1 girders being set between the west abutment and temporary support structures located between the abutment and Pier 1. The contractor lifted the first two girders of this segment as one pick, as shown in Figure 15.

Figure 15: Erection of the First Two Girder Pieces in Span 1

After the first two span 1 girder pieces were set, the remaining 8 girder pieces were placed with all cross frames as well. Steel erection progressed, field section by field section from the west abutment towards Pier 2, until all the girder pieces above Pier 2 were placed. Steel erection then began at the east abutment and continued field section by field section towards Pier 3. After the girder field sections above Pier 3 were placed, the three field sections that make up the center span were to be erected (See Figure 12).

The three separate center span stand jack lifts occurred between mid October 2013 and early November 2013. Figure 16 shows the structure just prior to the final center span lift, which included three longitudinal girder segments for the final four girder lines. Like all center span lift segments, the girder lines and cross frames were constructed on a nearby floating barge and moved into place below the steel superstructure.

Figure 16: Steel Superstructure Prior to the Final Center Span Strand Jack Lift

At the time of writing this paper (November 2013), the entire steel superstructure was in place. Deck forming, deck reinforcement placement, and deck construction will take place in early to mid 2014. It is anticipated that the new I-270 bridge will be opened to traffic in the summer of 2014.

Summary

The existing I-270 bridges are currently being replaced with a long-span haunched steel plate girder bridge with a total length of 1970′. The new structure consists of 5 continuous spans, with span lengths of 250′ - 440′ - 490′ - 440′ - 350′. The span arrangement was dictated by the need to span the Chain of Rocks Canal and adjacent east flood protection levee. The bridge can accommodate a future lane arrangement of 6 total lanes, and consists of 10 variable depth steel plate I-girders. The new I-270 bridge represents the largest steel plate I-girder bridge in Illinois.

The bridge design utilized a 3D finite element model in order to capture all superstructure component demands for design, including the girders, cross-frame members, and top flange level lateral bracing. The bridge was analyzed for construction loads, including a conceptual steel erection sequence, the proposed deck placement sequence, wind on structure during construction, and future part-width deck replacement.

In long span steel plate girder bridges, particular attention must be given to the use of lateral bracing to reduce wind load deflections during construction, while also providing necessary torsional stiffness and global stability during steel erection. In addition, the designer should consider how the structure may be built by proposing a conceptual erection sequence, and verify that the structural components are satisfactory for that conceptual erection sequence. In the case of the new I-270 bridge, the canal navigational restrictions resulted in the center span segments being lifted into place by stand jacks located at the cantilever tips of the previously erected girders. This stage of erection was investigated during design, to verify that the partially constructed steel superstructure would be sufficient for the center span segment lift.

Furthermore, in long span steel plate girder design, the repetitive use of flange thicknesses, web thicknesses, and details such as cross frame member sizes and connections can lead to time and monetary efficiencies in fabrication and construction.

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References

1. American Association of State Highway and Transportation Officials (AASHTO), AASHTO LRFD Bridge Design Specifications, 5th Edition, Washington, D.C. (2010).
2. Illinois Department of Transportation (IDOT) Bridge Manual. IDOT, Bureau of Bridges & Structures, Springfield, IL, (2009).
3. Yura, J., Helwig, T., Herman, R., and Zhou, C., “Global Buckling of I-Shaped Girder Systems,” ASCE *Journal of Structural Engineering*, Vol. 134, No. 9, September 2008.