BROADWAY BRIDGE TIED ARCHES

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BIOGRAPHY

Ms. McCombs is a Senior Technical Advisor with 22 years of experience in HNTB's Kansas City office. Her project experience includes analysis and design of tied-arch bridges, deck arch bridges, cable-stayed bridges, curved girder bridges and conventional I-girder bridges. She enjoys spending her time mentoring younger engineers and helping people by volunteering as a member of the Technical Advisory Committee for Bridging the Gap Africa, a non-profit organization designing and constructing pedestrian bridges in rural Africa.

Ms. Sarah Larson is an Engineer III with nine years of experience in HNTB's Kansas City office. Her project experience includes analysis and design of tied-arch bridges, deck arch bridges, curved-girder bridges and conventional I-girder bridges. She enjoys working on challenging, complex bridge projects and sharing her passion for engineering with those around her.

SUMMARY

Arkansas Department of Transportation replaced the Broadway Bridge over the Arkansas River along the existing alignment in downtown Little Rock, Arkansas. The bridge was closed to traffic for 6 months to allow construction of the approach spans and to float the new arches into place. This paper will discuss the design considerations of the two basket-handled 440 foot tied network arch bridges over the Arkansas River. During the design process, the designers were met with challenges to satisfy design criteria that compounded and increased construction costs. The initial design criteria allowed for installation of the River Rail Trolley required a floor system with longitudinal composite stringers and transverse non-composite floorbeams. Combined with the initial aesthetic inclination of the arch, the length and depth of non-composite floorbeams drove the height of the floor system and significantly increased the weight of structural steel. An informal value engineering study was performed to evaluate the option of allowing for future expansion of the River Rail Trolley, and the aesthetics of an inclined arch. The study revealed ways to save costs to the project and still satisfy the owner's aesthetic appeal with slightly modified design criteria.

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Project Overview

The existing Broadway Bridge, built in 1922, that connected North Little Rock and Little Rock, Arkansas served the community as a vital transportation link and a tribute to World War I veterans. The existing structure, shown in Figure 1: **Existing Structure**Figure 1, included one steel arch span, three additional concrete deck arch spans and several concrete beam spans. Extensive deterioration of the concrete superstructure caused the sidewalk to be closed. Chunks of concrete were falling from the structure onto parking lots, trails, roads and the river below.

Figure 1: Existing Structure

The structure crosses the Arkansas River, a navigable waterway on a lock and dam system, and carries vehicular traffic into the downtown area with nearly 24,500 vehicles a day and 1% truck traffic. Needing to construct a bridge over the navigation span would contribute to the overall design approach to the project.

In addition, the neighboring communities have an extensive trail system and one of the top enhancements for the new bridge was to add pedestrian and cyclist access with a new 16-foot wide shared-use-path and bridge-to-ground access ramps on both sides of the river. The existing traffic capacity was adequate with four 11-foot traffic lanes, two in each direction, and two 4-foot shoulders.

Constructing a project in the heart of downtown Little Rock would be a challenge. Just like most urban areas, open areas for construction lay-down yards are minimal. Additionally, there are several bridges crossing the Arkansas River nearby with height restrictions further impeding construction access options. Figure 2 shows an aerial view of the project site indicating the number of adjacent bridges and available staging areas for the project. The red highlight shows the Broadway Bridge and the yellow highlights depict the available construction staging areas. The alignment for the new structure did not have any options for relocation so the new structure would be built on the current alignment. Furthermore, the communities wanted to limit the closure time of the Broadway Bridge to six months or less.

Figure 2: Project Limits

Input from the Project Stakeholders was gathered and they wanted the new structure to provide a tourist destination and a gathering place for local users. Pulaski County decided to contribute $20 million to the project for a signature structure.

Structure Type Selection

Many factors were considered when selecting the structure type. A recap of the concerns includes a 6-month closure window, the same alignment as existing structure, a gathering place for the local users, aesthetics, navigation channel restrictions, vertical clearance for the navigation channel, the lock and dam system, and limited construction laydown areas. Structure types considered for the replacement structure included a cable-stayed structure, a plate girder bridge, a delta-plate girder bridge and a tied arch structure.

The cable-stayed option provided aesthetic appeal and could span the entire river. However, the traffic closure window would be more than 6-months and would not be acceptable. Additionally, the costs for this structure were not feasible and it was not selected.

The plate girder option called for an increased structure depth and required a grade raise to meet the navigation clearance. This option lacked aesthetic appeal but was easy to fabricate. Accelerated bridge construction techniques were considered but ultimately the plate girder option was not selected.

A delta plate girder has some aesthetic appeal but restricted the navigation span. The construction of the delta girder extended the construction time and was not selected.

The benefits of the tied arch structure included the aesthetic appeal of the basket-handled arches and a shallower superstructure to minimize the effects of the vertical clearance issues in the navigation span. The construction advantages of using a steel tied arch benefit the project in several ways. An accelerated bridge construction technique, floating the arches in place, significantly reduces the traffic closure window and can be accomplished within the 6-month closure period. The arches can be fabricated off-site. The erection of the structure on barges can occur on the water and does not tie up the limited construction laydown yards. The arches can be floated into place within a 24-hour window in the navigation channel thus reducing the overall impact to the water traffic.

The tied arch steel framing system consisting of two tie girders, two arch ribs, bracing, ten stringers and thirteen floorbeams per span, is a stable structure that can be moved. Steel stay-in-place form work was added to the structure to allow the slab to be cast once the arch spans were set in place. This method keeps the weight of the structure as low as possible to make the float-in a success.

Ultimately, the best bridge type for the site and site restrictions was the tied arch. Two 440-foot basket-handled steel tied arch spans were chosen for the main span river crossing and plate girder spans were used for the approach.

A general layout of the floor system includes stringers spaced at 7-foot 6-inches and floorbeams spaced at 36-foot 8-inches along the length of the tie span.

Original Design Criteria

The initial stakeholder input indicated the original design criteria for the project would include accommodating the future River Rail Trolley and an aesthetic feature of the “nearly touching” two inclined arches.

A River Rail Trolley system is used to connect Little Rock and North Little Rock. The system currently only has one bridge route that crosses the Arkansas River. The stakeholders wanted to be able to accommodate the future River Rail Trolley expansion on the new structure in the future. They wanted a minimally invasive construction method that would allow for lightweight construction equipment to make the modifications to the structure should the need arise. The minimally invasive construction method was to use a thickened slab and allow concrete notches to be cut in the deck to allow for the trolley rails. This idea worked well for the steel plate girder approaches. The girders were parallel to each other and allowed the slab to be thickened over two plate girders to accommodate the additional 8-inch concrete deck as shown in Figure 3.

Applying the same provisions for the future trolley system posed challenges for the tied arch spans. The

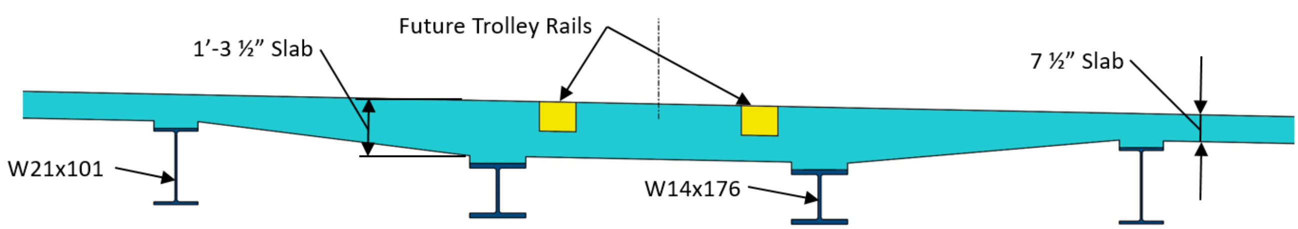

Figure 3: Future River Tail Trolley Cross Section – Approach Span and Original Design Criteria

floor system consisted of longitudinal stringers and transverse floorbeams. The transverse floorbeams interfered with the thickened slab.

The first of three options included a 10-inch uniform haunch. This would allow a 2” haunch over the two stringers under the thickened slab and a 10” haunch over the floorbeams and the eight additional stringers. This posed challenges for the composite beam design with a less than desirable load path and it was deemed unreasonable and not selected.

The second option presented was to use only transverse members that were notched to match the desired slab thickening. The stringer/floorbeam option allowed for a 36-foot 8-inch spacing between floorbeams. The floorbeam only option resulted in at least 3 times the number of floorbeams for a 12-foot floorbeam spacing and a thicker deck to span between the floorbeams. This option increased the overall weight of the structure. Additionally, if stringers were not used, longitudinal slab post-tensioning would be required to carry the elongation (tensile) stresses in the deck. This option was not considered feasible and was not selected for this project.

The selected and last option was to use a floating slab and stringer system. The stringers sit on top of floorbeams as shown in Figure 3. The stringers were composite with the slab and ranged in height from 21-inches to 14-inches under the thickened slab. The stringers sat on top of bearings and bolsters to allow the arch ribs and tie girders for each span to be the same and still account for the profile grade. The floating slab and stringer system did not provide dependable bracing of the compression flange and led to a 7-foot deep non-composite floorbeam design with 7-foot deep tie girders.

In addition, the aesthetic appeal of the “nearly touching” basket-handled arches were appealing to the stakeholders. Due to the vertical clearance on the shared-use path and the roadway, the inclination of the arch ribs required a 25-degree angle shown in Figure 4. This angle combined with the floor system framing defined above, resulted in a center-to-center spacing of the tie girders of 99-foot.

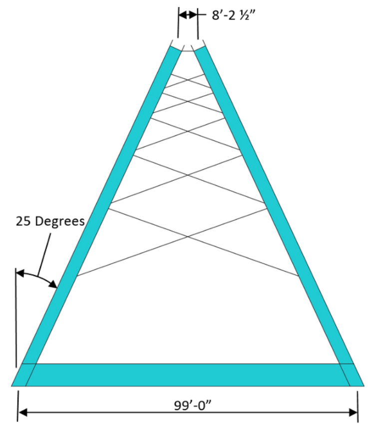

Figure 4: Typical Section - Original Design Criteria

The arch rib inclination lead to an unusual pattern for the arch rib lateral bracing. The spacing of the arch ribs at the apex were near 8-foot from centerline to centerline of arch rib. The bracing continued up the arches with X-bracing with one lateral tie at the apex of the arch as shown in Figure 5. This led to a longer unbraced length of the arch towards the center of the span.

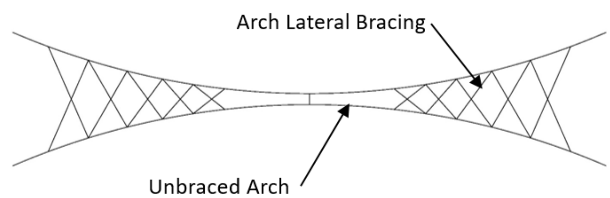

Figure 5: Arch Rib Bracing - Original Design Criteria

Revised Design Criteria

It became apparent that the original design criteria of the minimally invasive future trolley modifications and the aesthetic inclination of the arch was driving the design. An internal value engineering study was performed to see what could be modified to save costs to the project.

The inclination of the arch was modified to an 18-degree angle and the spacing at the apex of the arch ribs increased to about 22-foot. The change in angle reduced the center-to-center spacing of the tie girders to 88-foot as shown in Figure 6. A total reduction of 11-foot for each floorbeam.

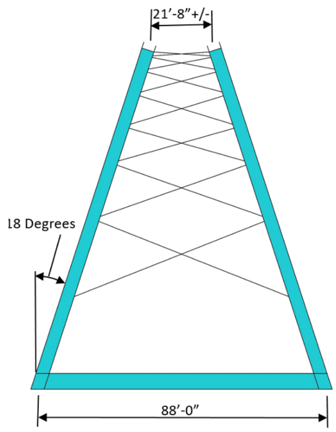

Figure 6: Typical Section - Revised Design Criteria

Increasing the spacing between the arch ribs allowed a traditional approach to the arch lateral bracing to fully brace the arch as shown in Figure 7.

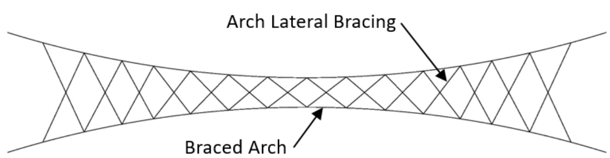

Figure 7: Arch Rib Bracing - Revised Design Criteria

Provisions for the Future River Rail Trolley system were also reviewed to explore other options. The "minimally invasive" criteria were relaxed to allow light-weight equipment to place the trolley support system.

The revised option required the 7 ½-inch concrete deck to be completely removed between two stringers. Each future trolley rail would sit directly on a new stringer that would be installed specifically for the trolley. The slab between the new stringers would be supported on new diaphragms connected to the new stringers as shown in Figure 9.

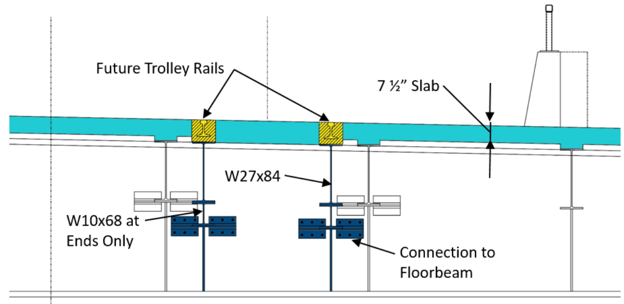

Figure 9: Future River Rail Trolley Modifications - Revised Design Criteria

The benefits provided by this option allowed the stringers and floorbeams to be designed as composite members as shown in Figure 8. The compression flange of the floorbeams were designed as braced which lead to a more efficient design. This resulted in the depth of the floorbeams and tie girders being reduced by two feet.

However, this revised structural arrangement created a new design concern with the floor system. As the tied arch structure is loaded, an elongation occurs in the tie girder. Now that both the slab and the floorbeams were composite, this meant the tie girders and stringers would undergo the same axial elongation from the slab dead load and live load. Using $\Delta = PL/AE$, it was determined that a 15ksi stress would be induced in the stringers due to elongation of the structure. To help save costs and decrease the stringer weight, a special detail was created to relieve the axial elongation in the stringers

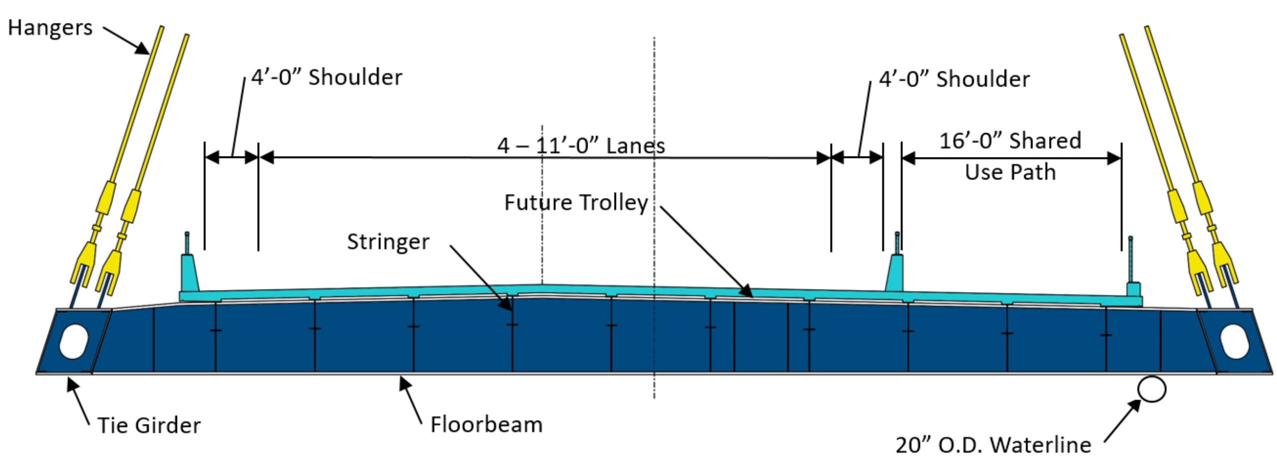

Figure 8: Typical Section of Revised Design Criteria

due to the slab weight.

The stringer to floorbeam connection is shown in Figure 10 and Sections A-A and B-B are shown in Figure 11. The top of the stringer and top of the floorbeam flanges are in the same plane. The plans were detailed to allow elongation to occur at every other floorbeam location. At the fixed connections, all the holes in the connection plates are standard holes. The expansion connection locations have short horizontal slotted holes in the top, bottom and web plates. At these locations, the bolts were installed only finger tight with oversized holes and double washers for a majority of the slab pour. These connections were designed as slip critical connections. The areas over the expansion connections were left open to allow the bolts to be tightened prior to the closure pour placement. This method alleviates the axial stress in the stringers induced by the slab dead load.

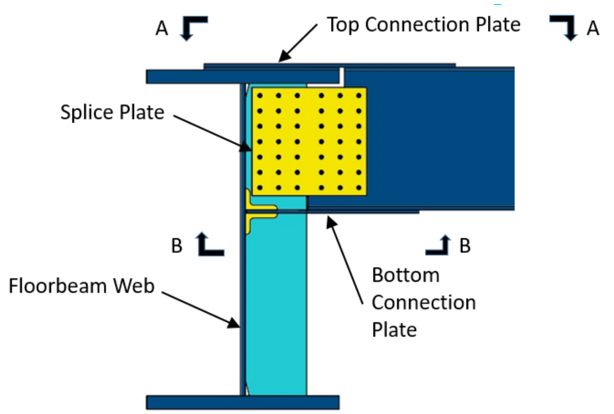

Figure 10: End Floorbeam to Stringer Connection – Fixed Connection Shown

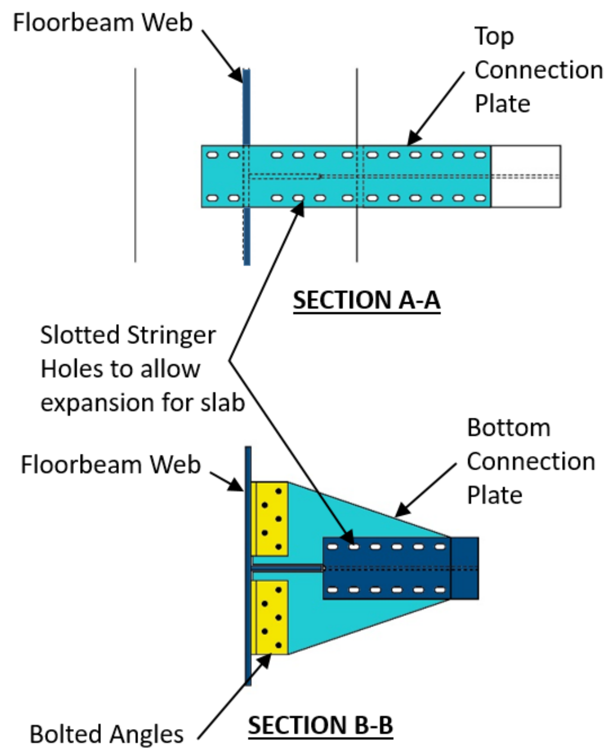

Figure 11: Stringer to Floorbeam Connection Sections – Expansion Connection Shown

To summarize, the revised design criteria helped the structure become more efficient. Changing the inclination of the arch reduced the length of the floorbeams by 11-foot. Altering the future Trolley system design criteria allowed for a composite design of both the stringers and the floorbeams allowing for a 2-foot decrease in the depth of the floorbeams and tie girders. In addition, since the stringers no longer needed to sit on top of the floorbeams, deeper and lighter stringers were utilized resulting in a total project savings of 5,000,000 pounds of structural steel.

The erected structural steel was bid in 2014 for a cost of $3.36 per pound. That cost included the user costs for the closure period and two sets of falsework to construct the arch spans. The schedule did not allow for the reuse of falsework.

Hanger Type Selection Studies

Two options were considered in the selection of the hanger types, bridge strand and stay-cables. Both types are used in arch structures in the United States.

Cost comparisons of stay-cables and bridge strand were performed in the early stages of the design for this project. Three strand diameters and their equivalent capacity stay-cables were sent to manufactures for bid in 2013. The costs per pound were gathered for ASTM A586[1] strand with Grade 2 wire and Class A coating for a 1 ½-inch diameter, 2 ½-inch diameter and a 3 ½-inch diameter strand. The equivalent stay-cable size of 6-strand, 15-strand and 29-strand tendons were compared in the cost study. The cost per pound of material was provided by a stay-cable supplier. For the 1 ½" strand, the cable stay was 2.6 times the cost of the strand. The cable stay equivalent to the 2 ½-inch diameter strand was 1.3 times the cost. For the larger diameter evaluated, 3 ½inch diameter, the cost of the stay-cable was comparable. Looking at the overall area, approximately two 2 ½-inch diameter strands is structurally equivalent to one 3 ½-inch diameter strand.

The cable-stayed strands are encased in a tube to protect it from corrosion. Each stay is made up of several strands that can be stressed to obtain the desired tension at each location. The benefits of these stays include aesthetic features since the coating can be different colors and more studies of fatigue resistance has been performed on these stays. The downside to these stays is that the strands cannot be easily inspected inside the protective coating, short adjustments are more difficult to maintain without causing reseating or bit locations from occurring between the anchorages.

Bridge strand is a standard material used for zinc coated steel wire structural strand and is defined by ASTM A586. Sockets are required at each end of the strand to allow for field connection and small adjustments can be made in the field.

The determination to use either stay-cable or bridge strand was based on the best cost and redundancy factor. The 3 ½-inch diameter strand and equivalent stay-cable were similar in cost. Using two 2 ½-inch diameter strands, the cost difference was about 10% higher but the redundancy factor was gained by using 2 strands per hanger location. The overall hanger cost of the project, as bid, was $2.8M. Ultimately, the bridge strand was selected as the hangers for this project for added redundancy.

ASTM 586 allows provisions for coating the wire with a Class A and Class C coating. Although not specified in the ASTM, Class C coating throughout can be fabricated with a reduction in strength compared to that of a Class A coating throughout. A 2 3/8" diameter strand was specified in the plans and ASTM 586 provides a minimum break strength for a Grade 1 wire with a Class A coating through out of 688 k. A Grade 2 wire with a Class A coating throughout has a maximum strength of 792 kips. A Grade 2 wire with a Class C coating was specified in the special provision with an ultimate tension of 714k. The special provision for the hangers required the strand and socket assembly to be taken to rupture in a tension test. The test set up is shown in Figure 12 and Figure 13.

Figure 12: Hanger Tension Test Setup

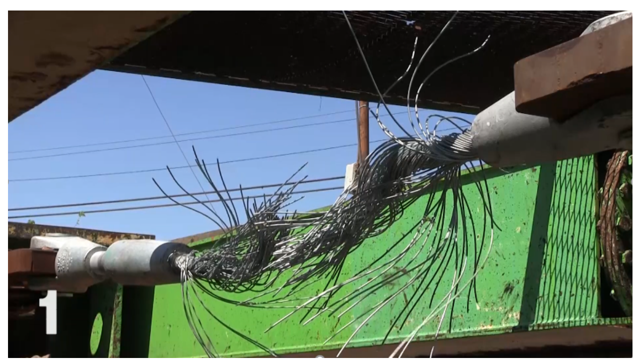

Figure 13: Hanger Tension Test Rupture

The actual break strength obtained for the in-place strand was 789k and exceeded the specified strength of 714 kips for the Class C coating throughout.

The arch rib hanger connection is shown in Figure 14 and the tie girder hanger connection is shown Figure 15. Both the hanger connections include a pin to attach the hanger to the structural steel.

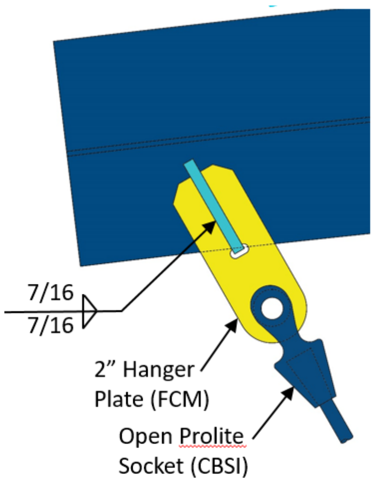

Figure 14: Arch Rib Hanger Connection

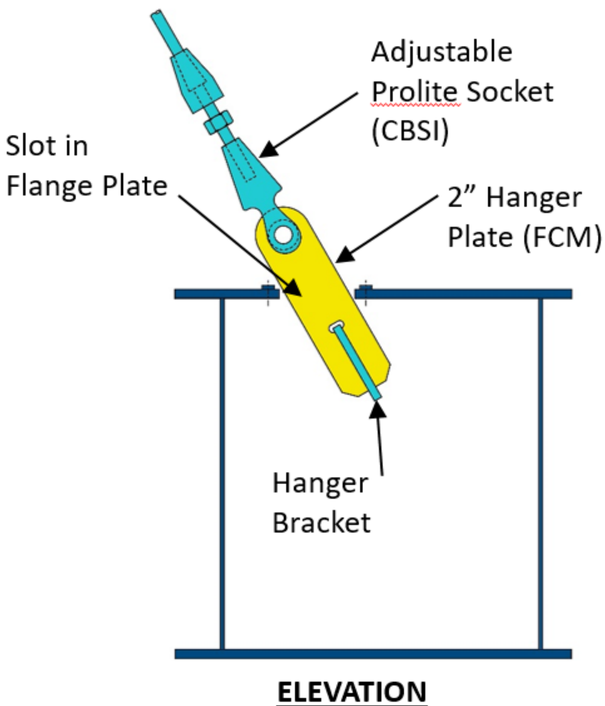

Figure 15: Tie Girder Hanger Connection

The tie girder hanger connection allows for adjustment with the use of a double threaded rod that connects the hanger strand to the hanger plate. To install this socket in the field, the threaded rod, upper hanger socket and lower hanger socket must be aligned in the field. The weight of the socket and sag of the strand added a challenge to be able to insert the threaded rod into both sides of the socket and allow the double threaded rod to pull the two pieces together. Jacks were used to pull the two sockets together while allowing for the threaded rod to be turned.

The selection of the bridge strand hangers allowed for an increase in redundancy for the hangers at a comparable price.

Summary

The informal value engineering study performed provided slight modifications to the stakeholders' design criteria and saved the project 5,000,000 pounds of structural steel. The bridge strand hanger selected provided increased redundancy, increased durability on the strands with the Class C coating throughout and improved ease of inspection for the owner.

References

1. ASTM Standard A586-04a, 2004 (2014), "Standard Specification for Zinc-Coated Parallel and Helical Steel Wire Structural Strand," ASTM International, West Conshohocken, PA, 2004, DOI: 10.1520/A0586-04AR14, www.astm.org.