EVALUATION AND RETROFIT FOR THE SECOND WIDENING OF THE P.R. OLGIATI BRIDGE

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BIOGRAPHY

Dr. Frank A. Artmont is an Engineer with Modjeski and Masters, Inc. He received his BS in Civil Engineering and PhD in Structural Engineering at Lehigh University. Dr. Artmont has worked on numerous design, rehabilitation, research, and forensic investigation projects. He is an AISC member and an active member of the AASHTO/NSBA Collaboration.

Dr. Philip Ritchie is a Technical Manager with Modjeski and Masters, Inc. He received his BS from Drexel University, and his MS and PhD from Lehigh University. His over 25 years of experience includes engineering a variety of bridge design types including arches, cable-stayed, suspension, truss, and girder. Dr. Ritchie's expertise encompasses finite element analyses of complex structures, and steel bridge analysis, design, and detailing.

Dr. Thomas Murphy is a Vice President and the Chief Technical Officer of Modjeski and Masters, Inc. His professional experience has included the analysis, design, and detailing of a variety of bridges with special emphasis on seismic analysis and design. Dr. Murphy has been involved in all stages of the bridge design process; from the development of design specifications, to the completion of conceptual studies, preliminary and final design, and construction stage issues.

SUMMARY

The P.R. Olgiati Bridge is a steel multi-girder bridge that carries US 27 over the Tennessee River in Chattanooga, Tennessee. In 1998, the bridge was widened, and pairs of steel cap beams were added to the faces of the four existing concrete river piers which extended transversely beyond the edges of the piers and supported the new girders. Each pair of cap beams was supported by six saddle beams which spanned between the cap beams and rested on the top of the piers. The cap beams were long enough to support an additional girder line for a future second widening.

In 2016 during the second widening, Modjeski and Masters (M&M) assisted the Tennessee DOT after concerns that the saddle beams supporting the cap beams may be overstressed under unlikely, but possible, loading scenarios. M&M’s detailed evaluation revealed that the concerns were valid and three different retrofit options were subsequently developed to address the issues.

Of these, the addition of concrete filled steel tube (CFST) struts extending from the ends of the cap beams to the piers was ultimately chosen. The CFSTs were designed to be preloaded, thereby redistributing dead load from the existing saddle beams and improving their performance.

This manuscript will cover all aspects of the project, including 1) the evaluation of the existing saddle beams; 2) the evaluation of the retrofit alternatives; and 3) the design, installation, and jacking of the CFST struts.

EVALUATION AND RETROFIT FOR THE SECOND WIDENING OF THE P.R. OLGIATI BRIDGE

Background

The P.R. Olgiati Bridge is a 2645-foot-long, fifteen span steel multi-girder bridge over the Tennessee River in Chattanooga, Tennessee. The original bridge was built in 1953 and consisted of two side-by-side two girder structures of uniform width supported on common substructures, with an open joint between them, each with 26-foot-wide roadways and 5-foot-wide sidewalks. The river crossing portion consists of five spans of lengths 177 feet 9 inches – 276 feet 6 inches – 375 feet 0 inches – 276 feet 6 inches – 177 feet 9 inches, with the 177-foot 9 inch-spans simply supported and the middle three spans continuous. Figure 1 shows the plan, elevation, and representative cross-sections of the original construction.

In 1998 the bridge was rehabilitated and widened. The four southern river spans were widened 50 feet, 25 feet on each side. Each side was supported on two new girders which run continuously over the five spans. The articulation was altered as well, with most of the original expansion bearings converted to pinned bearings, leaving only the expansion bearings at Bents 7 and 12 for the river spans. The open joint between the two original structures was also eliminated. The drawings in Figure 2 show the representative cross-sections of the 1998 construction. Note that accommodation was made for adding an additional girder on each side for future widening.

To support the new girders, two steel cap beams extending 35 feet beyond the ends of the piers were supported by each of the four existing solid concrete river piers. The cap beams in turn are each supported by six saddle beams running between the original girders and bearing on the top of the piers.

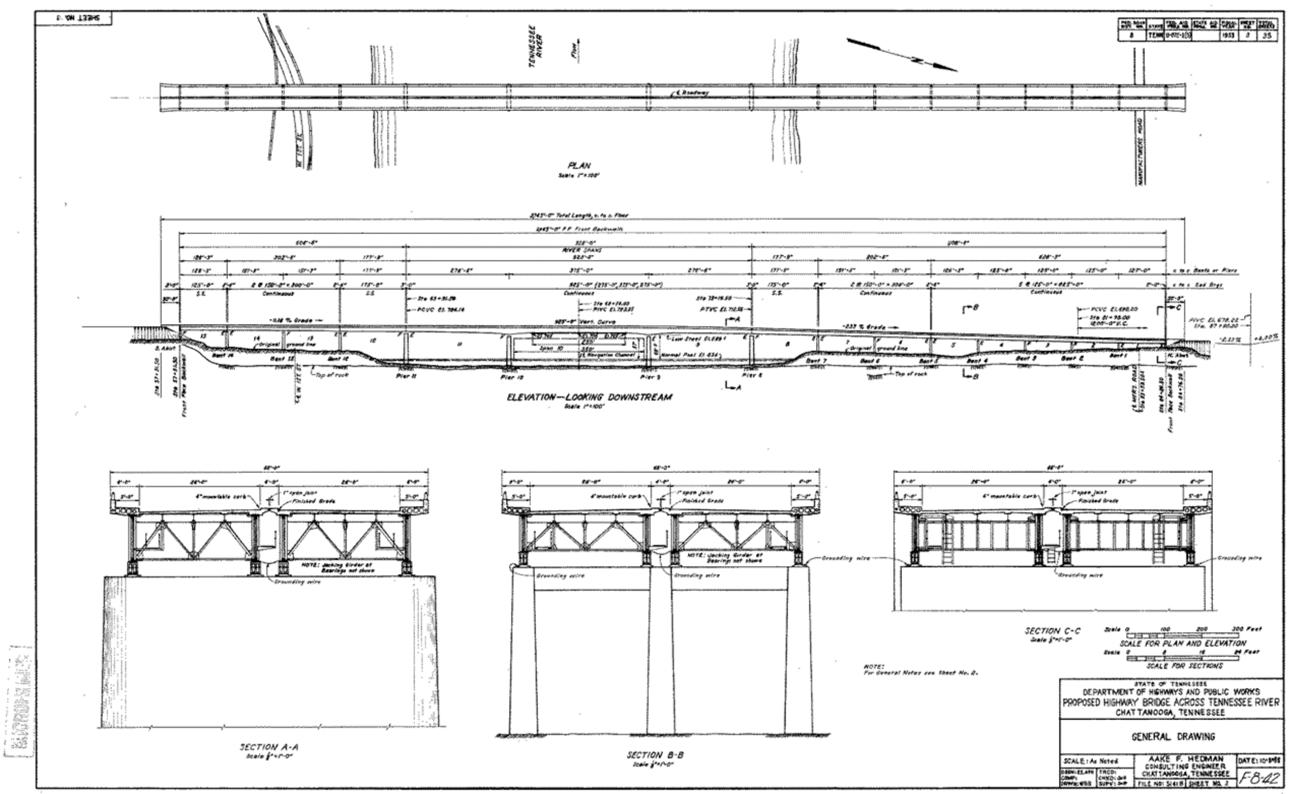

Figure 1 – Plan, elevation, and selected cross-sections of original structure

The new girders bear on support beams spanning between the cap beams in the cantilever portions. Once the new steel was erected, the top of the piers between the cap beams was filled with concrete, encasing the new saddle beams. Anchor rods were installed through the webs of the cap beams into the existing piers. Although not designed as composite, the cap beams would be expected to exhibit composite behavior, particularly under service loadings. Two different depths of cap beams and two different sizes of saddle beams were used: one at piers 8/11, and one at piers 9/10. The cap beams and saddle beams at piers 9/10 were larger due to the larger reactions at these piers due to the longer supported spans. Figure 3 illustrates the details of the steel cap beams at piers 9/10, and Figure X shows a rendering of the cap beam system with labeled components.

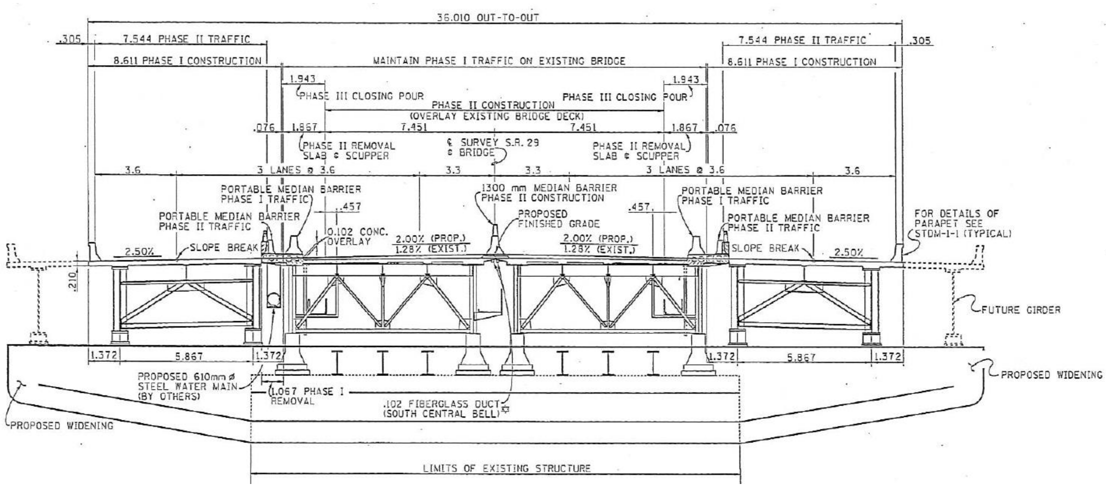

Figure 2 – Cross-section of 1998 widening of structure at piers 9, 10, and 11

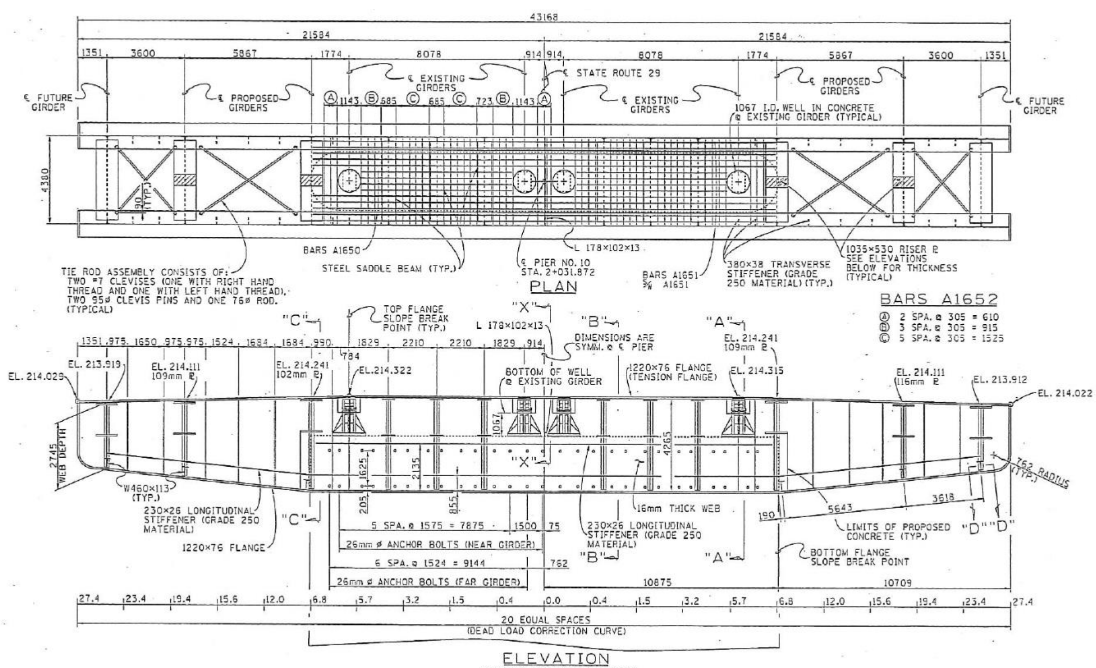

Figure 3 – Cap beam and saddle beam details at piers 9/10

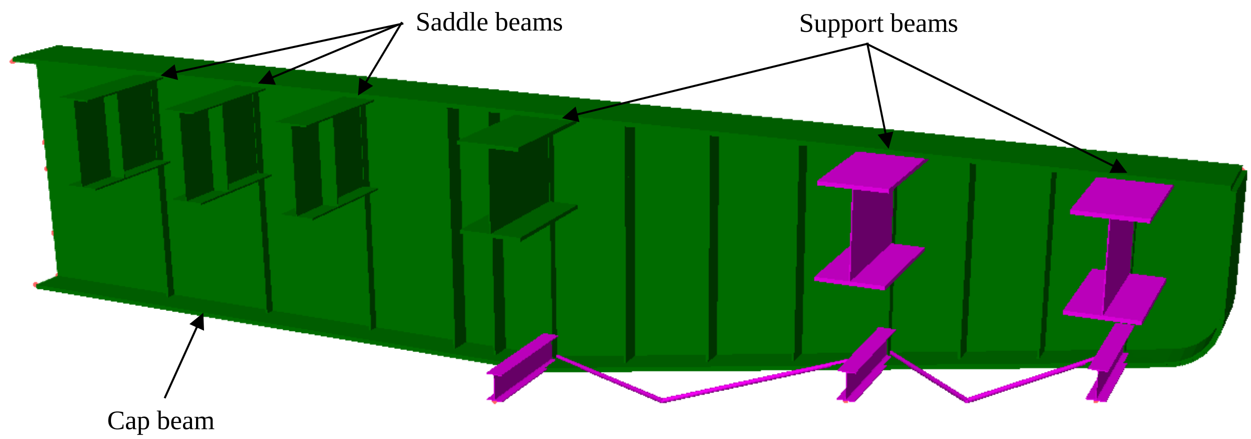

Figure 4 – Rendering of cap beam details at piers 9/10

In 2016, the planned future widening was undertaken. Prior to performing the widening, concerns were raised about the distribution of live loads in the saddle beams, particularly under an eccentric loading, with only vehicles in the cantilever lanes on one side. Two supplementary saddle beams were proposed by TDOT for each pier for additional support, as illustrated in Figure 5. Modjeski and Masters, Inc. was asked to assist TDOT by performing a refined analysis to assess the demands on the substructure as designed.

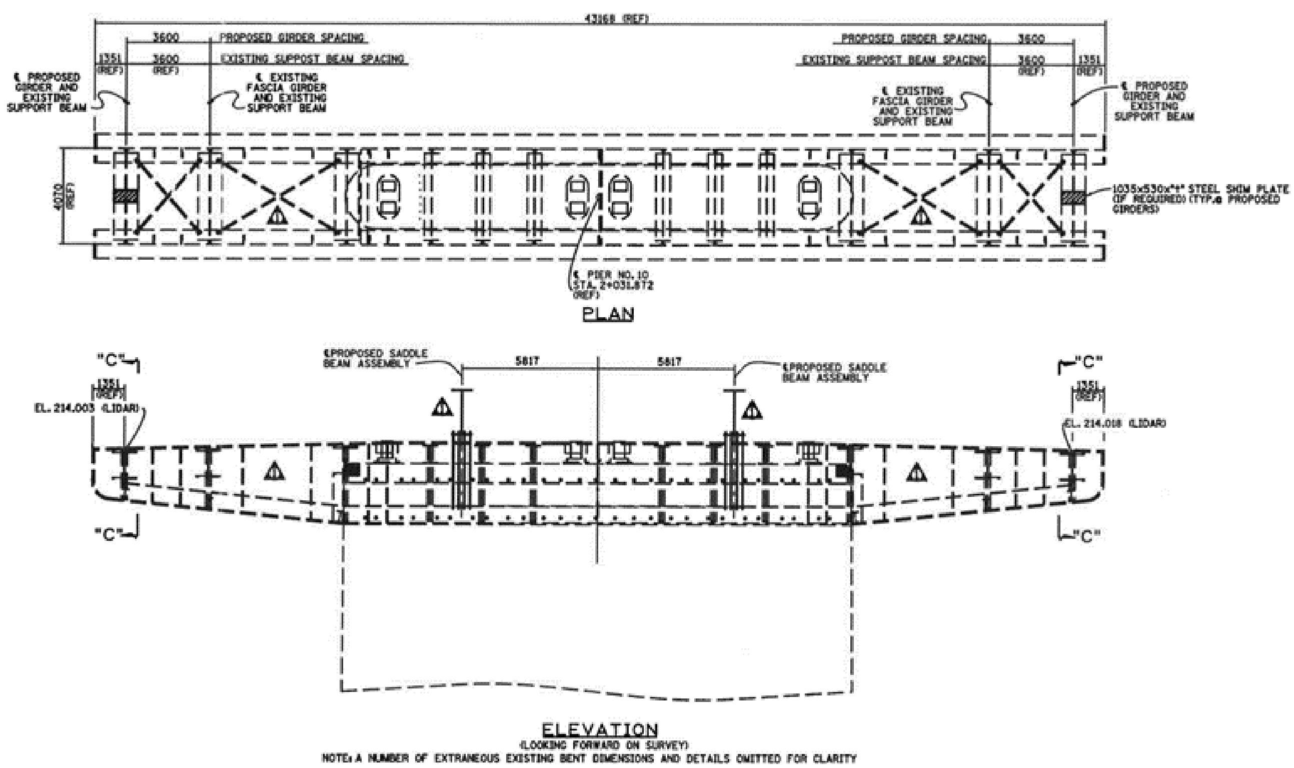

Figure 5 – Details of supplementary saddle beams

Evaluation of As-Built Condition

To determine the level of overstress (if any) in the saddle beams, the pier system was investigated in a series of three-dimensional finite element (FE) analyses using LUSAS. An example of one of these models is shown in Figure 6. The cap beams, saddle beams, and lower portion of the pier (b-region) were represented by shell elements (green), the upper portion of the pier (d-region) was represented using solid elements (blue), and the load beams and lower cap beam bracing were represented by beam elements (pink). Line girder analyses were used to estimate dead and live load reactions from the longitudinal girders. The camber of the cap beams was also included, as it would affect the distribution of load between the saddle beams. Since the pier and loading were symmetric about the pier centerline, symmetric boundary conditions were utilized to reduce the overall computational demands.

Initial analyses, which conservatively neglected the concrete fill between the cap beams, demonstrated that the outermost saddle beam (closest to the pier edge) was overstressed by approximately 2 times under the AASHTO LRFD Bridge Design Specifications' Strength I limit state. If the first saddle beam were to fail, the second would become highly overstressed and so on, leading to a progressive failure of the support of the cap beam. During the original design of the system, refined analysis was not commonly used, and an assumption was made that the saddle beams, being much smaller and exhibiting less stiffness than the cap beam, would distribute the total load evenly between all saddle beams. The FE results did not corroborate this design assumption. To verify the FE results, hand calculations using a "beam on elastic foundation" type approach were performed, which demonstrated that the saddle beams would need to have an order of magnitude less stiffness than their actual stiffness for the load to be distributed evenly.

The finding that the outermost saddle beams would likely be overstressed led to the investigation of three existing alternate load paths: 1) shear capacity of the existing anchor rods, 2) bearing of the cap beam top flange on the concrete fill, and 3) composite action between the cap beam webs and the concrete fill. The first alternate load path was found to be insufficient due to the limited number of existing anchor rods, and the second alternate load path was found to be insufficient due to the limited capacity of the top flange-to-web weld of the cap beam. Additionally, the relatively large stiffness of the concrete fill compared to the cap beam would lead to a progressive failure in either of these load paths (similar to the saddle beams).

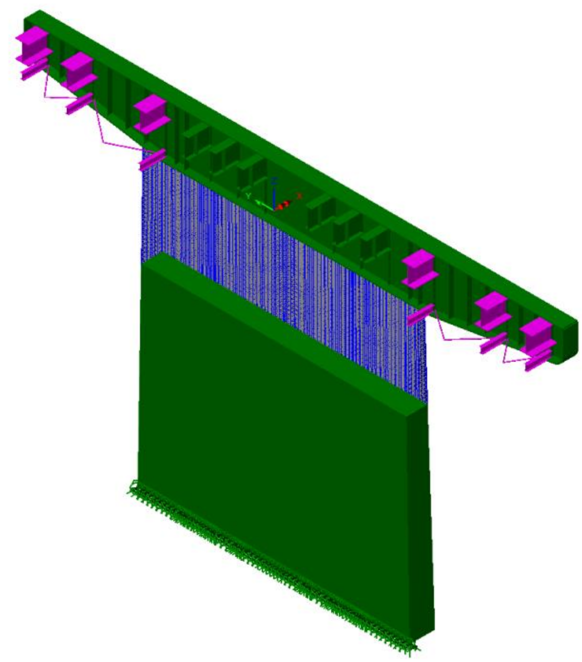

Figure 6 – LUSAS FE model of cap beam/saddle beam system

Good behavior was observed for the third alternative load path, when the cap beam web and the concrete fill were assumed composite. However, composite action could not be guaranteed due to the lack of dedicated shear connectors between the cap beam web and concrete fill.

Given the results of these evaluations, it was clear that some type of modification was required.

Evaluation of Proposed Supplementary Saddle Beams

Prior to M&M's involvement, TDOT had developed a system of supplementary saddle beams, which would be added as shown in Figure 5. In order to evaluate whether this system could resolve the problems with the existing saddle beams, the FE model was modified to include four supplementary saddle beams. For this model, non-composite cap beam behavior was assumed and the rods fixing the supplementary saddle beams to the cap beams were post-tensioned to provide for the best distribution of forces throughout the system. This model is shown in Figure 7.

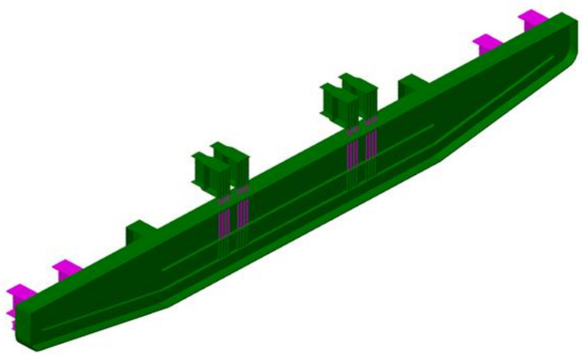

Figure 7 – LUSAS FE model of cap beam with supplementary saddle beams

The results of the analyses indicated that the supplementary saddle beams would not be difficult to design for the required loadings. However, issues would arise with compatible displacements between the cap beam and the concrete fill. If the post-tensioning rods were tensioned too much, the top surface of the cap beam bottom flange would press against the bottom of the concrete during the unloaded condition. If the post-tensioning rods were not tensioned enough, the bottom surface of the top flange would press against the top of the concrete fill. Therefore, in order to properly balance the loads in the supplementary saddle beams while also resolving the compatibility issues, it would be required to remove some of the concrete fill above the bottom flange and below the top flange. Even if concrete fill was removed, it would still be difficult to properly balance the preload in the supplementary saddle beams. Given the difficulty in accessing these areas, alternate retrofit schemes were developed.

Development and Evaluation of Alternate Retrofit Options

Given the practical difficulties with the installation and balancing of the supplementary saddle beams, two alternate retrofit options were developed. During the evaluation process, it was found that composite action between the cap beam webs and concrete fill would be sufficient for carrying the applied loadings. Therefore, the first alternate was based on attempting to enforce composite action between the cap beams and concrete fill. The most practical way to accomplish this was to externally post-tension the cap beams to the concrete fill, by drilling through the cap beam webs and concrete fill and installing post-tensioned rods or strands. However, given the number of holes required and the practical difficulty in drilling the holes, this option was abandoned.

The second alternate retrofit option was to provide large struts that run diagonally from the ends of the cap beams down to the pier edge, thereby turning the cantilevered cap beams into propped cantilevers. This option would provide an alternate load path, while mostly avoiding the compatibility issues that came with the other retrofit options. After consideration of the different types of struts which could be used, the required loading based on preliminary analysis, and aesthetics, a concrete-filled steel tube (CFST) was chosen for final design of the retrofit. The typical section of the bridge with added CFST struts is shown in Figure 8.

Design of the Concrete-Filled Steel Tube Struts

The first step in the design process was to determine the loads acting on the proposed struts. If the strut was installed on the bridge without any jacking, the strut would only carry additional dead load and live load applied to the strut from the point of installation onwards. Given that the saddle beams may have already been slightly overstressed when neglecting the concrete fill, it was desirable to have the struts carry some of the existing dead load as well. Therefore, the struts were designed to be jacked into compression, allowing them to carry some of the existing dead load and reducing the existing load acting on the saddle beams.

To explore the amount of strut jacking load required to provide enough relief to the existing saddle beams, the analytical model was modified to include the struts, and temperature loads were used to provide different levels of preload in the struts. After some iterations, preloads of 1300 kip and 1700 kip were chosen for the struts at piers 8/11 and piers 9/10, respectively. The critical axial loading on the struts for the Strength I load combination was approximately 4400 kip, which included the jacking load. To expedite the design, fabrication, and installation of the struts by using only one set of details, only one size strut was designed.

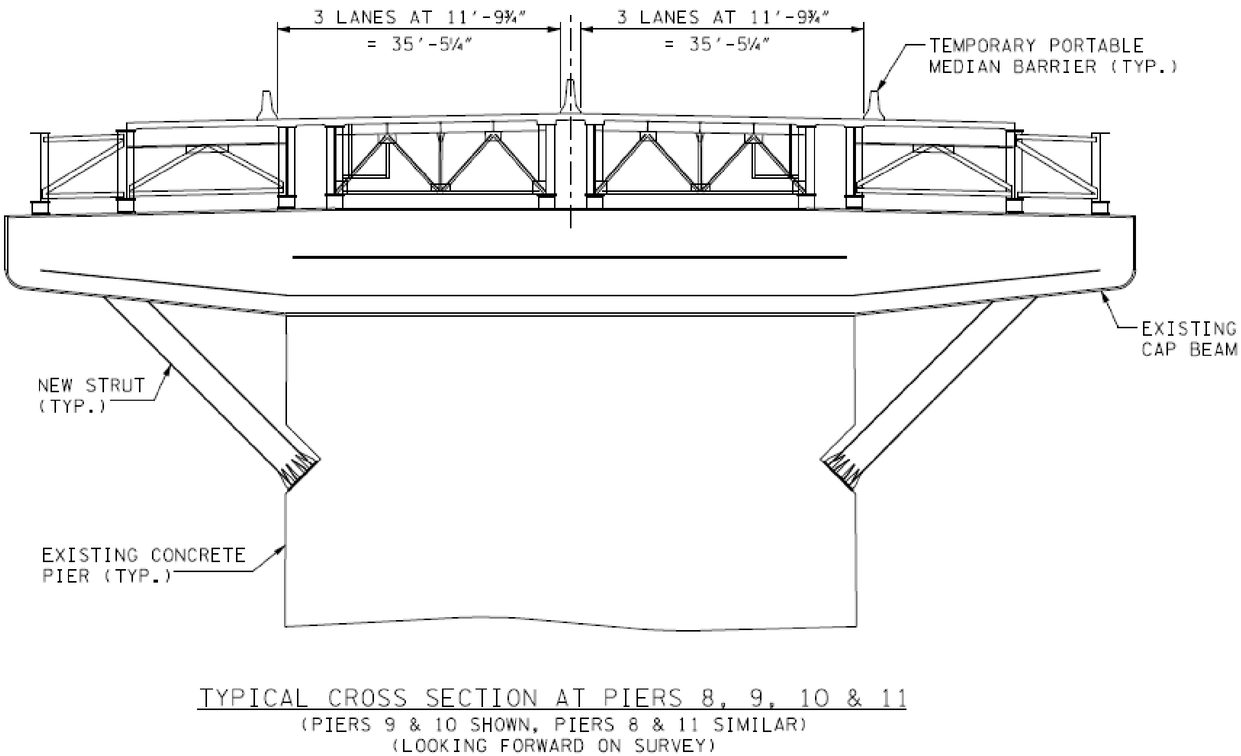

Figure 8 – Typical cross section with proposed CFST struts

The CFST struts were designed using the AASHTO LRFD Bridge Design Specifications, 7th edition, 2014, with 2015 and 2016 interims. The design load and considerations of readily available steel tube led to the use of a 42-inch diameter, $^5/_8$-inch thick ASTM A252 Grade 3 tube filled with 4000 psi self-consolidating concrete. No reinforcement cage was used within the tube.

To accommodate the circular base plate of the new struts, the pier edges were notched, as shown in Figure 8. The concrete was over-demolished by a depth of 6 in. and new concrete and reinforcement was provided to provide a smooth surface at all cut locations, including for the base of the strut. The strut base plate was 62 inches in diameter and $2^1/_4$ inches thick, and was stiffened using 1-inch stiffeners, much like the base of a large light mast. Twelve $1^1/_2$-inch diameter ASTM F1554 Grade 55 anchor rods were used to anchor the base plate to the pier. Leveling nuts and a 3-inch gap between the strut base plate and the concrete were provided to facilitate alignment of the strut during installation. The gap would later be filled with high strength grout after the strut was positioned but before the jacking.

The top connection of the strut to the cap beams consisted of two separate elements. First, an 80-inch deep welded load beam was installed between the cap beams, as shown in Figure 9. This load beam was inclined at the same angle as the strut and bolted to the cap beam webs using 8x8x3/4 angles. A bolted field splice was provided within the load beam to allow for installation between the existing cap beams. The second element of the connection consisted of two matching cruciform shapes which would facilitate the jacking process. The first cruciform protruded from the top of the CFST and the second was welded to the bottom flange of the load beam. These two shapes would be bolted together after jacking, as shown in Figure 10. The use of the cruciform shape allowed four 250-ton jacks to be placed between the strut and the load beam, and for enough bolts of the splice plates to be installed to avoid slipping when the jacking load was released from the jacks and transferred to the cruciform. A rendering of the jacks in one of the quadrants of the cruciform is shown in Figure 11

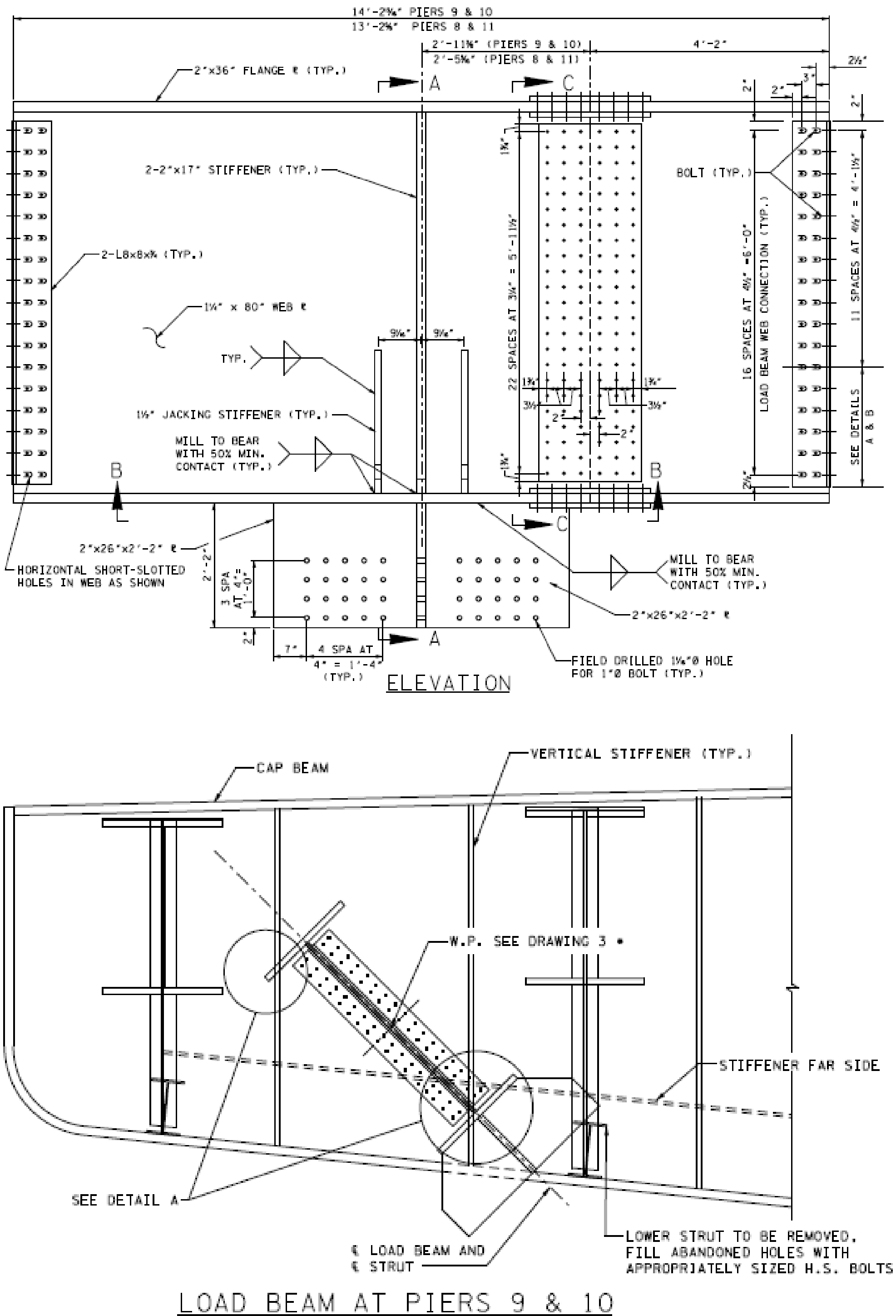

Figure 9 – Load beam details

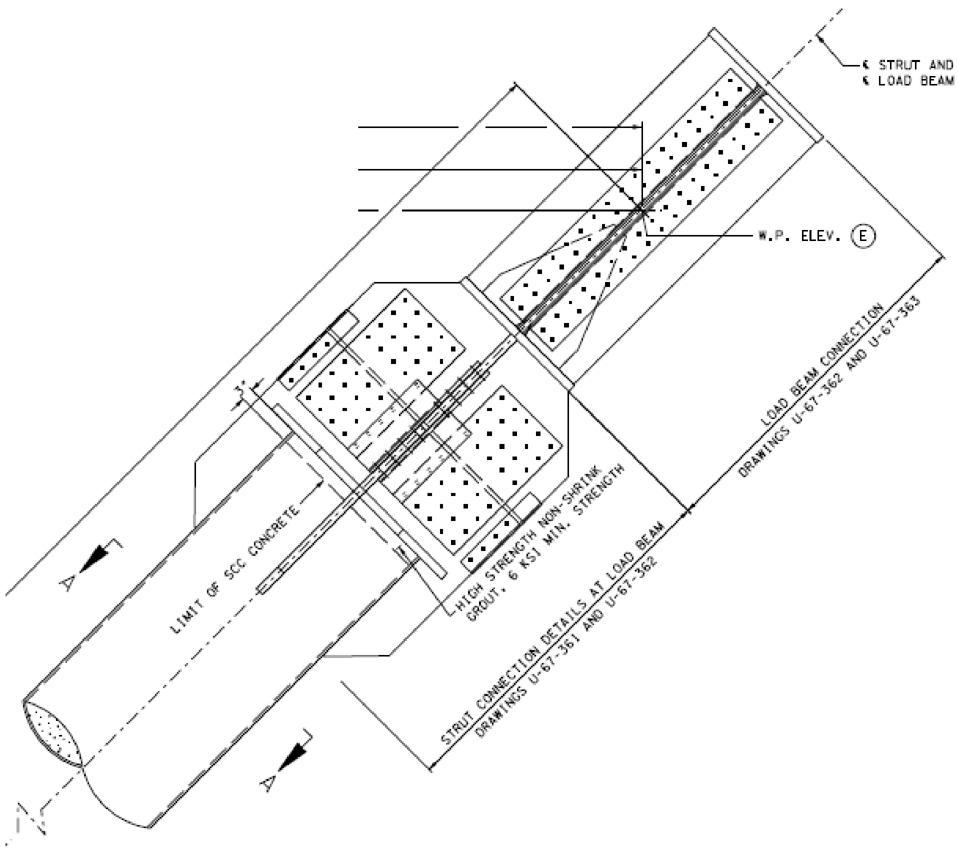

Figure 10 – Connection between strut cruciform and load beam cruciform

Installation and Jacking of the Struts

The first step in constructing the struts was cutting the notch in the pier face. The notches were cut using a large concrete wire saw, which had pulleys mounted directly to the sides and edge of the pier. A pilot hole was drilled at the intersection of the two final cut surfaces, and the cuts were made from this hole to the outer edge of the pier. A third cut was made horizontally from the pilot hole to reduce the weight of concrete required to be lifted out of the cut region at one time.

Following the notch cuts, placement of new concrete, and installation of anchor rods, the struts were lowered into place in their notches. The tops of the struts were suspended from the cap beams until the load beams were aligned and installed.

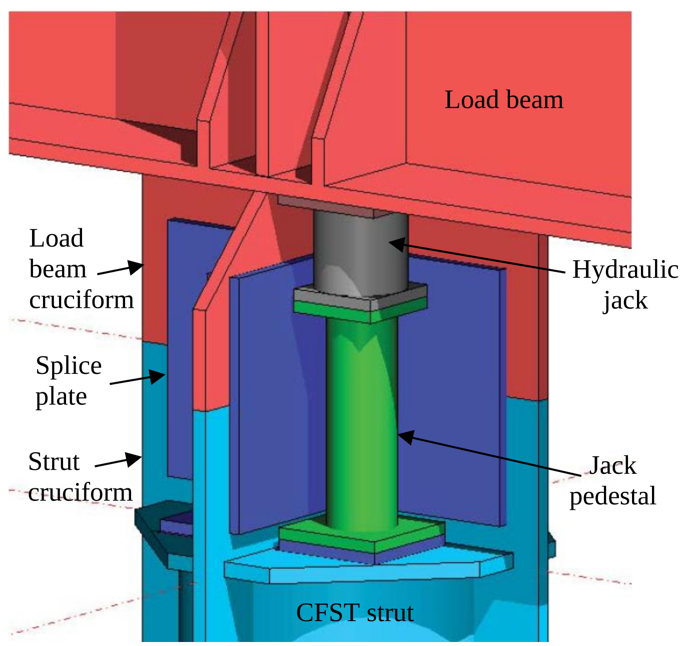

Figure 11 – Rendering showing jack placed in cruciform quadrant

The load beams were installed in two stages. In the first stage, the two portions of the load beams were moved into position between the two cap beams and spliced together. Then, the single spliced load beam was oriented, aligned with the strut, and bolted to the cap beam webs. Once the struts and load beams were in place and aligned with each other, the struts were pumped full of self-consolidating concrete.

After both the strut and load beam were in place, the cruciform splice plates were installed and bolted to the strut portion of the cruciform. The load beam cruciform did not contain any holes, as there was no method to predict the upwards deflection of the cap beams accurately enough to pre-locate the holes. Therefore, the splice plates, which had pre-drilled holes in their top portions, were used as a template for field drilling the cruciform holes once jacking had been completed. After the splice plates were in place, jacks were placed in the four quadrants of the cruciform and connected to a single hydraulic system to ensure even loading across all four jacks. The installed splice plate and jacks in one of the four quadrants are shown in Figure 12. Also shown in this figure is a mini-CFST designed by the contractor to be used as pedestals for each of the jacks.

Figure 12 – Splice plate and jack in one quadrant of the cruciform

To avoid large unbalanced horizontal loads in the cap beams due to the inclination of the struts, the struts on each edge of a pier were jacked simultaneously. The jacking process began by loading the strut on one side of the structure to a load of 100 kips, followed by loading the opposite strut to 200 kips. This process continued in 200-kip increments, keeping the total load at each side of the pier within +/- 100 kips, until the specified jacking load was reached.

Prior to jacking, the gaps at each leg of the cruciform between the top and bottom portions were measured, and measurements were also taken after each increment in the jacking process. The initial gap was to be set at approximately 5/8 inches, and 3/8 inches of movement was expected based on the results of a weighted average of LUSAS analysis results, where 75% of the average was based on the fully composite results and 25% of the average was based on completely non-composite behavior, i.e. completely neglecting the existence of the concrete fill. However, the real behavior of the cap beam-concrete fill system was unknown, so the envelope of allowable movements was between fully composite behavior and completely non-composite behavior. The gaps were plotted in real time by an onsite M&M engineer, and the jacking process was permitted to continue only if the gaps were within the expected envelope. An example of one of these plots is shown in Figure 13. While the first few measurements at each of the piers were generally not within the envelope, later measurements and the average increase of the gap across the four legs of the cruciform generally matched the predicted behavior. Jacking was allowed to proceed to the specified value at all four piers, and the final elongations of all struts were in reasonable compliance with the expected outcome. The pair of completed struts at pier 10 are shown in Figure 14.

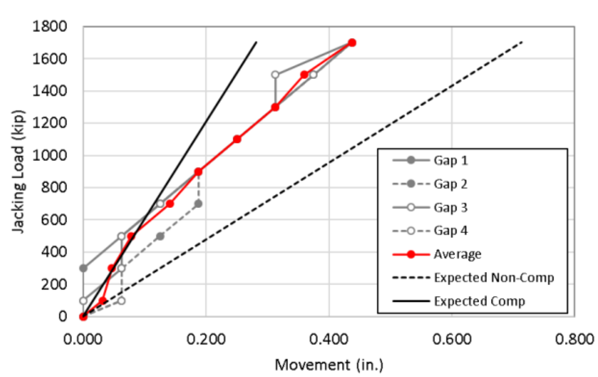

Figure 13 – Typical plot of movements during jacking of strut

Conclusions

This project is summarized in three phases. First, the existing structure was evaluated using advanced analysis techniques and reasonable engineering assumptions regarding the behavior of the structure. This process confirmed that the existing saddle beams could potentially be overstressed. The second phase determined if alternate load paths already existed to accommodate this potential overstress. In order for these alternate load paths to carry the loads from the cap beams to the piers, displacement compatibility between the cap beams and concrete fill was required to avoid significant ductility demands on various portions of the structure. The third phase established a retrofit that could ensure that safe function of the structure without the need to rely on a series of incompatible load paths, none of which could carry the full load unaided by the others. Out of the alternatives developed, the addition of CFST struts to alter the behavior of the cantilevered cap beams to propped cantilevers was determined to be the best solution from a design, construction, and aesthetic standpoint.

Figure 14 – Completed struts at pier 10

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CEI: Volkert, Inc.

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1. AASHTO LRFD Bridge Design Specifications, 7th Edition 2014, with 2015 and 2016 interim revisions.