HEAVY MOVABLE STRUCTURES, INC.
SEVENTEENTH BIENNIAL SYMPOSIUM

October 22-25, 2018

Retrofit of a Rolling Leaf Bascule Bridge – Laurel, Delaware

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Design by AECOM in support of the Delaware Department of Transportation

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MARRIOTT'S RENAISSANCE HOTEL AT SEAWORLD
ORLANDO, FLORIDA

Abstract

Upgrading an old bascule bridge to handle modern loading can prove difficult when the original structure uses lower grade steel in comparison to modern varieties, and light sections without extra capacity to minimize weight of the bascule leaf.

This paper focuses on the bridge design concepts and detailing developed to retain aesthetic elements of an overhead counterweight rolling leaf bascule "Scherzer" bridge as part of the bridge conversion into a modern fixed crossing. The 100-year-old existing bridge carries Central Ave (Alternate Route 13) over Broad Creek in Laurel, Delaware. Recent inspection and rating of the existing structure required a posting of 13 tons and emergency vehicles were prohibited from access. The waterway serves only small craft without the need for bridge openings. The bridge is within the Laurel Historic District and the Delaware Department of Transportation (DELDOT) wanted to retain the major elements of the bridge aesthetics while providing a functional and low maintenance structure.

Design challenges and solutions presented included the retention and stability of heavily corroded bascule through girders, the temporary and permanent support and stability of the overhead counterweight with track and segmental girders, the development of a new micro-pile supported substructure without impacting bridge hydraulics, the squaring of the skewed-end bascule span, the use of standard bridge railings within a constrained cartway, and the use of concrete Bulb-Tee girders and concrete deck within the tight vertical profile of the existing floor system.

Construction on this project began in in February 2018 and project updates and progress photos will be available for the presentation.

Introduction

Figure 1: Bridge 3-152 Central Ave. over Broad Creek

The bridge (BR 3-152) carrying Central Ave. over Broad Creek in Laurel, DE was originally built in 1923. It is a rolling lift bascule bridge, designed by the Scherzer Rolling Lift Bridge Company. See Figure 1 for photograph of the existing structure. The most recent inspection findings identified several areas of deterioration requiring repair and rating of the structure required a posting of 13 tons. The low posting requirement meant emergency vehicles could not make use of the bridge. AECOM was tasked with inspection and design of repairs of the structure in order to remove the posting. As the structure is located within the Laurel Historic District, DELDOT wanted to retain the major elements of the bridge aesthetics while providing a functional and low maintenance structure, without reducing the vertical clearance or hydraulic opening. The waterway serves only small craft without the need for bridge openings.

Figure 2: Underside of BR 3-152, looking northeast

The existing structure of BR 3-152 is an efficient design, but as it is a two girder through bridge, it does not meet modern standards of redundancy and was not designed for modern highway loads. The existing roadway width is 25’-0”, measured to the inside faces of the guard rail. Span 1, the skewed lift span (56’-7” to 72’-3”) is connected directly to the curved segmental girders which support the counterweight. The segmental girders sit directly on top of the track girders which also serve to support the shorter Span 2 (20’-9”). Both Span 1 and Span 2 girders are built up sections, with angle flanges. The Span 2 girders are stiffened and have web cover plates in order to provide the shear capacity required to carry the

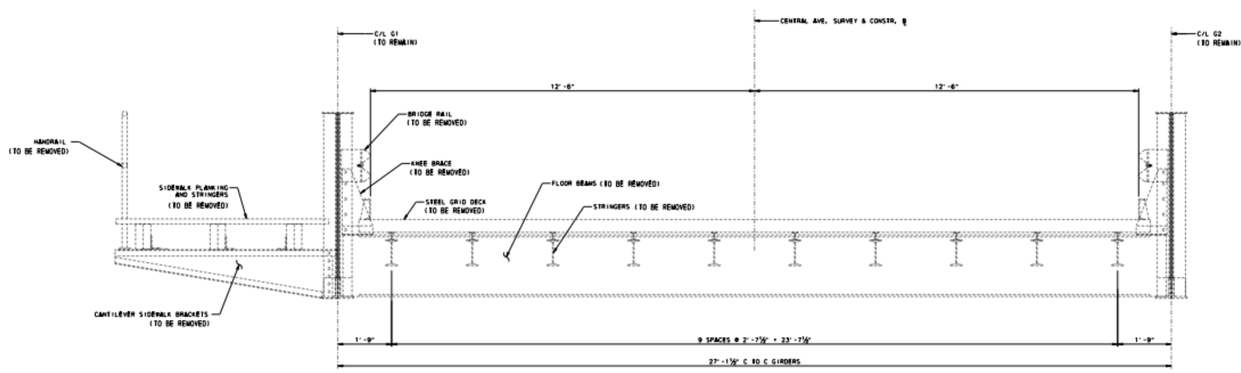

Figure 3: Existing Typical Section – Span 1

entire bridge as it rolls in the open position. In order to reduce load on the bascule leaf, Span 1 has a steel grid deck, 5 3/16” thick. Span 2 has a 9” thick concrete deck, although the stringers are embedded in it, the deck is not composite with either the stringers or floor beams. Both spans have rolled section stringers (S10X25.4) which are bolted to the webs of the rolled section floor beams (WF24X79.9 and WF24X90). The timber deck sidewalk with steel pipe handrails on the west side of the bridge is supported on steel brackets, which are cantilevered off the through girders. The brackets are connected to girders at the floor beam locations, which serve to counteract the overturning moment that would otherwise cause the west through girder to be unstable.

The existing substructures consist of a north abutment, south abutment and two unconnected pier columns, all supported on timber piles, as per existing plans. The size and layout of the piles is unknown.

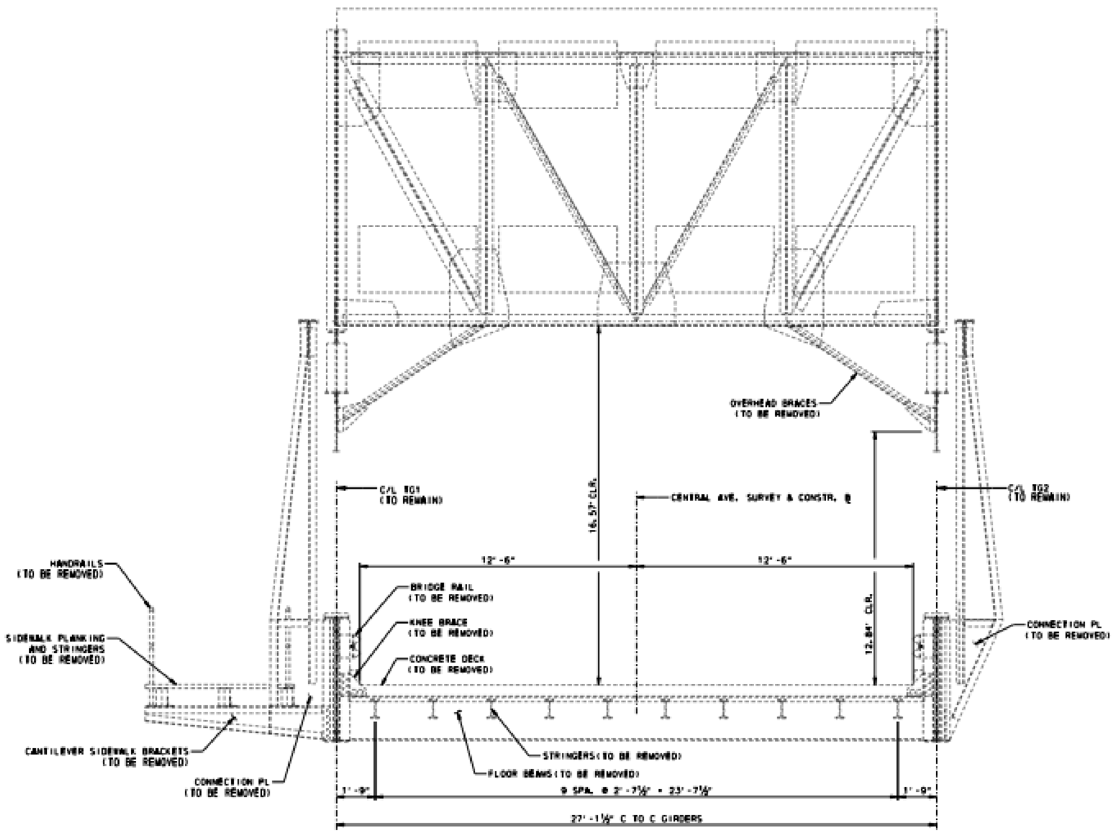

Figure 4: Existing Typical Section – Span 2

The guard rail on the existing bridge consists of a W beam rail directly attached (bolted) to the stiffeners on the through girders and the decorative concrete balustrades on the wing walls of the abutments. The configuration with the guard rail directly connected to the through girder is not a crash rated system and moreover, since it would transfer any force from an impact to a fracture critical member is not a safe system for the long-term.

Initial Investigations

Inspection

Inspection in preparation for repairs was conducted in November 2013. As a whole, the superstructure was in fair condition. Above the roadway level, the stringers, floor beams and girders are in good condition, but below the deck deterioration and section loss are readily apparent.

The stringers and floor beams in Span 1 have been replaced in a previous rehabilitation and are in good condition. The stringers and floor beams in Span 2 had not been replaced, most likely due to concerns over stabilizing the track girders during construction, while the floor beams were removed. Instead, the floor beams in Span 2 had cover plates welded to the bottom flanges. There were also cover plates located at the top flanges, but during inspection, these were determined to be ineffective as they are not actually connected to the top flanges of the floor beams. Interestingly, the floor beam at the north abutment is actually bearing on the beam seat for its full length. The stringers and floor beams in Span 1 and the stringers in Span 2 are in good condition. The floor beams in Span 2 have section loss to the webs near their connections to the girders and the bottom flanges at the bracing connections. Similarly, the bottom flange bracing in Span 1 is in decent condition, while the bracing in Span 2 is in poor condition, including areas of severe deterioration and loose connections.

Figure 5: Bracket Connection to Through Girder

The bascule girders in Span 1 received several repairs during a previous rehabilitation, which included replacement of rivets with high strength bolts, complete replacement of the bottom flange angles and the complete replacement of the last few feet of the girders adjacent to the south abutment. Despite the previous rehabilitation, the Span 1 girders still exhibits minor section loss to the web above the bottom flange angles. Additionally, there is significant section loss to the web where the floor beam connections bolt on. During the original construction, holes were fabricated in the webs of the west through girders in order to allow for the passage of splice plates between the sidewalk brackets and floor beam top flanges (See Figure 5). These holes exhibit significant section loss, with knife edging.

Figure 6: Bottom of West Span 2 Girder at Pier

The track girders in Span 2 are original construction, with no previous repairs, with the exception of jacking stiffeners which were added when bearings at the abutment were replaced (See Figure 6). The webs of the track girders adjacent to the bearings exhibit areas of holed through section loss. The bottom flanges of both girders exhibit areas of 100% section loss at all floor beam connection locations and above the substructure.

Figure 7: Counterweight, Rack Frame and Segmental Girder

The overhead portion of the structure is also in fair condition overall. The segmental girder is in good condition, with no significant areas of deterioration or section loss. The main truss members connecting the counterweight to the bascule span girders are also in good condition. However, the bracing between the two trusses exhibits pitting, with areas of the 100% section loss in some of the machinery support frame members.

The substructure is in fair condition. There are no signs of settlement or other global overstresses in the substructure. However, there are areas of map cracking at the pier columns and north abutment directly beneath the girder bearings which require repair.

Finally, the decorative balustrades on the abutment wing walls, which have actually already been fully replaced in a previous rehabilitation, also exhibit heavy cracking. This could be from a number of causes, possibly including impact damage, poor concrete mix used during construction and/or a lack of adequate shrinkage and temperature reinforcement.

Rating

Reevaluation of the ratings based on the inspection conducted for this rehabilitation did not result in a change to the 13 ton posting. Ratings were performed using BRASS LRFD, which is DELDOT's preferred rating software. The track girders in Span 2 had ratings over 1.0, despite the significant section loss to the web and bottom flange, because they were originally designed to carry the full weight of the bridge as it rolls into the open position, which significantly outweighs typical design vehicles. The stringers, and floor beams in both spans and the bascule girders in Span 1 all had ratings below 1.0. The controlling ratings were closely matched by the floor beams in Span 2 and the bascule girders in Span 1. Despite the section losses in the webs of both members, the ratings were actually controlled by flexure.

Design

Superstructure Alternative Investigation

The initial alternative investigated was the repair of the existing structure, while maintaining its existing structural configuration. The stringers and floor beams could successfully be replaced with rolled sections made of modern grade steel, and made composite with a concrete or filled grid deck. The bascule girders would have required the installation of multiple flange cover plates, on both the top and bottom flanges even when reinstalling a steel grid deck instead of a concrete deck.

The benefit of this option was that it would maintain the existing look of the structure with minimal changes in the dead load. However, installation of cover plates would be challenging as it would require the temporary removal and modification of the double angle tension members which run from the center of the bascule girders to the counterweight support structure. As the tension ties help to support the bascule girders, the bascule girders would require further strengthening without them. Even complete one for one replacement of the girders would have been equally challenging. With this option, all three substructures would also need to be strengthened, as they were not designed for modern live loads. This would have also left the Department with a structure that was still fracture critical.

Other alternatives included the removal of the existing deck, stringers and floor beams, and installation with a composite deck beam bridge, contained between the existing through girders. The new superstructure system would remove the fracture critical designation of the superstructure, while minimizing changes to its outward appearance. The difficulties of this system were in providing sections stiff enough to meet AASHTO live load deflection criteria, while staying within the existing structure envelopment and not significantly reducing overhead clearance. Changing from a grid deck to a concrete deck will also add depth due to the 2% minimum roadway cross-slope required for drainage. Due to the difference in depth between any new girders and the existing through girders, it is not feasible for the new girders to match the stiffness of the existing girders. This differential stiffness could result in unequal deflections between the longitudinal members and overstress the deck or diaphragms. Therefore, the new system would need to be isolated vertically from the through girders, while still allowing the through girders to be braced by the new superstructure. This lateral bracing system is shown in Figure 10. By using short slotted holes in the connection, the superstructure is allowed to deflect without shedding load to the through girders. In order to minimize the size of the new girders, it is necessary to reduce the span length by installing a new wall pier between the two existing pier columns. Due to the unequal span arrangement, it is not possible to take advantage of continuous construction, as this would result in an uplift force on the north end of the girders. Instead, a link slab would be used in order to avoid putting a joint that will need to be maintained and could leak into the new superstructure.

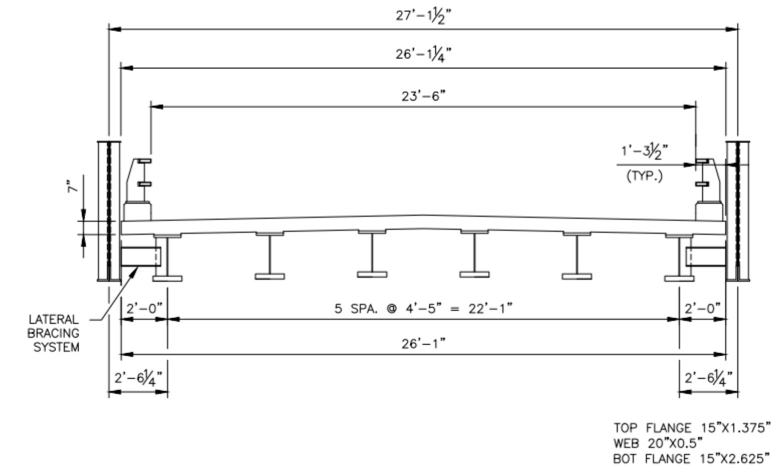

Figure 8: Steel Girder Option (Not Selected For Final Design)

The design team investigated steel beams first. The result was short heavy sections, with thick bottom flanges. These weren't efficient sections, and the depth of the girders would make diaphragm installation challenging. As one alternative, the design team investigated NEXT beams, due to their shallow profile. However, further investigation determined that after including final beam camber, NEXT beams would be deeper than the steel girders or other prestressed girder options. After further investigation it was determined that a 29 inch deep Bulb-Tee girder with a 7 in composite concrete deck was the more efficient prestressed girder option. Initial price investigation also determined that it would be more efficient than the steel girder alternative. Although the second span is significantly shorter than the first, the same beams were used for consistency and due to the fact they were the shortest standard Bulb-Tee's available. The new superstructure depth would be 36 inches, not including haunches, camber, or cross-slope. This resulted in a depth significantly

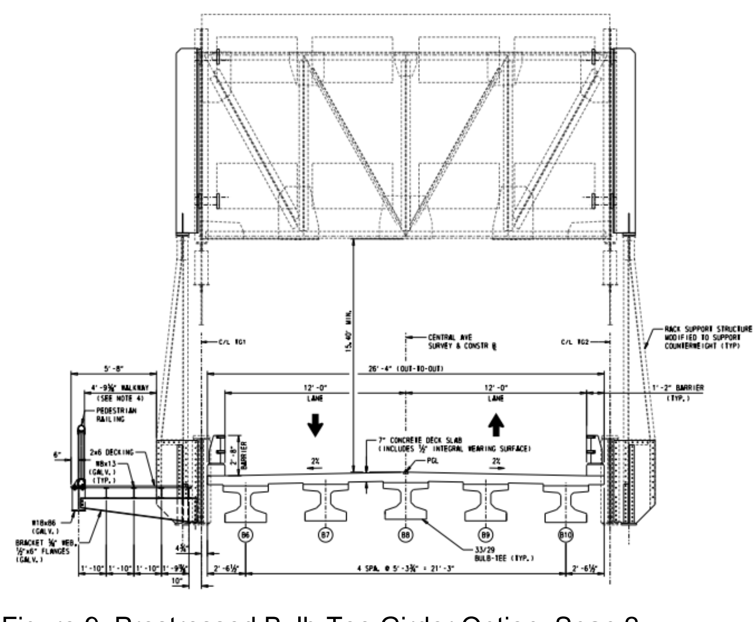
Figure 9: Prestressed Bulb-Tee Girder Option, Span 2

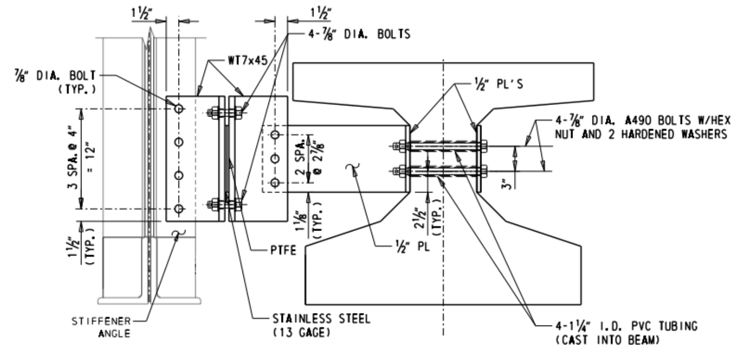

Figure 10: Lateral Bracing System

deeper than the distance from top of deck to bottom of girder in the existing structure, reducing the vertical clearance to the underside of the existing counterweight. However, the clearance on the existing bridge is actually controlled by two sets of braces supporting the overhead structure. These braces served to help stabilize the structure before the counterweight concrete cured, support the machinery which would no longer be used, and provided stability to the structure while it was in the open position. After transforming the bridge into a closed structure, the overhead counterweight and machinery trusses only need to support themselves. After analysis, it was determined that these braces could be removed without compromising the integrity of the existing structure, and therefore the overhead clearance of the structure would actually be improved, even with raising the roadway profile.

The new barrier will consist of a crash tested, curb mounted rail and post. The new barrier is TL-3 rated and was selected in order to minimize any reduction to the roadway width. In order to maintain 12'-0" lanes, it was necessary to notch the barrier at the locations of the through girder stiffeners, which would otherwise be cast into the barrier.

The sidewalk on the west side of the bridge will be composite timber deck to match the timber deck on the existing structure. Due to the isolation of the existing through girders from the new superstructure, the west girder could no longer support the sidewalk with cantilevered brackets. Instead, a stringer and tapered floor beam system designed to resemble the existing tapered brackets will support the new sidewalk. In turn, the floor beam will be supported by the existing west girders and a new edge beam. This edge beam will be supported on extended beam caps which will be added to the existing substructures (See Figure 12).

Although the existing bridge deck is flat, the roadway at the south end slopes down towards the bridge. The original bridge was an open deck structure, so drainage from the approaches was not a concern. Now that the bridge will be a closed deck, scuppers were added to provide drainage. With the reduced roadway width, the scuppers would need to be placed partially underneath the barrier. This was achieved by fitting the scuppers within the standard side slot detail for this barrier.

Counterweight Modifications and Substructure Design

The isolation of the new superstructure from the existing through girders leaves the existing structure in an unbalanced condition. Per the existing plans, the entire rolling leaf weighs 440,800lbs, including the counterweight (See Figure 11). This does not include 15,000lbs of balance blocks that were previously removed from the structure. The counterweight itself weights 273,400lbs and is centered 11.80ft from centerline of pier for an eccentric load of 3,226.12ft*kips. This eccentric counterweight load would either need to be supported, or removed. Permanent removal would modify the look of the existing structure, which is against the goals of the project and would require the construction of a false counterweight. Additionally, the segmental girders would need to be stabilized during demolition and reconstruction.

After additional investigation, it was determined that the existing rack frame could support the counterweight with minor modifications. These included directly connecting them to the counterweight (See Figure 9), adding a tension tie and building new bearings at the base of the rack support columns. A second redundant system to resist the counterweight eccentricity is provided by new anchor bolts at the existing south abutment, which are designed to resist the full uplift force from the counterweight. A third redundant system is the new bridge superstructure itself. Although the lateral braces allow for differential movement, any large movement of the girders would be stopped by the connection to the new prestressed girder superstructure, which outweighs the old floor beam and stringer system.

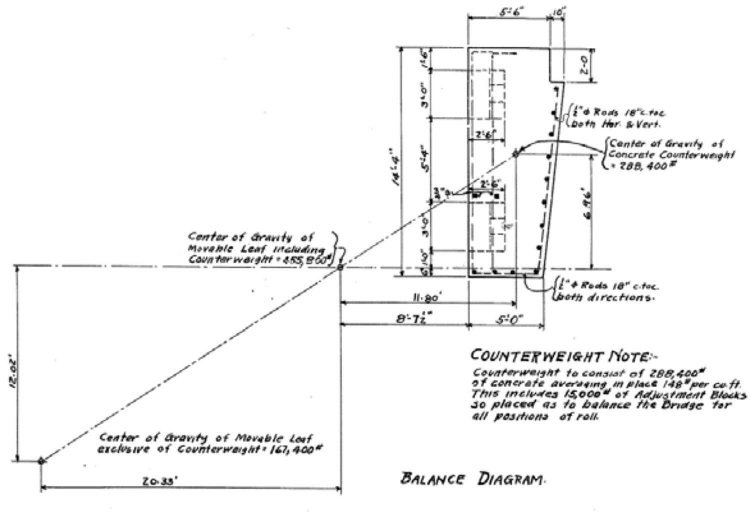

Figure 11:Balance Diagram for Existing Structure

The modification to the rack frame spreads the counterweight load between the pier and north abutment. Both the existing pier and north abutment were designed for the entire weight of the bridge superstructure. The pier columns when the bridge is in the "closed" position and the north abutment when the bridge is in the open position. These two substructure units are in decent condition, and by spreading the load out between them, it is possible to add the additional load from the new superstructure and increased live load from removing the posting without actually increasing the design load to either unit. As the north abutment is already full width, it can be adapted to accommodate support for the new superstructure with minor beam seat modifications. As previously mentioned, a new infill wall between the existing pier columns will support the new superstructure at the pier. This infill wall will be supported by micropiles, which have been arranged so they can be installed through the existing bridge deck.

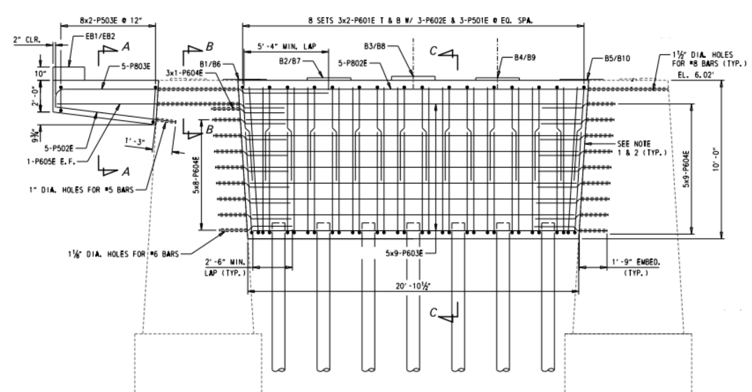
Figure 12:Pier Modifications

The existing south abutment was only designed to carry a tributary portion of the bascule span dead load, outdated lower live loads, and impact from the closing of the bascule span. Without accurate knowledge of reinforcement and pile placement, the capacity could not be evaluated or relied on to carry additional loads. Therefore, more extensive modifications were required. The easiest way to modify the existing south abutment was to build the new abutment behind the existing abutment, while using the existing abutment as shoring for any excavation. However, this extended the span length to 75ft, resulting in a deeper superstructure. In the end, this was still the preferred option because other options would have more extensive demolition and excavation around the existing structure, more challenging temporary supports and more work in the creek. By building a new abutment it was possible to remove the skew from the superstructure. The new abutment would also be supported on a single line of micropiles, which could be installed between the existing wing walls. Although the south abutment is at the expansion end of the span, and therefore only sees minor longitudinal loads, with only a single line of micropiles, there is not much longitudinal stiffness to the pile system. Therefore, soil straps were added to the back of the abutment to resist the longitudinal loads.

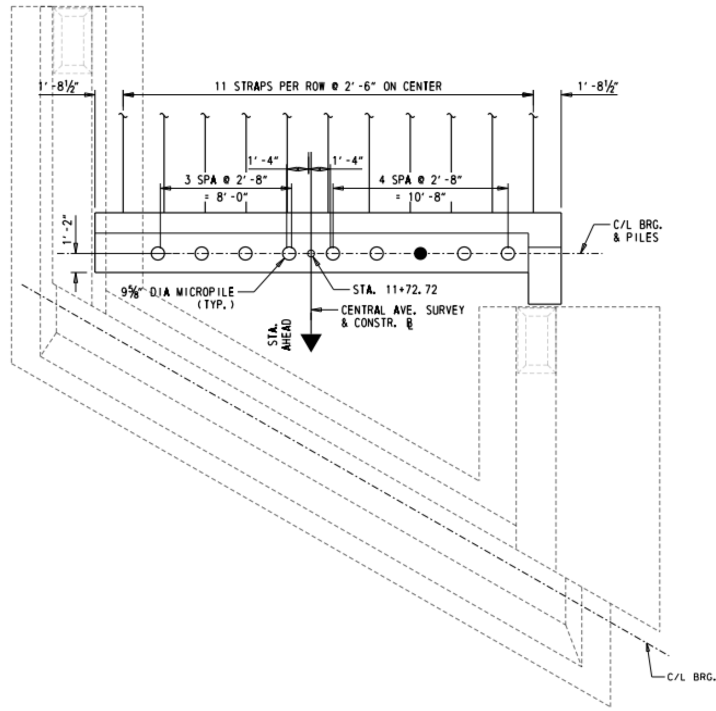

Figure 13:New South Abutment

Conclusion

Design of a new bridge within such a limited envelope was challenging. It was only possible due to a fortuitous geometry which made it possible. While no one design element is unique to this project, each one had to be detailed specifically to meet the unique requirements and challenges presented by this structure.

The construction contract was put out to bid in fall of 2017 and the contract was awarded to Eastern Highway Specialists. Construction began in February 2018 and updates and pictures will be available for the HMS presentation in fall of 2018.