Construction of the John Greenleaf Whittier Memorial Replacement Bridge

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IBC 16-14

KEYWORDS: Launched Girders; Overhead Gantry Crane; Network Arch; Arch Erection; Falsework Towers; Hilman Rollers; Girder Erection; Girder Stability; Sand Jacks; Temporary Crane Ramps; Jack-up Barge

ABSTRACT: With daily tide cycles of 9ft and with areas inaccessible by cranes, an erection method utilizing two launched girders supporting overhead gantry cranes was implemented. This unique method was used to erect the approach spans, the arch floor system, and a 200-ton crane which traversed the arch floor system to erect the upper arch. The launch girders were moved and will be reused to remove the existing bridge and erect the SB Bridge.

PROJECT OVERVIEW

The John Greenleaf Whittier Replacement Bridge is a 480-ft long network arch with 11-ft deep plate girder approach spans that currently carries three northbound and three southbound lanes of I-95 across the Merrimack River between Amesbury and Newburyport, MA.

A Best Value Design-Build (BVDB) Procurement process was used for the Project, which included a two-phase selection process. The first phase consisted of creating a short list of qualified Design-Build Entities (DB Entities). The second phase consisted of the submission of Technical and Price Proposals. The construction joint venture team won the project with a $230,000,000 bid based on the technical merit of their unique approach to constructing the arch and approach structures.

The project scope includes the erection of two new bridge structures (one SB & one NB) and one new secondary bridge that crosses over I-95, demolition of the existing trussed arch bridge, widening of I-95 which required tall and long retaining wall structures, wetland and riverfront mitigation, utility relocation and construction of noise walls.

This paper will focus on the challenges the team faced constructing the new bridges over the Merrimack River as well as removing the existing bridge.

PROJECT CHALLENGES

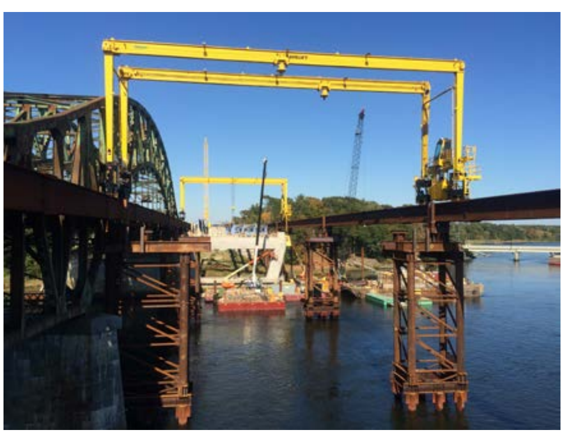
The project team faced a number of challenges when analyzing possible construction methods for the sub-structure foundations, as well as the super-structure steel and concrete deck:

- Tide cycle variations of +/- 9-ft on a normal day, leaving some areas of the river bottom (rock) exposed
- Maintaining a 150-ft navigable channel at all times for pleasure boats
- Environmentally restricted wetland areas along each bank of the river, directly below and extending beyond the footprint of the proposed bridges
- Extremely difficult slopes along the river bank, where the abutment structures were to be located
- Limited crane access from land and/or water
- Limited available area and access for material staging and storage
- Demanding construction schedule
- Maintaining 100% of the existing traffic lanes, except for limited "rolling lane closures"
- Demanding crane capacity restrictions

LAUNCHED GIRDER SYSTEM

Given the unique site challenges, a launched girder system supporting overhead gantry cranes was implemented (Figure 1). This system added a great deal of flexibility to the erection process and served multiple functions in each phase of construction.

Figure 1 – Launched Girder & Gantry Crane System



During the erection of the replacement bridges, the launched girders provided access to the approach staging area, allowing the overhead gantry cranes to unload steel members directly from delivery trucks when possible. Steel members could then be transported from the staging area out onto the bridge for installation (Figure 2).

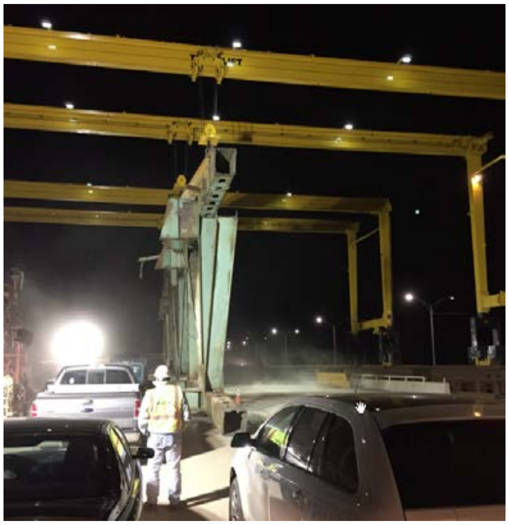
Figure 2 – Overhead Cranes Deliver Steel

Once the approach spans and floor system for the arch were erected, the overhead gantry cranes were then utilized to deliver and erect the 200-ton crane that was used to construct the upper arch (Figure 3).

Figure 3 – Overhead Cranes Deliver Crane

Finally, the launched girder system allowed for access to the existing bridge during the demolition phase, enabling the contractor to remove the existing truss in large segments and thus, minimize the time required for demolition (Figure 4).

Figure 4 – Overhead Cranes Used for Demolition



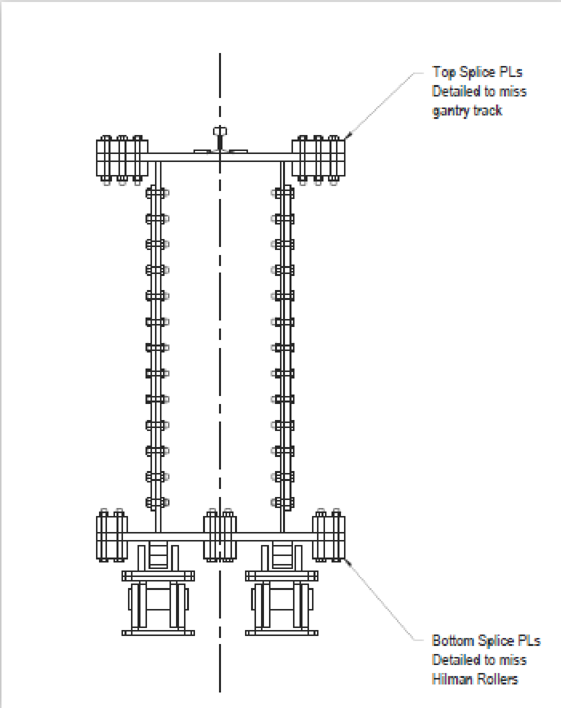
The launched girder system, however, did not come without its own unique set of design challenges. In order to reduce the number of falsework towers required on the project, the launch girder itself had to span up to 160-ft between supports. This meant that not only did the launch girder have to be designed to support the gantry crane reactions over these long spans, but it also had to be able to cantilever that far during the launch before reaching the next support point. Given the large distance between supports (and therefore, long unbraced length), a rectangular fabricated box section was chosen for the launch girder. The box section reduced concern for lateral torsional buckling over the long unbraced lengths, provided torsional resistance for the long cantilevers during the launching process, and provided larger weak axis section properties required to resist external lateral loads from wind and base reactions from the overhead gantry cranes.

A tapered steel section was provided at the leading and tailing ends of the main box portion of the launch girder to accommodate cantilevered tip deflections of up to 4-ft during the launching process (Figure 5). The tapered section allowed the leading end of the launched girder to deflect under the maximum cantilever and then ease back onto the rollers of the next falsework support without the use of a “king-post”.

Figure 5 – Launch Girder Nose

The launch girder consisted of 50-ft long segments which, when connected together, provided an effective 850-ft long pathway for the gantry crane system. The 850-ft length was not sufficient to traverse the entire width of the river valley so the launch girders were moved in two “pushes” for each phase of the project requiring the contractor to think about building the bridge in “halves”. The splices connecting these segments (Figure 6) had to be specially designed to accommodate other aspects of the launched girder system design.

Figure 6 – Launch Girder Splice



First, the splice at the bottom flange had to be detailed to not interfere with the Hilman Rollers used at the falsework towers to support the girders and minimize friction during the launching process (Figure 7).

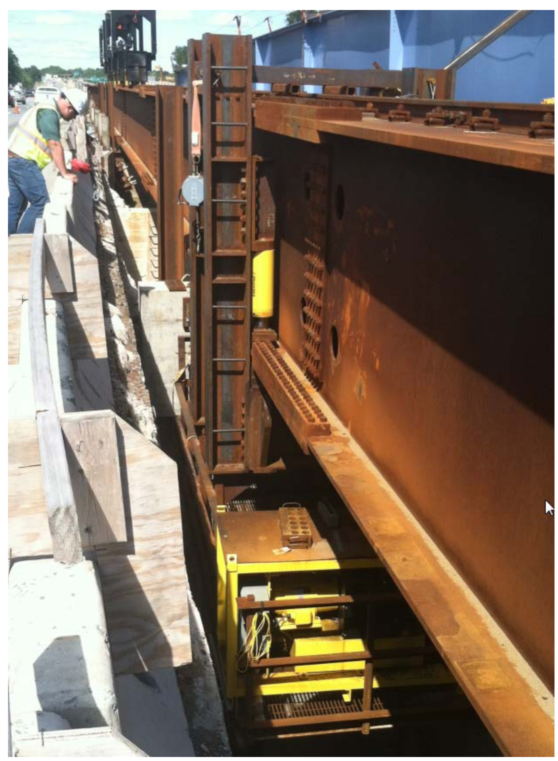
Figure 7 – Launch Rollers & Splice Interface

Second, the splice at the top flange had to be detailed to not interfere with the overhead gantry crane rail. Both of these constraints made for a unique looking splice detail, comprised of splice plates well over 2 inches thick of Grade 70 material to make up for the limits imposed on the plate width (Figure 8).

Figure 8 – Overhead Crane Rail & Splice Interface

To move the launch girders ahead during the launching process, a push/pull system was employed at the tail end of the girders. In addition to the friction in the Hilman Roller system, the push/pull system was designed to accommodate friction due to any mis-alignment in the rollers along the longitudinal axis of the beam, the extra force required to push the girders along an uphill grade, as well as the additional grade of the nose section of the girder itself, and any braking forces required to slow/stop the girders from creeping on a downhill grade (Figure 9).

Figure 9 – Push/Pull System



Once launched into position, the girder had to be able to handle the maximum design reaction of 110 kips induced by the gantry crane system. The local effect of this reaction on the top flange of the launch girder was studied in detail to ensure that the top flange was sized and/or stiffened appropriately to handle the large point loads.

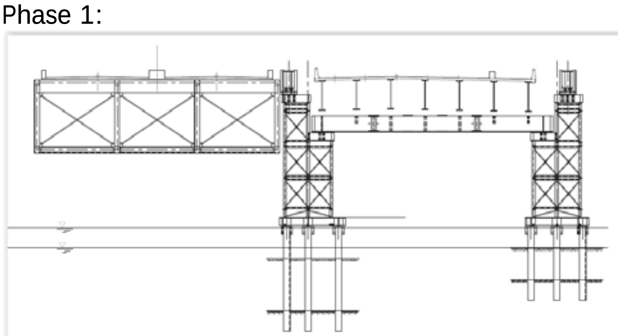
FALSEWORK TOWER DESIGN

Similar to the launched girder system, the falsework towers used on the project were intended to perform multiple functions throughout the construction process. As previously mentioned, they were designed to support the launch girders and the gantry crane system during all phases of erection and demolition, but they were also utilized to support the approach girders and the arch floor system as they were being constructed. The need to simultaneously support both the launched girder system and the new bridges during construction resulted in vertical design forces of up to 2000 kips for the towers. Water depths of up to 30-ft to mudline and 40-ft to rock coupled with the need to design for impact of debris and ice over multiple east-coast winters added to the complexity of the design.

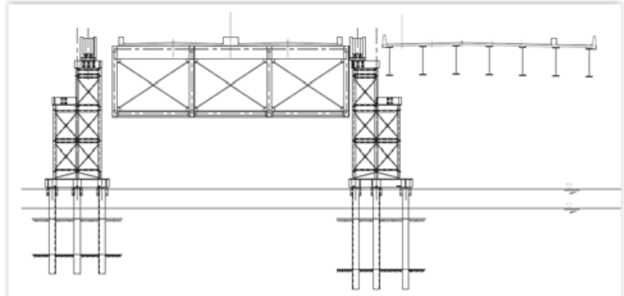
To further maximize the function of the towers, they were detailed to allow for reconfiguration and reuse during different phases of construction (Figure 10). The eastern-most towers used in Phase 1 (NB construction) were relocated to the west to be used in Phase 2a (Demolition). In addition, the middle towers were detailed to be used for Phase 1 and 2a without modification, and then shifted over 10-ft to transition to Phase 2b (SB construction).

Figure 10 – Tower Re-use

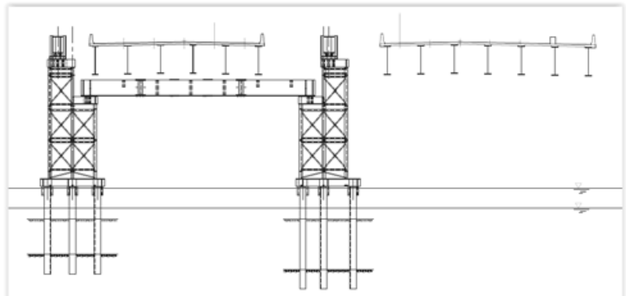
Phase 1:



Phase 2a:



Phase 2b:



The main falsework towers were divided into two types. The first type utilized W-Shape columns with more traditional tension-only X-bracing that was originally planned to match the contractor's existing system (Figure 11).

Figure 11 – W-Column Falsework Towers

The second type of towers utilized 30 inch diameter pipe columns with more rigid tubular tension/compression bracing to support the arch floor system as well as the cranes used to erect the arch and install the precast deck panels (Figure 12). In the end, the heavier pipe column towers were actually more cost-efficient due to the number of bolted connections provided in the bracing.

Figure 12 – Pipe Column Falsework Towers

The foundations for all towers were comprised of 30 inch diameter pipe piles that were drilled and grouted into rock and then filled with concrete for additional weight and axial capacity. The additional weight was required to reduce the uplift resulting from extreme design load combinations with large design winds acting on the structures supported at the tops of the towers. Detailing of the tower to foundation connection was a two-fold challenge. The steel grid of the upper tower did not always align with the foundation piles, yet the connection between the two needed to be rigid enough to act as a “frame”. In addition, the connection had to accommodate the need for field modifications based on pipe pile installation tolerances of up to 6 inches in any direction (Figure 13).

Figure 13 – Tower Foundation

At the tops of the towers, the towerheads had to be detailed to accommodate the multiple functions of the towers. At the launch girder support locations, the towerheads were detailed to incorporate the roller/swivel bearings required for the launched girder system. The pipe-column towers supporting the arch floor system were detailed to incorporate 8-ft diameter sand jacks, which provided a vertical bearing point for the arch tie girders, as well as a pin connection point for the temporary strut members used to construct the upper arch. The W-column towers were detailed to support an additional temporary falsework girder that served as the support point for the approach girders during erection.

ARCH ERECTION

The erection of the 480-ft main span network arch took place in several stages. As previously mentioned, the floor system was erected on the heavy pipe-column falsework towers in the main navigation channel. The main tie girder and floor beam members were stick-built using the overhead gantry crane system (Figure 14).

Figure 14 – Arch Floor System Erection

Once the arch floor system was complete, a temporary crane mat system was installed. This mat system was a combination of heavy H-pile steel mats to support the two cranes used during the arch erection, and hardwood crane mats to support delivery trucks and lifts used to transport and install the arch steel members. With the crane mat system in place, the gantry cranes were used to assemble the 200-ton crane that would be used to erect the arch (Figure 15).

Figure 15 – 200-ton Crane Assembly

The upper arch was constructed on top of large trussed strut members that transferred the weight of the arch members directly to the falsework towers below (Figure 16). These strut members also provided jacking capability to aide in member fit-up and installation of the final arch keystone segment.

Figure 16 – Upper Arch Erection

After all of the major arch segments were in place, the overhead crane system was again employed to disassemble the 200-ton crane and replace it with a 100-ton telescoping boom crane. This smaller crane was used to install the precast deck panels and the cable hangers for the arch (Figure 17).

Figure 17 – Precast Deck & Cable Hangers Installed

The cable hangers were stressed in sequence to target forces established by Genesis Structures. This sequence was selected to prevent buckling of the unbraced arch and to minimize the need for the contractor to adjust cable forces after the initial stressing. Midway through the initial cable stressing process, the sand jack bearings at the falsework towers were released, and the arch was free to span the full 480-ft across the navigation channel. Once the remaining cables were stressed, the deck closure pours and post-tensioning was completed and barrier and overlay was installed to conclude the erection process.

ACKNOWLEDGEMENT

The author would like to acknowledge the General Contractor, Walsh Construction, for their excellent execution of the work throughout the project and the Engineer of Record, HNTB Corp., for their input and collaboration during the construction and review process.