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The Gateway Bridge and The Complete City Street: A Merger

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Abstract

The North Washington Street Bridge, currently under construction and scheduled to open in 2023, is a multimodal transportation structure designed to address the needs of a modern and ever-evolving city. In its heyday, the bridge that it replaces carried horse drawn carts, vehicles as well as elevated trains across the Boston Inner Harbor. After a century and two decades of service, it had become functionally obsolete and structurally deficient, constantly in need of costly maintenance and with its unique features like the central swing span permanently fixed and inoperable. Still burdened with carrying more than 42,000 vehicles and heavy pedestrian traffic across the harbor each day and supporting essential and critical utilities, the time had come to replace the bridge. The new bridge will accommodate not just pedestrians and the Freedom Trail but also cyclists in separated bike lanes; carry not just cars but also buses including those on a southbound dedicated Bus Rapid Transit (BRT) Lane; serve not just as an observation deck for the adjacent iconic Leonard P. Zakim Bunker Hill Memorial Bridge but also as a destination in itself; be not just a gateway to the harbor but also complement its surroundings. The design of the replacement North Washington Street Bridge emerged from a confluence of deliberations between the owner, agencies, and the stakeholders. In this article, we will delve into the constraints and solutions that accompanied the context sensitive design of this livable, walkable, sustainable, and multimodal bridge with the flair of a complete city street.

Keywords: North Washington Street Bridge, Complete Streets, V-Pier, Architectural Trellis, Post-tensioning, Prestressing

1. The Existing Bridge

The Charlestown Bridge was originally constructed in 1898 to carry North Washington Street, two trolley car tracks and two elevated street railway tracks from Keany Square in Boston to City Square in Charlestown. The elevated railway tracks and their support framing had subsequently been removed and the street railway tracks had been paved over. The Charlestown Bridge (Bridge No. B-16-016) consisted of three different structures and carried North Washington Street over the Charles River, Water Street and the adjacent warehouses underneath the bridge deck in Boston. The focus of this article is the bridge over the Charles River or the Inner Boston Harbor that is being replaced. The other two bridges are being rehabilitated. The existing, now demolished, bridge over water was a twelve span structure with five approach spans on either side of the main, two span swingable structure. The central swing, two span structure consisted of four steel through trusses, floorbeams, stringers and an open steel grid deck. The swing spans had an overall length of 240 ft. The swing spans were fixed permanently in the closed position in 1962.

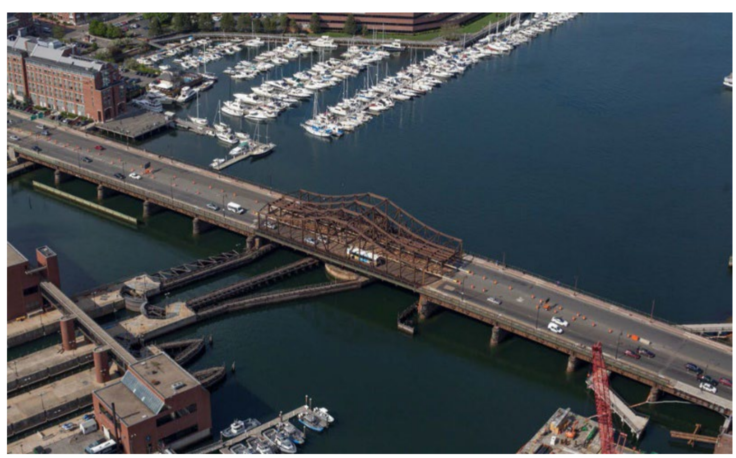

Figure 1 The Old North Washington Street Bridge

The roadway over the swing spans was divided into three 22 ft wide sections due to the center two trusses being within 4 ft wide raised medians in the roadway. The west roadway section carried two southbound lanes, the east roadway section carried two northbound lanes and the center section, which was closed to traffic in 2003 due to advanced deterioration to the stringers and floorbeams, used to carry one lane in each direction. This center section had a minimum vertical clearance of 14 ft-3 in in due to overhead bracing members between trusses. There were impact attenuators and gore lines approaching the medians at the central two trusses. There were two 11 ft wide floorbeam brackets cantilevered beyond the exterior trusses that supported a sidewalk on each side of the bridge. Most of the swing span floorbeams and all of the stringers were replaced in 1956 and the deck was replaced in 1992. Structural steel repairs were made to many of the floorbeams and stringers in 1991, 2003, 2008, 2009, 2011 and beyond.

The approach spans consisted of a girder-floorbeam-stringer superstructure which supported an 80 ft wide roadway with three lanes of traffic in each direction. The 6 ft wide cantilevered floorbeam brackets on either side supported 8 ft 7 in wide sidewalks. These spans were each 82 ft 1 in long and consisted of six riveted built up steel plate girders that support steel floorbeams, stringers, a non-composite reinforced concrete deck and hot mix asphalt wearing surface. The original floorbeams, stringers and deck were replaced in 1956. Structural steel repairs were made to many of the stringers and floorbeams in 1991, 2003, 2008 and 2009. There was a 4 ft wide raised median on the southerly three spans creating four lanes of traffic in the southbound direction for turning lanes at the adjacent signalized intersection.

The ten piers supporting the approach spans were constructed of granite stone masonry. The piers were founded on unreinforced concrete pile caps on timber piles, except for Piers 9 and 10 which were built on spread footings. Repairs were made to the concrete footings in 1951, which included the installation of steel sheeting, repair of eroded concrete, and the addition of a concrete apron which typically encased the lower 3-1/2 courses of the stone masonry. The pivot pier for the swing span was constructed of a 75 ft diameter unreinforced concrete and was founded on timber piles. The upper portion of the pivot pier was encased in 1951 and further modifications were made with the construction of walkways in 1973 at the time of the Charles River Dam construction.

2. Project Parameters and Constraints

2.1. Hydraulic Data

The bridge hydraulics at the site are mostly governed by the fluctuating tidal flow within Boston harbor and the pump discharge from the Charles River Dam. The Charles River Dam is a flood control dam that blocks high tidal inflow to the upstream portions of the river and maintains a recreational pool upstram of the dam. The flow throught the dam is completely regulated. The harbor width is nearly 1080 ft wide between the face of existing abutments. The riverbed elevations vary from -3.0 ft NAVD88 vertical datum near the outer approach spans to -23 ft in the main channel under the main span. The mean high water elevation is +4.32 ft and the mean low water elevation is -5.17 ft.

In the 1940s and 1950s, steel sheeting and concrete aprons were placed around the pier footings to repair eroded locations. In 1973, the Charles River Dam was constructed just upstream of the bridge. The Charles River Dam project installed stone riprap around Piers 6-10 for protection against the high-speed discharge of their water pumps that controls the level of the Charles River to the west of the dam. The dam added to the scour potential at Piers 8 and 9, as they were located adjacent to the discharge flow. A 200 ft long training wall is located in the northern portion of the channel upstream of the bridge. The purpose of the training wall is to deflect flow from the sluice gates. Navigational fenders were located at the downstream end of the large lock and extended throught the width of the bridge.

2.2. Geotechnical Data

The subsurface conditions at the site consist of fill or river sediment along with organic soils above relatively thinner layers of granular soil and locally distributed silty clay. These layers are underlain by glacial till, weathered bedrock and bedrock. The geotechnical issues which dictated the use of drilled shafts for pier foundations were the presence of relatively thick layers of soft or loose overburdern soils, including river sediment and organic soils, variable soil depth to the top of bedrock, variable quality of bedrock, presence of cobblers and boulders in the natural overburden soils and abandoned submarine cables and rock fill associated with dam construction. Shallow foundations were therefore not considered feasible for supporting the new bridge piers, temporary bridge and pedestrian walkways near the north and the south abutments.

2.3. Constraints from Approach Roadways

South Approach - The approach roadway south of the bridge extended from the Keany Square signalized intersection in Boston to the bridge. The NB roadway width leaving the Keany Square intersection toward the bridge consisted of of two 16 ft wide travel lanes and 1-ft shoulders/curb offsets on either side and narrowing to two 11 ft travel lanes at the bridge truss mid-span. The SB lanes at Keany Square provided storage for the signalized intersection and consisted of four storage lanes and 1-ft shoulders/curb offsets on either side. The south approach alignment to this structure consisted of a short horizontal tangent section which extended approximately 165 ft from the end of the bridge to a signalized intersection with Causeway Street and Commercial Street in Boston.

The roadway on the structure over the harbor was on a horizontal tangent. The vertical profile on the structure was a crest curve with the peak of the curve at the swing spans. Bordering the North Washington Street south approach on the west side is the ten-story Converse Headquarters building. The building and the two-level pavilion at Lovejoy Wharft extend along the bridge for 160 ft. There are two entrances and a pavilion stair case landing leading to the sidewalk on the bridge. Bordering North Washington Street south approach on the east side is the Department of Conservation and Recreation (DCR) Prince Street Park. The existing south approach also provided an Massachusetts Bay Transportation Authority (MBTA) bus stop. The bus stop was located along the right shoulder of the NB roadway just north of the Keany Square intersection.

North Approach - The approach roadway north of the bridge extended from the City Square signalized intersection in Charlestown. The southbound roadway width leaving the City Square intersection toward the bridge consisted of three 15 ft wide travel lanes and 1-ft shoulders/curb offsets on either side and transitioned to two 20 ft travel lanes at the project bridge structure. These two SB lanes further narrowed to two 11 ft wide travel lanes at the bridge truss mid-span. In the NB direction, the NB lanes approaching City Square provided storage for the signalized intersection and consisted of four storage lanes. A concrete median separates the three SB lanes and four NB lanes at the intersection. The roadway width varied from 131 ft at City Square to 80 ft at the bridge truss mid-span.

The north approach to this structure also includes adjacent bridge spans over warehouses and Water Street in Charlestown. These adjacent structures are located between City Square and the project bridge structure. The approach roadway and adjacent bridge spans are on a horizontal curve alignment that extends 460 ft (including 325 ft of bridge spans) to the City Square intersection.

Bordering North Washington Street north approach on the east side is Charles River Avenue which provides access to the Constitution Marina and the Residence Inn Hotel. Bordering North Washington Street north approach on the west side is the DCR Paul Revere Park.

2.4. Constraints from Features Crossed

The existing bridge crossed the Boston Inner Harbor between Boston and Charlestown which is approximately 1080 ft wide, approximately 15 ft above water level at the mean high tide and 25 ft above water level at the mean low tide. The Charles River Dam is located approximately 200 ft west of the bridge with one 40 ft wide large lock and two 22 ft wide smaller locks that allow for navigation between the Charles River and Boston Inner Harbor.

The main navigational channel was located under the swing span of the bridge where marine traffic was directed through the locks located at the dam. An existing fender system was installed on both sides of the pivot pier and along the adjacent piers. Navigational lighting was attached to the underside of the swing span and along the fender system to help direct the marine traffic. The Massachusetts State Police Marine Unit barracks are also located within the Charles River Dam building.

Constitutional Marina, with capacity to hold approximately 250 vessels, is located approximately 65 ft east of the northern three spans of the Charlestown bridge.

2.5. Constraints from Pedestrian Facilities

Existing Pedestrian Facilities – Continuous sidewalks existed along both sides of North Washington Street within the project limits and extend beyond each approach. The Freedom Trail ran the entire length of the project along the east sidewalk. Boston's Freedom Trail connects significant historic sites in Boston's North End and Charlestown. The Freedom Trail is well traveled and a core feature to Boston's pedestrian community, and economically critical to Boston's tourist industry.

Walkways under the North Washington Street Bridge – Two existing pedestrian walkway structures cross under the North Washington Street Bridge within the project limits, The Tundor Wharf Walkway under the north end of the bridge and the Lovejoy Wharf Walkway under the south end of the bridge, both connecting DCR Parks on one side to properties on the opposite side of the bridge, without requiring pedestrians to cross North Washington Street at roadway grade.

Existing Bicycle Facilities – The existing bridge and its approaches had inadequate bicycle accommodation. While both recreational and bicycle use occurred on the roadway and sidewalks crossing the structure, bicyclists were limited to either utilize the existing sidewalks or share the roadway travel lanes with the vehicular traffic. The existing shoulders measured only 1 foot on either side of the roadway within the project limits.

2.6. Constraints from Utilities

The exisiting bridge supported numerous utilities owned by National Grid, Eversource, Comcast, Massachusetts Bay Transportation Authority (MBTA), Boston Water Sewer Commission (BWSC) and Boston Transportation Department (BTD). The configuration of utilities was slightly different in the central swing spans. All of these utilities were to be accomodated in the replacement bridge.

<u>Approach Spans</u>

- 27 in x 23 in concrete encased electrical distribution utility ducts in the east and west girder bays of spans 1 through 5 and in the west girder bay of spans 6 through 10
- Large concrete manhole structures in the west girder bay of spans 5, 6 and 10 and in the east girder bay of spans 5 and 10
- Steel manholes in the center girder bay of span 4, and below the west sidewalk in spans 4, 6 and 10
- 8 in diameter water pipe on east sidewalk
- 36 in diameter gas line in the center girder bay
- 1½ in diameter BTD traffic signal interconection duct and 3 - 2½ in diameter Comcast ducts attached to east pedestrian railing

- 2 - 6 in diameter high pressure fluid filled transmission lines attached to west face of the west girder
- 1 - 3½ in, 1 - 3 in and 2 - 1 in diameter lighting ducts on top of the floorbeams below the east sidewalk
- 1-2½ in, 1-3 in diameter lighting ducts and 3-4 in diameter MBTA electrical distribution ducts on top of the floorbeams below the west sidewalk
- Several abandoned cables near piers 5 and 6
- 8- 4 in diameter ducts hung from stringers between manholes in span 10

<u>Swing Span</u>

- 8 in diameter water pipe on east sidewalk
- 1½ in diameter BTD traffic signal interconection duct and 3 - 2½ in diameter Comcast ducts attached to east pedestrian railing
- 8 - 4 in diameter electrical ducts hanging at both the east and west sides of the center truss bay
- 1 in diameter duct on the east side of the center truss bay extending south of the center pier
- 2 in diameter duct and 3-1 in diameter ducts on the west side extending south of the center pier
- 3 - 3 in diameter galvanized pipes below the west sidewalk
- 2 - 6 in diameter high pressure fluid filled transmission lines below the west sidewalk
- 6 - 12 in diameter gas lines below the east truss bay. There is an abandoned 36 in diameter gas main under the channel bed of the swing span as well

At the time of bridge design, it was determined that the transmission lines had to not only stay in place but also be supported at 10 ft intervals throughout construction. The constraints imposed by this requirement led to both the choice of tub girders for bridge superstructure and their transverse spacing and layout.

2.7. Constraints from Cultural Resource Areas

The existing bridge was listed in the Massachusetts Cultural Resource Information Systems inventory for historic status although it was not in an historic district. the Preliminary Structure Report recommended its replacement due to advanced structural and functional deficiencies and substandard load rating. At all four corners of the bridge, there were additional project constraints:

Southwe–t - The 10 story Converse Headquarters building has its foundation connecting to and integral with the existing bridge south abutment and wingwalls, two entrances are accessed from the west sidewalk as well as staircases leading to the pavilion of the LoveJoy Wharf.

Southeast - DCR Prince Street Park with tennis courts adjacent to the south abutment east retaining wall

Northwest –DCR Paul Revere Park with stair access from the west approach sidewalk of warehouse bridge spans.

Northeast- Charles River Avenue which provides access to the Constitution Marina and Residence Inn Hotel

2.8. Other Constraints

An evaluation of the bridge paint system concluded that the existing coating contained lead and that it was in very poor condition and could not be overcoated.

3. Design

3.1. Superstructure Design

Of the several alternatives considered, the superstructure cross-section with four steel box girders was selected (Figure 2). This option had several benefits over other alternatives considered in the Bridge Type Study phase. The steel box girders, because of their greater torsional rigidity as well as for their ability to provide a more stable cross-section during construction stage with only half the bridge constructed longitudinally, became the ideal candidates for bridge's superstructure. Other advantages included a shallower girder depth allowing the provision of a greater vertical clearance from water level in a corrosive environment and minimizing the amount that the roadway profile had to be raised over the navigation channel.

The design parameters required that the bridge carry five lanes of vehicular traffic totaling 57 ft 6 in in width, two 7 ft wide bicycle lanes separated from the vehicular lanes by 3 ft 6 in wide traffic barrier and two 10 ft 6 in wide sidewalks to comfortably accommodate pedestrian traffic. The sidewalks would be widened to a maximum width of 19 ft at the main span above the navigable channel. This required large overhangs from the exterior girders. The girders themselves could not be spaced further apart because of the constraint imposed by the two high pressure fluid filled 115 kV transmission lines supported from the existing bridge, which were required not only to stay in place throughout the bridge replacement but also needed to be supported every ten feet.

The central swing spans on the existing bridge consisted of four trusses with three two lane roadways located between each truss. The center two lanes were closed to traffic years ago due to detrioration of steel structural members. It would also not have accommodated traffic during construction without requiring extensive repairs. It was also determined that constructing the bridge in three phases was not feasible as it would have resulted in three narrow longitudinal construction zones. Based on the traffic counts, two lanes were needed in the southbound direction and one in the northbound direction throughout contruction. Design-phase plans included construction of a temporary bridge to replace the easterly swing spans after demolition of the two easterly trusses in the first stage. After relocating the traffic to the eastern side of the bridge, the western side of the bridge would be demolished entirely and reconstructed while the utilities would be relocated to a temporary utility bridge constructed west of the bridge in the second stage. The third stage proposed demolition of the remaining eastern existing structure along with removal of the central temporary bridge and reonstruction of the entire eastern side of the bridge in the third stage.

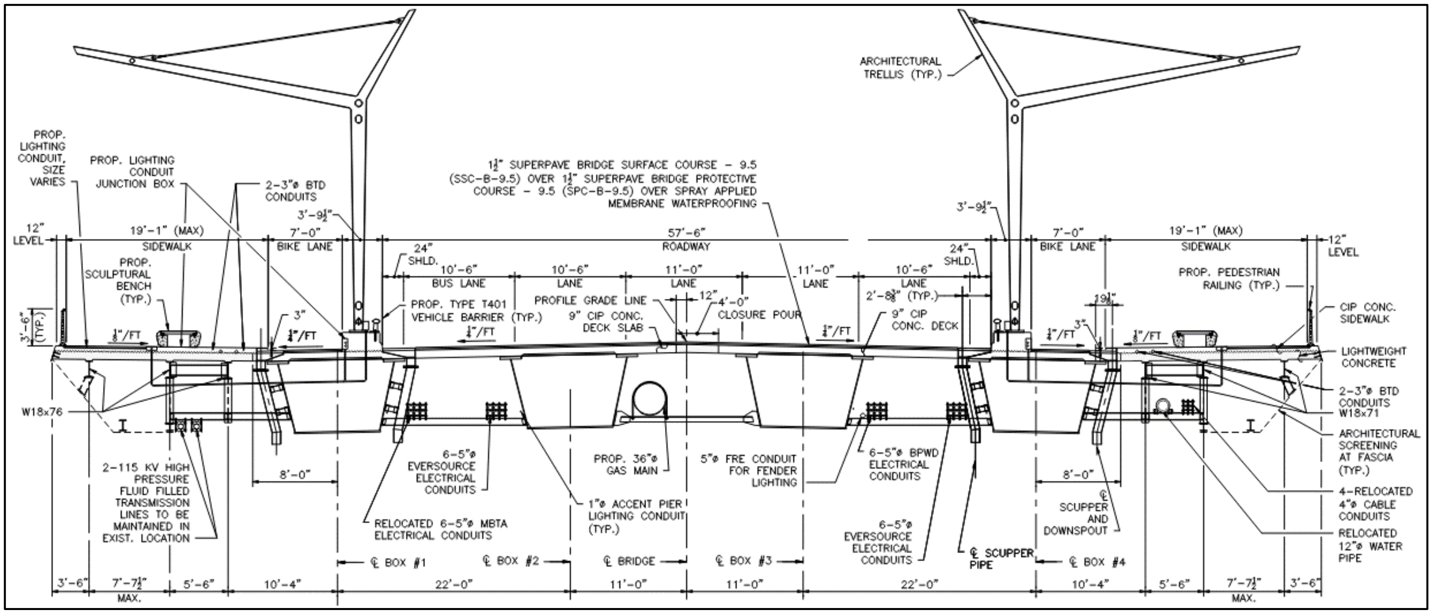

Figure 2 Bridge Cross-Section at Span 7

This scheme involved the substructure and superstructre to be built in two major stages, each consisting of two lines of box girders and a substructure unit below each of the girders. The box girders were the ideal choice for the proposed construction sequence because of their stability, even in a partial cross-section configuration with two lines of girders, and high torsional rigidity. However, during construction phase, the staging sequence was modified to construct a nearly full length temporary bridge alongside the existing bridge to carry three traffic lanes and one sidewalk. This temporary bridge would lie to the west of the existing bridge on temporary steel pile bents. The temporary steel bents would also support the temporary utility bridge to maintain the numerous utilities crossing the bridge throughout construction. This utility bridge would be adjacent to the existing structure after demolition of west sidewalk of the existing bridge. These two temporary bridges would allow the construction of nearly the entire bridge cross-section to progress from the north abutment to the south abutment (Figure 3).

Midas Civil and MDX software were used for creating the bridge's structural model and their analysis results were compared to envelope the design forces for superstructure elements. The results converged as much as the two programs would allow based on their methods of load distribution. One key difference was in the way the programs develop demands for eccentric load.

Figure 3 Existing Bridge (Left), Temporary Utility Bridge (Middle) and Temporary Bridge (Right) looking South

Midas relies more on the torsional resistance of the tub girders while MDX uses self-equilibriating shears in each girder to resist eccentrically applied loads. And while MDX only permits the superstructure to be analyzed with rigid supports, Midas Civil was used to capture the effects of substructure rotation on the forces developed in superstructure elements.

3.1.1. Tub Girders

Each of the four steel tub girders consist of 8 ft wide bottom flanges and 6 ft 6 in deep webs with the top flanges spaced 11 ft apart. This configuration permitted employing a conventional reinforced concerete deck with the top flanges of the tub girders supporting the deck uniformly at 11 ft intervals. These tub girders would span continuously over the substructure units and have a total length of 1087 ft. The individual span lengths would vary and the longest span length would be 190 ft. which support the overlooks and span over the main navigation channel.

Internal (inside the tub girder) and exterior (between the tub girders) crossframes are provided between each transverse floorbeam (not shown in Figure 2). The internal crossframes are to resist the distortional stresses in the tub girder from eccentrically applied loads. The exterior crossframes are to allow load sharing between adjacent tub girders and restrain non-composite dead load rotations. A top flange truss system is also provided inside the tub girders to resist torsional demands before hardening of the concrete deck.

3.1.2.Floorbeams and Cantilevers

The framing plan of the bridge shows 36 transverse and continuous floorbeams to support the wide sidewalks proposed on this bridge. The intermediate floorbeams are partial depth in relation to the tub girders while the floorbeams acting as end diaphragms are full depth at intermediate and end bridge supports.

Figure 4 Fit-up of Span 7 Steel Framing in Construction Yard

A typical intermediate floorbeam at the interior and exterior tub girder is shown in Figure 5 and Figure 6. A bolted tie plate was designed for the continuity of the top flange of the floorbeam and en plate moment connections were provided for the web and bottom flange connections through the web of the tub girder.

omposite action was not relied on for the floorbeam design, however, the floorbeams were attached to the deck with shear stud conectors. The floorbeams cantilever out 13 ft from the outside web of the exterior tub girders. The length of the cantilevered floorbeams further increases to 21 ft at the main span where the sidewalks widen from 10 ft 6 in to 19 ft creating observational overlooks. Figure 7 and Figure 8 show the floorbeam cantilevers at the approach and the main spans.

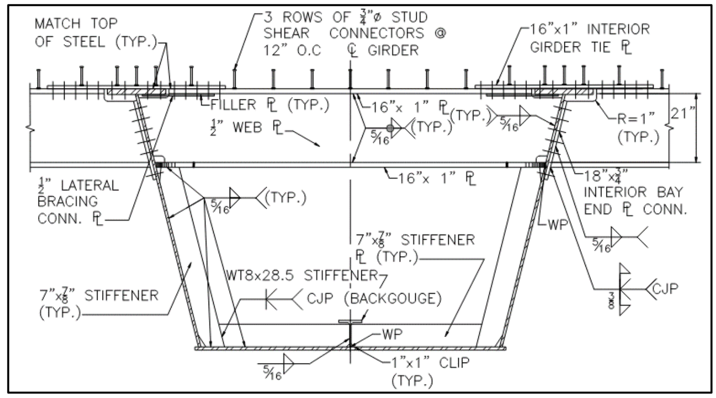

Figure 5 Interior Floorbeam and Interior Girder Connection in Approach Spans

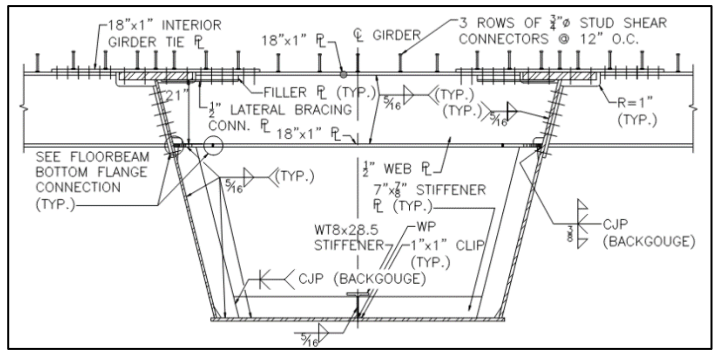

Figure 6. Interior Floorbeam and Interior Grid Connection in Span 7

3.1.3 End Diaphragms

The floorbeams designed as full depth end diaphragms at the bridge supports are made continious by utilizing bolted top and bottom flange splices as shown in Figure 9 and Figure 10. Similar to the intermediate floorbeams, the end diaphragms are also connected to the deck with stud shear connectors.

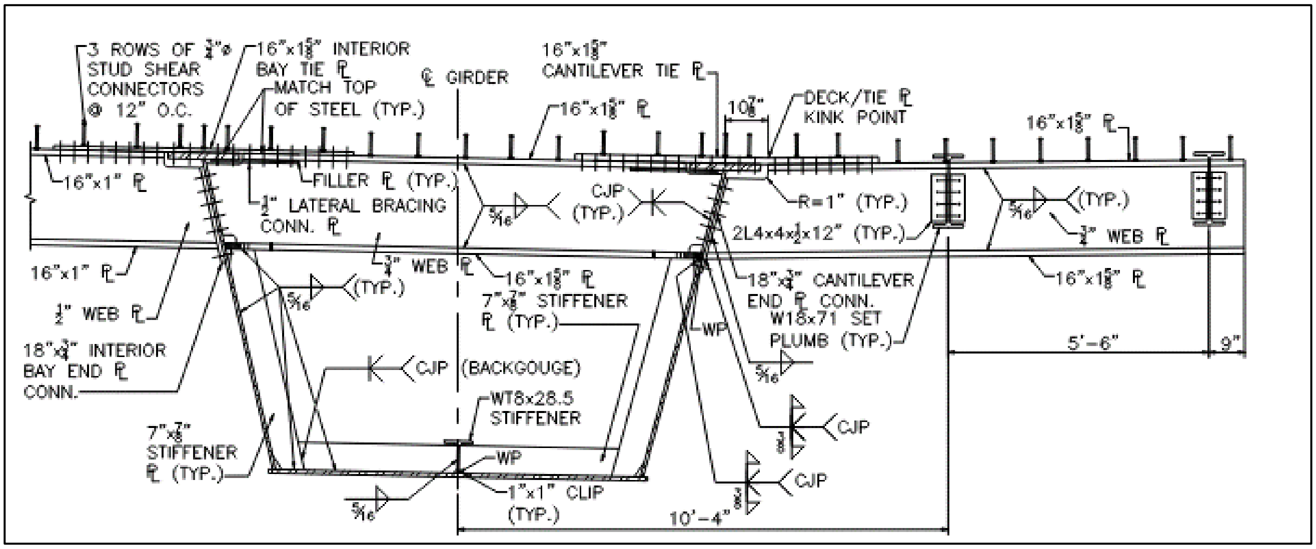

Figure 7. Intermediate Cantilever Floorbeam at Approach Spans

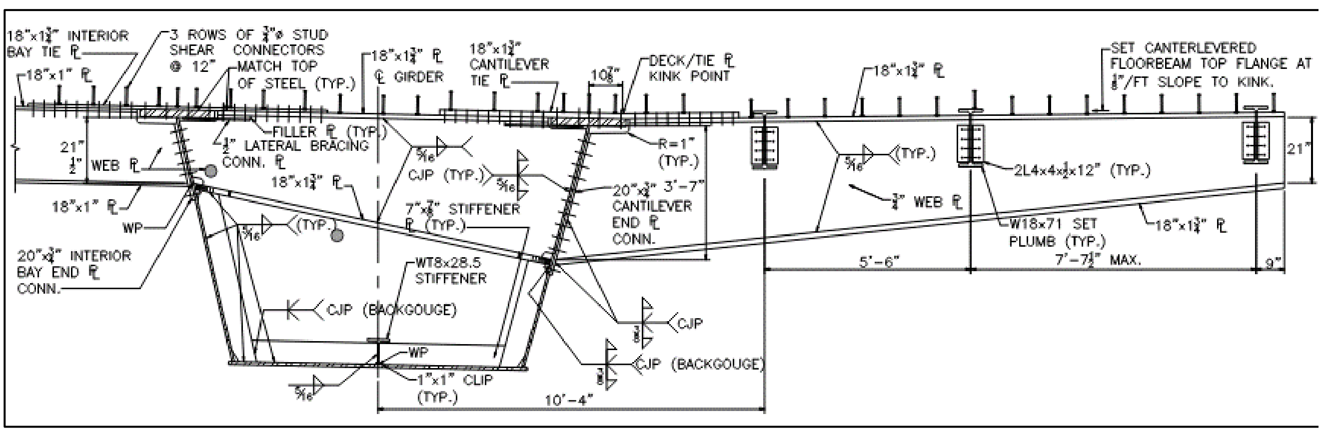

Figure 8. Intermediate Cantilever Floorbeam at Span 7

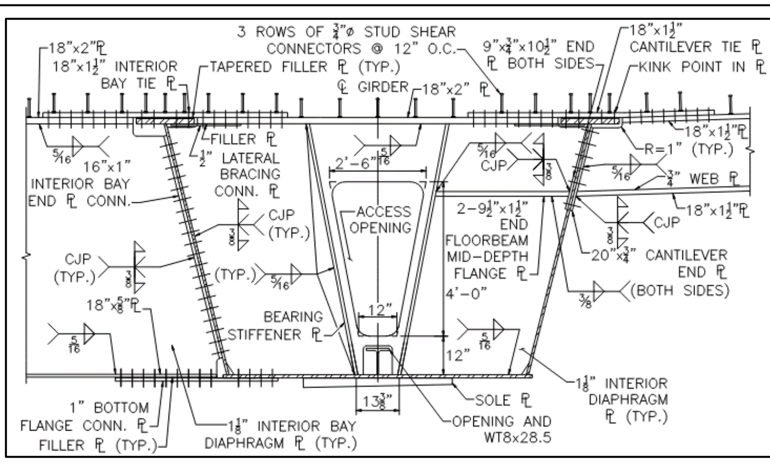

Figure 9. End Floorbeam at Exterior Girder at Approach Span Piers

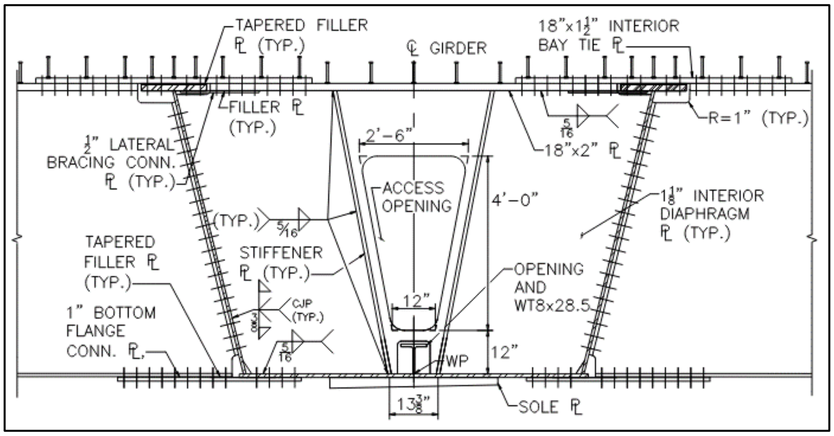

Figure 10. End Floorbeam at Interior Girder at Approach Spans

3.2 Substructure Design

The Leonard P. Zakim Bunker Hill Memorial Bridge is an iconic structure known for being the widest cable stayed bridge in the world at the time of its construction. The Zakim Bridge is to the west of the North Washington Street Bridge carrying the I-93 interstate highway across the Charles River. It is also easily recognizable by its two 330 ft tall inverted Y-pylons designed to honor the Bunker Hill Monument. Designing a gateway bridge next to an iconic structure meant designing a bridge to complement the Zakim Bridge and blend its design harmoniously with its surroundings. The design of the North Washington Street bridge is intended to function as a viewing platform for the Zakim Bridge. The uniqueness of an otherwise simple and lean superstructure would come from its substructure. It was decided to use five V-Piers to support the superstructure. The V-Piers would be akin to the reflection of the inverted Y-Pylons of the Zakim Bridge (Figure 11). The accent lighting would illuminate these piers to match the lighting scheme of the Zakim Bridge during a holiday or a special event, creating a symphony.

The existing abutments were modified to support the new superstructure. The layout of the piers was determined by the requirement to provide a navigational span, maintain or minimize the increase in loads to the existing abutments and to balance the loads on the pier arms of the V-Pier as much as it was possible. The existing approach span piers and the central swing span pier also limited where the new piers could be founded.

A V-Pier at pier locations 2 through 5 consists of four V-Pier columns (Figure 12 and Figure 13), arms and post-tensioned ties. The arms of each V-Pier extend out 25 ft from the centerline of the column and rise up to 30 ft from the base of the column. Owing to shallow depth of Pier 1, it was designed as a solid V-Pier without the triangular opening between the tie beam and the arms. High performance concrete would be used in the construction of column stem and arms. Connecting the arms together is to be the post-tensioned concrete tie beam.

The typical bearing at all supports with the exception of Pier 1 North is a unidirectional guided expansion disc bearing. At Pier 1 North, a fixed bearing is proposed. During construction phase however, the bearing at Pier 5 was temporarily made fixed as the construction of the superstructrure began from the north abutment proceeding south. A strip seal joint is proposed at the South Abutment because of its proximity to the fixed bearing location on Pier 1. At the North Abutment, the deck joint will be modular to accommodate a movement of ¾ in per 10 degrees Fahrenheit change in temperature.

Figure 11. Proposed Bridge Elevation Showing V-Piers Reflecting the Pyloos of the Zakim Bridge (Rendering)

The four pier columns at each pier location share a footing which would be supported on eight 6 ft diameter drilled shafts. The drilled shafts extend 40 ft. to 80 ft through layers of silt and till before being socketed into competent bedrock. The length of the rock sockets in competent bedrock vary from 10 ft to 27 ft between different piers.

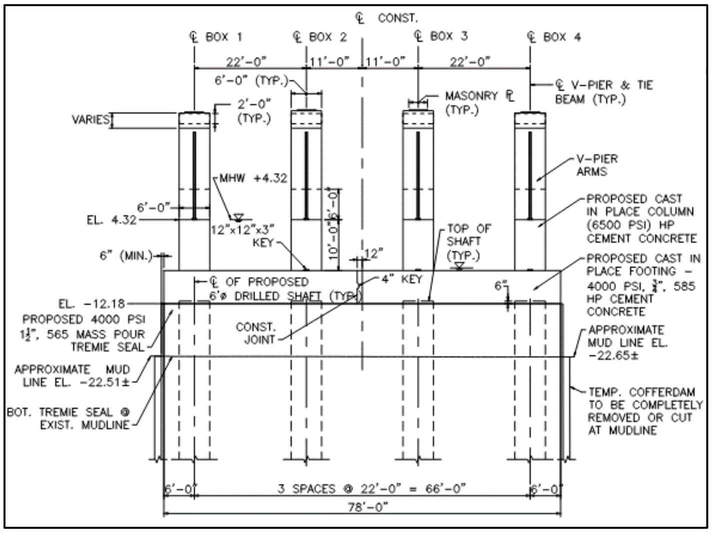

Figure 12. V-Pier 3 Transverse Elevation

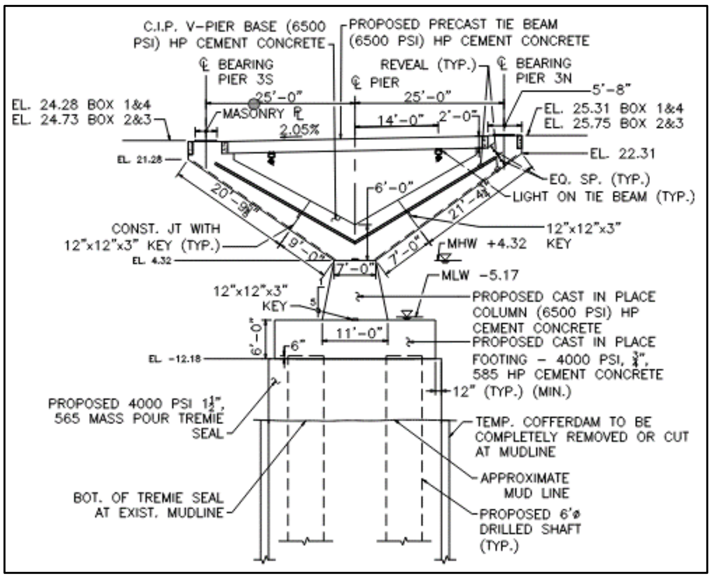

Figure 13. V-Pier 3 Longitudinal Elevation

Several flood scenarios were considered in the hydraulic modeling to represent a 100-yr event for hydraulic design, 200-yr event for scour analysis and a 500-yr event for scour analysis check. In addition to these, an extreme flood scenario during the highest tide of record plus sea level rise (SLR) was also considered in the Hydraulic Modeling and Scour Analysis Report.

Midas Civil and STAAD.Pro software were used for structural analysis for pier design.

3.1.3. V-Pier Arms

Each V-Pier arm extends 25 ft longitudinally from the center line of the pier column to the center line of the bearing atop the arm. The arms have a constant width of 6 ft. The maximum depth of the arms cross-section is 6 ft 9-⅞ in for Piers 2 and 4 and 6 ft $10^7/_8$ in for Piers 3 and 4 (Figure 1412). The cross-section depth tapers down to 3 ft 4¾ in for Piers 2 and 4 and 3 ft $11^7/_8$ in for Piers 3 and 4 at the top of the pier arms. The longitudinal reinforcement in the pier arms consists of two (Piers 2 and 5) or three (Piers 3 and 4) top and bottom rows of #11 rebars with 13 rebars in each top row and 12 rebars in each bottom row. The side faces of the arms consists of 5 #11 rebars.

3.1.4. Tie Beam

The tie beam is a 6 ft wide and 2 ft deep post-tensioned concrete beam connecting the tops of the north and south pier arms together (Figure 15). Each tie beam has ten 1¾ in diameter 150 ksi steel post-tensioning bars placed in 4 in diameter high density polyehtylene (HDPE) ducts.

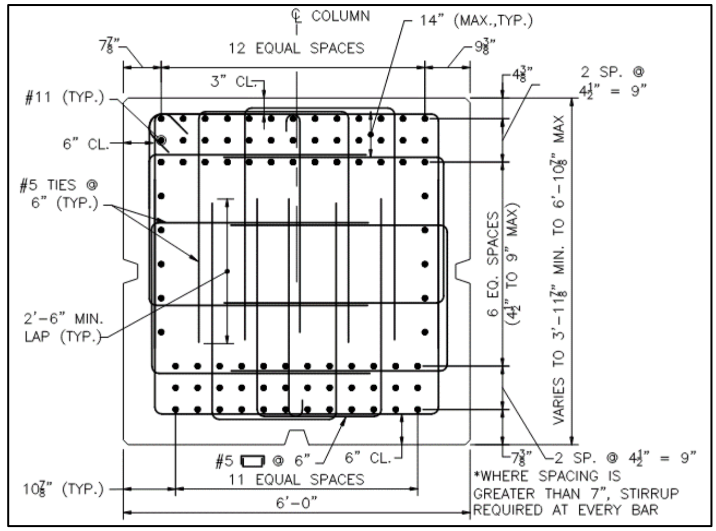

Figure 14. V-Pier 3 Arm Cross-Section

At Pier 1, the entire post-tensioning jacking force of 2200 kips is applied in one stage with estimated post-tensioning loss of 9%. At Piers 2 through 5, the post-tensioning force was applied in two stages (Figure 16). The first stage of post-tensioning force was applied when the precast concrete tie was placed on the ledges constructed at the top of the pier arms and when the concrete in the 12 in gap between the arms and the tie at both ends intended for closure pour was still not poured. The magnitude of initial post-tensioning force was 300 kips at the exterior piers (under exterior girders 1 and 4) and 150 kips at interior piers (under interior girders 2 and 3). This initial post-tensioning had the effect of off-setting the downward pier arm deflection due to self-weight of the arms and the tie beam and pulling the arms closer to induce tension in their bottom fibers such that the arms would be capable of resisting greater magnitude of subsequently applied loads. After the initial post-tensioning, the concrete pour filled the gap between the tie and the arms.

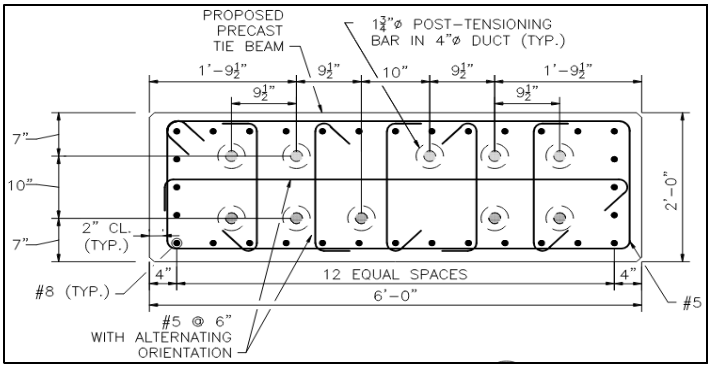

Figure 15. Post-tensioned Concrete Tie Beam

After the closure pour had cured, additional post-tensioning force of 2,000 kips was applied to the tie beam at exterior piers and 2,150 kips at interior piers. Total post-tensioning losses were estimated to be no more than 14% at the exterior piers and 20% at the interior piers. The post-tensioning system was required to be designed by the contractor such that no more than 12.5% of the pre-stressing force would be eccentric at any time. All subsequent loads would be applied after the closure pour.

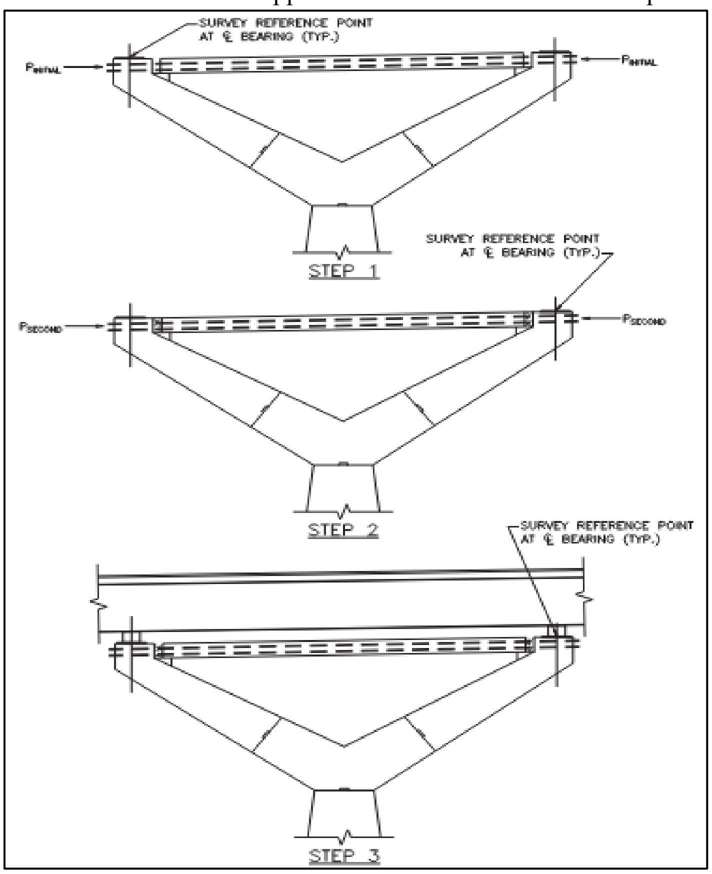

Figure 16. V-Pier Post-tensioning Sequence

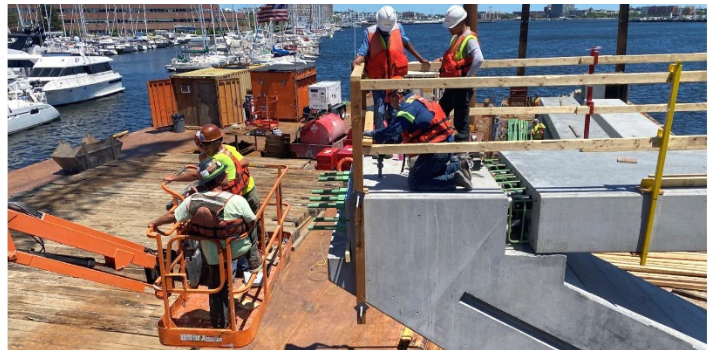

Figure 17. Post-tensioning Preparation at Pier Arm - Tie Beam Joint

3.1.5.Pier Column

The base of the 'V' and the pier column were constructed monolithically placing the construction joint in the arms at a distance of 8 ft horizontally at Pier 1 and at 8 ft 6 in to 9 ft 6 in for Piers 2 through 5. The reinforcement in the arms is mechanically spliced to the rebars extending out of the pier column. The reinforcing bars from the 'south' of the column section were bent and extend into the north arm of the pier while the rebars from the 'north' of the column section bend and enter the south arm (Figure 18). Due to the dense reinforcement at the interface of the arms and the pier column, 3D models of the pier reinforcement had to be developed to detect and avoid clash between the rebars going into the pier column from the arms (Figure 20).

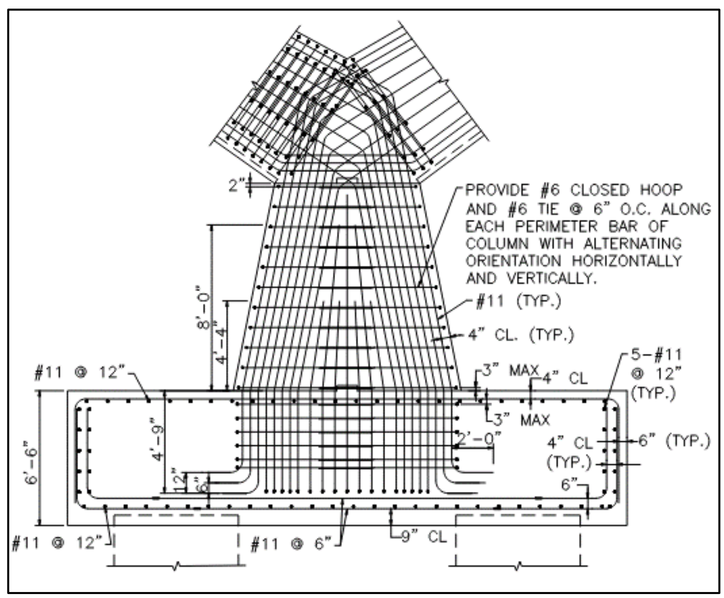

Figure 18. Typical V-Pier Column and Drilled Shaft Cap Reinforcement

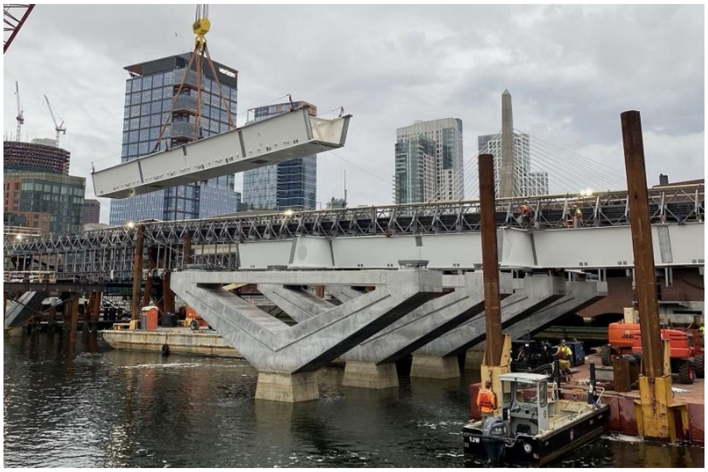

Figure 19. Girder Erection over V-Pier

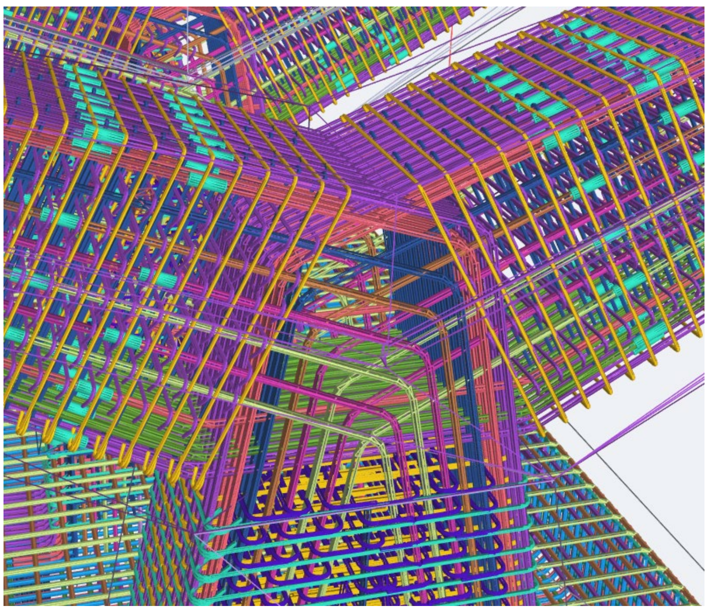

Figure 20 3D Model for Pier Reinforcement Clash Detection

The four pier columns at a V-Pier were constructed on a 6 ft 6 in deep drilled shaft cap. In concept and design, it is similar to a pile cap, only with larger drilled shafts instead of piles. It measured 78 ft wide and 27 ft long. The top of the drilled shaft cap were all set to an elevation below mean sea level. The caps of Piers 1 and 5 were in the riverbed, however, the caps for Piers 2 through 4 are perched at 10 ft. to 15 ft. above the riverbed (Figure 21). This design minimized the depth of the cofferdam and dewatering operations in the water and also minimized the handling and removal of contaminated soils from the riverbed. Eight 6 ft diameter drilled shafts supported the cap (Figure 22). The centerline of each pier column matched with the centerline of the two drilled shafts, just north and south of the base of the pier column. The drilled shafts were laid out in two rows with 16 ft 6 in spacing between them longitudinally and 22 ft transverse spacing to match the spacing between the centerline of the box girders and the pier columns.

3.1.7.Drilled Shafts

Drilled shafts had several advantages over other type of deep foundations including their ability to support large axial loads. Drilled shafts could also penetrate through possible obstructions like cobblers and boulders and their lengths could be revaluated and adjusted based on the subsurface conditions encountered during drilling. The drilled shafts also allowed for the perched caps and benefits previously described.

The drilled shafts were designed to support the proposed vertical loads by a combination of side shear resistance of 2.4 ksf from the glacial till and weathered bedrock layers and a side shear resistance of 7.0 ksf and end bearing resistance of 20.0 ksf from the competent Argillite bedrock layer. Additionally, the geotechnical report also recommended socketing the drilled shafts a minimum of one drilled shaft diameter into the competent bedrock and reducing the diameter of the drilled shafts rock socket by 6 in to 5 ft 6 in.

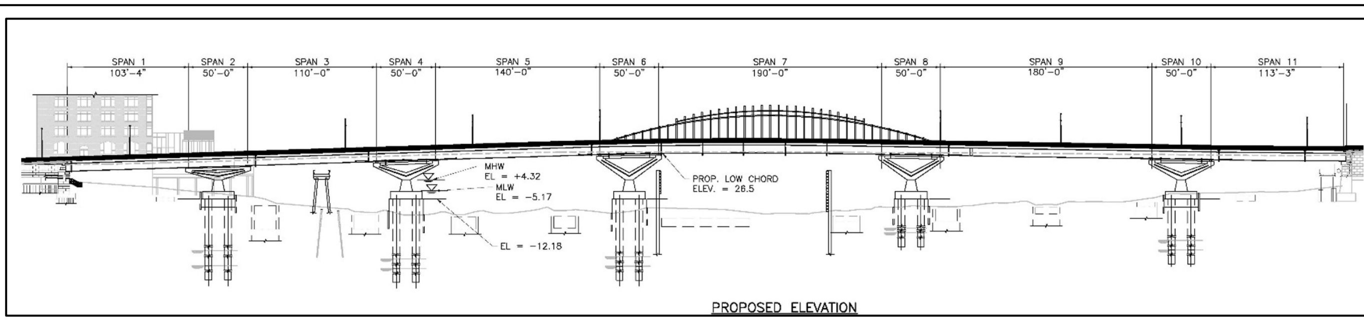

Figure 21. V-Pier Drilled Shaft Caps Below Mean Low Water

3.1.6.Drilled Shaft Cap

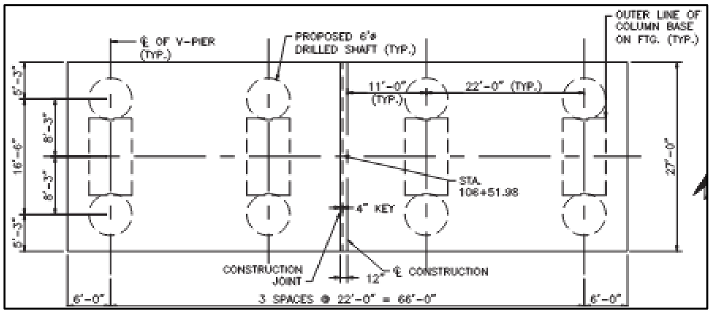

Figure 22. V-Pier Drilled Shaft Cap Plan

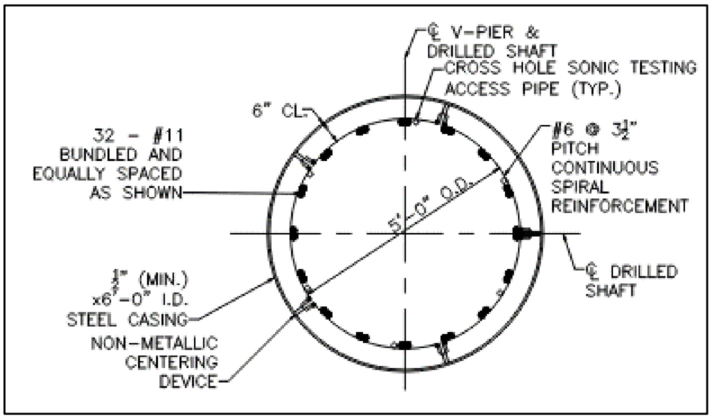

Figure 23. Drilled Shaft Cross-Section

Each drilled shaft is comprised of 32-#11 rebars bundled in pairs of two and arranged in a circular geometry. #6 rebars were used for continuous spiral reinforcement at 3½ in pitch (Figure 23). Clear cover to the spiral reinforcement was maintained at 6 in which reduced to 3 in (Figure 24) in the rock socket portion of the drilled shafts. Based on the loads, the depth of the rock sockets varied from 10 ft to 27 ft.

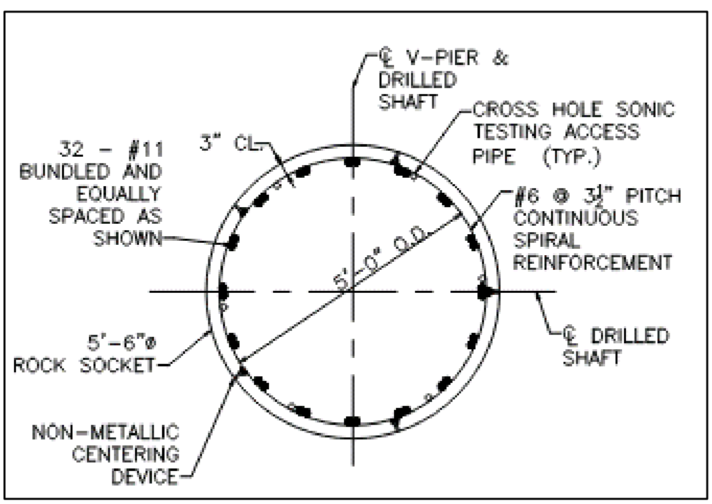

Figure 24. Drilled Shaft Cross-Section in Rock Socket

3.2. Architectural Trellis and Fascia Screening

A trellis was located along each side of the bridge in the main span to mark the crossing of the navigable channel underneath. It also provides a physical separation from the vehicular traffic and provides partial shade for the widened overlook areas on the span which contain sculptured benches, tree plantings and landmark plaques describing the views from the bridge. It resembles a circular arch in both plan and elevation as seen in Figure 25 and Figure 27.

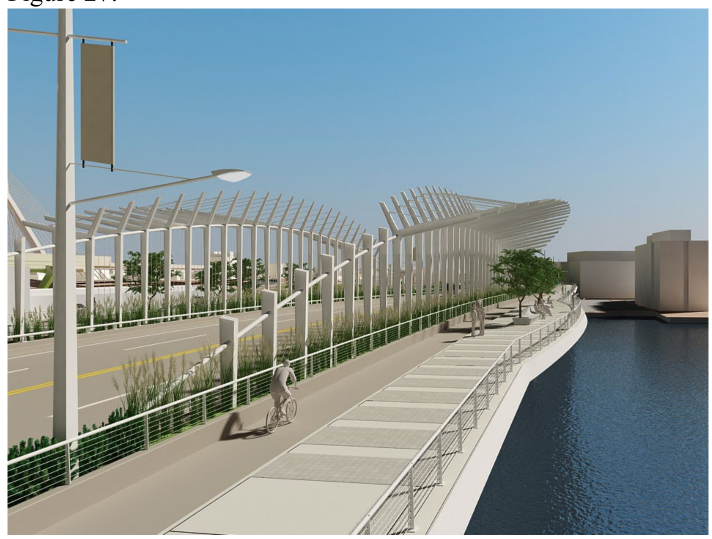

Figure 25. Architectural Trellis (Rendering)

Each trellis is composed of 14 straight and 23 'Y' shaped poles fabricated from welded plate members (AASHTO M270 Grade 50) and main and front linking members fabricated from steel pipe (AASHTO A53 Grade B) and round HSS (ASTM A500 Grade B). The arms of the 'Y' shaped poles are tied together by a $^3/_8$ in diameter S.S. Type 316 cable. All structural members would be hot dip galvanized. The tallest pole at the center is over 30 ft high and 32 ft across. The poles are spaced 7 ft 3 in on center and are tied together by 8 in diameter strong pipe main link and a 5 in diameter strong pipe front link.

Another architectural element to improve the bridge's aesthetics is the proposed bridge fascia screening. The screening slopes at a varying angle from the edge of the deck down to the level of the girder bottom flange and then extends horizontally just past the second stringer from the girder. The dotted line at the bridge fascia (under the deck) in Figure 26 is representative of the architectural screening. The structural elements that will support the weight of the FRP panels for screening can also be seen.

Six landmark plaques on the bridge railings in the Span 7 will also adorn the structure as will the two historical interpretive panels (TBD) and two dedication plaques the north and south approaches. The navigational lighting will be supported from the deck in Span 7.

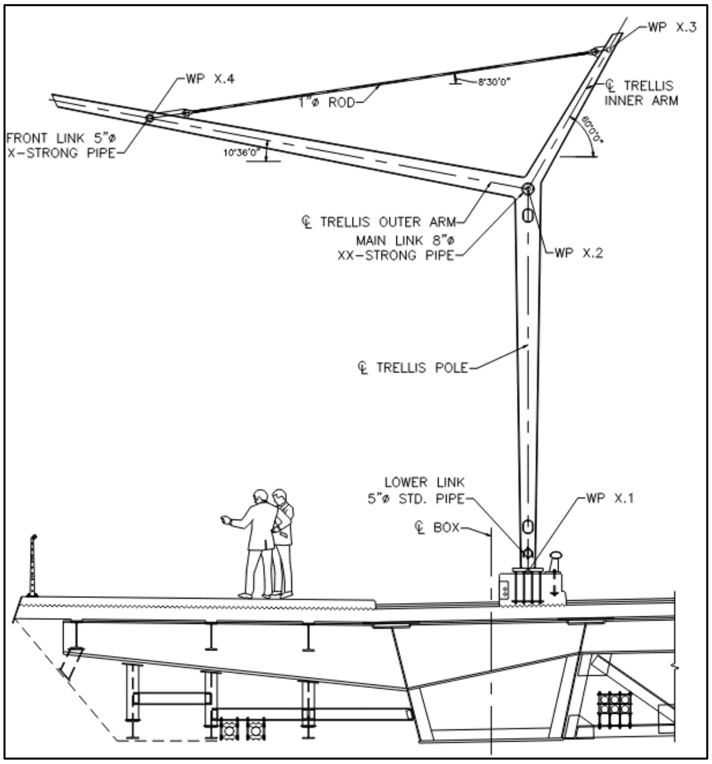

Figure 26. Tallest Trellis Pole

3.3. Fender System

The existing fender system was installed on both sides of the pivot pier and along the adjacent piers as seen Figure 1. The fender system was generally in fair to poor condition with minor collision damage, deteriorated wooden and steel piles and the walkways along the fenders had several floorboards missing.

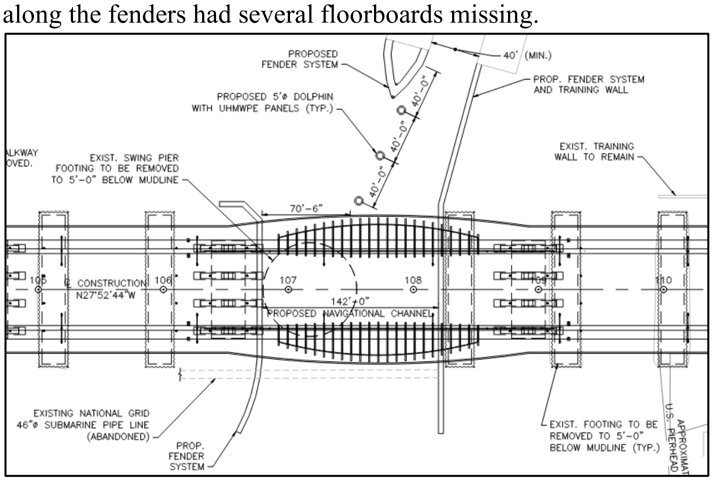

Figure 27. Proposed Fender System Plan

The proposed fender system would be constructed north of Pier 3 and south of Pier 4 with the latter extending further west as a training wall to meet with the large lock of the Charles River Dam (Figure 27). A bullnose fender system would also be constructed at the west wall of the large dam lock in addition to three 5 ft diameter dolphins with UHMWPE panels spaced 40 ft apart. The pipe piles in the fender system will be 20 in diameter ASTM A252 Grade 3 steel with minimum ½ in wall thickness. The 8 in x 10 in timber would be made of sawn lumber S4S with a minimum bending strength of 1200 psi. The whalers would be installed between elevations -8.5 to +13.5 at the south fender and from mudline to elevation +13.5 at the north fender.

3.4. Utilities

All utilities, but the two 115 kV transmission lines will be relocated twice during construction, first to the temporary utility bridge and then to the replacement bridge under the west sidewalk. Once the construction of the new superstructure is completed the utilities would be relocated to the three bays between the box girders with the exception of the high voltage transmission lines which would remain in place throughout construction. The following utilities will be carried by the replacement bridge:

- One set of 6 - 5 in diameter Eversource electrical conduits in both the west and east (exterior) girder bays.
- 6 - 5 in diameter MBTA electrical conduits in the west girder bay.
- 1 in diameter accent pier lighting conduit
- 36 in diameter gas main the middle girder bay
- 6 - 5 in diameter Boston Public Works Department electrical conduits in the east (exterior) girder bay
- 12 in diameter water pipe under the east sidewalk
- 4 - 4 in diameter Comcast cable conduits under the east sidewalk
- 2 - 3 in diameter BTD Traffic Signal conduits embedded in each sidewalk
- Multiple 2 in diameter lighting conduits embedded in the sidewalks and bike barrers

4. Current Project Status

The bridge is under construction with projected completion in 2023 (Figure 28).

5. Conclusion

The existing structure was designed to serve the functional needs of a 1898 industrial era. Complete with trolley cars and street cars, narrow lanes for slow moving horse drawn carriages and a swing span that allowed for tall vessels to pass upstream to the Charles River. The structure provided this valuable service for 120 years.

The proposed bridge is designed to meet the needs of our city's innovation hubs and urban areas, complete with a Bus Rapid Transit Lane, 4 vehiclular lanes, 2 protected bike lanes, 2 wide sidewalks and accomodations for the numerous utilities that keep our city connected. The proposed bridge will function as a Complete City Street while providing a place to congreagate on the main crossing and simultaneously marking the gateway between the Chalres River and the Boston Inner Harbor.

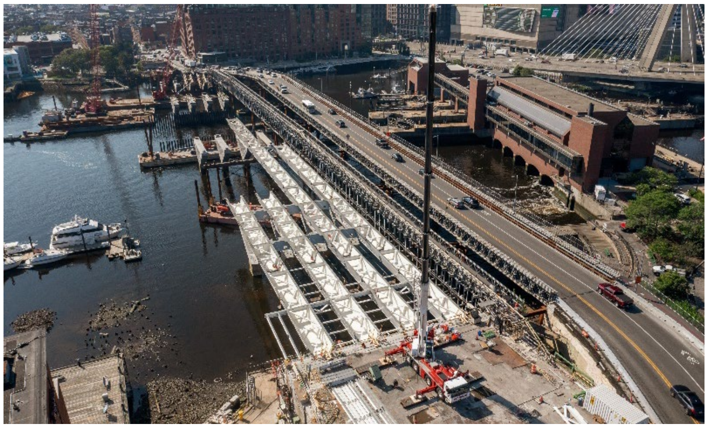

Figure 28. Construction Progress (left, white), Temporary Utility Bridge (middle) and Temporary Bridge (right) looking South

It is a testament to how the public and the community collaborates to advocate and add value to our public transportation projects to meet the demands of users and stakeholders. This bridge will connect the residents to their workplaces, schools, healthcare facilities and other essential businesses and services; the recreational cyclists and the pedestrians to the parks along the Charles River and waterfront; the tourists and the visitors to the sites along the Boston Freedom Trail and support the numerous utilities including electric, gas and communications that cross from Boston Proper to Charlestown for the next century.

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Alfred Benesch & Company – Project Management, Design and Construction Phase Engineering
Rosales + Partners – Bridge Architecture
GZA – Geotechnical Engineering and Fender Design

References

Bridge drawings – Bridge plans by Alfred Benesch & Company modified text size and sheet references removed for clarity.
Bridge photos – Alfred Benesch & Company (Engineer)
Bridge renderings – Rosales + Partners (Architect)
Bridge 3D model – J.F. White Contracting Co. (Contractor)

