DESIGN AND CONSTRUCTION OF THE NORTH BANK BRIDGE

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Summary

The North Bank Bridge is a 213 m long bridge being built as part of a new riverfront park in Boston and Cambridge, Massachusetts, USA. As part of a large scale restoration of the Charles River Basin, the project will transform a 60,000 m^2 industrial site into inviting and vibrant parkland, while facilitating the flow of non-motorized traffic through this heavily travelled area. Ammann & Whitney designed a "sinusoidal bridge" for the highly constrained site which was conceptualized in cooperation with Buro Happold in a highly community oriented process. The tubular steel truss bridge features fibre reinforced polymer decking. Analysis of the bridge included a thorough investigation of lateral vibrations and global buckling. The project was initiated as a remediation measure for the Central Artery / Tunnel Project and was ultimately constructed with American Recovery and Reinvestment Act funds. Construction is distinguished by the collegial atmosphere and the Contractor's efficient method of producing the irregular truss geometry.

Keywords: footbridge; tubular; truss; lateral vibrations; global buckling; fibre reinforced polymer (FRP)

1. Background

1.1 Site History

The Alster-damm in Hamburg made that city "one of the most beautiful in the world," according to a Boston resident who in 1859 published an etching of the Alster River in a memorial addressed to the state legislature. He was convinced that Hamburg was an ideal model for Boston's future. Instead of reeking, dangerously polluted mudflats exposed at every low tide, a dam near the mouth of the Charles River would create a great "water park" in the heart of the city. Fifty years later an earthen dam was completed near the mouth of Boston Harbor, and landscaped esplanades were laid out on both sides of the river. The Charles, once a foul threat to public health, became the centrepiece of America's first regional park system. With the river as foreground, Harvard University, the Massachusetts Institute of Technology, and Boston University erected new campuses along the esplanades between 1913 and 1939.

At the mouth of the Charles, however, the esplanades were cut off from the harbour by a half dozen railroad bridges and trestles that crossed the river to connect Greater Boston with northern New England. And when the Central Artery, a six-lane elevated highway, was built through the centre of the city in the 1950s, it further isolated the Esplanade from the harbour and divided and isolated neighbourhoods throughout the city.

Less than twenty years later, engineers began discussing the demolition of the highway. Long expanses of brick and granite warehouses had been transformed into residences and new residential towers followed. The city had begun linking sections of the harbour's edge into parkland, and the regional parks agency drew up plans to connect the Charles with the harbour for the first time. If the highway were replaced by a tunnel, according to these visionaries, the city would be made whole once more.

This vision became reality with the execution of Boston's Central Artery / Tunnel Project (CA/T). The elevated highway through the city was relocated to a tunnel on the same alignment, and transitioned to gorgeous new bridges over the Charles River Basin. Federal regulations required mitigation for the impact of this work on existing and planned parks. Further, the new bridges required the use of land owned by the regional parks agency. To settle these state and federal requirements, the state highway department agreed to a program of pedestrian and bicycle paths and the construction of 800,000 m^2 of new public spaces, including the construction of the North Bank Bridge.

1.2 Project History

The bridge design began with the derivation of multiple concepts and engagement with a very active local community. Once a concept was chosen, the final design phase began. The unorthodox nature of the design required a thorough analysis with several layers of review, including an independent review by Buro Happold. Responsibility for the project was unavoidably transferred between Authorities, creating managerial challenges. Project partners faced significant hurdles in transforming the industrial site into parkland related to existing infrastructure and contaminated soils.

In 2007, the Authority solicited bids for the park project and the low bid was approximately 20% higher than estimated. Asserting that the bids exceeded the funds on hand, the Authority determined not to proceed with the work. When the American Recovery and Reinvestment Act (ARRA) became law in 2009, high priority was given to projects that were “shovel-ready,” with completed bid documents ready for advertisement. The North Bank Bridge was chosen to receive funding. This time, the low bid was below the estimate and construction began in 2010.

2. Design

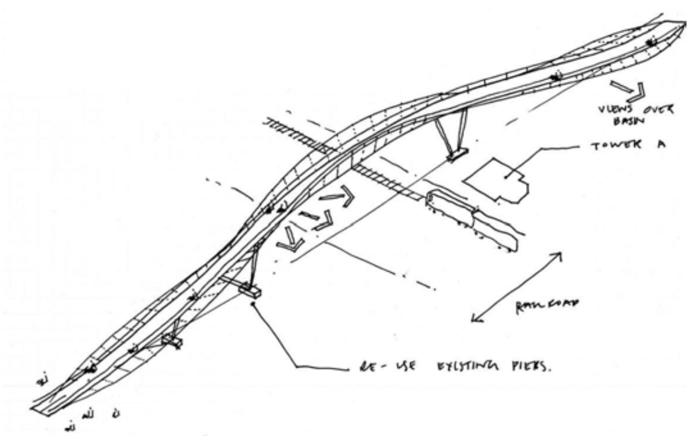

Figure 1 Concept sketch of bridge and site, from southwest looking northeast

2.1 Conceptual Design

The first phase of design was carried out in collaboration with Buro Happold and Julian Hakes. Through a series of site visits and design charettes, several concepts were developed and presented to the community.

A number of options were studied in an attempt to address the multitude of geometric constraints. The constraints, taken from west to east, are: clearance above an amphibious vehicle (Boston Duck Tours) launch ramp, clearance above railroad tracks bisecting the park, clearance between a historic building (Tower A) and a parallel highway ramp (Ramp CT), clearance below the Leverett Circle Connector Bridge, clearance above the Millers River, and clearance below the Zakim Bridge. Taken together, these and other physical features represented a significant challenge in arranging the structure.

The initial options included a “rainbow arch” and asymmetric “solid industrial” design but the preferred solution was dubbed the “sinusoidal bridge” because of its snaking, undulating form in plan and elevation (see Figure 1). The bridge is supported by a truss structure that is positioned alternately below and above the deck level according to the site constraints.

At the west and east, approach embankments are provided to a height of approximately 3 m at either end of the alignment. The structure commences with a minimal depth as the trusses sweep below the deck in the approach spans. They then rise above the deck over the railroad so that the necessary 5.6 m clearance is achieved while minimising the overall length of the structure. Through the transition, the trusses fold in close to the walkway, accommodating the narrow gap between Tower A and Ramp CT. The alignment was also set to allow for the re-use of existing foundations, minimising cost, risk, and disruption.

The selection of steel pipes for the truss members was made as these sections can be readily bent to form complex curves in plan and elevation without a concern that imperfections will be noticeable. Above the railroad, a high sided protective mesh screen is required and fit neatly within the elevated trusses.

The design is satisfying for an engineer for a number of reasons. Firstly, it is a simple form that is born out of the site constraints. The bridge is a direct response to the project brief and would not work on another site. Secondly, the bridge’s beauty comes from the expression of the structure alone and not because of any add-ons or cladding. Thirdly, the collaborative team work that occurred in evolving the design concept was an enjoyable process but one which also enabled the relevant stakeholders to give input at an appropriate stage to produce a memorable structure.

2.2 Final Design

2.2.1 General

The final design was founded on a holistic, iterative approach encompassing overall geometry; foundation and substructure design; and analysis of global buckling, lateral vibrations, and individual members and joints; all while prioritizing preservation of the approved concept, the user experience, maintainability, and constructability.

The tubular steel truss is made up of deck chords and sweeping outer chords connected by posts (dubbed vertical members) spaced at 3.2 m on centre, and intermittently reinforced with diagonals. At each vertical, there is a floorbeam that supports the fibre reinforced polymer (FRP) deck. The deck acts as a composite member with the truss. The shear action of the deck is augmented with steel lateral braces as required. Throughout the length of the bridge, the sweeping outer chords of the truss remain outboard of the deck chords so that the angle of the verticals to the walkway varies as the outer chord follows its sinusoidal path.

The 7 span continuous structure has a total centreline length between abutment bearings of 212.73 m (see Figure 3). The span arrangement is symmetrical about the main span with span lengths from west to east of 25.40 m, 25.40 m, 31.75 m, 47.63 m (railroad span), 31.75 m, 25.40 m, and 25.40 m.

The horizontal alignment of the bridge centreline consists of a simple reverse curve, with the point of reverse curvature located at the middle of the bridge. The first curve has a radius of 304.8 m and the second has a radius of 396.2 m.

The profile of the bridge deck is symmetrical about the apex at the midpoint of the bridge. The profile consists of constant approach grades of 4.9% (in accordance with the Americans with Disabilities Act), connected by a 41.3 m long crest vertical curve (see Figure 4). The approach ramps leading up to the bridge spans at each end of the bridge carry the pathway from grade up to the bridge at approximately a 5% slope.

The truss chords are 324 mm diameter pipes with a thickness of 25.4 mm. The verticals are 324 mm diameter pipes with a thickness of between 9.5 mm and 12.7 mm. The diagonals are 168 mm diameter pipes with a thickness of 14.3 mm. The floorbeams are 254 mm wide x 152 mm with a thickness of 12.7 mm. The lateral braces are 127 mm x 127 mm with a thickness of 12.7 mm. The FRP decking is approximately 127 mm thick and is composed of top and bottom skins joined by closely spaced webs, oriented longitudinally.

The original concept called for Vierendeel trusses which would require rigid joints. The guideline for rigid joints is to use a ratio of 1:1 for chord:vertical diameter, and a ratio of 2:1 for chord:vertical thickness. Though the Vierendeel concept was ultimately found to not be feasible, the use of the above ratios was continued to the greatest extent possible after adding diagonals to the trusses to capitalize on the resulting joint strength.

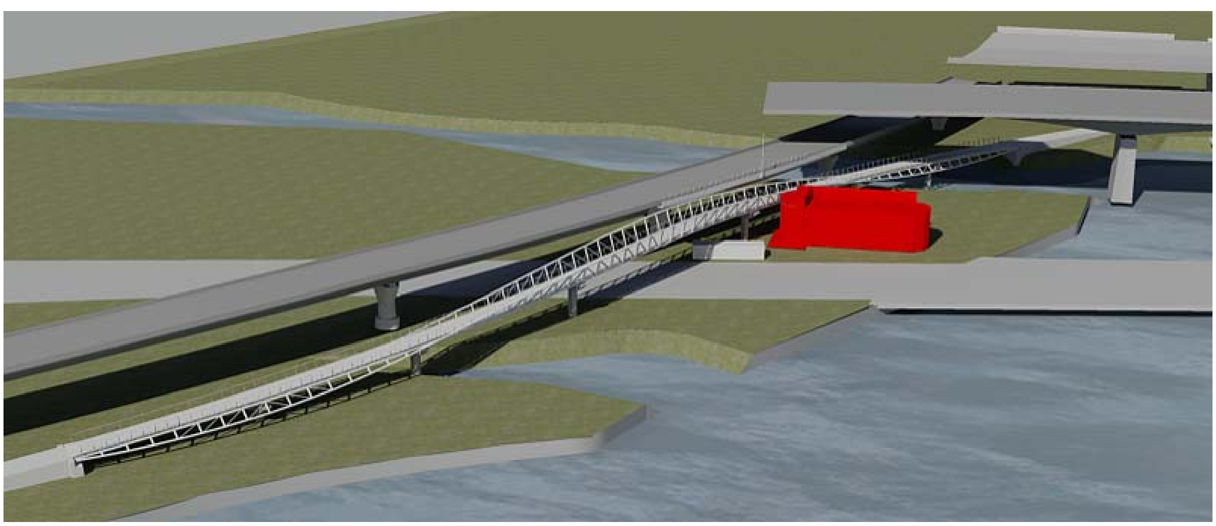

Figure 2 Rendering of bridge and site, from southwest looking northeast

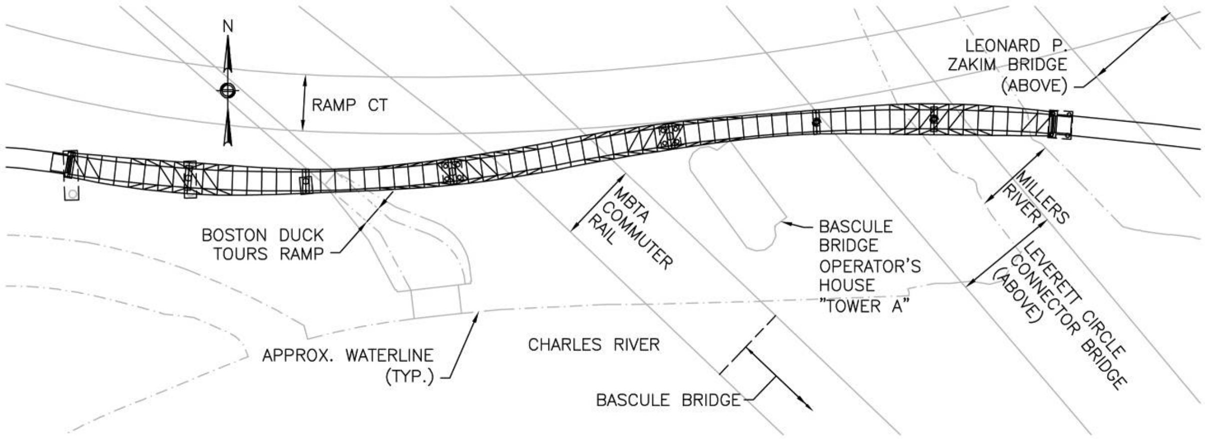

Figure 3 Plan of bridge and site

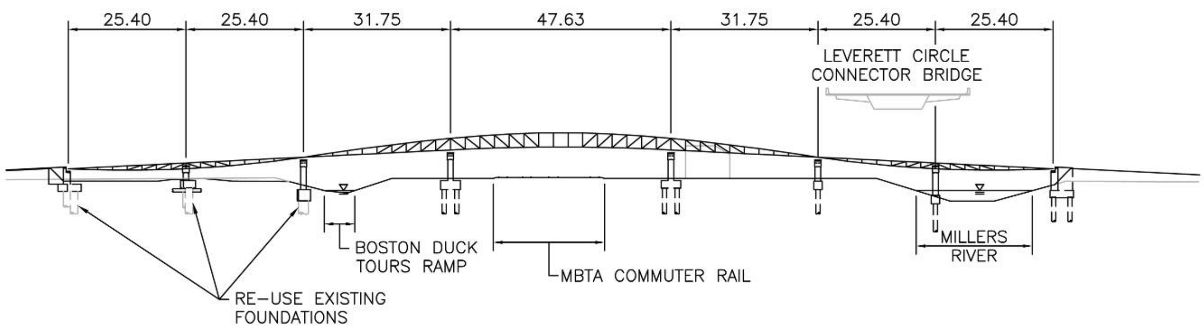

Figure 4 Elevation of bridge and site

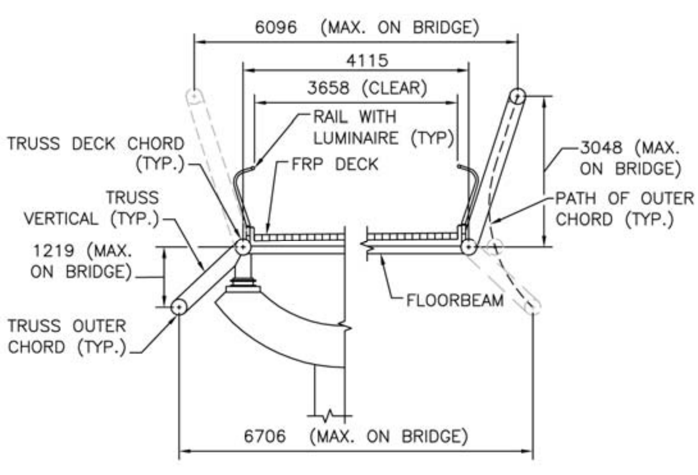

Figure 5 Typical section of bridge

2.2.2 Setting the geometry

In determining the geometry, there were several major goals: follow the conceptual design, avoid site constraints, re-use existing foundations, and meet the accessibility requirements of the Americans with Disabilities Act. Further, the geometry is a major determinant of the structure's behaviour, requiring an iterative approach.

To address the above a controllable, repeatable approach was adopted in which the geometry was determined parametrically. The centreline was set as two circular arcs in plan, set in relation to the existing foundations and major site features and constraints. Accessibility requirements largely determined the vertical geometry of the centreline. The deck chords were set to simply follow the centreline. The outer chord geometry was then determined using an interesting algorithm.

First, a series of "sinusoidal" arcs were set in elevation relative to centreline. Next, a set of lines joined by an arc were defined in cross section (see Figure 5). Given a longitudinal position along the centreline, the vertical location of the outer chord was determined from the sinusoidal arcs. Given this vertical location, the horizontal location in section was then determined from the set of lines joined by an arc. These sections were then radially applied to the three dimensional centreline to get a set of three dimensional working points. All of this work was performed with a simple spreadsheet.

The data was used to automatically draw a solid model of the bridge. This model was then inserted into a model of the site which included laser survey at the point of minimum clearance at the control tower. With this composite model, the aesthetics and clearances were examined. The data was also used to rapidly generate the analysis model. Any issues with the geometry found in the site model or the analysis model were easily addressed by adjusting the parameters of the algorithm in the spreadsheet and performing another iteration. The finalized working points were presented as a table in the Contract Drawings.

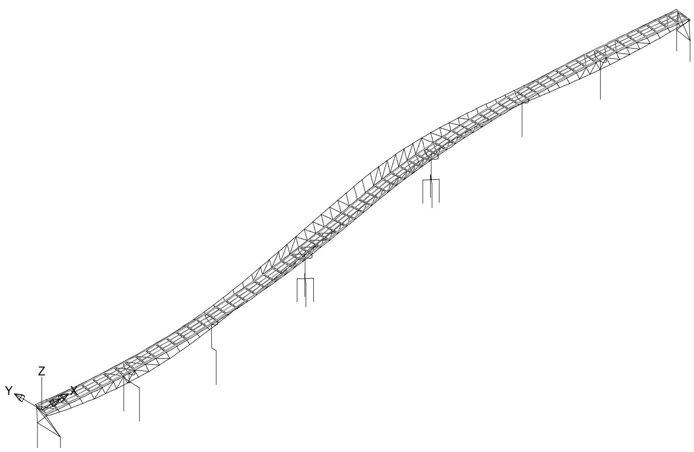

Figure 6 Lusas model of bridge

2.2.3 Global analysis model

The bridge was modelled with finite elements using Lusas software. Foundations, substructure, and steelwork were modelled as beam elements and the deck was modelled with shell elements (see Figure 6). The deck acts compositely with the truss, and the model includes the effects of staged erection of the deck. The model was used to determine member forces and to examine the structure's lateral vibration characteristics and stability. Further details on the model can be found in the relevant sections below.

2.2.4 Foundations

The site has relatively poor soil conditions which are unsuitable for shallow foundations. Numerous subsurface obstructions are known to exist, and vibrations due to pile driving were a concern at Tower A. Also, the new structure incorporates existing drilled shafts that were constructed during the CA/T project for temporary ramps. It is generally undesirable to mix multiple foundation types for a single structure because of the significant differences in lateral stiffness. Drilled shafts were used extensively on the CA/T, so there was knowledge and equipment available in the area. For these reasons, drilled shafts were chosen for the new foundations.

Analysis of the shafts used methods specified for use on the CA/T. Non-linear soil / shaft interaction was examined in a stand-alone program. The program was first used to develop equivalent free standing cantilever lengths of shafts. These lengths were used in the global analysis model. Forces thus found with the global analysis model at the top of the shafts were then applied to the stand-alone models. Design forces were extracted and standard moment / axial force interaction methods were used for design.

Leading up to the ends of the bridges, pathways are on embankments which vary in height from 0 m to approximately 3 m. Construction of these embankments was anticipated to result in consolidation of the underlying organic deposits and subsequent large settlements. The soil was therefore pre-consolidated to avoid supporting the embankments on deep foundations or using a lightweight fill. Unlike on the bridge itself, any future settlement on the approaches can be addressed rather simply.

2.2.5 Deck

The bridge incorporates an FRP deck for reasons discussed in Section 2.2.6 Vibrations. It is typical to attach FRP panels to steelwork with a relatively lightweight clip detail. This approach results in non-composite behaviour between the decking and the steelwork. On the North Bank Bridge, the FRP deck is made composite with the truss with a high strength bolted detail in which the lower skin of the deck is bolted to steel angles which are welded to the floorbeams. The bolts are tensioned after the bridge is erected and the deck has been surveyed and shimmed into proper position. The deck is therefore not composite for dead load. The analysis model reflects this staging.

As mentioned above, the deck was modelled with shell elements. The out-of-plane bending and shear behaviours are not important to the global behaviour of the bridge, so the shells were made to represent axial, in-plane shear, and in-plane bending behaviours only. The deck elements use orthotropic properties to reflect the actual material properties in each direction. An equivalent thickness was calculated for each of the three desired behaviours. The calculated thicknesses were very close to one another and the lowest was used. A comparison of hand calculations and results from stand-alone FEM models of the deck verified this approach.

2.2.6 Vibrations

Conscientious of the ongoing interest within the engineering community in pedestrian induced vibrations, this issue was discussed at length with the Authority at the earliest stages of design. After a thorough discussion, we were directed to design for a minimum frequency of 3 Hz for the first vertical mode per the American Association of State Highway and Transportation Officials (AASHTO) Guide Specifications for Design of Pedestrian Bridges. Since the frequency of the horizontal cyclical load due to walking is half that of the vertical, we were directed to design for a minimum frequency of 1.5 Hz for the first horizontal mode (see Figure 7).

Meeting these requirements required several iterations of truss geometry, truss member sizes, location of truss diagonals, bearing layout, and substructure and foundation design. It was also found beneficial to substitute an FRP deck for the previously assumed concrete deck. Due to FRP's high stiffness to weight ratio, this had a dramatic effect on the vibration behaviour of the bridge.

At the end of this process, the bridge met the above criteria, but it was deemed prudent to perform a more thorough analysis. As is now widely known, on a bridge with a low lateral frequency and the correct distribution of pedestrian lateral loading, a resonant response can occur. If a small group within a large crowd of pedestrians happens to be walking in phase, this group can set up a resonant response, which will encourage others to synchronize with them. Eventually, this feedback will produce motions that can become disturbingly large.

Armed with this knowledge, we applied a horizontal load due to walking as a distributed cyclic load at a range of frequencies and found a maximum response at 1.6 Hz. The load applied was roughly equivalent to a dense crowd walking with 100% synchronization. Scaling the response to 20% was taken to represent a random synchronization of 20% of the dense crowd. This scaled reaction was very small. We could find no published guidelines, but we judged that at these very small levels of movement no feedback would occur. In searching the literature, we did find serviceability limits for horizontal vibration in ISO 2631 (1980). We found that in order to produce an unacceptable structural response per these guidelines, a synchronization rate of 75% of the dense crowd would have to occur.

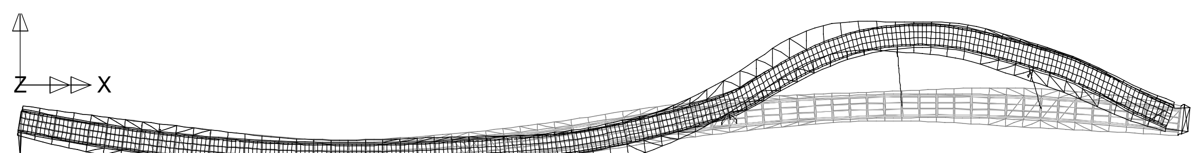

Figure 7 Plan showing first lateral mode of vibration, f = 1.6 Hz

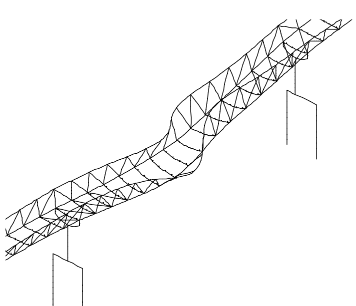

Figure 8 Detail of buckled outer chords at main span

2.2.7 Buckling

At several locations along the bridge (most notably at the main span), the outer chord is in compression and bracing against buckling is provided only by the truss verticals. The degree of lateral restraint provided by the verticals is dependent on their cross sectional properties, length, and distance from a support. Since these factors vary along the bridge, the bracing they provide to the compression chord also varies. The compression chords themselves follow a non-planar path. The buckling behaviour of the bridge is therefore of concern and is non-trivial to determine. A full nonlinear buckling analysis was therefore carried out.

The nonlinear analysis used a distributed load approximating the self weight of the bridge (approximately equivalent to live load) applied in a variety of patterns. The load was incrementally increased. At each iteration, the deformed geometry for the previous iteration was used as the basis of the stiffness matrix. For the purposes of this analysis, materials were assumed to be elastic. The deformations at each iteration were examined and the structure was found to behave linearly up to approximately fifteen times the starting load (see Figure 8). This demonstrated that the behaviour of the structure is linear well beyond the material limits. Therefore, a linear analysis was used for the general analysis and stresses were limited to the elastic range. Similar analyses were carried out on segments of the bridge to verify constructability.

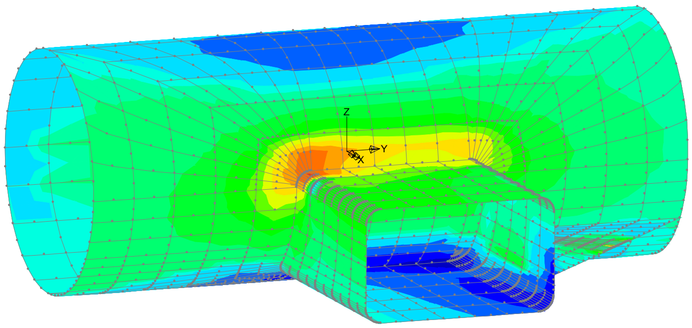

Figure 9 Shell element model of floorbeam to chord joint

2.2.8 Fatigue

Welded tubular joints typically have pronounced local stress concentrations, commonly referred to as hot spot stresses. The range of stress at the hot spot must be kept within a given allowable range. For most locations, a stress concentration factor was found in the literature. At floorbeam to deck chord connections, no stress concentration factors could be found so a finite element model was used (see Figure 9). The rectangular floorbeam and circular deck chord were modelled with shell elements. Member loads taken from the global model were applied to find the hot spot stress ranges directly.

2.2.9 Low maintenance

The steelwork is composed of closed sections and the joints are completely sealed. This limits interior corrosion by preventing the cycling of air and intrusion of water. To minimise maintenance of the exterior protection system, the steelwork is metalized, rather than painted. Further low maintenance details include the FRP deck, stainless steel safety infill, and LED lighting. Dissimilar materials were scrupulously isolated to protect against galvanic corrosion.

2.2.10 Pedestrian experience

The pedestrian experience was treated with great care. All butt welds are ground smooth. The multi-piece curb was carefully detailed to allow access for the various trades during construction and a pleasing appearance to pedestrians in service. Ammann & Whitney's railing concept is bridge specific. To minimise visual clutter, the posts are layed out with the truss bays, leaning away from the user at the ultimate slope of truss. The horizontal infill of tensioned wires maximizes transparency to the user, but creates a "ladder" effect which can be dangerous for children. A return at the top of the railing post simultaneously eliminates this ladder effect and presents the handrail back to the user. Lighting is provided by an LED light strip integrated into the handrail. Required safety infill is provided in each truss bay at the railroad span. A tensioned stainless steel mesh infill is used, eliminating additional cross frames required by rigid infill.

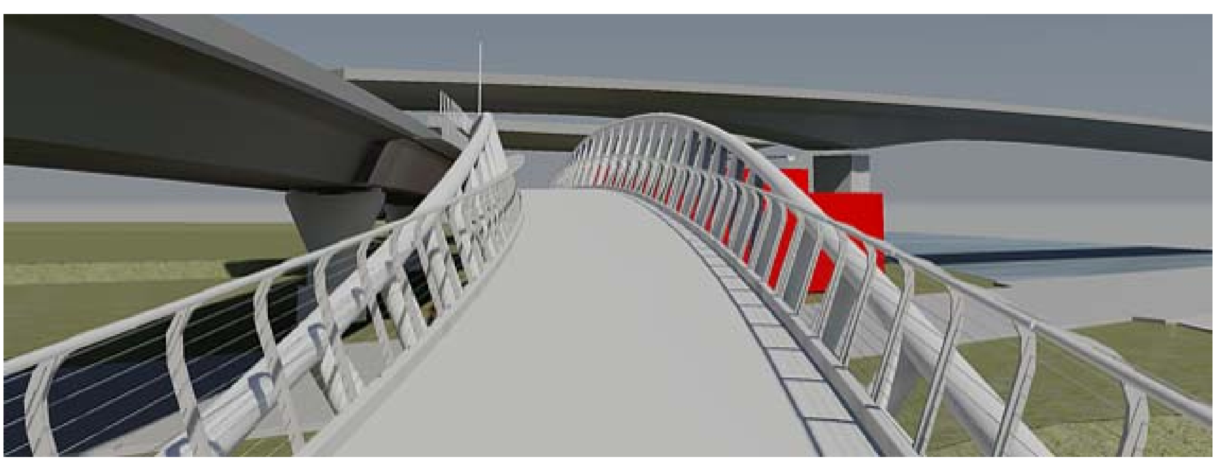

Figure 10 Rendering of bridge and site from bridge user's perspective

3. Construction

The specific capabilities of the Contractor suggested a series of small, but important modifications to the plans. The Contractor, Consultant, and Authority worked closely to resolve each issue with either no reduction in quality or an improvement. This spirit of cooperation carried over to more mundane aspects of construction, such as dealing with unforeseen buried conditions. The Authority deserves a large share of the credit for this success for fostering an environment of open communication.

The Contractor proposed a simple and effective method for fabricating the complex geometry of the truss chords. First, a non-uniform rational basis (NURB) spline was defined using the working points provided on the drawings. This spline was used as the reference geometry. The spline was then broken down into a series of lengths which matched the length of pipe which could be easily procured. Each segment was then approximated by a planar element, consisting of discrete bends and tangents. Each planar element is connected to its neighbour with the ends “timed” together such that the overall reference geometry is approximated with these planar elements to within 2 mm. A three dimensional solid model of the proposed finished geometry to be produced by this method was provided to the Authority by the Contractor, examined by the Consultant for aesthetic integrity, and approved.

For ease of handling the pieces, the trusses are fabricated on their sides. The coordinates supplied in the contract documents are converted into relative shop coordinates. A target and total station are used to precisely locate pipe supports. Each chord segment is set in a minimum of three supports. Bending takes place on site so that any segment found to not fit properly into the surveyed supports can be easily returned to the bending process. Completed truss halves (north and south) are temporarily connected into sections and surveyed. Adjacent bridge sections are then pre-assembled prior to shipping.

The sections of the bridge are shipped over the road with a field joint at the middle of each floorbeam. The bridge was designed such that the full cross section could fit through all marine obstructions, but an optional floorbeam splice was provided to expand the pool of bidders. All field splices are detailed with a retractable backing ring.

4. Acknowledgements

This project could not have been completed without the hard work of the people at the Central Artery / Tunnel Authority, MassDOT, and the Department of Conservation and Recreation; nor without the support of Governor Deval Patrick and the MassDOT Board; nor without the dedication of the community, including the Citizens Advisory Committee for the New Charles River Basin, the Charles River Watershed Association, MassBike, the Charles River Conservancy, WalkBoston, and the Conservation Law Foundation; nor without Ammann & Whitney’s design partners: CRJA / Carol R. Johnson Associates, Stantec, GPI / Greenman-Pedersen Incorporated, and Buro Happold.

5. Conclusion

The Central Artery / Tunnel Project removed a cleft in the city and allowed it to become whole again. As one of Central Artery’s mitigation commitments, the North Bank Bridge removes a similar cleft between the riverfront parks and the harbour. It is our sincere hope that the bridge is found to be useful and enjoyable to the public.

Figure 11 Rendering of bridge and site