# EXISTING NORTHERN AVENUE BRIDGE EVALUATION MEMO

## PROJECT BACKGROUND

The Northern Avenue Bridge over the Fort Point Channel in Boston MA was originally constructed between 1905 and 1908 and has been repaired / rehabilitated numerous times throughout the years. Due to severe deterioration, the bridge was closed to vehicular traffic in 1997 and closed to pedestrian traffic in 2014. The 2014 closure was prompted by a new revelation that several floor beams supporting the pedestrian walkway had a calculated live load rating capacity of zero tons. This finding was the result of an inspection and rating effort provided by TranSystems. Since then, the bridge has been out of use and left in the swung open positon.

Through the 2017 consultant selection process, AECOM was selected by the City of Boston as the Consultant for the project. As the first step of the Contract, AECOM performed another iteration of the bridge inspection as an independent effort to compare with the results of the 2013 inspection performed by TranSystems. Based on the inspection and structural analysis, AECOM has further evaluated the feasibility of rehabilitating or preserving the bridge.

## CONDITION INSPECTION SUMMARY

In 2017 AECOM performed a hands-on structural inspection of the Northern Avenue Bridge and submitted an Existing Conditions Report to the City of Boston on March 30, 2018. The purpose of this inspection was twofold: compare the existing conditions found in the 2013 Routine & Special Members Inspection Report prepared by TranSystems, and evaluate the potential steps necessary to rehabilitate or reuse existing structural members.

### Floor System Condition

The deck and floor system, including all deck and structural framing elements as well as the sidewalk cantilevers and lower lateral bracing, were inspected in 2013. These members were not re-inspected in 2017 based on the severity of the condition noted during the previous inspection. The deck and floor system were found to be in critical condition due to widespread deterioration, and a portion of the sidewalk cantilevers were noted as a risk for imminent failure at the time of the 2013 report. Refer to Images 1 and 2 below, showing sample floor beam and stringer conditions.

Image 1: 100% Section Loss in a Swing Span Floorbeam

Image 2: 100% Section Loss in an Approach Span Stringer

Truss Elements Condition

The truss members themselves varied in condition based on their location along the bridge and by element. The lower chord members exhibit moderate-to-severe corrosion and deterioration (up to 100% section loss) concentrated around the ends of the members near the pin assemblies (See Image 3). The upper chord members are observed to be in generally satisfactory condition, with a few scattered deficiencies and corrosion with no significant visible deterioration (See Image 4).

The condition of the vertical truss members vary along their length, with the areas below the deck possessing moderate-to-advanced corrosion and the areas above the deck in generally satisfactory condition with minor deficiencies and scattered corrosion. In general, the portions of the verticals which extend below the deck and pin joint are severely deteriorated, as seen in Image 5.

Similar to the verticals, the diagonal truss members generally show moderate-to-advanced corrosion below the deck level, especially concentrated around the pin joint areas. An example of section loss of the diagonals at the lower pin joint can be seen in Image 6.

The upper lateral bracing members are generally in satisfactory condition with isolated locations of moderate to advanced deterioration concentrated mainly at the ends of the members. The upper sway bracing on the swing span is generally in satisfactory condition with a few scattered deficiencies.

Image 3: Severe Deterioration and 100% Loss of Lattice Bracing in the Swing Span Lower Chord

Image 4: Typical Top Chord Member in the Approach Spans

Image 5: Severe Deterioration of the Verticals Below the Deck and Pin

Image 6: 100% Section Loss of Eye Bars Diagonals in the Swing Span

Based on the existing conditions found during the inspection, not considering structural analysis, approximately 75% of the primary truss members in both the swing span and the approach spans are severely corroded and deteriorated, and/or would require significant repairs.

## STRUCTURAL ANALYSIS SUMMARY

A structural analysis was performed to determine if there are members which, even in good condition, would not be suitable for reuse in the trusses due to their structural capacity. The primary and secondary truss members were analyzed for their combined axial and flexural capacity, based on anticipated loading conditions. The analysis treated the swing span as fixed in the closed position and supported at the approach piers and center drum pier. Due to the severally deteriorated condition of the floor system, it was assumed that in any rehabilitation scenario the floor system would need to be completely replaced and thus the stringers and floor beams were not analyzed at this time.

Loading scenarios were based on the proposed future programing of the bridge, which considers the potential for vehicular traffic (HL-93 Truck), pedestrian traffic and lateral wind loading on the superstructure. Current specifications for vehicular, pedestrian and wind loading conditions vary from the original forces the bridge was designed for in the early 1900's. Due to the location of the structure and the possibility for large gatherings on the bridge, such as when there are fireworks in the harbor or when the tall ships come to town, the pedestrian load is treated as an assembly load of 100 pounds per square foot (psf), per the Building Code, rather than 75 psf per AASHTO design requirements.

Based on the loading and conditions described above, in all spans, the interior trusses performed better than the exterior trusses. This is likely due to the fact that the interior barrel was initially designed for railroad loading. The swing span has a greater percentage of members meeting capacity, as compared to the approach spans. This is likely due to the fact that the swing span had to be constructed with heavier members to withstand loads in the open cantilevered position in addition to loads in the closed position.

## COMBINED CONDITION AND ANALYSIS SUMMARY

Combining the results of the condition inspection and structural analysis, the elements of the bridge which may be potentially re-used in a rehabilitated structure have been evaluated. As previously discussed, the floor system is beyond repair and would require replacement in any rehabilitation scenario. Figure 1 below graphically shows the summary of the results of the combined capacity and condition analysis for the primary truss elements. The elements depicted in red indicate members which would need significant repair and/or do not meet current load capacity requirements. Members depicted in green would meet current load capacity requirements but may also require minor repairs. The diagram is shown for a typical truss in the structure; there is some minor variation among the spans and trusses. Overall, based on existing conditions and structural analysis, 75% of the exterior and interior swing span trusses, 90% of the exterior approach span trusses and 75% of the interior approach span trusses primary members would require significant repair and/or do not meet load capacity requirements. For the secondary truss elements approximately 25% of the upper sway bracing on the approach spans and less than 20% of the upper sway bracing on the swing span would require significant repair and/or do not meet load capacity requirements.

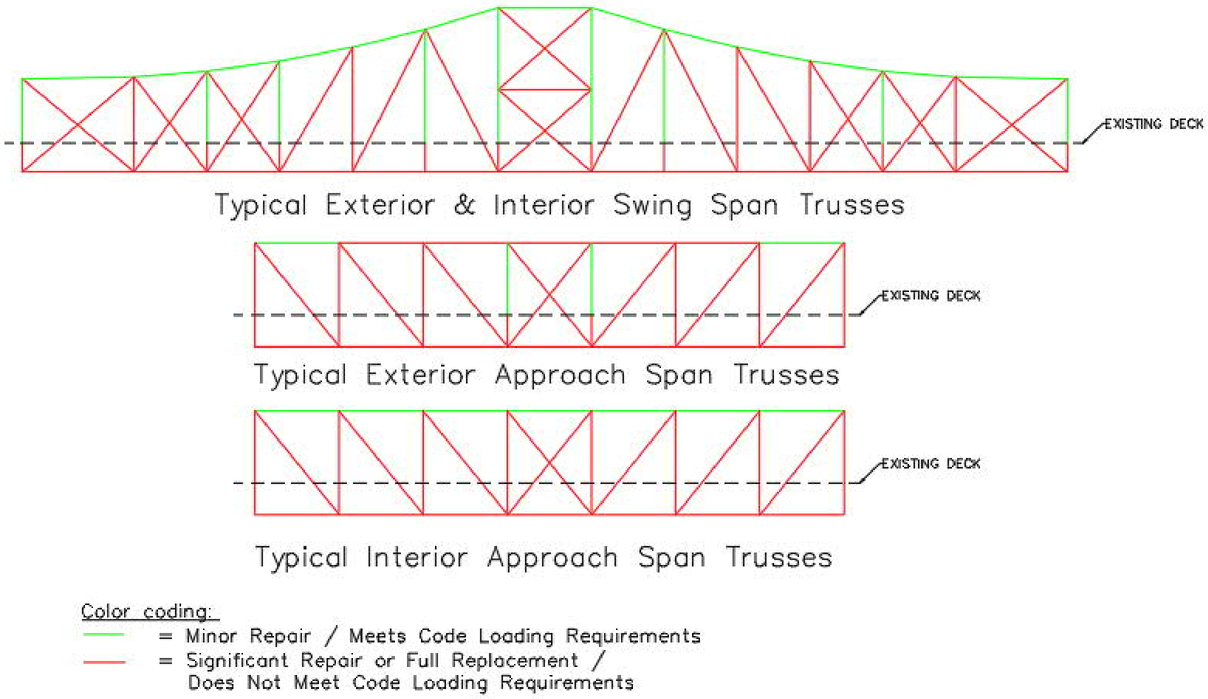


Figure 1: Truss Based on Combined Condition and Capacity

There may be rehabilitation strategies to reduce the amount of rehabilitated or replaced elements and these are discussed in the following section.

## REHABILITATION CONSIDERATIONS AND STRATEGIES TO REDUCE THE PERCENTAGE OF NON-USEABLE MEMBERS

Based on the above discussion, approximately 10% to 25% of the primary truss members have the potential to be reused in the new structure after minor repairs are addressed and the members are cleaned and re-coated. The remaining primary truss members will require significant repairs or full replacement in order to satisfy safety and service requirements of the new structure. To rehabilitate the individual members, the trusses have to be carefully disassembled and reassembled. This work entails moving the existing bridge offsite to a controlled environment.

### Fabrication and Rehabilitation Considerations

The majority of the truss members are comprised of unique, built-up shapes with intricate connection details and numerous blind spots. The majority of the repairs undertaken would be fabricated on a case-by-case basis, with limited typical repair details.  For these types of rehabilitations, standard repair designs require supplemental work to account for restrictions due to the distinct member cross sections and connections.

As is commonly associated with rehabilitation of this type of structure, there are uncertainties as to the full extent of deficiencies which cannot be confirmed until the bridge is disassembled. While visible surface defects were recorded during the inspection, until the members are deconstructed and observed more closely, the full extent of these defects is uncertain. This is particularly true at the pinned connections, where numerous members are stacked together, blocking the full view of all members. It is probable that hidden deficiencies will be uncovered at these locations during deconstruction. Thus, the potential for greater loss than previously observed is high. This may lead to additional repairs, design or analysis being needed and more members which will be deemed unsuitable for reuse or rehabilitation in order to safely complete the restoration. For example, on the recent Longfellow Bridge rehabilitation project, the original intent had been to retain and repair all of the columns on three of the eleven arch spans and all of the columns on the outside fascia for all of the other spans. Once this was attempted, it became obvious that it was not going to be feasible. The necessary repairs were too extensive and obtrusive, negating the historic aspect as well as not providing a 75-year useful life. The decision was made to replace them all as replica columns. As a result, the only original steel remaining is in the arches, which is only possible because they were over designed originally.

It can be challenging to fit the components back together when they are reassembled. For example, the swing span has been in the swung open position since 2014; however, in the 2013 inspection it was noted that the live load shoes for the swing span were missing. This means that since some unknown time before the 2013 inspection, the swing span has been resting solely on the drum pier in a cantilever condition. The original bridge was only designed to be cantilevered for short periods, during which times there was no live load on the bridge. Due to these conditions, during the time between when the live load shoes were removed and the pedestrians were allowed on the bridge, the load path in the truss was altered from its original design, and truss members that were designed to only carry dead load were now subjected to pedestrian live loads. Since the structure was not designed to be cantilevered for long periods of time, the swing span as a whole has experienced significant sagging. Photo 7 below shows the difference in vertical alignment between the approach span and the swing span one of the last times the bridge was closed. In fact, the alignment was so far off that timber walkways had to be constructed along the north bay to provide an even walkway for pedestrians when the bridge was still in use. Thus, work will need to be performed to ensure that once repaired and swung closed, the swing span will line up with the approach spans and that the required bridge geometry is attained.

Image 7: Vertical Alignment Differential between the Swing and Approach Span

More localized examples include elongation of individual members, which may be a hindrance when reassembling the truss, as they will not line up as intended. This is particularly challenging since all of the 248 pinned connections would need to be disassembled and reassembled. These joints are complex in the sense that there are many connecting elements and plates at the joint, as seen in Images 8 & 9. The possibility that all of the pieces will not fit back together properly after repairs are made is highly probable. Given the historical cyclic loading of the bridge, it is also possible that the holes in the members that encase the pin have experienced "egging" and are no longer uniform circles, and thus do not provide the same constraints as when they were originally designed. Examples of deformation in the pin and the surrounding members are observed in Images 10 & 11 below. In addition to repairing deterioration in the member cross sections, distortion of the pin holes would also need to be addressed and corrected in order to restore the structural integrity of the pinned connections. Such repairs could potentially be achieved via cover plates, splices, or full member replacements, all of which have the potential to further complicate the joint, particularly in regard to geometric constraints.

Image 8: Typical Lower Chord Pinned Connection

Image 9: Model of a Disassembled Pin Connection

Image 10: Deterioration of the Pin

Image 11: Egging around the Pin Hole

Materials and Fatigue Considerations

The material properties of the existing steel are an important consideration when evaluating rehabilitation options. There are at least two types of steel on the existing bridge: steel from the original construction between 1905 and 1908, and steel from the reconstruction of the swing span between 1934 and 1936. Given that both types of steel are over 80 years old, there are uncertainties as to whether or not the existing members will provide the proposed 75-year service life. In conjunction with the uncertainties relating to the as-built materials, there are also unknowns about the fatigue life of the as-inspected materials. Fatigue is the weakening of a material due to repeated cyclic loading and unloading, such as vehicular traffic or bridge openings. Damage due to fatigue is cumulative and permanent; it cannot be reversed with reduced loading. Fatigue failures are generally localized, and they typically occur suddenly at stress levels lower than the actual yield stress of the material.

Steel has an approximate fatigue limit, which refers to the number of stress cycles it can withstand before failure. It is difficult to estimate the amount of remaining fatigue life for a structure of this age, due to a lack of accurate traffic information since the bridge was constructed, and due to historical bridge opening logs being unavailable. The uncertainties associated with the fatigue evaluation are particularly concerning for the members which see tensile stresses due to live load, such as the diagonals, as fatigue is most often observed in tension members. Given that the remaining fatigue life of the steel cannot be accurately determined, it cannot be confirmed with certainty that if the existing steel in these components was reused or rehabilitated that it would last for the remaining service life of the structure. For these reasons, tension only members should not be rehabilitated or repaired and instead should be replaced.

Preservation Strategies

A strategy to increase the percentage of usable members may include splicing new sections onto the existing steel components. Details of this nature would potentially allow for more of the existing steel to be reused, by splicing new and old steel sections together. Splices on the lower chord are not practical given existing condition as well as fatigue considerations, and splices on the diagonals are not acceptable as described above; thus, this strategy could potentially be applied to verticals and selected secondary members.

Welded Splice

A welded splice may be desirable from a visual point of view, since, if done using full penetration welds, with the welds ground smooth, the splice would be nearly undetectable. Welding to tension members is not recommended on bridges due to the potential for fatigue cracking from added stress concentrations and failure at welded locations. Welding may be considered for non-tension elements, such as the majority of the truss verticals; however, there are challenges associated with welding to the existing steel.

The American Welding Society (AWS) first issued its Standard Specifications for Welded Highway and Railway Bridges in 1941. Bridge steels of the early 1900's era had little in their specifications in regards to chemical composition to control weld cracking other than limits on impurities (Phosphorous & Sulphur) related to the steel manufacturing processes typically employed. Therefore, there is uncertainty with the weldability of the existing steel and, as a result, laboratory testing, development of specific weld procedures, qualification of those procedures and non-destructive testing (NDT) would be necessary to ensure weld integrity.

When evaluating the feasibility of welded splices the differing physical shape of the members must be considered. Members of the bridge are built up from rolled steel shapes available at the time of construction. For many of these shapes, there is no modern equivalent shape, so creating an exact match for a welded splice is problematic. The built up shapes (i.e., multiple plates and rolled shapes combined) require prep work for creating acceptable weld joint details and weld sequencing to avoid member distortion (see following discussion regarding geometry control). This work requires specialized welding techniques akin to ornamental ironwork with unique set ups and control of operations. If the anticipated welding is not done properly and carefully, it will likely lead to weld defects or cracking and the associated re-welding to address these imperfections may create delays and added cost.

Bolted Splice

A bolted splice is a feasible alternative to a welded splice. Due to the intricate lattice work on the verticals, not only would a splice of the verticals need to be sized for capacity, but it would also need to be designed around the existing lattice pattern. Due to the combined axial and flexural effects on the verticals and the geometric limits based on lattice location, larger splice plates are required. The approximate location of the splice on the vertical truss members would be just above the deck level. A preliminary splice detail for a sample vertical is presented in Figure 2 below.

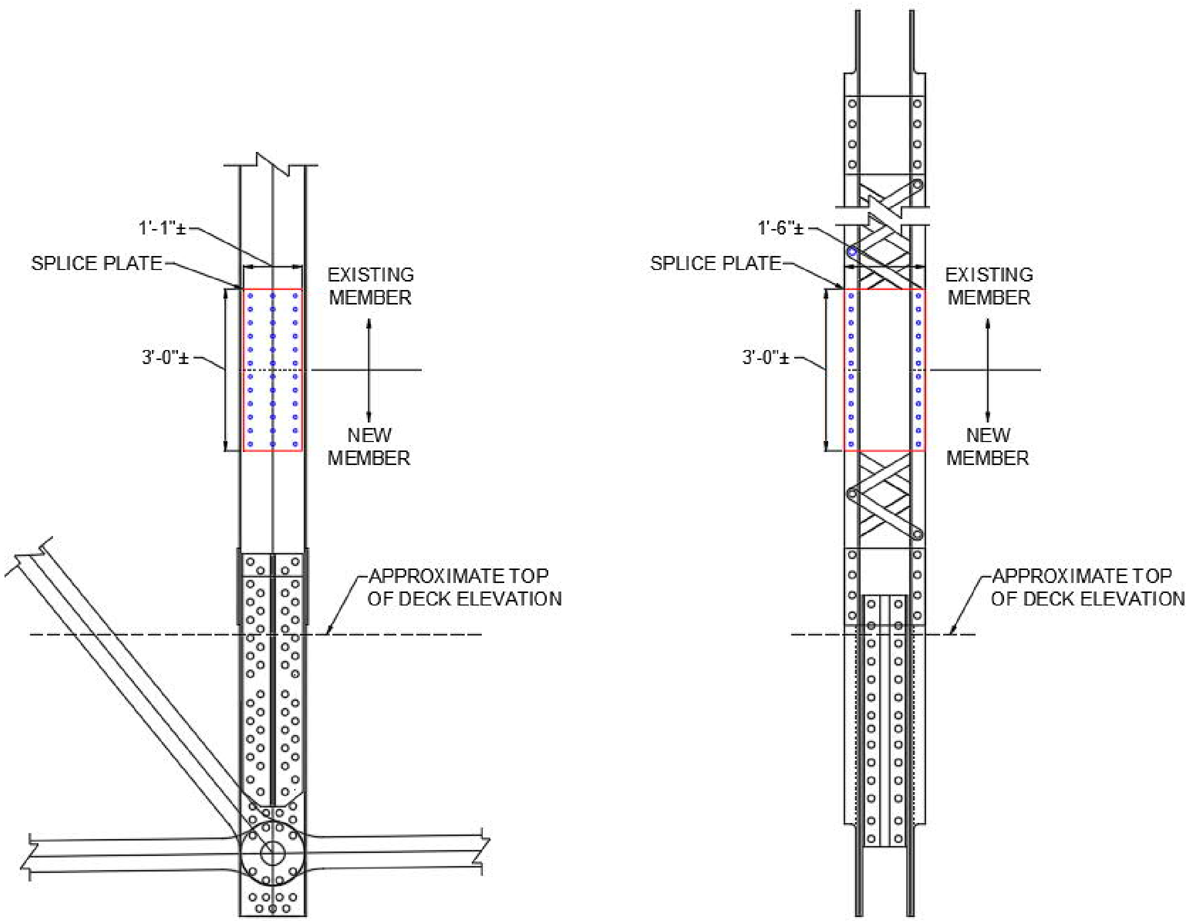


Figure 2: Approximate Splice Detail

Regardless if the splice is bolted or welded, there are challenges related to geometry control. It is critical on a truss bridge structure that the geometry is carefully controlled so the bridge profile is correct once dead loads are applied. This geometry is controlled by precisely setting the layout of the pin connected joints, which accounts for the elongation or shortening of the members under dead load. With a pin-connected truss, the holes for the pins are reamed in a shop environment in order to precisely control the pin hole locations and geometry. As a result, after splicing onto the existing member, additional operations to drill or straighten the members to these tolerances would be required.

It should also be noted that if the splice option is pursued, additional repairs would still be necessary. Since the truss members are primarily axial force members, they do not function like a typical beam where repairs can be focused in high-stress regions. Instead, the axial force travels through the entire member, and thus the cross section at every location along its length would need to possess adequate capacity. This means that a spliced member may still require additional strengthening outside of the splice region; this is particularly true at the pin locations. The inspection report indicated the majority of member deterioration was concentrated around the pins, and thus the majority of the members at these locations would need to be rehabilitated in order to restore the integrity of the pinned connection.

Coating System

When evaluating rehabilitation of the truss elements, the coating system required to provide a structure with a 75-year design life also needs to be considered. New construction of bridges over waterways in Massachusetts uses hot dip galvanizing in order to protect the structural steel and to provide the desired design life. For the rehabilitation of the truss elements, galvanizing is not an option, especially if using riveted connections in the rehabilitation. Thus, the steel would need to be protected via a coating system. A coating system for this structure, when exposed to the elements, would require additional maintenance and re-coating efforts in frequent intervals throughout its life.

Rehabilitation of the Trusses as Non-Structural Elements

An alternative to the rehabilitation of the trusses to be re-used as originally intended, with the trusses acting as the primary structural elements, is restoration of the trusses for non-structural use. In this scenario, the truss would act as an ornamental or architectural feature designed to withstand its own self-weight and lateral wind loads but it would not be subjected to live loads nor contribute to the structural capacity of the bridge span. As part of this option, a new girder bridge, designed for live load and, potentially, the additional weight of the architectural truss, would be designed and constructed.

This option eliminates many of the concerns discussed above regarding fatigue life and structural capacity of the truss members. The reduced loading of the non-structural truss will improve performance of the members; however, approximately 75% of the bridge will still require repairs to some degree based on condition alone to meet service and safety requirements. The challenges associated with member deformation, hidden deterioration, service life and coatings previously discussed would still apply.

A new girder bridge, either between the existing trusses or supporting the rehabilitated trusses, would have a deeper section below the deck. This increased structure depth would increase the overall profile, creating more impacts on the approaches in order to meet slope requirements.

Resiliency

Resiliency is one of the overall conceptual foundations of the Northern Avenue Bridge Project. One of the goals of this project is to be among the first structures in the area to follow the Climate Ready Boston guidelines for a sustainable future. Currently, the bridge underside is submerged in water during storm surges. This direct exposure to salt water only worsens the already declining condition of the floor system and the lower portions of the truss. Given that sea levels are expected to rise over the desired 75 year life of the structure raising the bridge to improve resiliency is essential. Regardless of whether the bridge is rehabilitated or replaced, the final structure will need to be raised in order to achieve resiliency and to meet the Climate Ready Boston guidelines.

COST CONSIDERATIONS

Order of magnitude costs have been developed to help evaluate the feasibility of rehabilitation. These were developed in a "bottoms up" fashion based on means and methods a contractor would need to use. This includes first removing the existing structure from the site, disassembling the truss elements, evaluating the pieces, replacing and/or repairing the elements as required, reassembling the trusses, transporting the trusses back to the site and re-erecting the trusses. The reconstruction work would also entail work to rehabilitate the existing foundations as well as work on the approaches to the bridge to transition the new profile to the existing grade.

The range of cost for the superstructure work alone (not including the substructure and approaches) is on the order of $100,000,000 to $105,000,000. These costs are escalated to future dollars assuming a start date of construction of spring of 2021. This considers the time, skills and precision associated with strategically disassembling and reassembling the truss. Extreme care needs to be taken to preserve as many members as possible, and the complexity of details to match existing elements would add to the overall cost. Given the high probability of finding further deterioration once the bridge is disassembled, additional costs to account for unforeseen repairs are probable and contractors will account for these risks with higher bid costs. This factor has been considered in the cost evaluation.

The non-structural rehabilitation option is comprised of two major stages, first constructing a new girder bridge and also rehabilitating the truss elements. Due to the added cost of a new structure to support bridge loadings, plus the aforementioned cost of truss restoration, the costs for this option are significantly higher – on the order of $110,000,000 to $115,000,000, not including foundation work and approach work.

PROS AND CONS OF REHABILITATION

The previous discussion has described the inspection and analysis conducted to date as well as a discussion of the technical challenges associated with rehabilitating a truss structure of this age and condition. To help the City of Boston evaluate whether rehabilitation is feasible, Table 1 below summarizes the pros and cons associated with rehabilitation of the truss structure.

Table 1: Pros and Cons of Truss Rehabilitation

| Pros | Cons |
| --- | --- |
| Bridge's character-defining features remain in place, including its triple barrel design, truss approach profile, and truss side profile | Cannot be galvanized which is the preferred coating method for the site to provide a 75 year design life |
| Maintaining the original designs, materials and workmanship allows users to experience the historic associations and feelings of the original bridge. | Difficult and lengthy process of removal and disassembly to evaluate components |
| | Associated risks in terms of cost and schedule regarding unknown and hidden conditions |
| | Large percentage of primary truss elements require significant repairs or replacement due to condition and/or capacity |
| | Given the mixture of new and existing steel the desired design life of 75 years is questionable and the bridge would require a vigorous maintenance schedule and additional costs. |
| | Raised profile will detract from the historical significance of maintaining the original truss shape |
| | Splices on the lower chord are not practical given existing condition as well as fatigue considerations, and splices on the diagonals are not acceptable as described above; thus, splicing would primarily be possible for the verticals and other secondary members |

## CONCLUSIONS & RECOMMENDATIONS

Based on our evaluation and analysis presented in this report, it is not recommended that the City of Boston pursue rehabilitation of the original truss structure. This recommendation is based on the condition of the bridge elements and structural analysis, as well as evaluation of the risks associated with rehabilitating the steel in terms of schedule, cost and design life considerations.

As options are further evaluated to meet the needs of the project, the costs and risks associated with rehabilitation will be compared to replacement options. Replacement options may range from reinterpretations of the crossing with a similar scale and profile of the existing truss to completely new and "bold" options. In the event that rehabilitation is not pursued and a new bridge is constructed there still may be options to salvage portions of the bridge for historic purposes such as displays or other acceptable preservations means. Regardless of the option selected, there is also an opportunity to conduct a 3-D laser survey of the bridge with the goal of providing a virtual reality tour of the original bridge, either on site or at a nearby museum. All replacement options will be evaluated in terms of how they may honor the history of the original bridge as well as the history surrounding the Fort Point Channel and the City of Boston.