JACOBS

Memorandum

Appendix #2

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Subject	**Foundry Branch Trestle Feasibility Study**
Project Name	Palisades Trail Feasibility Study
Date	December 2019

Executive Summary

This feasibility study report presents options and an order of magnitude cost estimate for the adaptive reuse of the Foundry Branch Trolley Trestle Bridge as part of a pedestrian/bicycle trail. The options were developed considering the results of the visual inspection and a detailed structural analysis of the bridge in its as-built condition. The visual inspection of the bridge identified the following:

- The approach trestles are in very poor condition. There is significant deterioration of the primary structural support members with complete section loss at some locations. Secondary members, that provide for alternate load paths, are also in an advanced state of deterioration with many having corroded through and are hanging loose from the bridge. Several the bracing members of the approach trestles would need to be increased in size to support the load from a pedestrian/bike trail.
- All the bridge footings are in poor condition and need to be replaced. Although the abutments are in reasonable condition, it is impossible to calculate their load carrying capacity.
- The suspended truss span is generally in better condition with minor rusting of the chord and web members. There is some corrosion of the gusset connection plates and of horizontal members where water has not been able to drain. The truss members have been shown to be able to support the load from a pedestrian/bike trail. The truss will need to be checked for conformance with code requirements for fracture critical members at the next design stage.

This study evaluated four options for adaptive re-use of the bridge as a pedestrian / bike crossing. Consistent across each option is that the truss span is currently in reasonable condition and able to be reused by first lifting from the supporting trestles and then rehabilitated either on site or transported to an offsite facility. New shallow footings will be needed for all options. The four options are:

- Option 1 - Rehabilitate the approach trestles steelwork
- Option 2 - Replace the approach trestles with new structures to match existing
- Option 3 - Replace the approach trestles with new longer spans
- Option 4 - Retain the approach trestles as facades supported by new structure

For each option, this report includes discussion of the risks resulting from the deteriorated condition of the steelwork; construction duration; historic review during the design and construction; life cycle risks; aesthetics; and a qualitative evaluation of comparative cost.

As new footings are needed for all options, all existing approach trestle steelwork will need to be dismantled. Reuse of that deteriorated steelwork for any rehabilitation option would present both schedule and cost risk during the construction phase of the project. Therefore, should it be decided to proceed with the design of the reuse of the bridge, it is recommended that Option 2 is selected with new approach trestles of similar geometry to the existing constructed on new footings. Option 2 would also result in a structure that is aesthetically closest to the existing historic structure. Options 3, although less costly, is considered undesirable as it would be aesthetically very different to the existing.

The order of magnitude cost estimate for this option, and the cost at the lower and upper accuracy range is presented below.

Low Range (-20%)	Estimated Cost	High Range (+50%)
$2,201,000	**$2,751,000**	$4,127,000

CONTENTS

1. Overview

The Foundry Branch Trolley Trestle Bridge is a historic bridge located in Foundry Branch Valley Park close to Georgetown in the District of Columbia (Figure 1 and Figure 2). It was originally constructed as part of the West Washington & Glen Echo Electric Railroad/Glen Echo Trolley Line. This trolley system served Washington, D.C. from Georgetown to Glen Echo, Maryland from its construction in 1897 to 1962 when it closed. The bridge has since remained out of use.

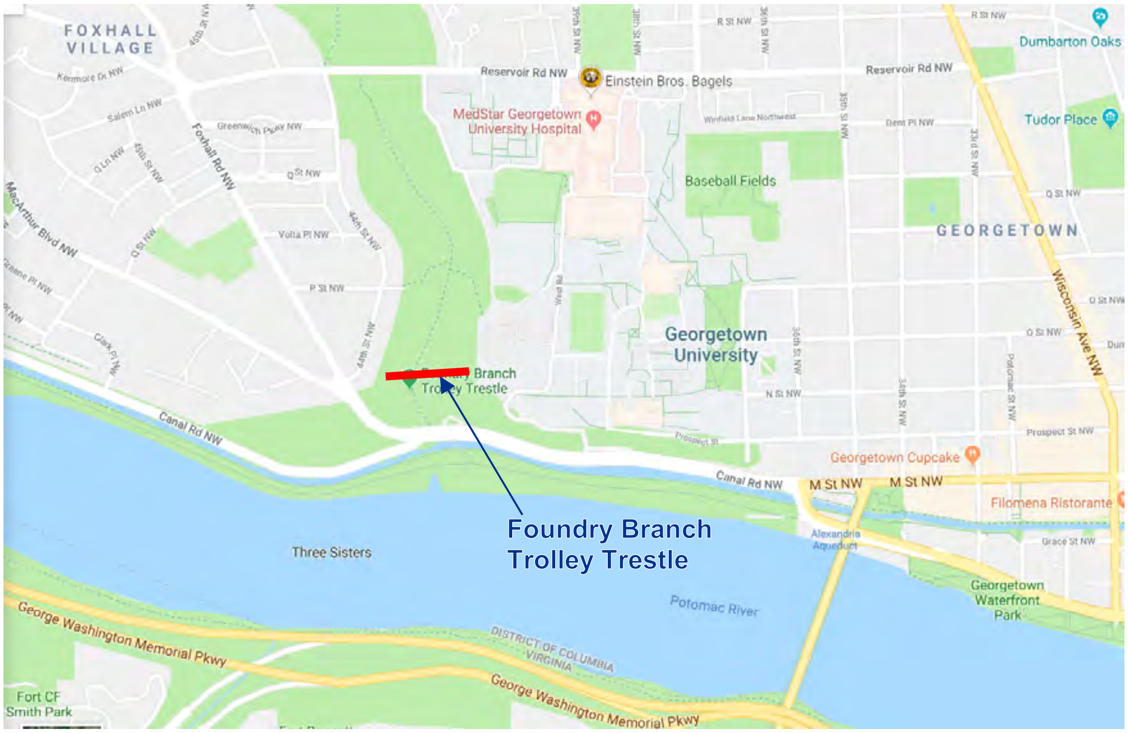


Figure 1: Bridge Location

Figure 2: Aerial Photograph of the Bridge

A pedestrian path was located under the main truss span until the entire structure was fenced off in 2016 when deemed hazardous because of potential falling debris.

The dimensions of the bridge are 252 feet long by 19 feet wide. The primary framing consists of a pair of underslung steel trusses spanning 100 feet (Figure 3) and a series of steel braced frame approach trestles and concrete abutments on the far east and west sides. The approach trestles are supported on rectangular concrete footings. The top of the bridge has zero grade from end to end. The approach trestles and truss span consist of a combination of I-shapes, channels, angles, WT's, and rods, with riveted connections.

The purpose of this feasibility study is to develop options and an order of magnitude cost estimate for the adaptive reuse of the bridge as part of a pedestrian/bicycle trail.

Figure 3: Bridge Elevation Photograph

2. Existing Conditions

Inspection of the bridge was performed through a combination of on-site inspection and use of a 3D model compiled through a laser scan. The survey data was collected over one week in April 2019 before full leaf out of the surrounding vegetation. Jacobs staff also performed site visits during the 3D scanning of the bridge to visually review the condition of existing structural components. A follow up site visit was made on August 22, 2019 after review of the 3D scanning results to confirm observations.

The Foundry Branch Trestle Inspection Report (Attachment A) provides complete documentation of the condition of the bridge at the time of the inspection. It also includes a complete description of the bridge geometry and observed section sizes. Below is a summary of the findings of the inspection:

- The approach trestles are in very poor condition. There is significant deterioration of the primary structural support members with complete section loss at some locations. Secondary members, that provide for alternate load paths, are also in an advanced state of deterioration with many having corroded through and are hanging loose from the bridge.
- The suspended truss span is generally in better condition with minor rusting of the chord and web members. A large part of the original paint system is still intact. There is some corrosion of the gusset connection plates and horizontal members where water has not been able to drain.

Certain parts of the structure were not able to be inspected due to the significant overgrowth of vegetation that covers parts of the bridge. Physical access onto the remaining trolley rail ties that cover the top of the bridge was also not feasible due to safety concerns.

3. Structural Model

A structural model of the existing bridge was compiled using Midas Civil 2019 Finite Element Analysis Software (https://en.midasuser.com/product/civil_overview.asp). A structural model is used to quantify the deformations and forces in the structure under applied loads (Figure 4).

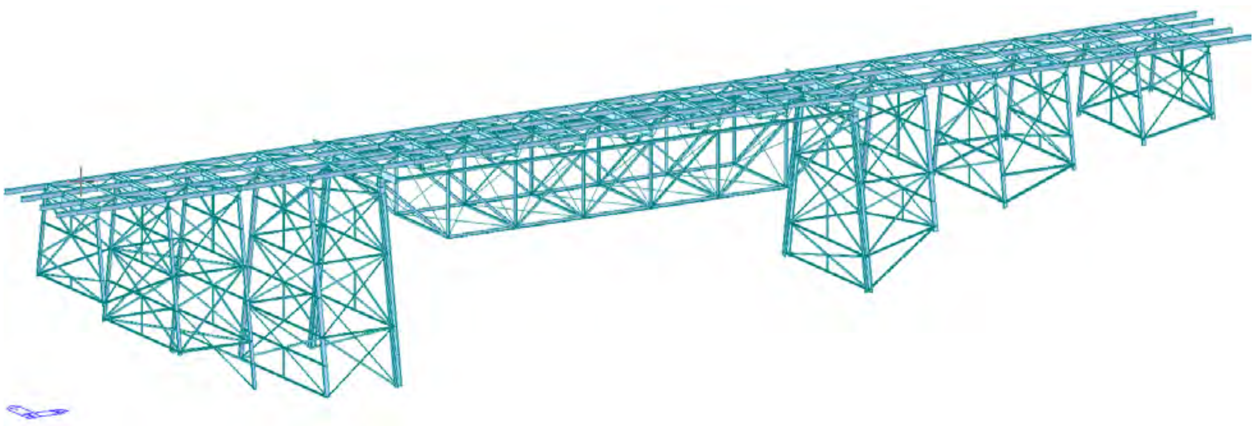


Figure 4: Global 3D Model

The starting assumption for the structural model is that all members are restored to their as-constructed condition (i.e. no corrosion). The purpose of this is to identify the parts of the structure that, even without considering corrosion, would need to be entirely replaced to support a new deck and pedestrian / bike loading. The results of this analysis will be combined with the findings of the inspection report which identifies the members that would need to be replaced or strengthened due to their deteriorated condition.

3.1 Geometry

The bridge geometry and section sizes for the structural model were taken from the Bridge Inspection Report. Attachment B provides additional printouts from the structural model, including node/member numbering, section properties and material properties.

3.2 Loading

Loading is applied to the structural model as described by the Foundry Branch Trestle - Design Assumptions (Attachment C). The existing rail ties and other material that remains on the top of the bridge are assumed to be replaced by new structure that will support a concrete deck for a 12-ft wide pedestrian/bike trail, as described in the Design Assumptions memorandum.

3.3 Results

3.3.1 Deflections

The maximum deflection under unfactored pedestrian live loading is 0.34 inches (Figure 5). The maximum allowable deflection is span/360 (AASHTO Guide Spec for the Design of Pedestrian Bridges) or 3.33 inches (100-ft trestle span). Truss deflections are therefore well within that allowed by AASHTO LRFD.

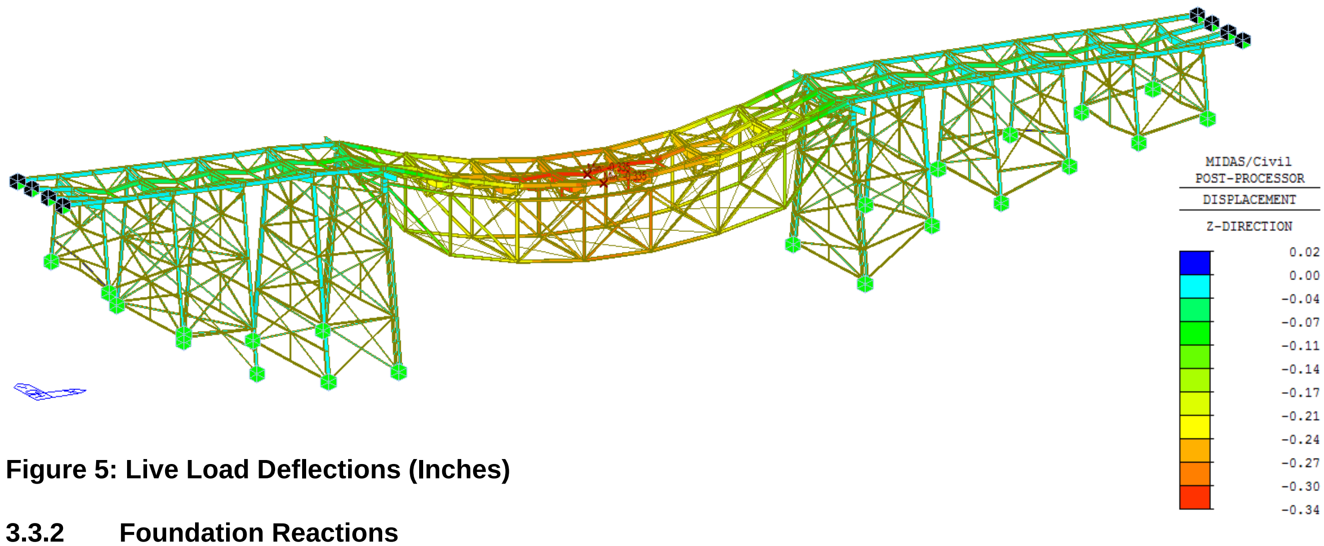


Figure 5: Live Load Deflections (Inches)

3.3.2 Foundation Reactions

Foundation reactions at the top of the concrete footings (not including the weight of footing) are given for the north half of the bridge at each gridline (Table 1). Reactions at the south side will be similar. All results are in kips and are unfactored.

	Unfactored Reactions at each gridline (kips)										
Loadcase	Gr-01	Gr-02	Gr-03	Gr-04	Gr-05	Gr-12	Gr-13	Gr-14	Gr-15	Gr-16	Gr-17
Steel Self Weight	2.0	4.2	4.9	5.7	20.4	20.0	5.2	4.6	3.1	4.0	2.0
New Deck + Railings	7.4	10.1	10.5	10.6	36.4	35.4	11.4	10.7	8.9	11.2	7.1
Pedestrian Load	6.2	8.4	8.8	8.8	30.3	29.5	9.5	8.9	7.4	9.4	5.9
H10 Truck	9.7	7.2	7.1	6.7	10.0	10.0	8.0	8.7	9.3	9.7	7.5

Table 1: Foundation Reactions (Unfactored)

3.3.3 Member Section Checks

Midas Civil includes the ability to perform checks to various design codes. The bridge would originally have been designed using an Allowable Stress Method (ASD) or design tables, such as those included in the Carnegie Steel Pocket Companion, 1903. Through discussion with DDOT it was agreed that the current AASHTO LRFD (American Association of State Highway and Transportation Officials - Load and Resistance Factor Design) should be used. The basis of this decision is to allow a proper understanding of the work required to bring the structure up to a condition to meet current standards.

Midas Civil provides the code check results as member utilizations where a value greater than 1.0 indicates the member is over stressed from a combination of axial loads, flexural loads and shear. Axial loads typically drive the member capacity for a truss bridge type, which is also the case for this bridge

Figures 6, 7, 8 and 9 provide the graphical results of the code checks to AASHTO LRFD. The results are summarized below.

- Figure 6 – Bracing members in the plane of the supporting columns at Gridlines 1 to 4 have utilizations greater than 1.0. The maximum utilization is 2.73.

- Figure 7 – Bracing members in the plane of the supporting columns at Gridlines 13 to 17 have utilizations greater than 1.0. The maximum utilization is 4.37.
- Figure 8 – The maximum member utilization in the truss is 0.68 in its top chord.
- Figure 9 - The maximum member utilization in the deck is 0.99 at a diagonal bracing member. The 10-inch deep longitudinal deck members typically have utilizations less than 0.7.

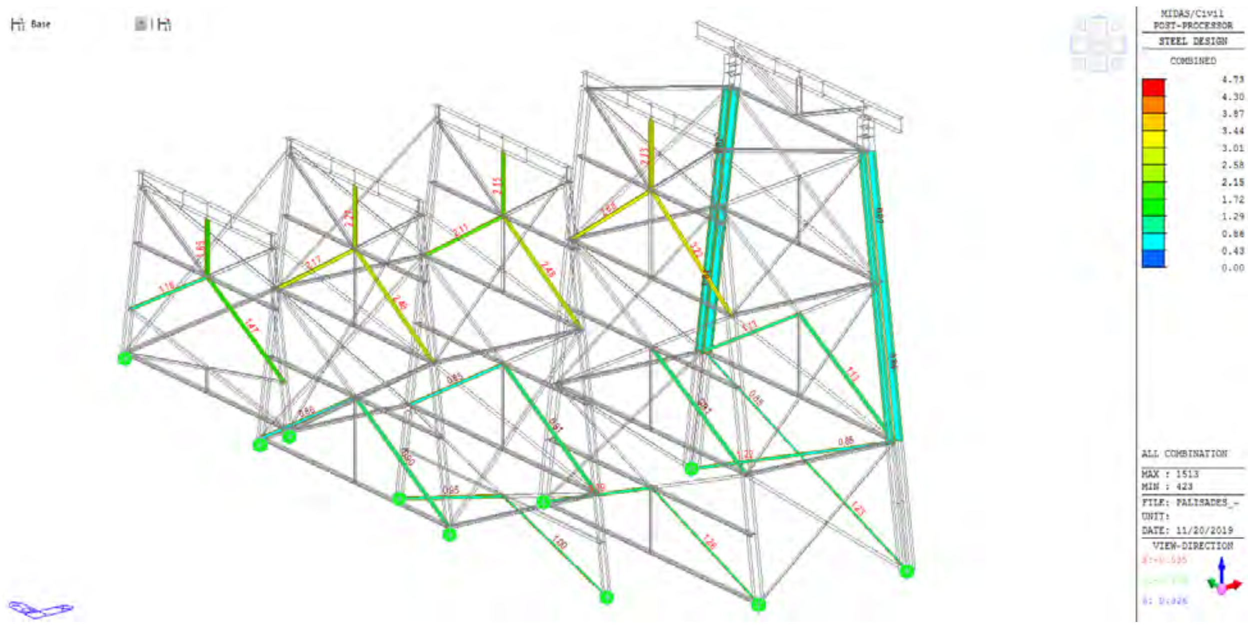


Figure 6: West Approach Results (Utilizations > 0.8 Shown)

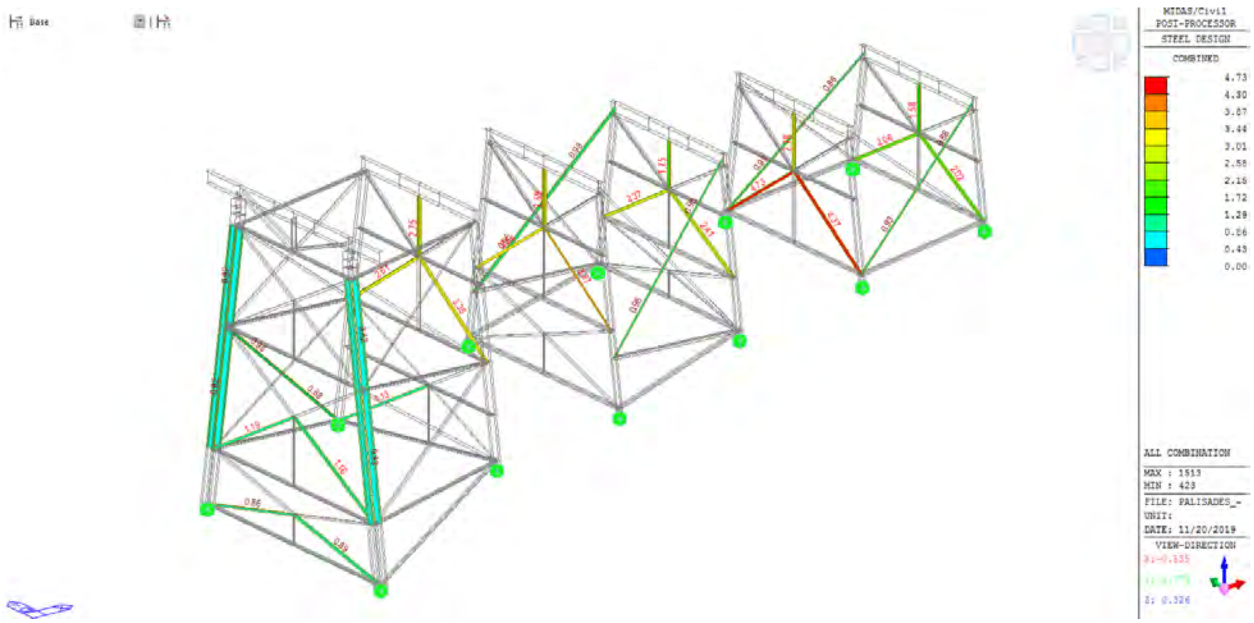


Figure 7: East Approach Results (Utilizations > 0.8 Shown)

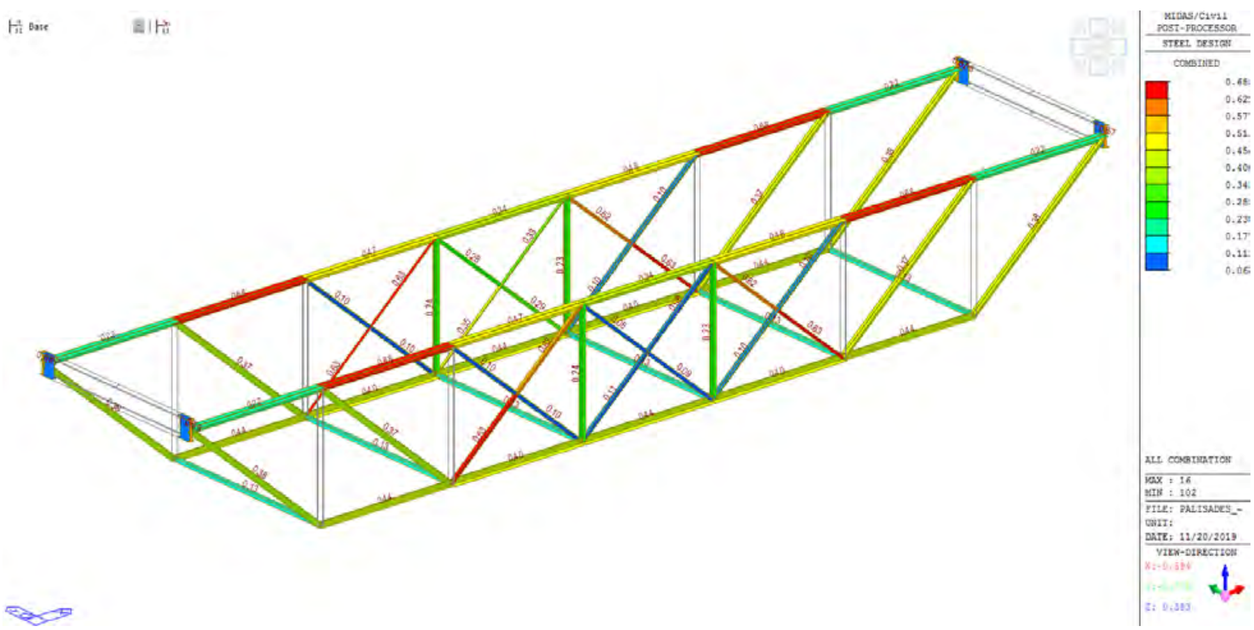


Figure 8: Truss Results (All Utilizations Shown)

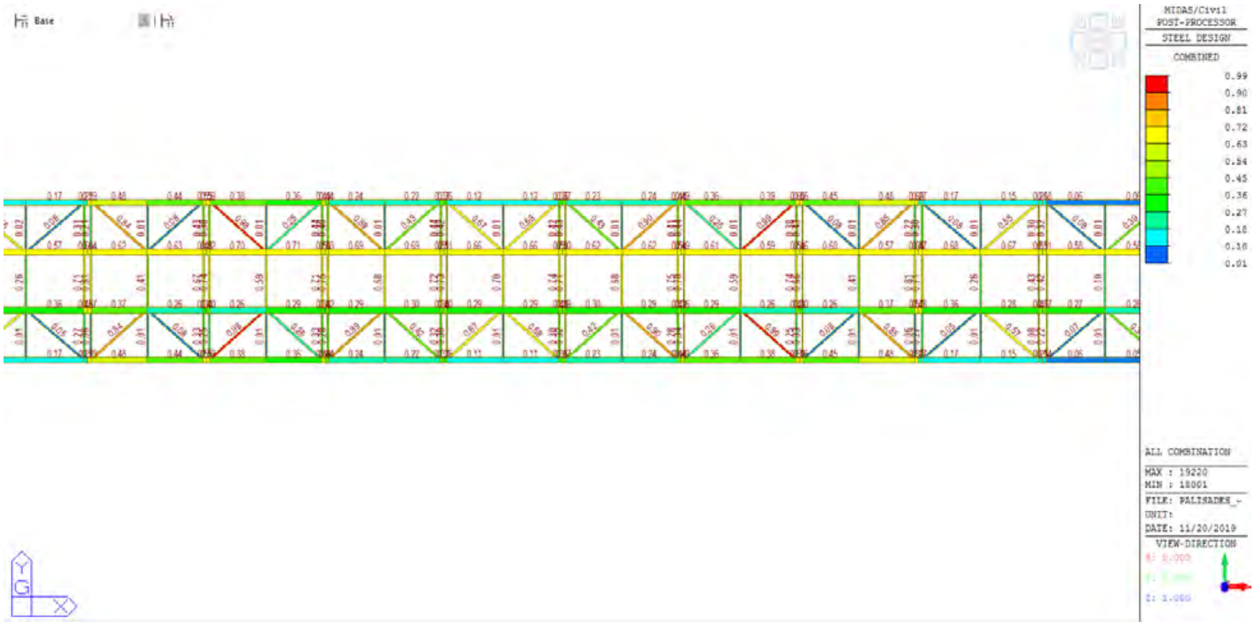


Figure 9: Deck Members above Truss (All Utilizations Shown)

3.4 Summary

The following summarizes the findings of the structural analysis of the bridge and the bridge inspection.

- The 100-foot truss span can support the loads resulting from a 12-ft wide pedestrian/bike trail.
- The truss bottom chord and some diagonals are tension members that are classed by FHWA as Fracture Critical members. A fracture critical member is defined as a component in tension whose failure is expected to result in the collapse of the bridge or the inability of the bridge to perform its function. Since the truss members are built up from double angles or double T-sections, they allow for internal member redundancy where the built-up member detailing provides mechanical separation of elements to limit fracture propagation across the entire member cross section. The truss has not been checked as part of this study to ensure that the internal member redundancy is adequate for the applied loads. The detailing of the members will also need careful review to ensure loads are adequately redistributed. This should be completed at the next design stage.
- Many diagonal bracing members of the approach trestles need to be strengthened to carry the applied loads from a 12-ft wide pedestrian/bike trail. The approach trestles are also in very in poor condition:
    - Multiple bracing members need to be replaced due to their deteriorated condition.
    - Multiple connection gusset plates need to be replaced due to their deteriorated condition.
    - Of the ten vertical supporting members of the western approaches, four are significantly corroded and in need of replacement. Of the remaining six supporting members, two are buried and so are assumed to also be in poor condition and the remaining four show corrosion at their connection to the footing and are recommended to be replaced.
    - All twelve vertical supporting members of the eastern approaches show corrosion at their connection to the footing with holes visible through the steelwork in multiple locations and are recommended to be replaced.
- Deck members that are visible (longitudinal stringers and cross-members) show significant deterioration, it likely there is more where moisture is trapped beneath the rail ties. As a result, it is assumed that all the deck members would need to be replaced.

4. Review of Structure in its Current Condition

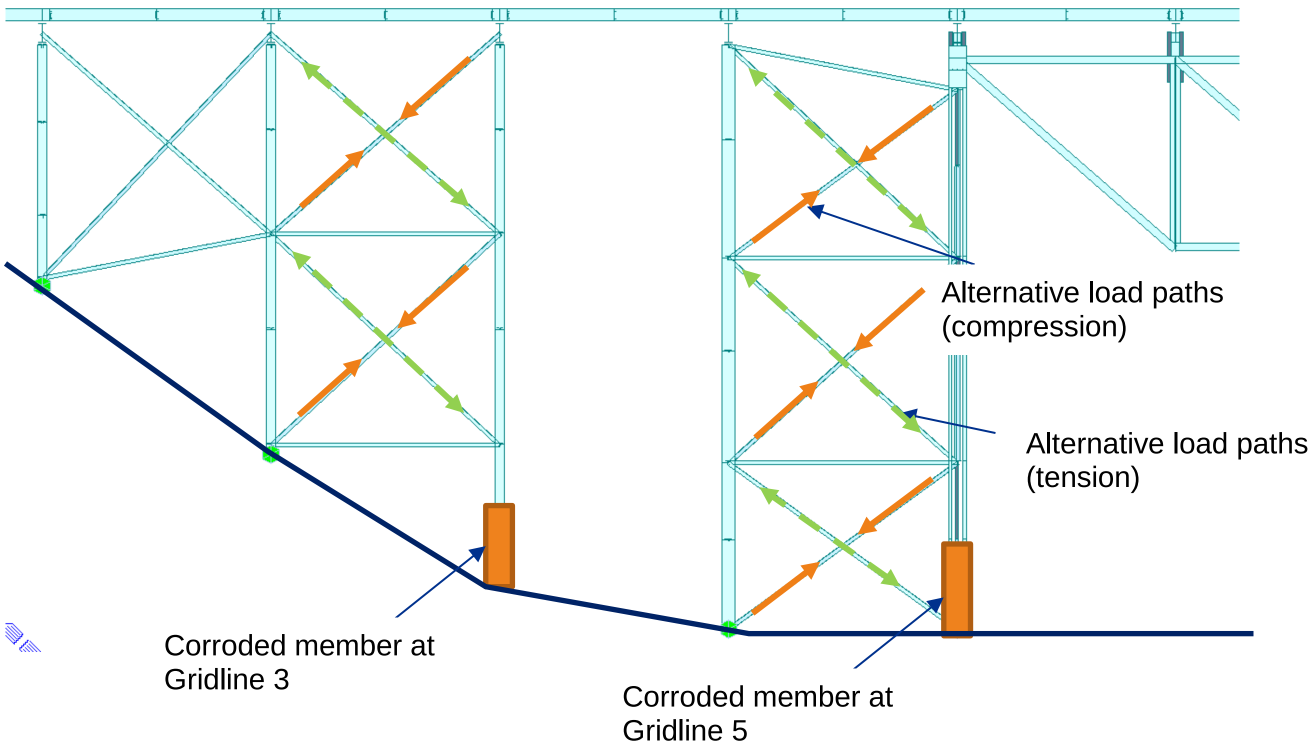
The existing structure is in poor condition. Of most concern is the base of Gridline 3 north and south (Figure 10) and Gridline 5 north and south (Figure 11). Gridline 5 provides the primary load path for support of the main truss span.

Figure 10: Column Base Gridline 3 North and South

Figure 11: Column Base Gridline 5 North and South

To assess how the above steelwork deterioration impacts on the stability of the existing structure, the structural model was modified to include reduced plate thicknesses to simulate corrosion. Two new sections were created with the full section geometry maintained, but the plate thickness reduced in increments. The new sections were assigned to the base of Gridline 3 north and south and Gridline 5 north and south. The applied loading included only steel self-weight and a 250 lb/ft allowance for rail ties and other material that remains on the top of the bridge. The analysis resulted in the following conclusions:

- Once the section thickness at Gridline 5 reached approximately 0.1 inch (from the starting 0.38 inch), the member no longer has the capacity to support the applied load.
- The model confirmed that as the section thickness is reduced, load is re-distributed through the diagonal bracing to alternative load paths (Figure 12).



Note: The secondary load path through the bracing will also result in an axial load in the deck longitudinal girders. The truss should therefore be shored up before any part of the deck is removed.

Figure 12: Alternate Load Paths

Given the amount of corrosion at Gridline 5 has already advanced to well beyond 0.1 inch of material remaining, it is reasonable to assume that the structure is being supported by loads being re-distributed through the diagonal bracing members as shown in Figure 12.

Since the diagonal bracing members also have corrosion, both within their length, and at their connection to the main verticals there is concern that any further reduction in alternate load paths could lead to collapse of the bridge. This analysis supports the concern over the stability of the main truss span and that it be either shored up or lifted from the bridge.

5. Foundation Options

The discussion in this section is preliminary in nature based on the limited information available through visual inspection of the existing bridge. At this stage, site-specific subsurface information is not available to make a complete assessment and definitive decision on the bridge foundation alternative. Consequently, the discussions here are qualitative in nature and more detailed evaluation will be needed during the future design stages.

The subsurface conditions are extracted from the historical borings performed in 1981 for the design and construction of a 9' diameter crosstown watermain (see Attachment C). The closest available boring (B-2A) shows a top layer consisting of stiff to very stiff silt and clay fill with a thickness of about 15 feet and with SPT blow counts between 9-18 blows-per-foot (bpf). This fill layer is underlain by a medium dense to very dense sandy layer with blow counts between 11 and >50 bpf. The top of the rock in this boring is approximately 26 ft below the existing ground surface. However, it appears that the top of bedrock elevation at the site varies substantially as based on field observations, the bedrock appears to be outcropping at the ground surface in the area of the existing west abutment (Figure 13).

Figure 13: Rock Outcrop at Gridline 2/3

5.1 Abutments

The far east and west side of the existing bridge structure are supported by concrete abutments. While the abutments are generally in relatively good condition for their age, several cracks and small concrete spalls are present. The observed cracks start at top of the abutment under the steel beams and extend vertically up to full height with other cracks extending horizontally. Cracks are up to 1/16" wide with minor moisture stain and efflorescence. The east abutment has its footing exposed for 90% of its length with minor spalls and honeycombing. The west abutment has the footing exposed and is undermined for approximately 8 ft long. In addition, there are no as-builts or historical design drawings for these abutments. Calculation of their safe loading capacity would therefore need to include assumptions about their design and condition leading to potentially unreliable results. Accordingly, it is recommended at this stage to not reuse the existing abutments to carry the loads of the rehabilitated or replaced bridge. The existing abutments can be either kept in place if their long-term stability can be assured with underpinning or they can be demolished, and new abutments can be constructed right behind them. The bridge load will then be carried by the new abutments and sections of the new replacement bridge could potentially be spanned over the existing abutments.

5.2 Approach Trestle Footings

The approach trestles are supported on rectangular concrete footings located on the slopes in front of the abutments. All observed footings are experiencing deterioration in the form of concrete spalls and many have full width horizontal cracks. Many footings on the west side are completely buried and not visible. Concrete cores taken at Gridline 13 for the 2014 Washington Metropolitan Area Transit Authority (WMATA) inspection were not able to be tested due to being too fractured. The south trestle on Gridline 15 has a full width - 2 feet high spall that exposes inside concrete with no sign of reinforcement rebars. Also, inspection of the cores taken as part of the previous WMATA study show no signs of reinforcement. It is possible that the footings are unreinforced.

Due to the lack of regular maintenance, there is significant erosion of the slopes at either side of the bridge which has led to the undercutting of some of the footings (Figure 14). On the west side, the footings at the bottom of the slope are entirely buried with debris and not visible. The slopes will need to be reestablished during re-construction to ensure their stability.

Figure 14: Undercut Footing

It is recommended that the existing trestle footings are removed and replaced with new footings constructed within properly stabilized slopes with the top of the footing extending at least 2-ft above ground so that they do not become buried.

Shallow foundations are generally the most economic option when feasible. The geometry and size of the shallow foundation elements needed to properly transfer the load to the ground will depend on the loading and the subsurface conditions. Given that the existing bridge is supported on shallow foundations and that replacement foundations and abutments are being planned at or very close to the existing locations, it is reasonable to conclude that the rehabilitated bridge may also be supported on shallow foundations.

There is an existing 9 feet diameter crosstown watermain owned by DC Water near the bridge area. The watermain is at least 25 feet away from the closest existing bridge footing and it is at a depth of at least 125 feet below ground surface. The watermain is in bedrock and therefore it is unlikely that the new bridge foundations will adversely impact it.

5.3 Geotechnical Investigation

A more detail geotechnical investigation will be necessary before advancing the design of the bridge's foundations any further. Geotechnical investigation of the subsurface conditions, including laboratory and field testing, are required to be performed to describe the features of the soil and rocks as well as measuring the groundwater level. The investigation should be adequate enough to determine the subsurface profile, shear strength parameters, compressibility parameters and unit weights of each of the soil layers. In addition, rock coring and laboratory testing will be needed to determine the characteristics of the rock mass. Geologic mapping including orientation and characteristics of rock discontinuities may be necessary since bedrock is shallow to properly assess the stability of the existing slopes under the bridge loads.

6. Bridge Options

Four options are considered for how the existing bridge may be rehabilitated for adaptive re-use as a pedestrian / bike crossing. Each option assumes that the truss span is lifted from the supporting trestles and dismantled either on site or transported to an offsite facility. The steelwork should be cleaned, and any corrosion repaired. Connection gusset plates should be replaced. The truss would then be re-assembled using bolted connections in place of rivets and then re-painted.

- Option 1 - Rehabilitate the approach trestles
- Option 2 - Replace the approach trestles with new structures to match existing.
- Option 3 - Replace the approach trestles with new longer spans.
- Option 4 - Retain the approach trestles as facades.

Each of the above options is described further in the sections below.

6.1 Option 1 - Rehabilitate the Approach Trestles

This option would seek to retain as much of the original steelwork of the approach trestles as possible (Figure 15). Construction would include:

- Carefully dismantle the approach trestles and catalog each member for its location.
- Clean the steel to bare metal.
- Replace members that are corroded beyond repair. This would be on a case by case basis dependent on the anticipated loading for the member vs. level of corrosion. This will require continuous design oversight during construction.
- Replace gussets with bolted connections to replace the rivets.
- Replace bracing members with larger sections based upon the results of the structural analysis.
- Construct new footings and abutment support. Because the top of footing would be at a higher level than existing, there will need to be reconfiguration of the steelwork to accommodate the new geometry.
- Re-assemble the new/rehabilitated approach trestles.
- Lift the rehabilitated truss on to the approach trestles.
- Add new deck steelwork, and deck slab and railings to support the pedestrian / bike trail.

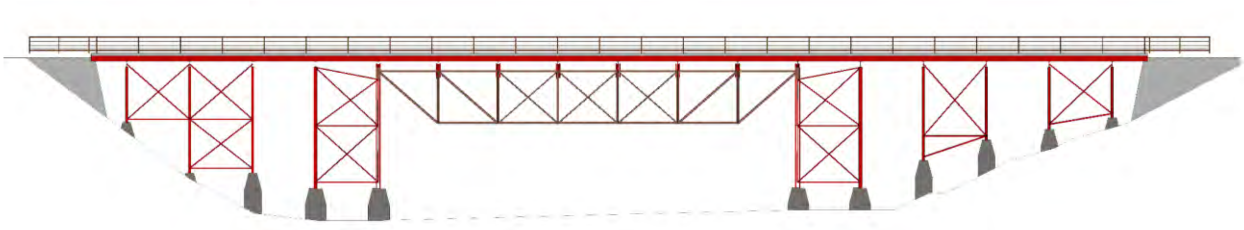


Figure 15: Option 1 - Rehabilitate the Approach Trestles

6.1.1 Risks

- It is possible that very little of the original steelwork from the approach trestles will be able to be salvaged for re-use. The full amount of corrosion will not be known until connections are dismantled and the steelwork is blast cleaned.
- There will need to be continuous design input throughout construction to agree the members that are able to be salvaged vs. those that should be replaced. This would include continuous coordination with the DC State Historic Preservation Office, or other assigned advisor, which may extend the construction schedule. In some cases, the original steel section is no longer available and so there may be differences in the sizes of the replaced steel members.
- There will need to be changes to the geometry of the steelwork to accommodate new footing elevations on the west side which may not be aesthetically acceptable. This coordination would occur during the design phase.
- Steel bridges are typically re-painted every 20 to 25 years. The rehabilitated steelwork will require more maintenance during its life time if it is to meet the AASHTO LRFD 75-year design life with more frequent repairs and repainting being necessary compared with a properly detailed new structure. Figure 16 shows the Glen Echo Trolley Bridge over Minnehaha Creek that was rehabilitated in 2014 where areas of the paint system have already begun to fail.

Figure 16: New Corrosion at the Rehabilitated Glen Echo Trolley Bridge (August 2019)

6.2 Option 2 - Replace the Approach trestles with new Structures to Match Existing

This option would replace the approach trestles with new steelwork using similar geometry as the existing (Figure 17). Construction would include:

- Dismantle the approach trestles and remove from site.
- Construct new footings and abutment support.
- Assemble new steelwork for the approach trestles.
- Lift the rehabilitated truss on to the new approach trestles.
- Add new deck steelwork, and deck slab and railings to support the pedestrian / bike trail.

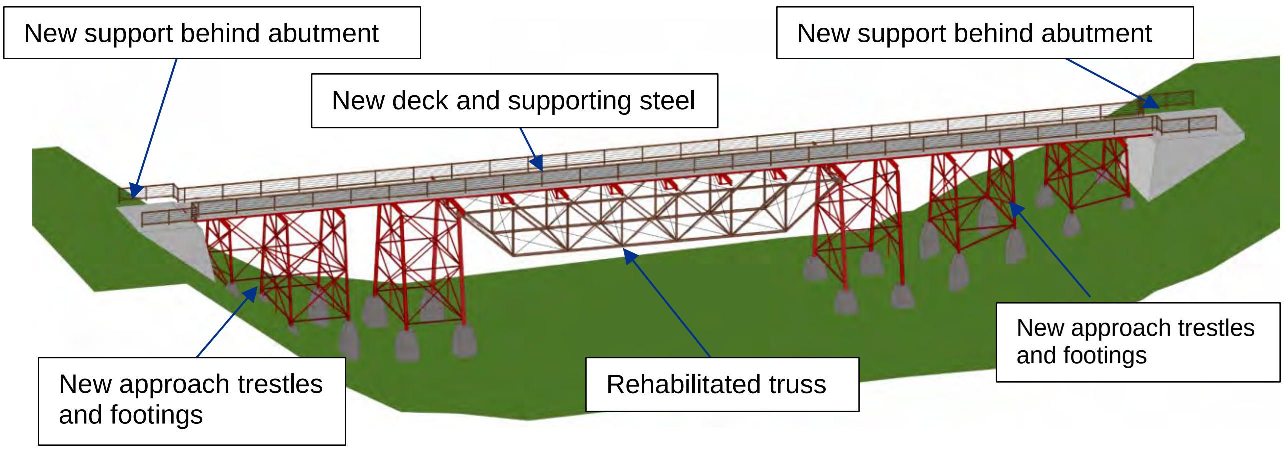
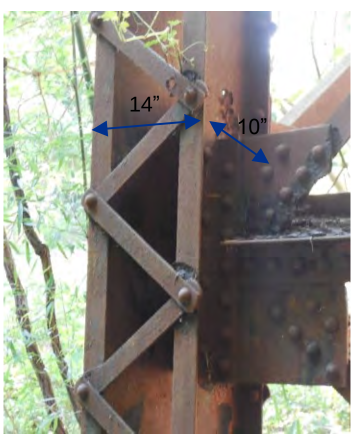


Figure 17: Option 2 - Replace the Approach trestles with new Structures to Match Existing

Although closed box shapes are generally preferred for maintenance and durability, the majority of section shapes could be kept as close as possible to the existing shapes using I-sections and angles. Replacing the existing riveted gusset connections with welded connections would also provide for a structure that is easier to inspect and maintain as there are fewer areas where moisture and dirt can be trapped. A similar structural approach of welded I-sections is used for a footbridge over Route 50 in Arlington (Figure 18).

The exception would be the main truss supports at Gridlines 5 and 12 which are currently lattice members. It is recommended that these are replaced with sections that would be easier to inspect and maintain. A hollow rectangular shape (see Figure 19) of similar outside dimensions could provide adequate capacity. This would be preferred to I-section shapes would need to significantly larger shapes to achieve the same capacity.

Figure 18: Footbridge over Route 50 in Arlington



Comparable HSS10x10x0.375

Figure 19: Replacement Section Shapes

6.2.1 Risks

- Less of the original structure would remain with this approach. This may be acceptable since the main suspended truss is the most visible element of the bridge.
- Coordination with the DC State Historic Preservation Office, or other assigned advisor, is needed on the selection of section shapes and the geometry of the approach trestles where they need to be modified due to the new concrete footing elevation. Most of this coordination would occur during the design phase before construction begins.

6.3 Option 3 - Replace the Approach Trestles with new Longer Spans

This option is similar to Option 2, with the approach trestles being entirely replaced, except that this Option uses fewer, lower cost, concrete piers (Figure 20). The approaches would resemble a more modern bridge with only the truss span retained. Construction would include:

- Dismantle the approach trestles and remove from site.
- Construct new footings and abutment support.
- Construct new concrete piers.
- Lift the rehabilitated truss on to the new approach trestles.
- Lift new approach spans, using steel plate girders.
- Add new deck steelwork, and deck slab and railings to support the pedestrian / bike trail.

Concrete piers would result in a structure with a much lower life cycle cost than the steel trestles. The footings would also be much easier to construct with only one located on the eastern slope.

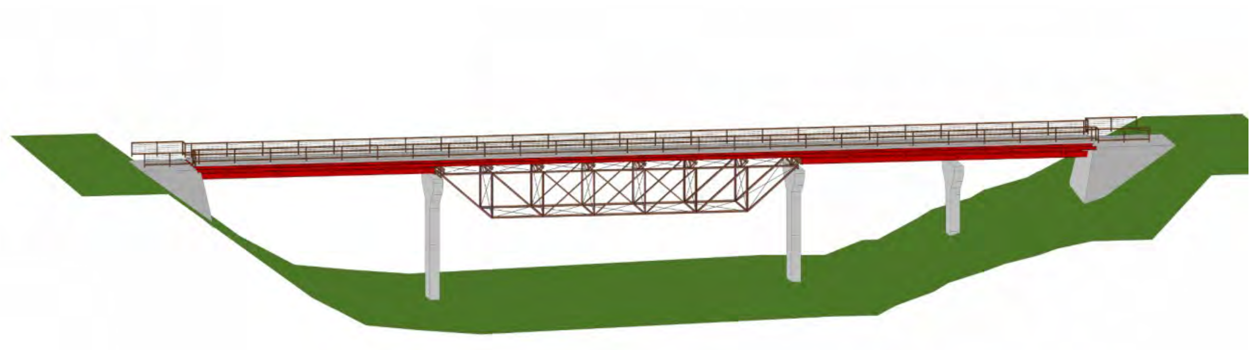


Figure 20: Option 3 - Replace the Approach Trestles with new Longer Spans

6.3.1 Risks

- The concrete piers are very different to the existing approach trestles and may not be acceptable to the DC State Historic Preservation Office.
- The bridge is less aesthetically attractive compared with Options 1 and 2 due to the dissimilarity between the approach piers and truss span.
- The maximum span length between abutment and pier is approximately 80-feet resulting in deck sections up to 3-feet deep compared with the existing 10-inch deep sections over the approach trestles and truss span.

6.4 Option 4 - Retain the Approach Trestles as Facades

This option seeks to maintain as much as possible of the existing approach trestles as a non-structural façade to the bridge (Figure 21). Support to the bridge would be by means of new structural members, which could be either steel or concrete, constructed within the footprint of the existing approach trestles. Construction would include:

- Carefully dismantle the approach trestles and catalog each member for its location.
- Clean the steel to bare metal.
- Construct new footings and abutment support.
- Construct new approach trestles (or piers), either steel or concrete.

- Salvage as much of the historic parts of the approach trestles and re-construct in their original location. They may need to be attached to the new approach trestles to provide long term stability.
- Lift the rehabilitated truss on to the new approach trestles.
- Add new deck steelwork, and deck slab and railings to support the pedestrian / bike trail.

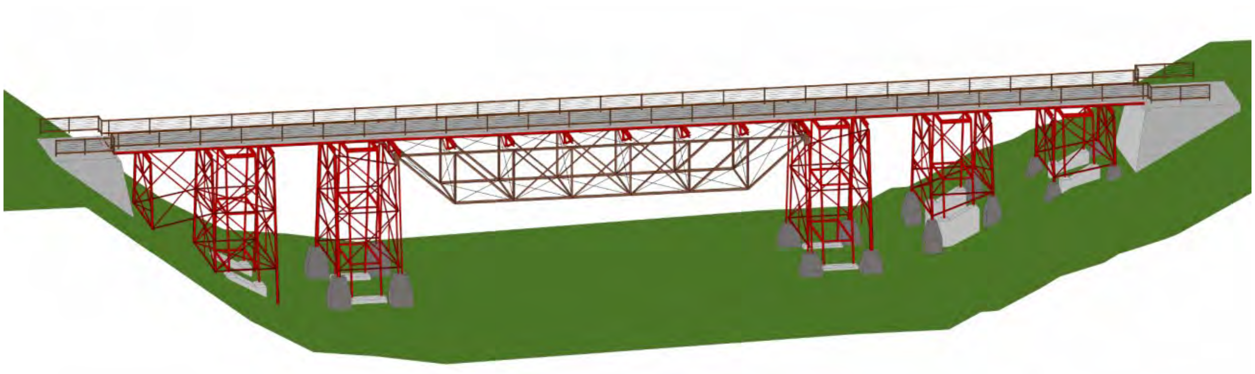


Figure 21: Option 4 - Retain the Approach Trestles as Facades

6.4.1 Risks

- The resulting structure would be the most difficult to maintain with areas of the new structure potentially hidden within the non-load bearing façade.
- Depending on the geometry of the new approach trestles, the structure may look aesthetically unattractive with multiple redundant members.
- As with Option 1, it is possible that very little of the original steelwork from the approach trestles will be able to be salvaged for re-use. The full amount of corrosion will not be known until connections are dismantled and the steelwork is cleaned.
- There will need to be continuous design input throughout construction to agree the members that are able to be salvaged. This would include continuous coordination with the DC State Historic Preservation Office, or other assigned advisor, which may extend the construction schedule.
- Unless there is a great deal of care in detailing how the historic steelwork is supported from the new structure, it is possible that the additional support will result in a structure that bears little resemblance to the original structure.

6.5 Other Considerations

Additional considerations that should be included in considering if the Foundry Branch Trolley Trestle Bridge should be rehabilitated for adaptive reuse as part of a pedestrian/bicycle trail include:

- The Trolley Trestle Bridge would be the only bridge of its type in DDOT’s inventory. This would require DDOT to establish bridge specific inspection and maintenance plans resulting in higher life cycle costs than for a typical DDOT bridge.
- Before advertising a construction contract for the bridge rehabilitation there would need to be a Memorandum of Agreement (MOA) in place that clearly defines roles, responsibilities and approval periods that would apply during the construction phase of the project. The 31st Street Bridge project in Georgetown included a similar MOA between DDOT and the National Park Service. This set out the process for the rehabilitation during construction and established the role of an external expert responsible for review and approval of the rehabilitation of the historic parts of the bridge. The cost of the external expert is included in the project construction cost as an allowance.

7. Constructability

7.1 Access

Access to the site beneath the bridge would be from Canal Road, westbound lane only. There would need to be traffic control for vehicles to safely enter and leave the site which would also need to account for the safe accommodation of pedestrians and bicyclists using the existing sidewalk along Canal Road (24 DCMR § 3315). There would also need to be access to each abutment from Foxhall Road to the west and from Georgetown University to the east, both across WMATA owned property (Figure 22).

A construction work area would be established beneath and adjacent to the bridge and a lay down area in the space between Canal Road and the bridge (Figure 22). A permit from NPS would be needed for access and construction rights from WMATA to access their property (and bridge). Existing utilities would need to be protected in place and erosion and sediment control measures established in accordance with District Department of Energy and Environment (DOEE) requirements.

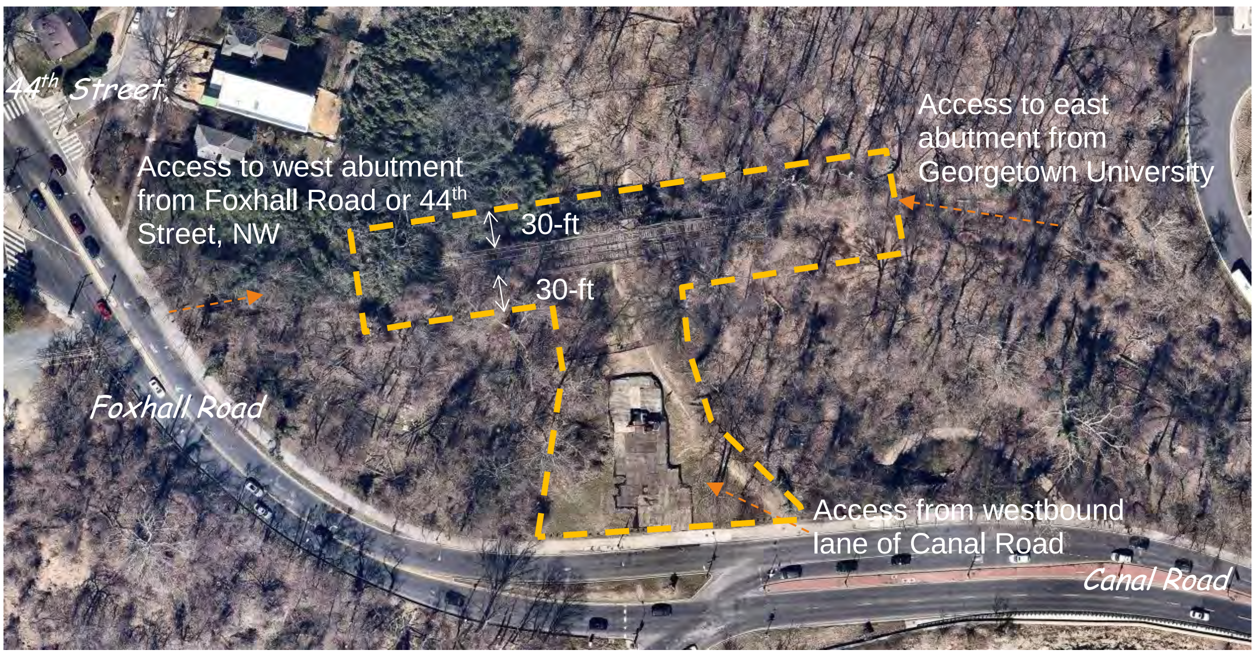


Figure 22: Site Access

7.2 Construction Approach

The approach to construction would be similar for all the options and include the follow major stages:

Stage 1: Establish access to project site. Construct erosion and sediment control measures and site security. Clear vegetation from around the project area. Protect existing utilities and construct site laydown area.

Stage 2: Erect temporary shoring to the truss span. Remove debris and existing rail ties from the top of the bridge. Access to the top of the bridge is from remote access platforms.

Stage 3: Remove existing deck steelwork (cross bracing members and longitudinal stringers).

Stage 4: The truss should be lifted from the supporting trestles and dismantled either on site or at an offsite facility. The steelwork should be cleaned, and any corrosion repaired. Connection gusset plates should be replaced. The truss would then be re-assembled using bolted connections in place of rivets and then re-painted.

Stage 5: Dismantle approach trestle steelwork. Depending on the option selected, the steelwork would either be removed to an off-site facility for cleaning and repair or disposed of.

Stage 6: Remove existing concrete footings.

Stage 7: Construct new footings. Underpin the existing abutments so they can continue to retain earth loads and construct a new foundation in the space behind the abutments to support the deck. Regrade the approach slopes and provide rock erosion protection.

Stage 8: Re-construct the approach trestles (or piers, per the option selected) on the new footings.

Stage 9: Lift re-habilitated truss onto the approach trestles/piers.

Stage 10: Install new deck steelwork, deck slab and railings.

Stage 11: Complete trail connections and re-establish the landscaping in the construction area beneath the bridge.

Stage 12: Open the trail on, and beneath, the bridge.

8. Order of Magnitude Cost Estimate

A planning-level order of magnitude cost estimate was completed in accordance with a Class 5 estimate as defined by the Association for the Advancement of Cost Engineering (AACE). The presented estimate has a range of accuracy, as defined by AACE and estimator judgment, of -20 percent to +50 percent based on the level of project definition and other factors such as project timing. The estimate was prepared based upon selection of Option 2 - Replace the Approach trestles with new Structures to Match Existing, as the preferred option.

The order of magnitude cost estimate for Option 2, and the cost at the lower and upper accuracy range is presented below.

Low Range (-20%)	Estimated Cost	High Range (+50%)
$2,201,000	$2,751,000	$4,127,000

A qualitative summary of the cost comparisons for Options 1, 3 and 4 is provided below.

Option 1 - Rehabilitate the Approach Trestles

This option would require a longer construction duration compared with Option 2 to assess which existing steel members may be retained. This would be dependent on their condition after removal and cleaning. There would be additional design cost and coordination with the DC State Historic Preservation Office, or other assigned advisor, throughout both the design and construction phases of the project due to members from the approach trestles needing to be assessed individually if they may be retained. There is unlikely to be any saving in material quantities compared to Option 2. Foundation costs and restoration of the main truss are the same as Option 2.

Option 3 - Replace the Approach Trestles with new Longer Spans

This option would require a similar construction duration to Option 2. Design and detailing of the new concrete approach piers would require the least level of effort. There would be the same coordination with the DC State Historic Preservation Office, or other assigned advisor, as Option 2 focused only on the main truss. Design of the approach piers would have been approved in advance, as with Option 2. There would be some savings in steel quantities. Costs for restoration of the main truss are the same as Option 2.

Option 4 - Retain the Approach Trestles as Facades

This option would require a similar construction duration to Option 1 to assess which existing steel members may be retained as part of the façade structure, and how they would be tied to the new supports. This would be dependent on their condition after removal and cleaning. There would be additional design cost and coordination with the DC State Historic Preservation Office, or other assigned advisor, throughout both the design and construction phases of the project due to members from the approach trestles needing to be assessed individually if they may be retained. There will be additional steel and foundation quantities needed to provide support to the façade steelwork. Costs for restoration of the main truss are the same as Option 2.

Ranking the anticipated cost for each option would have Option 3 as the least cost, followed by Option 2 and then Option 1. Option 4 would be the most expensive. The cost for each Option would likely fall somewhere within the estimate range described above.

Attachment A

Foundry Branch Trestle Inspection Report

Palisades Trolley Trail and Foundry Trestle Feasibility Study and Concept Plan Project

Foundry Branch Trestle Inspection

December, 2019

District Department of Transportation

Document history and status

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Palisades Trolley Trail and Foundry Trestle Feasibility Study and Concept Plan

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Executive Summary

The Foundry Branch Trolley Trestle Bridge is a historic bridge located in Foundry Branch Park close to Georgetown in the District of Columbia. It was originally constructed as part of the West Washington & Glen Echo Electric Railroad/Glen Echo Trolley Line. This trolley system served Washington, D.C. from Georgetown to Glen Echo, Maryland from its construction in 1897 to 1962 when it closed.

The sole purpose of this report is to document the results of the inspection of the bridge. Inspection was completed through a combination of visual inspection from ground level using telephoto camera lenses to view the upper parts of the bridge, correlated with a 3D reality mesh model of the bridge developed from a 3D laser survey. The report is intended to support the feasibility study that is under preparation for adaptive reuse of the bridge as part of the pedestrian/bike trail.

Certain parts of the structure were not able to be inspected due to the significant overgrowth of vegetation that covers parts of the bridge, the remaining trolley rail ties that cover the top of the bridge and physical access to the higher parts of the structure not being feasible due to safety concerns. This report represents the condition of the bridge at the time the inspection work was completed.

The approach trestles to the bridge are in overall very poor condition. There is significant deterioration of the members that provide primary structural support. Secondary members that provide for alternate load paths are also in an advanced state of deterioration. At the main vertical support to the suspended truss it appears that vegetation growing within the steel section is providing the only means of support to the suspended truss given the amount of section loss to the steelwork.

The suspended truss span is generally in better condition with minor rusting of the chord and web members. A large part of the original paint system is still intact. There is some corrosion of the gusset connection plates and of horizontal members where water has not been able to drain.

Given the condition of the primary supporting members of the approach trestles, there are concerns over the stability of the main truss span. The area beneath the bridge should remain closed and further signage added that identify the safety hazards beneath the bridge. Unless the truss is shored up on temporary piers, or lifted from the bridge, the structure is in danger of collapse in the short term.

1. Introduction

The Foundry Branch Trolley Trestle Bridge is a historic bridge located in Foundry Branch Park close to Georgetown in the District of Columbia (Figure 1.1). It was originally constructed as part of the West Washington & Glen Echo Electric Railroad/Glen Echo Trolley Line. This trolley system served Washington, D.C. from Georgetown to Glen Echo, Maryland from its construction in 1897 to 1962 when it closed (Figure 1.2 and Figure 1.3).

Figure 1.1: Project Location

Figure 1.2: View from a Trolley Approaching the Bridge from Georgetown and on the Bridge

Figure 1.3: Aerial Photograph of the Foundry Branch Bridge circa 1963 (DDOT Archives)

The overall dimensions of the trestle are 252 feet long by 19 feet wide. The primary framing consists of a pair of underslung steel trusses spanning 100 feet (

Figure 1.4) and a series of braced frame steel approach structures and concrete abutments on the far east and west sides (Figure 1.5 and Figure 1.6). The steel approach structures are supported on rectangular concrete piers. The top of the bridge has zero grade from end to end. The steel members consist of a combination of I-shapes, channels, angles, WT's, and rods, with riveted connections.

Figure 1.4: Suspended Truss – South Elevation

Figure 1.5: East Section Frames (Looking East)

Figure 1.6: West Section Frames (Looking West)

A pedestrian path was located under the steel truss span until the entire structure was fenced off in 2016 when deemed hazardous because of potential falling bridge elements (Figure 1.7).

Figure 1.7: Fallen rail tie beneath the bridge (August 2, 2016)

This inspection seeks to document the condition of the existing bridge to support a feasibility study for the adaptive re-use of the bridge as a pedestrian/bike trail. Inspection work included:

- Visual examination of the framing structural components.
- Since the bridge is in advanced state of deterioration, a 3D laser scan was performed by A. Morton Thomas (AMT) on April 2019 to create a 3D model of the bridge. Jacobs had representatives on site during the 3D scan to identify key areas to be scanned and document observed field conditions. The inspection method required no physical interaction with the bridge.

Both the 3D model and field notes were used to prepare this report and complete the bridge assessment.

2. Access to Site

The bridge main span crosses property owned by the National Park Service (NPS). The property on each side of the bridge main span, on which the approach span footings are located, and the approaches to the bridge are owned by the Washington Metropolitan Area Transit Authority (WMATA) (Square 1322, Lot 817 and Square 1321, Lot 822) – see Figure 2.1. The physical bridge is owned by WMATA.

Access to the bridge during inspection had to be coordinated with DDOT employees and the NPS personnel. An access permit issued by NPS for the dates during which access was also granted. The site is fenced off from public access with a gate that is locked with a chain and coded padlock. A warning sign advises trail users of the hazardous condition (Figure 2.2).

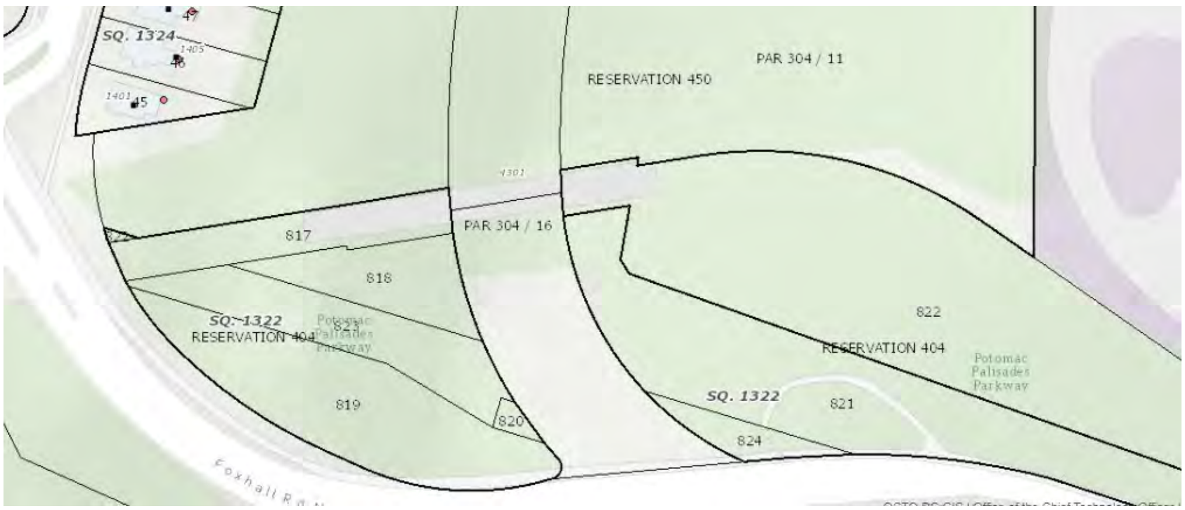

Figure 2.1: Land Ownership

Figure 2.2: Fence and Warning Sign

3. Results from the 3D Scan

A Morton Thomas collected survey data from the bridge using a 3D scanner. The work was performed over one week in April 2019 before full leaf out of the surrounding vegetation. Weather conditions were clear on each day of the survey.

Figure 3.1: Survey Team Performing 3D Scan of the Bridge

The purpose of the scan was to assemble a 3D point cloud of the bridge which could be used to develop the bridge inspection report without needing to bring access equipment to the site to reach the higher levels of the structure. The survey approach was selected over physical access to ensure the safety the inspection team following reasons:

- The timber rail ties on the top of the bridge are loose and have begun to fall from the bridge.
- Vegetation, including bamboo, ivy and some large trees has grown up close to each side of the bridge making physical access impossible. One tree has fallen and is leaning on the suspended truss.
- There is a large depression in the ground to the north west of the bridge that would make siting access equipment difficult, even if the bamboo were cleared.

Figure 3.2: Bridge Overgrowth

3.1 Post Processing of the 3D Cloud

The 3D cloud data was provided to Jacobs on an external hard drive due to the volume of data being transmitted. The volume of data also meant that it was extremely cumbersome, if not impossible, to try to use the point cloud directly in CAD software.

To simplify use of the data, Bentley's ContextCapture software was used to post process and develop a 3D reality mesh of the bridge which could then be used to support the inspection. Post processing took several weeks of computer time and still resulted in nearly 60Gb of data.

The resulting reality mesh proved to be extremely detailed. Using ContextCapture to view the model, we were able to take dimensions and identify areas where corrosion had gone right through a steel section. The model also allowed spot checks of information that would normally have needed a follow up visit to the site.

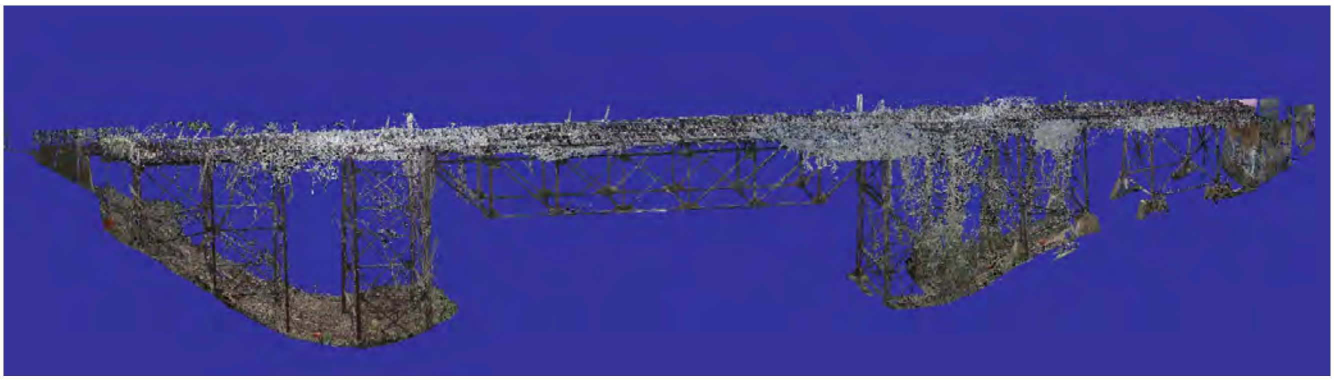

Figure 3.3: Reality Mesh of the Full Bridge

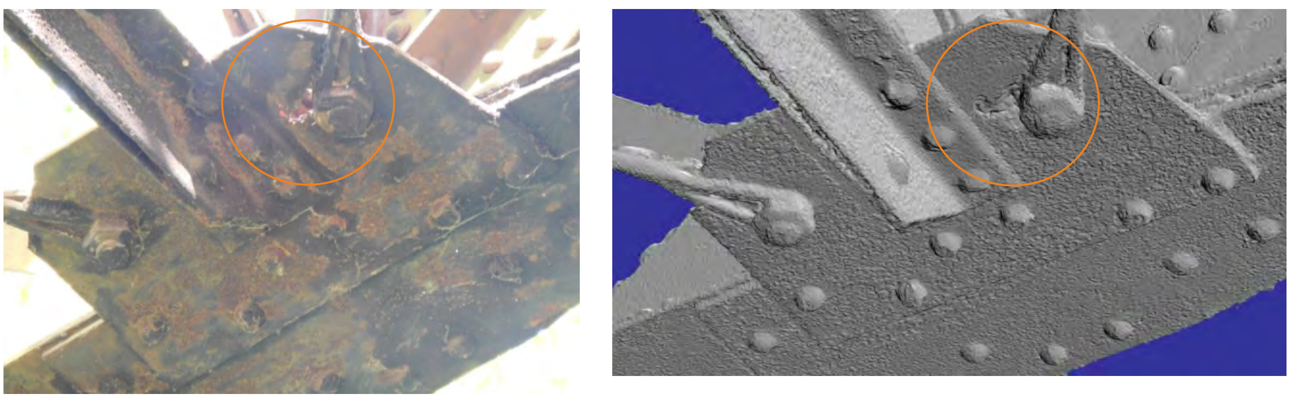

Figure 3.4: View of Gusset Detail from Photo and Model (Gridline 10 – North level -2)

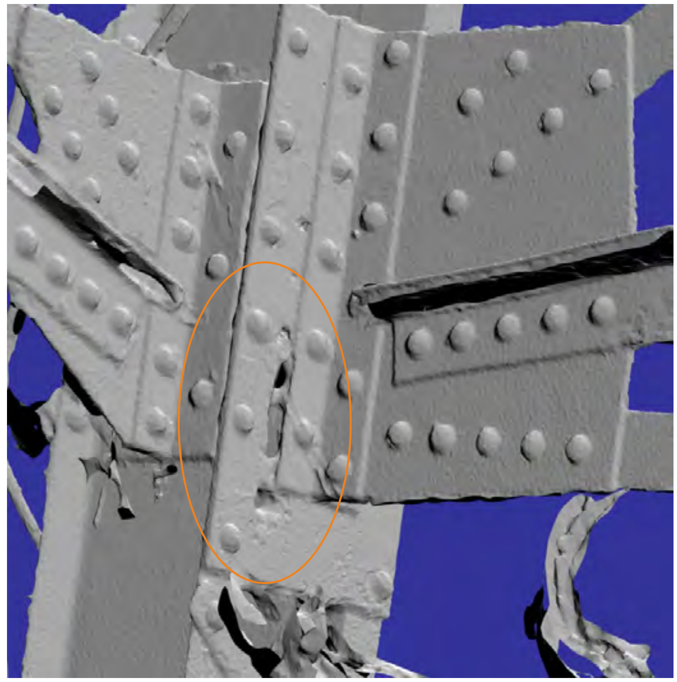

Figure 3.5: View of Gusset Detail from Photo and Model (Gridline 12 – South level -4)

3.2 Future 3D Model Improvements

The 3D cloud was obtained from data taken at thirty-six survey locations dispersed around the bridge and as high up the approach slopes as possible. The combined data allowed most of the 3D geometry of members to be shown in the resulting reality mesh. Back faces of members from one survey setup are caught at a different setup and combined in the reality mesh. The multiple survey setups are why there is so much data for the 3D cloud as one point in space may be covered multiple times. Areas that do not get covered by the survey are shown in the reality mesh as missing (Figure 3.6). This sometimes led to confusion over areas of the bridge that were corroded through, versus missing data. This was corrected through physical inspection on site and review of photographs.

Survey data, including photographs and lidar scans, obtained from a drone would have helped to fill in some of the gaps in the data, but unfortunately the heavy vegetation surrounding the bridge prevented use of a drone.

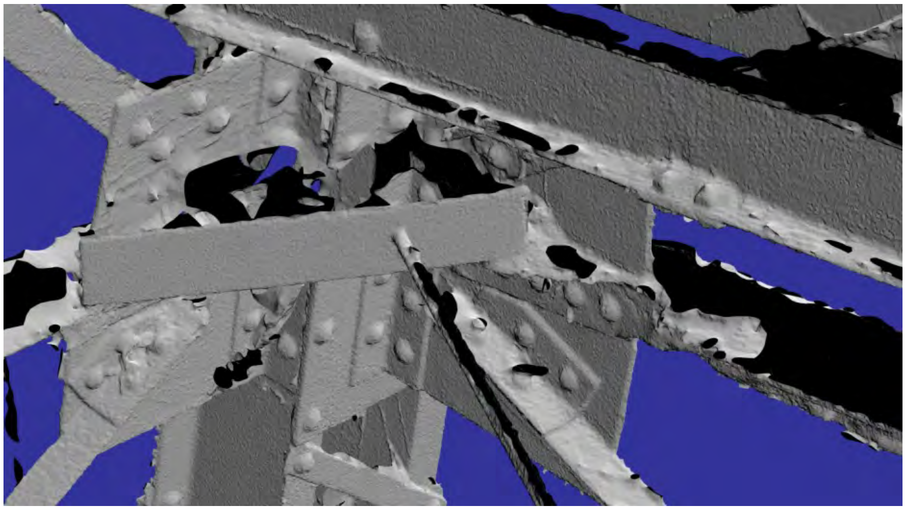

Figure 3.6: Missing Data

4. Bridge Geometry

The bridge sketches included in this report, were produced by Jacobs Engineering based on field measured data and the 3D bridge model. These include:

- Appendix A: Elevations and Sections of the bridge that show the referencing system used throughout this report to identify locations on the bridge (gridlines and levels).
- Appendix B: Dimensions and section designations.

Section sizes were based on measurements taken from the 3D bridge model. The member dimensions do not correlate with modern available section sizes so, the measured dimensions were cross-checked against the 1903 Carnegie Steel Company catalog of steel sections (Appendix C) and the closest matching shape selected. The 1903 section dimensions were then carried forward into the structural analysis of the bridge.

5. Inspection Observations

Jacobs performed site visits during the 3D scanning of the bridge (April 2019) to visually survey and review the condition of existing structural components of the trestle structure. A follow up site visit was made on August 22, 2019 after review of the 3D scanning to confirm observations taken from the reality mesh. There are no utilities visible on the bridge, but a Pepco power line is suspended above the bridge along its full length from poles located behind each abutment

Photographs and observations made are included in Appendix D. Marked up drawings identifying areas where deterioration of the steelwork was visible are included in Appendix E.

The following narratives summarize the observations.

5.1 Vegetation Growth:

The lack of regular maintenance of the trestle has allowed significant vegetation growth in and around the structure. Vegetation is intertwined with structural components, especially on the west section of the bridge structure (Figure 5.1).

Figure 5.1: Vegetation Growth Gridline 5 – South

5.2 Timber Rail Ties

The original heavy timber rail ties (measured on site as 9 inch wide by 7 inch deep) are significantly deteriorated and are not salvageable in their present state. Several wooden members were observed at the base of the structure, having fallen from the top of the bridge. A timber walkway was also originally present on each side of the tracks which is now in partial state of collapse. The steel tracks that the ties would have originally supported have been removed from the bridge. The railings along each edge of the bridge, visible in Figure 1.2, have been removed at some time in the past.

Figure 5.2: Timber Rail Ties (From West Abutment and East Abutment)

5.3 Concrete Abutments:

The far east and west sides of the structure are supported by concrete abutments. These structures serve as both retaining walls and gravity support of steel beams along the top of the trestle. Abutments are in overall satisfactory condition. Several cracks and small concrete spalls are present throughout. Cracks start at top of the abutment under steel beams and extends vertically up to full height with other cracks extending horizontally. Cracks are up to 1/16" wide with minor moisture stain and efflorescence. East abutment has exposed footing for 90% of length with minor spalls and honeycombing. West abutment has the footing exposed and is undermined for approximately 8 ft long (Figure 5.3).

Figure 5.3: West Abutment (Left) and East Abutment (Right)

The abutment bearing shelf supports the 10-inch deep longitudinal deck stringers (Figure 5.4). The shelf is 20-inch wide with a smaller bearing pedestal that supports the stringer. The stringer is restrained by a clip at the edges which is intended to allow it to slide freely in the longitudinal. It is no longer able to slide since the stringer is up tight against the back of the abutment.

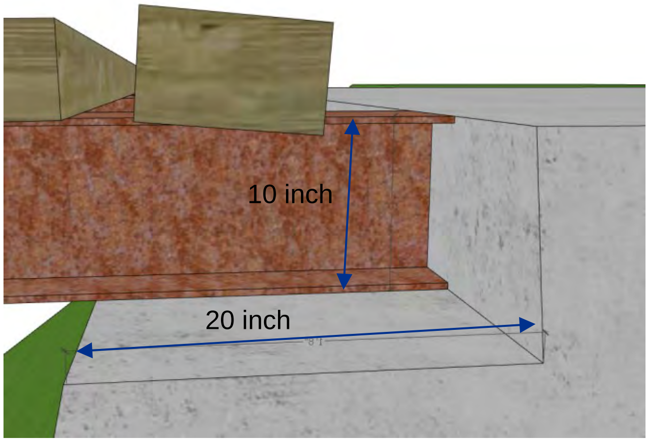

Figure 5.4: Abutment Bearing Shelf

5.4 Concrete Piers

Each of the trestle steel columns are supported by a concrete pier. Details of each pier are summarized below.

Ref.	Description
Gridline 1	South: Rectangular footing approx. 3-ft sq. Undermined at the front. Exposed aggregate. North: Not visible
Gridline 2	South: Rectangular footing. Not possible to measure dimensions. Undermined at the front. Exposed aggregate. North: Not visible
Gridline 3	South: Not visible North: Not visible
Gridline 4	South: Not visible North: Not visible
Gridline 5	South: Not visible North: Not visible
Gridline 12	South: 4-ft sq. at the top, approx. 1-ft height above ground North: 4-ft sq. at the top, approx. 1-ft height above ground
Gridline 13	South: 3-ft sq. at the top, tapering to 4-ft sq. Approx. 9-ft height above ground. Horizontal crack at half height North: 3-ft sq. at the top, tapering to 4-ft sq. Approx. 6-ft height above ground. Covered with ivy.
Gridline 14	South: 2-ft sq. at the top, tapering to 4-ft sq. Approx. 5-ft height above ground. Horizontal crack and exposed aggregate

Ref.	Description
	North: 2-ft sq. at the top, tapering to 4-ft sq. Approx. 8-ft height above ground. Horizontal crack and exposed aggregate
Gridline 15	South: 3.6-ft sq. at the top, tapering to 6.3-ft sq. Approx. 5-ft height above ground. Full width by 2-ft face spall North: 2-ft sq. at the top, tapering to 4-ft sq. Approx. 7-ft height above ground. Horizontal crack at mid height. Undermined as lower corner
Gridline 16	South: 3.6-ft sq. at the top, tapering to 5.6-ft sq. Approx. 4-ft height above ground North: 2-ft sq. at the top, tapering to 4-ft sq. Approx. 4-ft height above ground
Gridline 17	South: 3.6-ft sq.at the top, tapering. Approx. 2-ft height above ground North: 2-ft sq. at the top, tapering to 2.75-ft sq. Approx. 1.66-ft height above a 4-ft square rectangular base

All observed piers are experiencing deterioration in the form of concrete spalls and many have full width horizontal cracks. The south pier on gridline 15 has a full width - 2 feet high spall that expose inside concrete with no sign of reinforcement rebars. Also, inspection of the cores taken as part of the previous WMATA study show no signs of reinforcement. It is possible that the piers are unreinforced.

5.5 Approach Trestle Steelwork

The vast majority of the steel protective coating is no longer present. Some black paint is visible on the underside of the upper steelwork which, given its age, is likely to contain lead. No paint samples were taken due to the difficulty in safely accessing the higher areas of the bridge.

Both primary (columns and beams) and secondary (angles and tie rods) framing members have moderate to severe corrosion varying from to rusting of the surface to holes through flanges and/or webs, to complete loss of section allowing the member to fall to the ground.

Gusset plates that connect primary and secondary members also show corrosion from slight rusting of the surface to complete section loss. Connections are riveted throughout.

The following section summarize the findings for each part of the bridge.

5.5.1 Columns

The steel column sizes and sections vary based on the member length and spacing of steel angle bracing. From shortest to tallest, column sections present are 7” wide flanges, 10” wide flanges, and double 10” channel lattice columns at the support to the truss span.

All the steel columns were observed to have rust along full length of the member. The most significant deterioration is at the column base to concrete pier connection due to water sitting on top of the concrete, or in a depression in the ground where column members are buried.

Of most concern is the base of Gridline 3 north and south (Figure 5.5) and Gridline 5 north and south (Figure 5.6). At each of these locations there is very little steel section remaining. The load these members should be carrying is assumed to be re-distributed through bracing (which, as discussed below, is also in poor condition) to other members, and by built up vegetation. It is likely that the members which are buried, and not visible, have similar levels of deterioration.

Figure 5.5: Column Base Gridline 3 North and South

Figure 5.6: Column Base Gridline 5 North and South

Many of the column connections to bracing members are no longer effective (Figure 5.7).

Figure 5.7: Column Connection to Bracing Members

5.5.2 Bracing

Single and double angle framing is present throughout the trestle in the form of diagonal and horizontal bracing members. The member sizes vary depending on the length and orientation of the framing.

Some bracing members are fully corroded and either loose and not connected to primary members or broken and present on the ground underneath the structure. Many of the angles exhibit moderate to severe corrosion (Figure 5.8).

There is also some 0.7 inch diameter steel rods that provide in-plan bracing at the tops of the trestles (level 0), and first level down (level -2). Although the rods are typically in good condition, many of the gusset connections to the trestle vertical legs has corroded through allowing the rods to hang free.

Figure 5.8: Trestle Bracing Members

5.5.3 Top Elevation Steel Beams

A steel grillage that directly supports the timber rail ties sits on top of 15 inch deep I-Section girders that span transversely between the tops of the trestle legs. The parts of the 15 inch deep girders that are visible show significant deterioration to the webs and at some locations the webs are completely corroded through. Some top flanges also show significant corrosion.

Figure 5.9: Corrosion to 15-inch Deep Cross Girder

Four girders, each 10 inch deep, span longitudinally between the tops of the trestles. They are braced in plan by a combination of 6 inch deep channels that connect the girders transversely and diagonal 3 inch deep angles.

The longitudinal girders show signs of rusting on their under side. It was not possible to inspect their top side as they are covered by the rail ties which bear directly onto the girders. Based on prior experience, it is likely that moisture will have been trapped beneath the rail ties and caused to top flanges to corrode, but it will not be possible to verify this until the ties are removed.

5.6 Steel Truss Span

The truss assembly is composed of double angle chords members and single/double angle or double t web sections. The truss span was inspected through a combination of visual inspection using a telephoto camera lens, and inspection of the 3D reality mesh.

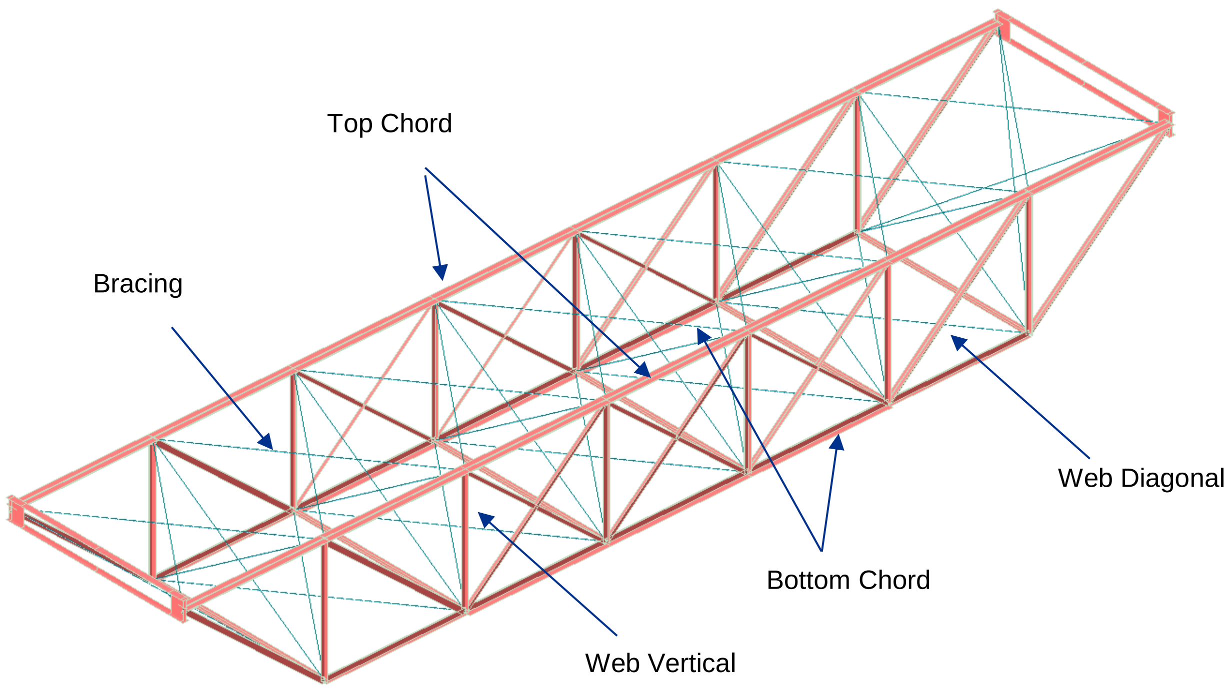

Figure 5.10: Suspended Truss

5.6.1 Truss Chords

Inspection of the chords did not identify any areas of corrosion greater than surface rusting. In many places the original paint coating is still visible.

5.6.2 Truss Webs

Inspection of the web verticals and diagonals did not identify any areas of corrosion greater than surface rusting. In many places the original paint coating is still visible.

5.6.3 Truss Horizontals

The double angles that connect the lower chords of the north and south trusses show signs of corrosion between the webs of the connected angles where expansion has pushed the steel apart between riveted connections (Figure 5.11). This is where moisture has been trapped in the space between the angles.

Figure 5.11: Rusting between Double Angles

The top chord of the truss is connected horizontally by the 15 inch transverse beams (see Section 5.5.3).

5.6.4 Truss Bracing

The truss is braced with 0.7 inch diameter round steel bars in plan in the top and bottom chords of the truss, and horizontally at each truss bay. The bars are arranged at each bay in an X configuration and will act as tension only members. There is slight rusting of the surface of the bars.

5.6.5 Connections

Riveted gusset plates join the members of the truss chords, webs and bracing. These appear to be generally in good condition with the exception of two of the plates connecting the lower chord of the truss to bracing where a hole has corroded through the plate. This may be due to ponding of water on the top of the plate and so the same condition is likely at other connections, but not yet right through the plate.

6. Summary

The approach trestles to the bridge are in overall very poor condition. There is significant deterioration of the members that provide primary structural support. Secondary members that provide for alternate load paths are also in an advanced state of deterioration.

The suspended truss span is generally in better condition with minor rusting of the chord and web members. A large part of the original paint system is still intact. There is some corrosion of the gusset connection plates and of horizontal members where water has not been able to drain.

Certain parts of the structure were not able to be inspected due to the significant overgrowth of vegetation that covers parts of the bridge, the remaining trolley rail ties that cover the top of the bridge and physical access to the higher parts of the structure not being feasible due to safety concerns.

Given the condition of the primary supporting members there are concerns over the stability of the main truss span. The area beneath the bridge should remain closed and further signage added that identify the safety hazards beneath the bridge. Unless the truss is shored up on temporary piers, or lifted from the bridge, the structure is in danger of collapse in the short term.