LONG BRIDGE STUDY

DISTRICT DEPARTMENT OF TRANSPORTATION

LONG BRIDGE STUDY

Existing Conditions Bridge Assessment Report

March 2013

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1. Background

1.1 History of the Long Bridge

The first version of the Long Bridge was constructed to connect the District of Columbia and Virginia side of the nation's capital and was authorized by Congress in 1808. First opened in May 1809, the Long Bridge, named for its length, was built on timber piles and included moveable/opening spans. The bridge was burned in 1814 by invading British forces and subsequently rebuilt and restored to service in 1816.

Until the 1850's, the bridge carried only foot and horse drawn traffic. Rails were first installed during the Civil War, yet the bridge was unable to carry railroad locomotives, rather, fully loaded freight cars were pulled by horse across the bridge. By 1863, a new parallel bridge was built to support full railroad use. The 1863 structure was about 100 feet downstream of the original alignment and again had two moveable spans. In 1870, perpetual use of the Long Bridge was ceded from the government to the Pennsylvania Railroad. By 1896, the bridge was carrying freight and interurban trolleys. An estimated 250 trains a day crossed the bridge, and the moveable span opened an estimated 20 times a day. Six different railroads plus the trolley line shared access to the bridge.

By 1904, the third Long Bridge was built about 150 feet upstream from the prior bridge (or 50 feet upstream from the original alignment). The bridge had a total length of 2,529 feet including a center swing span measuring 280 feet providing two 100 foot wide navigation channels. Of the 11 fixed spans, 10 were recycled from a bridge in Trenton, NJ, dismantled and shipped to D.C. for reuse at the Long Bridge site. In 1906, the construction of a new highway bridge (the 14th Street Bridge) 500 feet upstream of the railroad bridge allowed the trolley tracks to be placed on this new crossing until the opening of the Memorial Bridge in 1932. Sometime after 1906, the 1863 vintage Long Bridge was demolished. The 1906 vintage 14th Street Bridge was replaced with a fixed span in 1950, negating the utility of the opening span of its newer parallel northbound structure.

The Long Bridge and 14th Street Bridge southernmost spans were washed out between 1929 and 1932, and the spans were replaced with fill as part of the George Washington Parkway project. Between 1934 and 1935, the

Pennsylvania Railroad added electrified catenaries to the Long Bridge. Train electrification remained in use until the early 1960s. The unused catenary still remains in place today. In 1942, the fixed truss spans were removed, new piers were added to split the old truss spans in half, and the current girder spans were added. With the new load capacity, the bridge rating increased to E65 from E60, a reasonable design loading and considered appropriate for the time. (Note: The modern railroad loading rating is designated as E80, a proportional increase over E65 of 80 / 65, or a 23% increase). A new northbound only 14th Street bridge (the Rochambeau Bridge) opened in 1950 with a bascule span crossing the navigation channel.

The last known opening of the Long Bridge was March 3, 1969 when the bridge needed to open one last time to allow for construction equipment from the demolition of the 1906 14th Street Bridge to be floated down river. In the 1970's, due to vandalism, the operator house was removed from the bridge. In 1999, the bridge ownership was transferred from Conrail to the present owner CSX Transportation, Inc. Table 1 shows the historical ownership trail of Long Bridge.

Table 1 – Ownership of Long Bridge

Year	Owner of Long Bridge
1870	Federal government ceded control of Long Bridge to the Pennsylvania Railroad (Penn RR)
1918	Penn RR officially became owner after 50 years of control
1968	Combine ownership with the merger of New York Central and Penn RR (formally became Conrail in 1976)
1999	CSX Transportation, Inc. acquired ownership

The existing Long Bridge is comprised of multiple low level spans and a double span through truss Swing Bridge. Immediately downstream of the existing structure there are submersed timber piles and partial piers where prior Long Bridge alignments were constructed. The DC Chapter of the National Railway Historical Society (NRHS) provides a history of the Long Bridge, an online version of the history can be found at: http://www.dcnrhs.org/learn/washington-d-c-railroad-history/history-of-the-long-bridge.

1.2 Configuration and Layout of Current Bridge

The current Long Bridge is comprised of 22 through girder spans and a double span swing truss for a total of 24 spans over the Potomac River. It contains elements of the 1904 bridge (the swing span and twelve piers) and of the 1942 bridge (the girder spans and eleven piers). Long Bridge carries two tracks with a width of 36'-6" (measured between the centerline of the girders), but narrows down to 28'-8" at the swing trusses. There is no reserve width to add additional tracks. The vertical clearance is limited to 21' at the swing trusses (measured from the top of the track to the bottom of lateral bracing). Figure 1 shows several of the through girder "approach spans" as well as the main swing span truss over the navigation channel.

Figure 1 – Existing Through Girder and Truss Swing Spans

The through girder spans vary from 85 – 108 feet in length while the truss span measures 280 feet in total length and provides two 100 foot wide navigation channels. Note that there is an additional two span bridge that crosses the tidal basin between Potomac Island and the District of Columbia as shown in Figure 2.

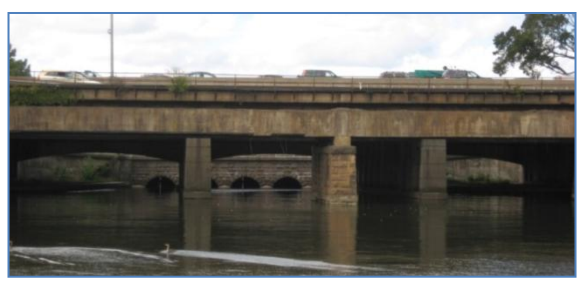

Figure 2 - Tidal Basin Spans

This additional two span bridge is included in the existing conditions assessment. The plan and elevation of the Long Bridge over the Potomac River are shown in Figures 3 and 4, respectively.

Figure 3 – Long Bridge Plan View

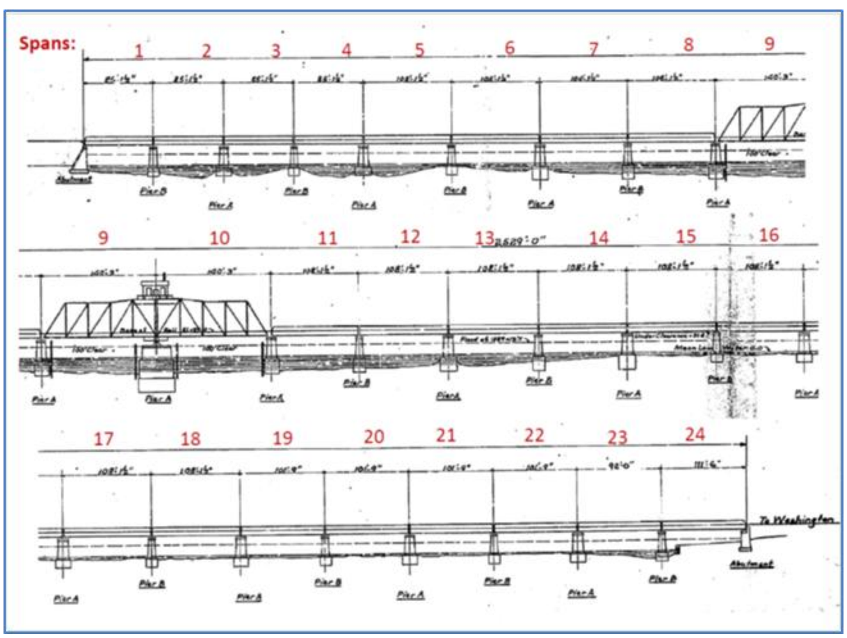

Figure 4 – Long Bridge Elevation

1.3 Through Girder Spans

The through girder spans on Long Bridge are typical of railroad bridge construction. A through girder type bridge is used when the objective is to minimize the bridge clearance. The use of through girder bridge and minimizing bridge clearance is advantageous on the Potomac River to maintain the view shed above the bridge to the monumental core of the District of Columbia. As opposed to a typical highway bridge where the main supporting beams are underneath the riding surface, a through girder supports the riding surface from the bottom of the beams and the traffic passes through the bridge. The majority of the beams are located above and adjacent to the traffic. This results in the structure remaining largely above and outside the navigation window.

The through girder spans consist of two large steel plate girders fabricated by riveting together plates and angle steel components. Web stiffeners reinforce the girders, and a system of lateral and cross braces tie the girders together and provide stability. The tracks are supported by a floor system that is located near the bottom flange of the girders. The floor system is made up of floor beams and stringers. The bridge has an open deck, which means that no floor other than the timber ties is provided. This type of framing allows corrosion to occur more rapidly and frequently due to the ability of rain and snow to corrode the exposed steel elements.

The railroad timber ties in an open floor system rest on the stringers, and these in turn are supported by the floor beams. The floor beams span in the transverse direction and are attached to the girders, thus completing the transfer of load from the ties, to the stringers, to the floorbeams, to the girders, and finally to the piers.

The type and grade of the steel are not known. However, based on the 2011 American Association of State Highway and Transportation Officials (AASHTO) Guidelines for Bridge Evaluation and Load Ratings – 2nd Edition, the yield strength of the steel is 33 ksi based on the year of construction for a typical through girder span.

Figure 5 shows the cross section view of a typical through plate girder span.

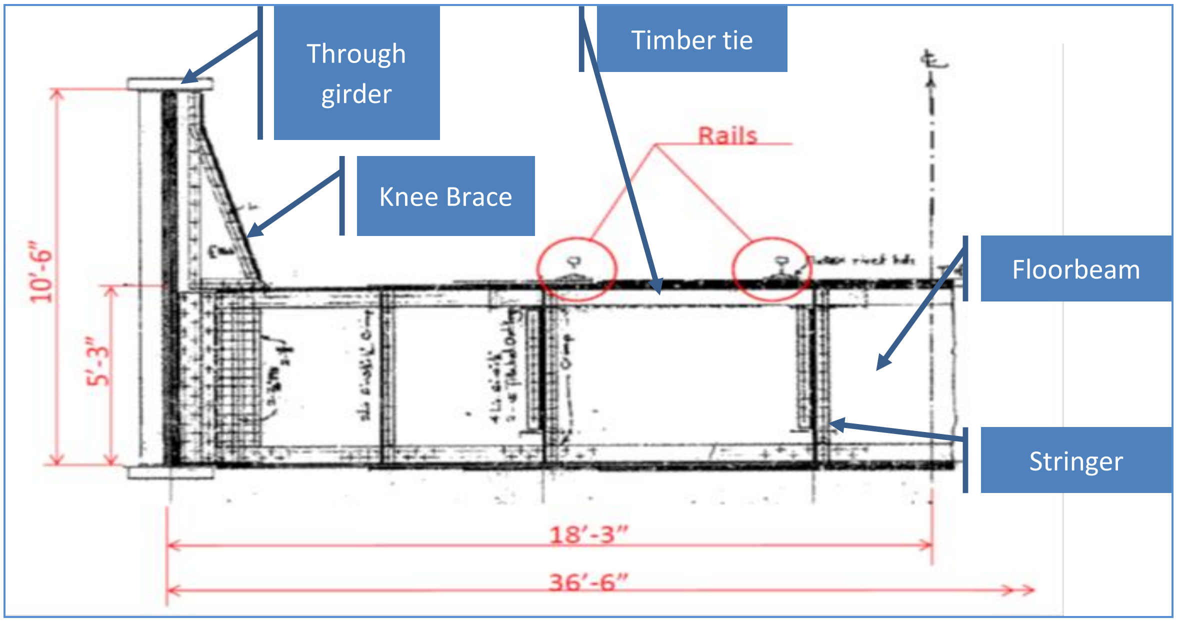

Figure 5 – Through Girder Span Profile, Soffit and Cross Section

1.4 Swing Truss Spans

The Long Bridge swing spans utilize two through trusses as the primary members of the superstructure. The through trusses are constructed of steel. The individual components of the trusses are sections built up from plates and angled steel. Rivets are used to connect the truss components. A truss is simply an assembly of triangular steel panels connected together at the intersection of the members and sized according to the span and loading demands. The perimeter members of a truss consist of a top chord, bottom chord, and end posts. The interior members of the truss that completes the triangular construction consist of diagonals, intermediate posts, and hangers. These members are connected to each other with gusset plates. This connection happens at what are called the panel points of the truss.

Like the through girder spans, the track is supported by a floor system, made up of floor beams and stringers. The track rests on ties and then the stringers; the stringers are framed into the floor beams. The floor beams span laterally and are attached to the trusses at panel points. The truss is laterally braced by sway bracing, top laterals, and bottom laterals.

The type and grade of the steel is not known. However, based on the 2011 AASHTO Guidelines for Bridge Evaluation and Load Ratings - 2nd Edition, the yield strength of the steel is 26 ksi based on the year of construction for a typical truss span.

Figure 6 below shows the profile and soffit of the Long Bridge through truss span.

Figure 6 – Through Truss Span and Soffit

The double span swing truss is designed to be supported solely on the pier at its center when the end supports have been released. It can be thought of as a balanced seesaw as it is opening and closing. It is equipped to be turned in a horizontal plane once it is released from the end supports in order to open the navigable waterway. When closed in the normal traffic position, lifts are inserted under the tips of the cantilevers, supporting the span at the center pier and resting on two end piers.

1.5 Piers of Trough Girder Spans

In 1942, the 1904 truss spans, except for the swing span, were each replaced with two shallower through girder spans by adding a new pier in the middle of the original spans. Consequently, the piers supporting the through girders spans are divided into two types: (1) those built in 1904 as shown in Figures 7 and 8 original pier drawings and (2) those built in 1942 as shown in Figures 9 and 10 original pier drawings.

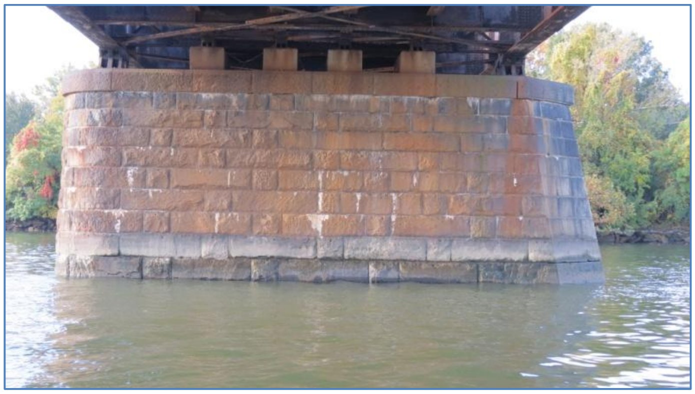

Figure 7 – 1904 Through Girder Span Pier

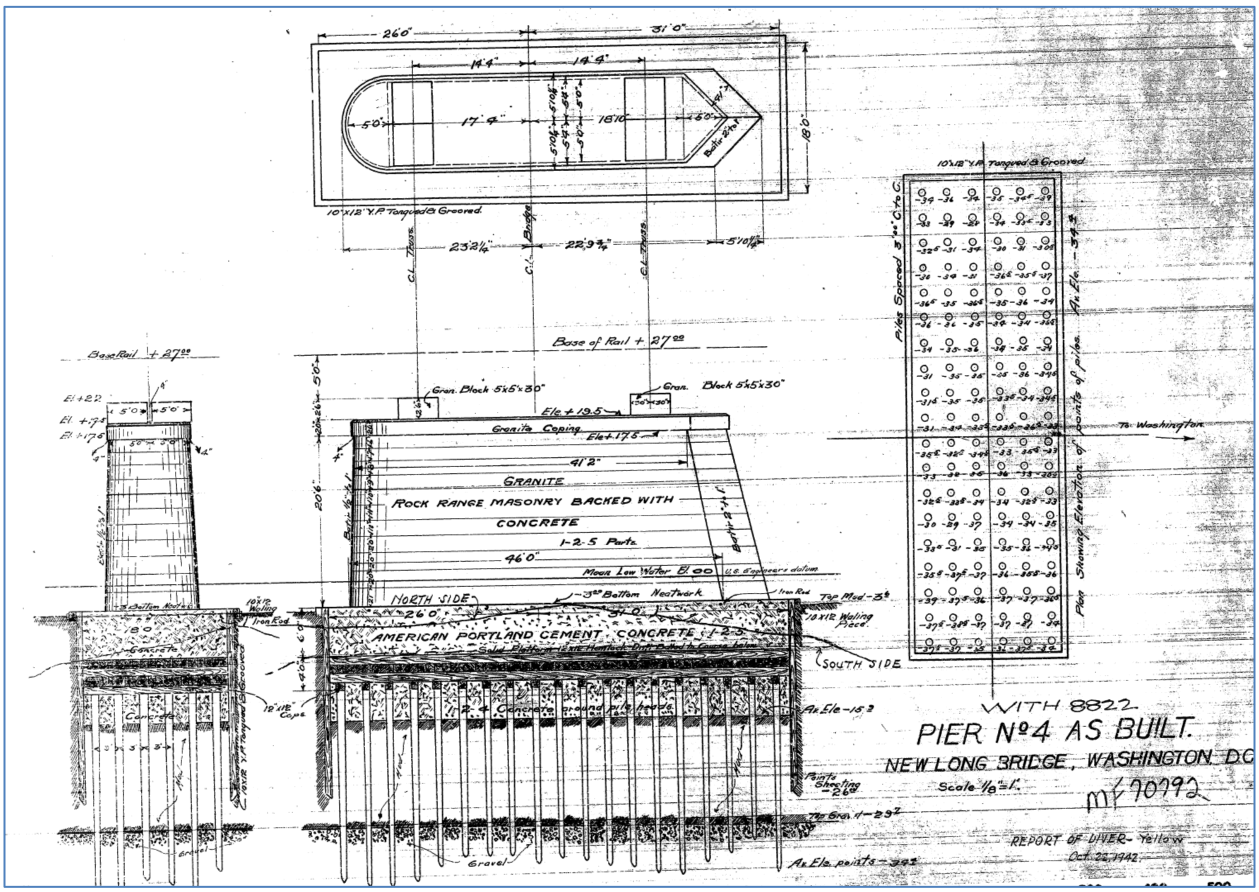

Figure 8 – 1904 Through Girder Span Pier Design Plan

Figure 9 – 1942 Through Girder Span Pier

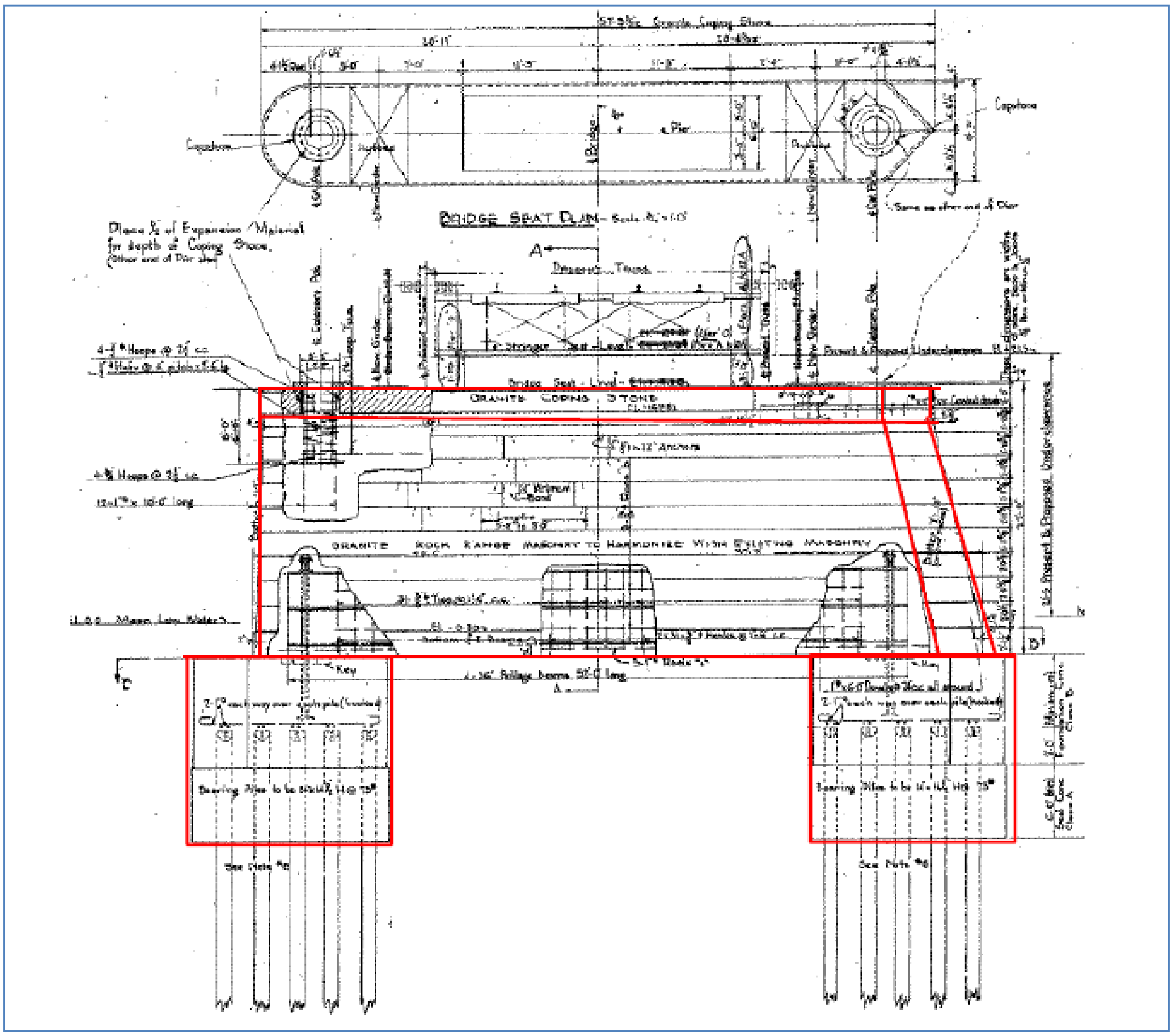

Figure 10 – 1942 Through Girder Span Pier Design Plan

Both types of piers were constructed of concrete with a facing of cut stones. Stone masonry facing presents a pleasing appearance and offers good resistance to the abrasion of the flowing river and protection against impact from floating debris. The stone facing is tied to the concrete by the use of steel anchor rods.

The piers are supported on piles that are below mud-level. Steel bearing piles are used in the 1942 piers and timber piles in the 1904 piers.

1.6 Piers of Swing Truss Spans

The piers of the two swing truss spans built in 1904 were also constructed of concrete with stone masonry facing as shown in Figures 11 (middle pier) and 13 (end pier). The piers are supported on a solid concrete filled caisson, a deep foundation type, common to river construction when high load carrying capacity is required. The actual concrete strength is unknown, but the concrete is designated as *Portland Cement Concrete* in the design plans with 1-2-5 mix ratios. For this type of mix ratio, the strength is approximately 3,000 psi. The caisson extends to a hard layer, about 40 feet below the water surface, as shown in Figure 12 for the middle pier and Figure 14 for the end pier.

Figure 11 – 1904 Swing Truss Middle Pier

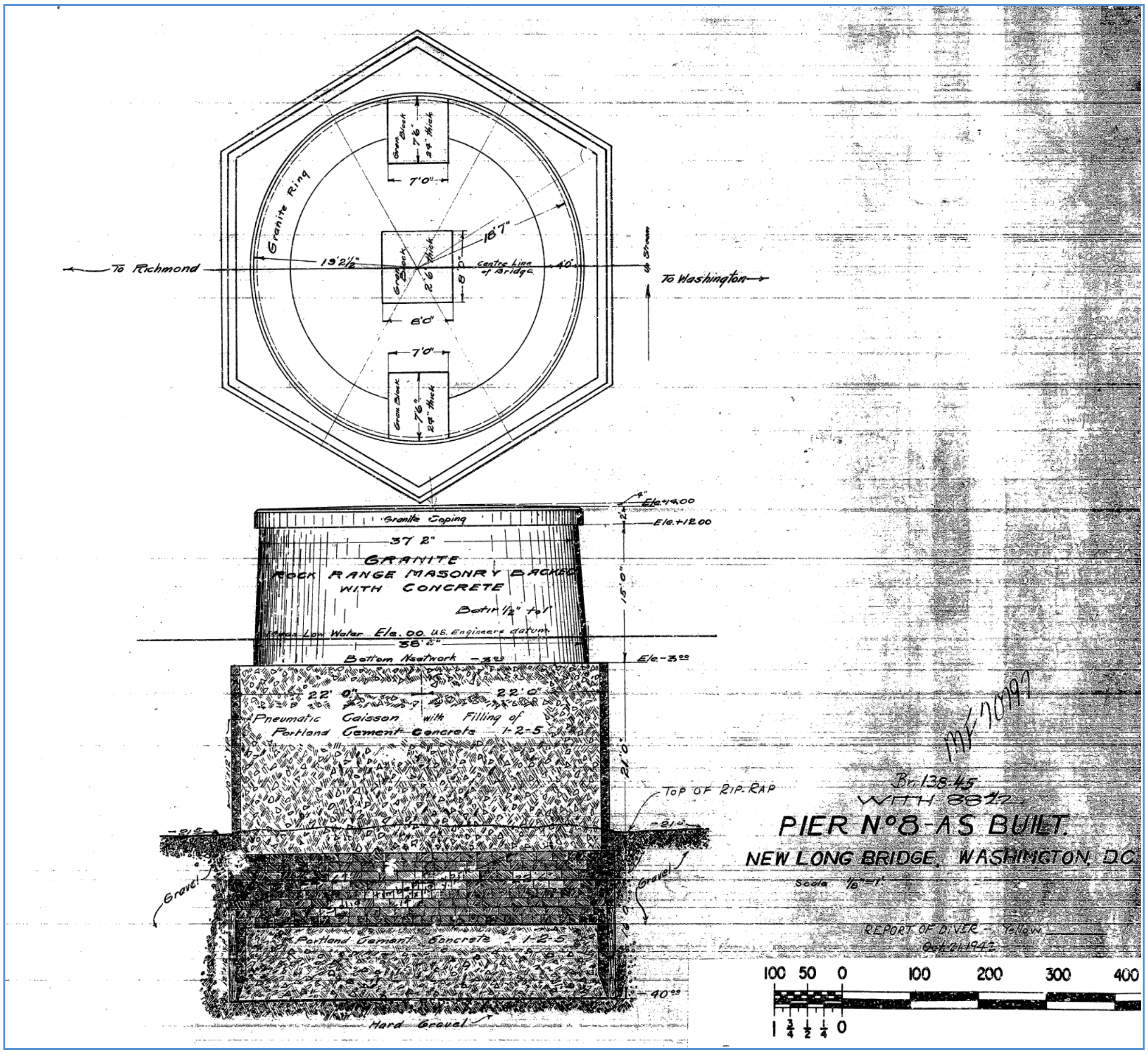

Figure 12 – 1904 Swing Truss Middle Pier Design Plan

Figure 13 – 1904 Swing Truss End Pier

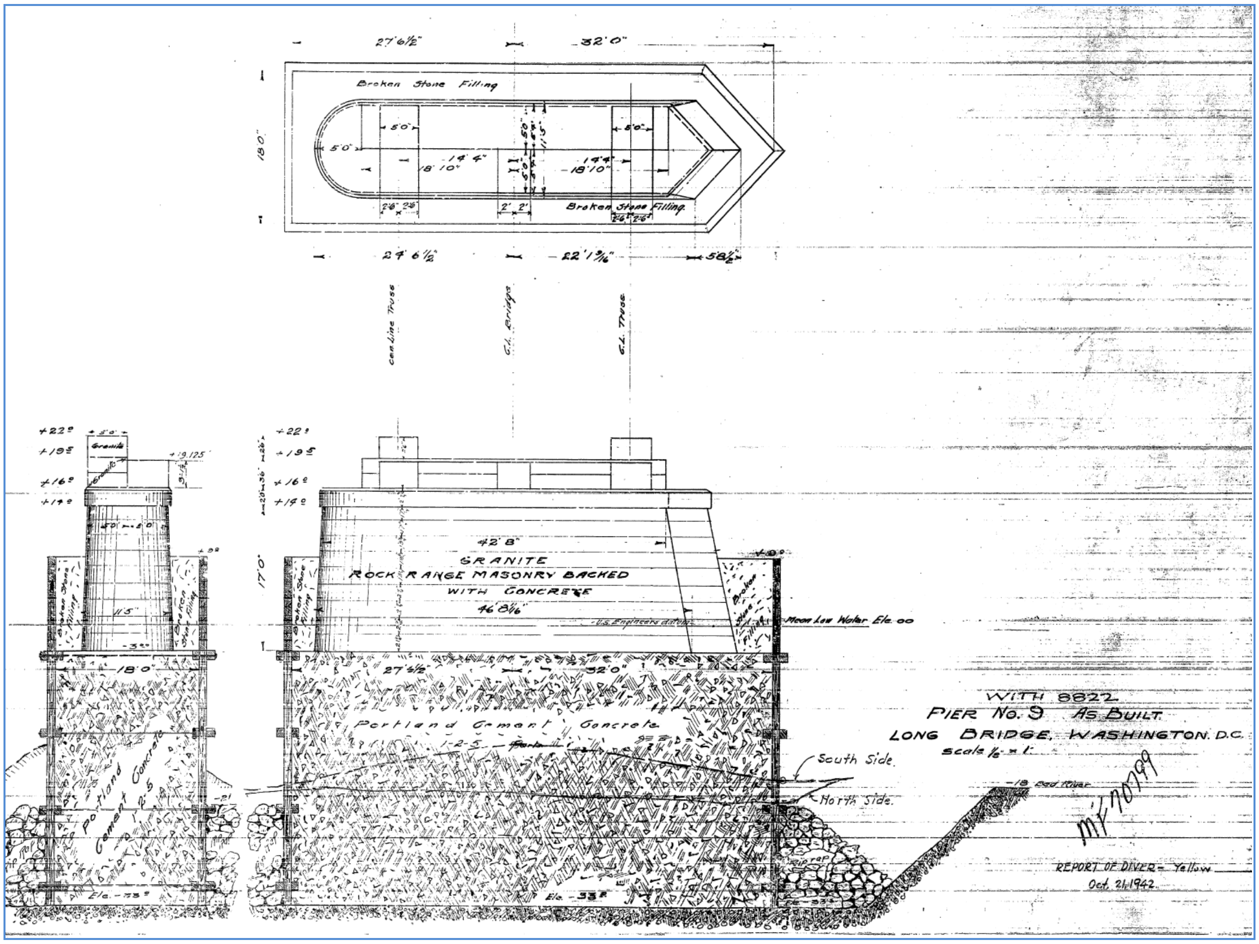

Figure 14 – 1904 Swing Truss End Pier Design Plan

1.7 Potomac River Hydrology

Washington, D.C. (District of Columbia) is located within the Chesapeake Bay drainage basin on the dividing line between the Piedmont and Coastal province. The topography within the District of Columbia ranges in elevation from sea level along the tidal portions of the Anacostia and Potomac Rivers to an elevation as high as 414 feet North American Vertical Datum of 1988 (NAVD88) at Tenleytown. Interstream ridges are highest in the part of the Piedmont within the northwest part of the city. These ridges descend gradually to the coastal plains to the south and east, where elevations rarely exceed 230 feet NAVD88. Average annual precipitation in the District of Columbia is about 43 inches with precipitation fairly well distributed throughout the year. Figure 15 shows the United States Geological Survey (USGS) hydrology map.

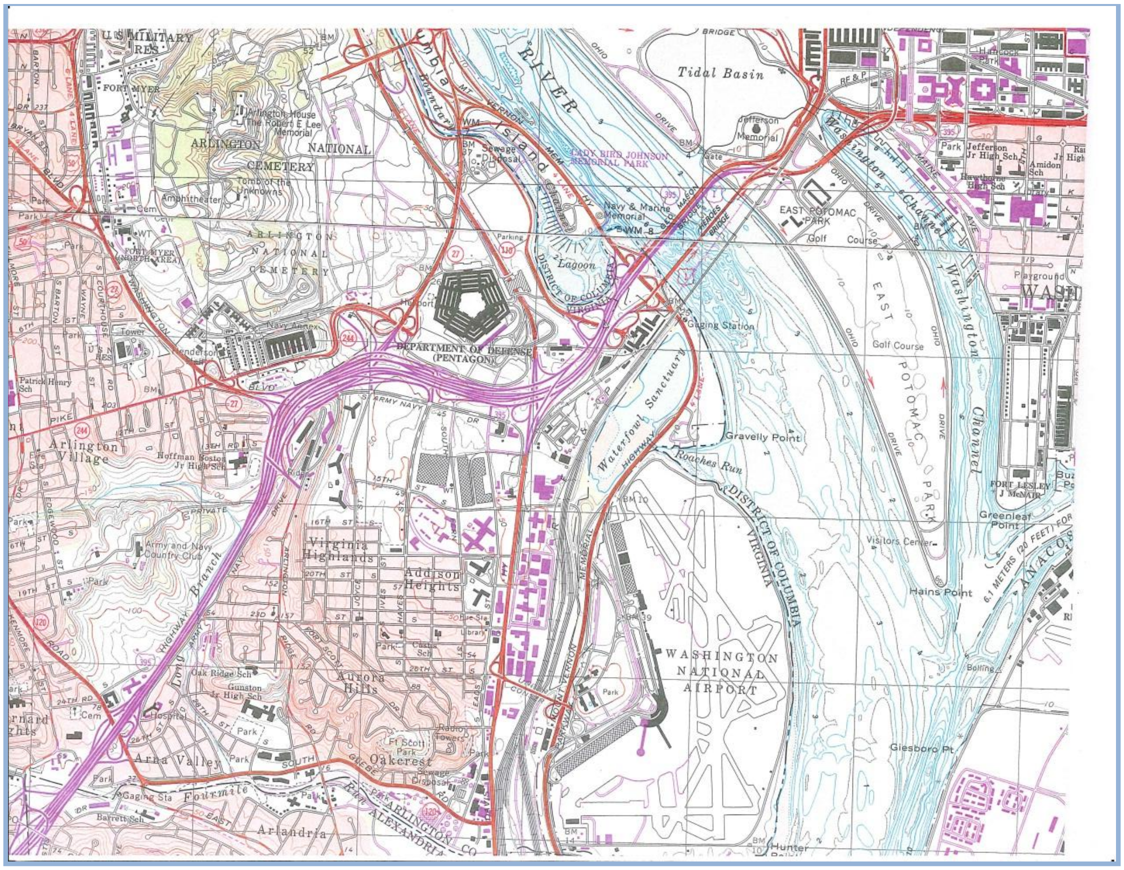

Figure 15 – USGS Hydrology Map

Water surface elevations on the Potomac reflect both riverine and tidal conditions. Tidal influences from the Chesapeake Bay, along the Potomac River, extend from the confluence with the Bay upstream to approximately 3,000 feet downstream of Long Bridge in the District, as reflected in the effective Flood Insurance Study (FIS) for the District of Columbia, Washington, D.C., dated September 27, 2010. According to the FIS, the flood frequency analysis of annual peak discharge for the riverine portion of the Potomac River is based on the USGS gage near Little Falls Pumping Station (USGS Station No. 01646500). The drainage area for the Potomac at this gage is 11,560 square miles. The peak discharges for various flood events associated with riverine conditions are listed in Table 2.

Within the tidal influenced portion of the Potomac River, the flood elevations as reported in the FIS are based on a stage-frequency analysis of water surface

Table 2 - Peak Discharges for Potomac River at Little Falls

Percent Chance Annual Exceedance	Recurrence Interval	Discharge (cfs)
10%	10	240,000
2%	50	395,000
1%	100	475,000
0.2%	500	698,000

elevations recorded at the National Ocean Service (NOS) Gage No. 8594900 located at Haines Point as shown in Table 3.

Table 3 - Water Surface Elevations for Tidal Influenced Portion of Potomac River at Haines Point

Percent Chance Annual Exceedance	Recurrence Interval	Water Surface Elevation (feet NAVD88)
10%	10	5.8
2%	50	8.9
1%	100	10.5
0.2%	500	14.7

1.8 Geological Structure of the Potomac River

The structure of the Potomac River bed is comprised of two layers of soil overlying rock in descending order. Review of historical geologic surveys and as-built drawings from the construction of the Long Bridge defines the composition of each of these three layers.

Based upon the soundings shown on the construction as-builts, the uppermost soil is composed of gravel, sand, silt, and clay as well as a stratum referred to as "dark mud," which is likely a very soft silt or clay. These soils have been identified as alluvium and artificial rock and can be found in the first 1 to 40 feet from the top of the riverbed. Geologic investigators historically have defined these soils as shown in Table 4.

Table 4 - Alluvium and Fill Soils

Source	River Bed Description
Fleming et al. (1994)	Along the edges of the river is primarily artificial fill. Within the river itself, the soil is composed of gray to gray-brown gravel, sand, silt, and clay derived from upgradient terrace, colluvium, saprolite, and fresh crystalline rock deposits – referred to as the Holocene Age Q1 Formation.
Froelich & Hack (1975)	This stratum is referred to as alluvial gravel, sand, silt, and clay with thicknesses ranging from a veneer to 25 feet or more, also intermixed with artificial fill, mainly river dredgings, along edges of the river.
Johnston (1958)	Johnston refers to this material as the Pamlico Formation and Recent alluvium, described as fine sandy loams, sands, and clays, and to a limited extent, gravels.

The second layer underlying the alluvium soils are the Coastal Plain sediments of the Cretaceous Age Potomac Group, including the Patuxent Formation, Arundel Clay, and Patapsco Formation. The three layers are typically defined by two units as shown in Table 5.

Table 5 - Potomac River Soil Layers

Soil Layer	
Upper Soil Layer – Patapsco Formation and Arundel Clay	Comprised of silty and sandy clays with minor amounts of sand and gravel. The clay is mainly mottled red and green, or gray to black and locally carbonaceous.
Lower Soil Layer – Patuxent Formation	Consists primarily of fluviatile, channel-fill, sand and gravel facies, with local lenticular bodies of silt and clay.
Rock Layer	

The soundings on the construction as-builts indicate primarily sand and gravel sequences, followed by hard white and red clay, and soft, micaceous rock (saprolite). According to Fleming et al. (1994), the soil formations are undifferentiated in the Washington West quadrangle area. These sedimentary layers can be found 40 to 100 feet from the top of the riverbed.

Underlying the Potomac Group sediments are the Piedmont basement rock formations as defined in Table 6.

Table 6 - Potomac River Rock Layer

Source	Basement Piedmont Rock Description
Fleming et al. (1994)	Metasedimentary Mather Gorge-Sykesville and Northwest Branch-Laurel motifs. The motifs are intruded by rocks of the Georgetown and Dalecarlia Intrusive Suites, Kensington Tonalite, Clarendon Granite, undifferentiated granitoids, and several quartz bodies.
Froelich & Hack (1975)	Referred to these rocks as metamorphosed igneous and sedimentary rocks of the Wissahicken Formation of the Glenarm Series, including quartzose boulder gneiss, mica schist and impure quartzite. The schists and gneisses are intimately associated with mafic igneous rocks of the Georgetown Complex, and with ultramafic rocks (soapstone, serpentinite, etc.). The metamorphic and mafic rocks are intruded by younger igneous rocks, mainly quartz diorites of the Georgetown and Kensington gneisses.

These igneous and metamorphic rocks are found in north-trending belts which plunge to the south. The crystalline rocks are variably cleaved and foliated, and jointed. The depth to the Piedmont basement rock is approximately 100 to 150 feet below mean sea level.

2. Survey Preparation

2.1 Assessment Objective

The visual assessment or survey is a precursor to a formal inspection that would include a hands-on inspection of each component of the bridge to gather information needed for an in-depth condition and load rating evaluation. The objectives of the visual survey were: (1) to gather as much information as possible without the ability to fully access the bridge superstructure; (2) to evaluate the condition and repair needs of the bridge; and (3) to better understand access issues, conditions, and to understand how these would influence a more formal inspection. The formal inspection can be used to identify additional needed repairs and better facilitate the ability to estimate associated repair costs. The decision is pending to complete a formal inspection of Long Bridge.

2.2 Assessment Procedure

The visual survey was a ride-by survey performed from a boat and was an opportunity to partially assess the structure. A portion of the survey route is shown in Figure 16 and provides an example of the route followed to assess the individual elements of each span and pier. Each span was assessed in a similar fashion. The route typically weaved in and out of each span so that an overview could be conducted. Each span of the bridge was observed from the boat in the following order: (1) upstream elevation, (2) soffit or underside view, (3) right pier, (4) left pier and (5) downstream elevation. Due to the nature of the observation, certain areas of the bridge, such as the deck, interior faces of the girders or truss members, and the underwater piers and foundations could not be observed.

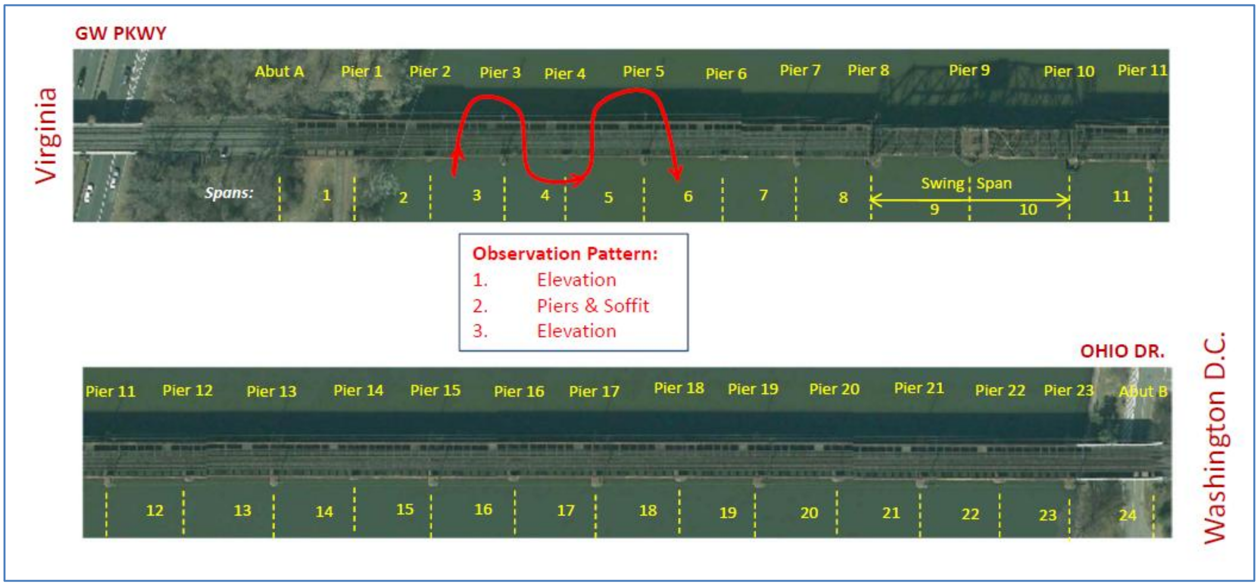

Figure 16 – Boat Survey Route

2.3 Survey Form

A survey form was prepared to document the condition information gathered for each span and to standardize the method of condition rating. The two-page form is shown in Figure 17 on the following two pages. Page 1 of the form documents the condition of each component of the span (girders, floor beam, and stringers) based on the extent of the deficiencies noted in the elements constituting the component. This is followed by the assessment of the overall condition of the span (Page 2).

LONG BRIDGE INSPECTION FORM (1 OF 2)

Date: ____________________

Team Leader: ______________

Team Member: ____________

Team Member: ____________

SPAN NO. : ____________

Weather: _________________

Temp.: ____________________

Total Hours: _______________

ITEM 58 DECK

Condition Rating: –

	Condition Rating	DEF
1. Structural Condition	X	X
2. Ballast		
3. Ties		
4. Deck Joints		
5. Walkways		
6. Drainage		
7. Fire Protection		
8. Handrails		
9. Utilities		
10. Approach Settlement		
11.		

APPROACHES

		DEF
a. Appr. rail condition		
b. Appr. Railway Settlement		
c. Appr. Sidewalk Settlement		
d.		

OVERHEAD SIGNS (Attached to bridge) (Y/N) ☐

		DEF
a. Condition of Welds		
b. Condition of Bolts		
c. Condition of Signs		

ITEM 59 SUPERSTRUCTURE

Condition Rating: –

	Condition Rating	DEF
1. Girders, Beams or Trusses		
a. Top Flange or Chord		
b. Bottom Flange or Chord		
c. Web or Diagonals		
d. Truss Joints		
e. Bearing Stiffeners		
f. Cross Frames, Diaphragms		
g. Knee Braces		
h. Pins		
i. Rivets or Bolts		
j. Welds		
k. Conn Plt's, Gussets & Angles		
l. Top Lateral Bracing		
m. Bottom Lateral Bracing		
n. Sway Frames		
o. Portals		
p. Hangers		
q. Bearings		
r.		
2. Floor Beams		
a. Top Flanges		
b. Bottom Flanges		
c. Webs		
d. Stiffeners		
e. Rivets or Bolts		
f. Welds		
g. Connections		
h.		
3. Stringers		
a. Top Flanges		
b. Bottom Flanges		
c. Webs		
d. Stiffeners		
e. Rivets or Bolts		
f. Welds		
g. Connections		
h. Diaphragms		
i.		
4. Superstructure (General)		
a. Paint		
b. Action Under Trains		
c. Collision Damage		
d. Member Alignment		
e.		

ITEM 60 SUBSTRUCTURE

	Condition Rating	DEF
1. Abutments		
a. Stems		
b. Wingwalls		
c. Backwalls		
d. Pedestals		
e. Bridge Seat		
f. Pointing		
g. Footing		
h. Erosion		
i. Settlement		
j.		
2. Piers or Bents		
a. Piles		
b. Footings		
c. Stem or Columns		
d. Cap Beams		
e. Top of Stem or Cap		
f. Pedestals		
g. Diagonal Bracing		
h. Fender System		
i. Erosion or Scour		
j. Settlement		
k. Pointing		
l.		

UNDERMINING (Y/N) *If YES please explain* ☐

COLLISION DAMAGE: None ( ) Minor ( ) Moderate ( ) Severe ( )

Any Fracture Critical Member : (Y/N) ☐

Any Cracks in Tension Plates : (Y/N) ☐

Year Painted :

COLLISION DAMAGE: None ( ) Minor ( ) Moderate ( ) Severe ( )

LOAD DEFLECTION: None ( ) Minor ( ) Moderate ( ) Severe ( )

LOAD VIBRATION: None ( ) Minor ( ) Moderate ( ) Severe ( )

X=UNKNOWN N=NOT APPLICABLE H=HIDDEN/INACCESSBLE R=REMOVED

Figure 17 – Survey Form*

***Adapted from Massachusetts Bay Transportation Authority Railroad Operations**

LONG BRIDGE INSPECTION FORM (2 OF 2)

ITEM 61
CHANNEL & CHANNEL PROTECTION

		DEF
1. Channel Scour		
2. Embankment Erosion		
3. Drift		
4. Channel Alignment		
5. Vegetation		
6. Rip-Rap		
7. Silt		
8. Debris in Channel		
9.		

ACCESSIBILITY: (Y/N/P)

	Needed	Used
Lift Bucket		
Ladder		
Boat		
Wader		
Inspector 50		
Rigging		
Staging		
Traffic Control		
RR Flagger		
Police		
Other		

CONDITION RATING GUIDE (for Items 58, 59, 60)

	CODE	CONDITION	DEFECTS
	N	NOT APPLICABLE	
G	9	EXCELLENT	Excellent condition.
G	8	VERY GOOD	No problem noted.
G	7	GOOD	Some minor problems.
F	6	SATISFACTORY	Structural elements show some minor deterioration.
F	5	FAIR	All primary structural elements are sound but may have minor section loss, cracking, spalling or scour.
P	4	POOR	Advanced section loss, deterioration, spalling or scour.
P	3	SERIOUS	Loss of section, deterioration, spalling or scour have seriously affected primary structural components. Local failures are possible. Fatigue cracks in steel or shear cracks in concrete may be present.
C	2	CRITICAL	Advanced deterioration of primary structural elements. Fatigue cracks in steel or shear cracks in concrete may be present or scour may have removed substructure support. Unless closely monitored it may be necessary to close the bridge until corrective action is taken.
C	1	"IMMINENT" FAILURE	Major deterioration or section loss present in critical structural components or obvious vertical or horizontal movement affecting structure stability. Bridge is closed to traffic but corrective action may put it back in light service.
	0	FAILED	Out of service - beyond corrective action.

DEFICIENCY REPORTING GUIDE

DEFICIENCY: A defect in a structure that requires corrective action.

CATEGORIES OF DEFICIENCIES:

M= Minor Deficiency - Deficiencies which are minor in nature, generally do not impact the structural integrity of the bridge and could easily be repaired. Examples include but are not limited to: Spalled concrete, Minor pot holes, Minor corrosion to steel, Minor scouring, Clogged drainage, etc.

S= Severe/Major Deficiency - Deficiencies which are more extensive in nature and need more planning and effort to repair. Examples include but are not limited to: Moderate to major deterioration in concrete, Exposed and corroding rebars, Considerable settlement, Considerable scouring or undermining, Moderate to extensive corrosion to structural steel with measurable loss of section, etc.

C-S= Critical-Structural Deficiency - A deficiency in a structural element of a bridge that poses an extreme unsafe condition due to the failure or imminent failure of the element which will affect the structural integrity of the bridge.

C-H= Critical-Hazard Deficiency - A deficiency in a component or element of a bridge that poses an extreme hazard or unsafe condition to the public, but does not impair the structural integrity of the bridge. Examples include but are not limited to: Loose concrete hanging down over traffic or pedestrians, A hole in a sidewalk that may cause injuries to pedestrians, Missing section of bridge railing, etc.

Figure 17 – Survey Form (continued)*

***Adapted from Massachusetts Bay Transportation Authority Railroad Operations**

As an example, for the girder component of a span, the deficiencies in flanges, web, and stiffeners determine the girder condition. Using the *Deficiency Reporting Guide* in the form, an element is rated by applying one of the following categories of deficiency for each component in the girder:

- None (Blank)
- Minor Deficiency (M)
- Severe Deficiency (S)
- Critical- Structural Deficiency (C-S)
- Critical – hazard Deficiency (C-H)

After all elements of the girder have been rated, the overall girder condition is rated as:

- Good (G)
- Fair (F)
- Poor (P)
- Critical (C)

Once each section of the bridge span have been rated using the same procedure for other span components (floor beams, stringers), the overall span condition is rated from 9 to 0, using the *Condition Rating Guide* on the form, with 9 representing an excellent condition and 0 representing a failed condition. This is systematically completed by assigning the following numerical ratings to the component condition:

- Good (9,8,7)
- Fair (6,5)
- Poor (4,3)
- Critical (2,1)

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3. Survey Results

The condition rating of each of the 24 spans of the Long Bridge over the Potomac River and the two span bridge that crosses the Tidal Basin are tabulated in Table 7 for both superstructure and substructure. For the superstructures, the majority of spans received a rating of 5 (Fair) with a rating of 6 (Satisfactory) in a few locations. The one exception was Span 20 with a rating of 3 (Serious) due to a cracking in the web of a stringer. For the substructures, the majority of spans rated a 7 (Good) with a rating of 6 (Satisfactory) in 3 locations.

Table 7 – Condition Ratings of Each Span

Span No.	Span Structure Type	Superstructure Condition Rating	Substructure Condition Rating
1	Through Girder	5	6
2	Through Girder	6	7
3	Through Girder	6	7
4	Through Girder	5	6
5	Through Girder	6	7
6	Through Girder	6	7
7	Through Girder	6	7
8	Through Girder	5	7
9	Swing Truss	5	7
10	Swing Truss	5	7
11	Through Girder	6	7
12	Through Girder	5	6
13	Through Girder	5	7
14	Through Girder	5	7
15	Through Girder	5	7
16	Through Girder	5	7
17	Through Girder	5	7
18	Through Girder	5	7
19	Through Girder	5	7
20	Through Girder	3	7
21	Through Girder	5	7
22	Through Girder	5	7
23	Through Girder	5	7
24	Through Girder	5	7

Specific recommendations that were derived from the visual survey are shown in Table 8.

Table 8 – Short Term Recommendations

Span	Superstructure	Substructure
1	Repair the bottom flange of corroded Stringer.	**Abutment A:** • Remove Vegetation growing around each end. • Repoint mortar joint between the masonry. • Epoxy inject the crack at the top stone. **Pier 1:** Remove a large tree growing at the upstream end of the pier.
2	Remove vegetation and large trees growing on the upstream side.	
3		
4	Repair the gusset plate with section loss.	**Pier 4:** Repoint mortar joint.
5		
6		
7		**Pier 6:** Repair the spalled pedestal under stringer exhibiting exposed reinforcing steel.
8	Repair the gusset plate with holes in it. Repair the consumed flanges of the stringers due to pack rust with the gusset.	
Swing Truss 9	Repair the bottom flange of the stringers that exhibit section loss. Repair the areas of pack rust which involves section loss in the turntable member. Repair corrosion on the bottom flange of the floorbeams.	**Pier 9:** Repoint/Repair top several layers of masonry, exhibiting cracking and spalling.

Span	Superstructure	Substructure
Swing Truss 10	Repair locations on the bottom flange of the stringers that exhibit corrosion & section loss. Repair the areas of pack rust which involves section loss in the turntable member. Repair the corroded web of the floorbeams. Also, repair pinholes through the web. Repair corrosion, steel flaking and section loss on the bottom flange of the floorbeams.	**Pier 9:** Repoint/Repair top several layers of masonry, exhibiting cracking and spalling.
11		**Pier 10:** Repoint mortar joint between the top stones. **Pier 11:** Repoint the mortar joint between stones.
12	Repair the corroded web of the floorbeams.	**Pier 11:** Epoxy inject the crack in the pedestal. **Pier 12:** Repair pedestals over showing spalling & exposed reinforcing steel.
13		**Pier 13:** Epoxy inject the cracked top stone.
14	Repair the bottom flange of the stringers with pack rust with the gusset.	
15	Repair the corroded web of the floorbeams.	
16	Repair the corroded web of the floorbeams.	**Pier 16:** Repair the spalled pedestal with exposed reinforcing steel.
17	Repair the corroded web of the floorbeams.	**Pier 16:** Repair the pedestal exhibiting spalling and exposed reinforcing steel.
18	Repair the corroded web of the floorbeams.	**Pier 18:** Repair the pedestal under with spalling and exposed reinforcing steel.
19		
22	Replace the corroded rivets.	**Pier 22:** Repair Pedestals that are spalling and have exposed reinforcing steel.
23	Repair pitting and section loss in the bottom flange of the cap beam. Replace the corroded rivets	**Pier 22:** Repair Pedestal with spalls and exposed corroded reinforcing steel.

Span	Superstructure	Substructure
24	Straighten the bent bottom lateral brace between Floorbeam 6 and 7. Repair the bottom flange of Stringers with heavy corrosion.	**Abutment B:** Grout inject the 1” wingwall/abutment separation. Epoxy inject the wide vertical crack at the wingwall.
Tidal Basin Span 1	Repair spalling at the bottom of the concrete deck. Repair the impact damage to the reinforced concrete fascia protection beams, which carry no load.	
Tidal Basin 2	Repair spalling at the bottom of the concrete deck. Repair the impact damage to the reinforced concrete fascia protection beam, which carry no load.	**Abutment B:** Grout inject the wide crack in the wingwall.

Some of the typical deficiencies as well as the more serious crack found in Span 20 are shown in Figure 18 on the following page**.** Detailed information included in the completed visual survey forms Long Bridge are provided in **Appendix A.** Field notes and photos that accompany the visual survey are provided in **Appendix B**.

Figure 18 – Typical Deficiencies Noted During Visual Survey

The overall condition ratings for the through girder spans of the bridge were determined to be 5 (Fair) for the superstructure and 7 (Good) for the substructure as shown Table 9. Substructure and superstructure ratings for the two swing truss spans were also determined to be 5 (Fair) and 7 (Good), respectively.

Table 9 – Condition of Long Bridge & Tidal Basin Bridge

Structure	Overall Superstructure Condition Rating	Overall Substructure Condition Rating	Remarks
Long Bridge (22 Through Girder Spans)	**5** FAIR	**7** GOOD	**Based on Observation from a Distance, only**
Long Bridge (2 Swing Truss Spans)	**5** FAIR	**7** GOOD	
Tidal Basin Bridge (2 Multi-Girder Spans)	**5** FAIR	**6** SATISFACTORY	

The visual survey also included an assessment of the Tidal Basin Bridge, located 1,000 feet to the east of Long Bridge on the same alignment. The condition was determined to be 5 (fair) for superstructure and 6 (Satisfactory) for substructure. The substructure rating of the Tidal Basin Bridge represents the average condition of the overall substructure of this 2-span bridge with two abutments and one pier. Two out of the three substructure units of this bridge were in good condition. For Span 1, the substructure is rated 7 (Good) while Span 2, with cracked wingwall, the rating is 5 (Fair), which results in the combined substructure condition of the piers and the abutment as 6 (Satisfactory).

The rating of the substructure included a wide shear crack observed in the wingwall of the east abutment. Although stone walls will occasionally develop cracks, these commonly occur along joint lines due to movement of the stones relative to each other. The crack at this abutment is more severe and includes multiple stones that have fractured through their cross section. The cause of this distress is unknown but is consistent with a settlement problem at the abutment causing portions of the stones to settle, while others remain in place, resulting in a fracture of the stones themselves.

The wingwall is only one component of an abutment. The condition of the east wingwall is reported as 4 (Poor) with a deficiency designated as 3 (Serious) due to the cracking. The other components of the same abutment are in relatively good condition. The backwall and stem breast wall are in 5 (Fair) condition and the seats are in 7 (Good) condition.

The Tidal Basin Bridge includes a crash protection concrete façade on the exterior of the bridge. The concrete façade is not in good condition with noticeable cracking and exposed rebar. The concrete façade is not a structural element of the bridge.

It is the professional opinion of the survey team, that if a formal inspection is conducted for this bridge that the overall ratings shown in Table 10 for the superstructure would drop one level to 4 (Poor). Based on the observations of corrosion and structural loss during the survey, it is anticipated that additional issues would be identified during a full inspection. Having made a closer examination of the substructure, the rating for the substructure remains 7 (Good).

Table 10 – Likely Condition of Long Bridge if Inspected

Code	Condition	Remarks
9	Excellent	
8	Very Good	
7	Good	Substructure Condition (by "Observation")
6	Satisfactory	
5	Fair	Superstructure Condition (by "Observation")*
4	Poor	↓ ↓ ↓
3	Serious	Monitor Deterioration
2	Critical	Close Facility, or Closely Monitor
1	Imminent Failure	Close Facility, Corrective Action Needed
0	Failed	Out of Service, beyond Corrective Action
*It is likely that the ratings would drop one notch if bridge was "inspected"		

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4. Recommendations

4.1 Short-Term Serviceability and Costs

Based on the results of the visual survey performed for this study, it is recommended that the following short-term repairs for elements identified in Chapter 3 will be needed:

- Steel section loss due to corrosion
- Steel cracking due to fatigue

The nature of these repairs can only be fully determined by a formal inspection, which requires permission to access the bridge from the CSX Corporation. The information obtained from a formal inspection would be sufficient for preparation of plans, specifications and estimates (PS&E) documents for the recommended repair work or recommendations for force account work should CSX desire to perform the work in-house or with on-call contractors. Additional information would be needed on the magnitude and exact location of the deterioration that requires repair. **Appendix C** provides the detailed scope and associated costs for performing a formal bridge inspection.

An additional option to extend the usable life of the bridge and provide improve the aesthetic appearance would be to paint the bridge. This would require surface preparation to remove mill scale, rust, and the existing paint that may increase the chance of failure and peeling of the new coating. Contaminant containment is needed to prevent both lead and other debris generated during surface preparation activities from entering the environment. Typically, up to three coatings of paint are applied to the structure.

Based on the visual survey, it is estimated that the cost of bridge repair for short-term serviceability will be approximately $450,000, as shown in Table 11. This estimate includes a sizeable contingency that is typically set aside for unforeseen issues that arise during repair. In this instance it represents an estimated cost of repair for each of the 22 spans over the Potomac River and is only an approximation.

The cost of painting the spans over the Potomac River is shown in Table 12. The costs associated with the preparation of the bridge surface and application of paint is estimate at $2,100,000. Additionally, the requirements for containment of lead and other hazardous surface materials escalates the cost an estimated 50% for a total of $3,150,000. Projects of this type also require a contingency for unforeseen repair issues, typically 30%, which would add an additional $1,050,000 to the total cost.

Table 11 – Approximate Repair Costs

Type of Repair	Quantity	Unit Cost	Repair Cost
Repair the shear crack in a stringer of Span 20.	1	$20,000	$20,000
Repair pinholes in the web of a floor beam in Span 10.	1	$10,000	$10,000
Assume in the absence of a formal inspection with access to bridge			
Repair deficiencies in each of the remaining 21 through girder spans, and 1 swing truss span resulting from corroded section loss, cracking, pin holes, etc.	22	$15,000	$330,000
		ESTIMATED COST	$360,000
		CONTINGENCIES, 25%	$90,000
		TOTAL ESTIMATED COST	**$450,000**

Table 12 – Painting Cost Estimate for Long Bridge

Segment	Approximate Steel Area per Span (square feet)	No. of Spans	Approximate Total Surface Area (square feet)	Surface Prep & Painting per square foot*	Cost of Surface Prep & Painting	Total Cost of Surface Prep & Painting	Cost of Containment (At 50% of Prep and Painting)	Total Painting Cost**
Through Girder	14,000	22	308,000	$5.00	$1,540,000	$2,100,000	$1,050,000	$3,150,000
Swing Truss	40,000	2	80,000	$7.00	$560,000			

* Cost does not include maintenance of train operations and boat traffic on the Potomac River.
**Cost does not include contingency of unforeseen issues that arise during painting operations.

4.2 Load Capacity/Demand Analysis

The service life of the bridge can best be answered by determining the current and future load capacities and load demands (this methodology assumes some short-term repairs as described above). The load demand of a bridge refers to the vehicle load that the structure will be subjected to while the bridge is in service. The load capacity of a bridge is the overall ability of the bridge to carry the imposed demand. For a satisfactory performance, the capacity must be greater than the demand meaning that the capacity-to-demand ratio greater than 1 (i.e. C/D > 1).

A load capacity/demand analyses was performed for Long Bridge as outlined in the following steps. This analysis follows a process of 10 and 20 year evaluation cycles based upon a capacity-to-demand ratio greater or less than 1. Table 13 illustrates the typical live loads (weights) that rail cars exert upon the rail bed.

Table 13 – Load Capacity/Demand Analysis

Projected Service Life	Capacity-to-Demand Ratio	
10 Years	**< 1** Plan Replacing Bridge	**>1** Determine 20 year Ratio
20 Years	**< 1** 1. Repair, but Plan Replacing in 20 years 2. Plan Replacing Now **Per Life Cycle Cost Analysis**	**>1** NOT LIKELY

Step 1: Determine Load Capacity

A. Determine original capacity using the sections and dimensions per the original bridge plans.

B. Determine the current capacity due to corrosion and fatigue. Assume the average overall steel section loss as 1/8” per current observation. Assume higher and lower limits of section loss as 3/16” and 1/16” and evaluate the sensitivity. Adjust the capacity for the average 1/8” section loss, if needed, per sensitivity.

Step 2: Determine Current and Future Load Demands

A. Estimate load effects using the magnitude and configuration of Cooper E-80, or alternate loading as the demand load, per AREMA standards (Figure 19).

B. Perform load repetitions for projected traffic for 10 and 20 year horizons.

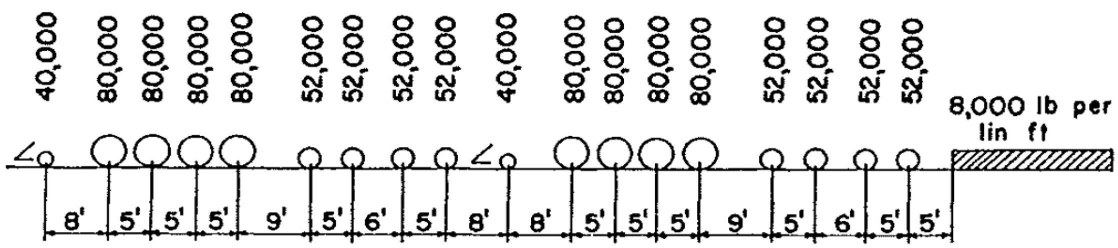

Figure 15-1-2. Cooper E 80 Load

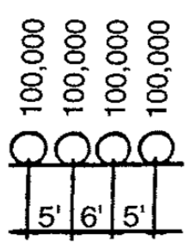

Figure 15-1-3. Alternate Live Load on 4 Axles

Figure 19 – Specified Design Live Load for Railroad Bridges (AREMA)*

***American Railway Engineering and Maintenance-of-Way Association**

<u>Step 3: Determine Capacity-to-Demand Ratio (C/D)</u>

A. Estimate 10 year C/D ratio such that if C/D < 1, plan on replacing bridge immediately or if C/D > 1 assess using 20 year C/D ratio.

B. Estimate 20 year C/D ratio such that if C/D < 1, perform a Life Cycle Cost Analysis to determine the most cost-effective option of either replacing the bridge immediately or replacing the bridge in another 10 years. For the 20 year analysis, it is highly unlikely that the C/D ratio >1, because of the increases load repetitions and age.

Depending upon the year and ratio, it can be determined if replacement of the bridge is necessary. The actual load rating report is provided separately from this document as it is a professional engineer sealed document.

4.3 Detailed Inspection Process

The execution of a detailed bridge inspection would include assessment of the topside superstructure as well as an underwater inspection.

<u>Topside Inspection</u>
The topside inspection would employ visual, physical and advanced inspection techniques to document deficiencies, identify critical deficiencies and recommend repairs to maintain short-term serviceability. Techniques used to perform the topside inspection include: barge with a lift, rail mounted under-bridge inspection unit, rail mounted bucket truck and ladders or scaffolding for bridge structure not over the Potomac River. The inspection will be primarily visual in nature performed up-close for individual elements of the structure. In some instances, more advanced methods may be used for inspection such as (but not limited to) dye penetrant, magnetic particle and ultrasonic testing.

The inspection would pay specific attention to fracture critical members. Fracture critical members are defined as steel members in tension or with a tension element, whose failure would probably result in a portion or full bridge collapse.

Overall management of the inspections will be the responsibility of an inspection manager supported by a quality assurance/quality control manager, team leaders and inspection team members. Personnel in the role of inspection manager and team leader are required to meet the qualifications as listed in the Code of Federal Regulations, Title 23 – Highways, Subpart G, Part 650 – Bridges, Structures and Hydraulics.

The inspection is conducted so that at no time will inspection activities be allowed to interrupt the flow of rail traffic. Scheduling of inspections must work around the schedule of rail traffic operations. A flagman from CSX will be required at all times during the inspection whether inspection activities will directly affect rail traffic or not. Coordination with CSX will take place well in advance of inspection activities so all parties are aware of the requirements and needs of the inspection process. To maximize inspection time, all inspections will be performed at night when rail operations are at a minimum. It is assumed to expect a maximum of only four hours per night of uninterrupted inspection time.

The duration of the inspection is determined by estimating the hours needed to inspect the truss and non-truss spans. For the two truss spans, the inspection of the lower members, the truss and the portion at and above the ballast is estimated at 20 hours. For the 22 non-truss spans, the inspection of the lower members and the portion at and above the ballast is estimated at 55 hours.

Underwater Inspection

The underwater inspection is completed for all 23 submerged piers and one submerged abutment. This is completed by a 3-person dive team consisting of a supervising engineer and two divers. The inspection includes a Level 1 visual / tactile inspection of the entire structure, combined with a Level II detailed inspection with partial cleaning on 10% of the structural elements. Underwater inspection data is collected in accordance with the *National Bridge Inspection Standards.* All data is recorded by the supervising engineer and included in a final inspection report. The duration of the underwater inspection is approximately two weeks.

The Long Bridge detailed inspection will be a month long effort coordinated for concurrent efforts of superstructure and underwater inspection as detailed in **Appendix C**. This inspection would provide a comprehensive assessment of the serviceable life of the Long Bridge and help shape the need for short- and long-term recommendations for bridge enhancement or replacement.

5. Conclusions

A bridge assessment via boat access included a limited visual review of the superstructure and substructure. It is the first step in determining the life and serviceability of a bridge. The objective of the boat survey was to identify any immediately observable critical issues but more so to plan a more complete inspection. Many railroad bridges of this age (from 70 – 110 years in age) continue to provide reliable service to their owners yet modern train weights and the high numbers of cycles of loading have an increasing effect on their deterioration and remaining life. This is due to a number of factors including the conservative nature of railroad bridge design, the high loads they were designed for when steam locomotives were common, and other factors that allow older bridges to remain in service past their expected / projected service life. The Long Bridge is no exception. It certainly has some short-term and long term rehabilitation needs, some of which are defined herein and others of which can only be determined following a more complete inspection. As to its long term viability, that will be determined after load ratings and more detailed inspections are complete.

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