Rehabilitation Evaluation Report

for the

Chestnut Street (S.H. 150) Pedestrian Bridge Over the Colorado River in Bastrop, Texas

Prepared for:

BASTROPTX
Heart of the Lost Pines
Est. 1832

Prepared by:

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1.0 Executive Summary

Kimley-Horn and Associates, Inc. (KHA) has conducted an evaluation of the Chestnut Street (S.H. 150) Pedestrian Bridge over the Colorado River to help quantify repairs and projected costs. Field evaluations were performed between November 5th and November 16th , and on December 5th in 2018. Evaluation methods included visual inspection, photo documentation, measurement and in-situ data collection, non-destructive testing such as sounding and scraping, and underwater diving for the pier foundations in the river channel. Access to the structure included on foot (above and below), by boom lift (upper truss), rope access (floor system below deck), and underwater (pier foundations).

Results of the field inspection were collated, and structure elements were assigned rating numbers in accordance with the National Bridge Inventory (NBI) rating (refer to table below for a description of the ratings for reference).

National Bridge Inventory Rating Information

9	Excellent condition
8	Very good condition – no problems noted
7	Good condition – some minor problems
6	Satisfactory condition – structural elements show some minor deterioration
5	Fair condition – all primary structural elements are sound but may have minor section loss, cracking, spalling, or scour
4	Poor condition – advanced section loss, deterioration, spalling, or scour
3	Serious condition – 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
2	Critical condition – 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
1	“Imminent” failure condition – 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 back in light service
0	Failed condition – out of service – beyond correction action

Based on this rating system, the following elements were identified and rated:

- **Approach Spans: Spans 1-3, 7-21**
  - ***Deck [NBI rating 6]***
  - ***Superstructure Beams [NBI rating 5]***
  - ***Substructure [NBI rating 6]***

- **Main Spans: Spans 4-6**
  - ***Deck [NBI rating 6]***
  - ***Superstructure [Overall NBI rating 3]***
    - **Upper Chord: Satisfactory Condition [NBI 6]**
    - **Verticals: Fair Condition [NBI 5]**
    - **Diagonals: Fair Condition [NBI 5]**
    - **Lower chord: Serious Condition [NBI 3]**
    - **Floor Beams: Fair Condition [NBI 5]**
    - **Stringers: Fair Condition [NBI 5]**
    - **Bearing Assemblies: Poor Condition [NBI 4]**
  - ***Substructure [NBI rating 7]***
- **Overall Structure: Serious [NBI 3].** Due to the current condition of the gusset plates along the lower chord, the overall bridge rating is an NBI 3 because of the fracture-critical nature of these connections. Eighteen gusset plates out of 60, about 30%, have localized areas with 100% section loss. The size of these areas ranges from 1 in^2 up to 24 in^2.

A coating evaluation was performed to determine the heavy metals content, including lead content of the coating on the bridge. Eight paint samples were collected from the structure on 11/7/18, and ten soil samples were taken below the bridge to test for lead on 12/5/18. The results of the soil testing determined that all lead content was within the allowable levels outlined in TCEQ's 2017 Protective Concentration Levels. The results of the paint testing from the structure determined that three of the eight locations exceeded the concentrations for lead as defined by the United States Federal Government. Therefore, the project must be treated as a lead abatement project.

The coating system on the bridge is in poor condition and has outlived it useful life. The coating is cracked at multiple locations and in other locations is peeling off. The coating has failed on most of the gusset plates and connection angles where the columns and diagonal braces are connected to the horizontal structural member at the bottom the bridge. Severe section loss, some layering of the steel and holes corroded through the plates and angles was observed. Corrosion layering was observed between the connection angles and the vertical truss members where the guardrail is attached to the bridge. The coating has also failed on the top flanges of the horizontal deck support members and stringers under the road deck. Section loss, severe corrosion and layering of the steel on the top flanges was observed. The connection bolts at each of the bridge were observed to be severely corroded.

The truss structure was analyzing for the following conditions:

- Scenario 1 – Original Condition (H15) – as it was designed
- Scenario 2 – Deteriorated Condition – current condition

  - Scenario 2a – based on original design loads (H15)
  - Scenario 2b – based on pedestrian live load only
- Scenario 3 – Proposed Condition
  - Scenario 3a – assumes repaired members allow structure to serve as a pedestrian walkway / viewing platform
  - Scenario 3b – assumes repaired members allow structure to serve as a deck park (as well as pedestrian walkway / viewing platform)

Evaluation of these scenarios led to three possible options:

OPTION 1: Restore the structure to an unrestricted pedestrian bridge by:

- Remove concrete deck.
- Remove existing coating.
- Repair required structural members.
- Recoat the steel structure.
- Replace concrete deck with lightweight concrete based on reducing the pedestrian width by 2 feet on each side. Further analysis may allow for full width replacement.
- Rehabilitate the approach spans (spalls, cracks, and exposed reinforcing only).
- Continue operating the bridge as a pedestrian bridge with no access restrictions.

OPTION 2: Restore the structure to be repurposed as a deck park with unrestricted pedestrian access by:

- Remove concrete deck.
- Remove existing coating.
- Repair required structural members. Depending on desired deck park features, additional strengthening and more extensive member replacement may be necessary.
- Recoat the steel structure.
- Replace concrete deck with lightweight concrete based on reducing the pedestrian width by 2 feet on each side. Further analysis may allow for full width replacement.
- Rehabilitate the approach spans (spalls, cracks, and exposed reinforcing only) Operate the bridge as a deck park after coordination with deck park designer to ensure deck park features are within tolerable load limits of the bridge as rehabilitated.

OPTION 3: Demolition

- Demolish truss spans and approach spans. This option assumes that costs and/or effort to rehabilitate the bridge do not meet the economical or functional goals of the City. For this option, the truss spans as well as the concrete approach spans would be demolished. As an alternative, the approach spans can be left in place and rehabilitated and repurposed into a deck park or viewing area and possibly connected to the newer pedestrian bridge at the river end of each side. It is assumed spans will be demolished by explosive measures or lowered and disassembled. Debris would be manifested and hauled off for disposal. If the bridge is demolished, the construction process will still need to be controlled to ensure no lead escapes into the riverbed or the surrounding soil but removing the coating is likely not necessary.

Cost ranges associated with the options described above are as follows:

Option Description	Conceptual Opinion of Probable Construction Cost Range
1. Pedestrian Bridge (Assumes Full Width Deck)	$8,000,0000.00 - $10,000,000.00
2. Deck Park (Assumes Full Width Deck)	$8,500,000.00 - $10,500,000.00
3.A Demolition of Full Bridge Structure	$3,500,000.00 – $4,500,000.00
3.B Demolition of Truss Spans Only	$3,000,000.00 - $3,500,000.00

The opinions and conclusions expressed in this report are based on the information provided by the City of Bastrop and data collected during the site evaluations identified above and may be amended or supplemented should new information become available. No warranties or guarantees, expressed or implied, are made or intended. This report has been prepared solely for the City of Bastrop and should not be relied upon by any other party or for any other purpose. Specifically, this report may not be used as construction documents. Any reliance on this report by any party other than the City of Bastrop shall be without liability to Kimley-Horn and Associates, Inc. or their employees.

2.0 Historical Background

The Colorado River Bridge in Bastrop, completed in 1923, showcases a historic route through Texas which has been critical to Bastrop's development since the beginning of the 19th century. The structure is an important surviving example of the work of the early Texas State Highway Department during a time when the automobile was emerging as the dominant mode of transportation. The bridge is an active listing on the National Register of Historic Places and obtained this classification in Transportation and Engineering. Related to transportation, the bridge was a link in the historic route of the Camino Real on which Bastrop was settled and served as a critical highway link between Houston and Austin. Related to engineering, it is a major bridge embodying the design and construction technology of the early period of highway construction in Texas.

Although the resident engineer on the project was R.E. Schiller, the bridge design reflects the influence of G.G. Wickline, State Bridge Engineer from 1918 until the 1940s, and is one of the earliest uses of the Parker truss surviving in Texas. The Parker truss was the truss design of choice form the 1920s into the 1940s because its efficiency of design allowed for a longer span with greater strength while using less steel, thus lowering the cost by reducing the weight of the of the bridge. The bridge was opened for use in January 1924.

The bridge was originally financed by Bastrop County along with federal and state aid. The ownership transferred to the State during the Great Depression when the inability of local governments to maintain roads led to their wholesale transfer to state governments (aided by federal dollars) around the country. The ownership then transferred again to the City of Bastrop in the 1990s when the parallel vehicular bridge was constructed, and the Colorado River Bridge became a pedestrian walkway.[1]

3.0 Structural Description

The Colorado River Bridge is a 1284-foot long concrete and steel structure bridge with three identical Parker through truss main spans. The bridge has an overall width of 21.5 feet, and it crosses the Colorado River as state highway Loop 150 two blocks west of the Bastrop Commercial Historic District. While the Colorado River is normally contained within a 200-foot wide channel 60-feet beneath the roadway, the bridge spans a much broader wooded floodplain.

The bridge consists of 21 total spans: 18 approach spans and the three truss main spans. The approach spans, spans 1 - 3 and 7 - 21, have a span length of 39-feet and consist of simply supported reinforced concrete slabs and girders (T-beams). The main spans, Spans 4-6, have a span length of 192-feet and are composed of simply supported steel Parker through trusses. The superstructure rests on reinforced concrete bent caps and columns over the approach spans and on concrete piers over the main spans (Photos 1-6).

The steel trusses consist of upper and bottom chord, vertical, and diagonal members with built-up cross sections composed of angles, channels, and plates connected by V-lacing and batten plates and joined by rivets. The trusses are braced with cross frames and diagonal bracing members connected to the vertical-upper chord joints. The floor system consists of a reinforced concrete deck, floor beams with built-up sections and stringers with rolled I-shape sections. The diagonal bracing consists of threaded rods connected to the floor beam lower chord joints (Photos 7-8).

The truss members, floor beams, stringers, cross frames and rods have a protective coating paint system. See the subsequent sections for discussion of the bridge coating.

The bridge was originally designed to carry vehicular traffic with a design load of H15, which is based on a two-axle single unit vehicle weighing 15 tons. By comparison, current bridges are typically designed based on HL-93 loading which consists of a three-axle vehicle and varying load/geometric combinations totaling 36 tons. Records indicate that the bridge was in service until early 1989. However, by then it had become functionally obsolete by modern bridge standards and was taken out of service and repurposed to accommodate pedestrian traffic. The bridge was added to the National Register of Historic Places inventory in 1990.

Specific truss members are identified by the sketch shown in Figure 1 for reference. The cross-section geometry for the truss and floor system members is listed in Tables 1 and 2.

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[1] <u>Reference:</u> *Colorado River Bridge at Bastrop United States Department of the Interior National Park Service National Register of Historic Places Registration Form*

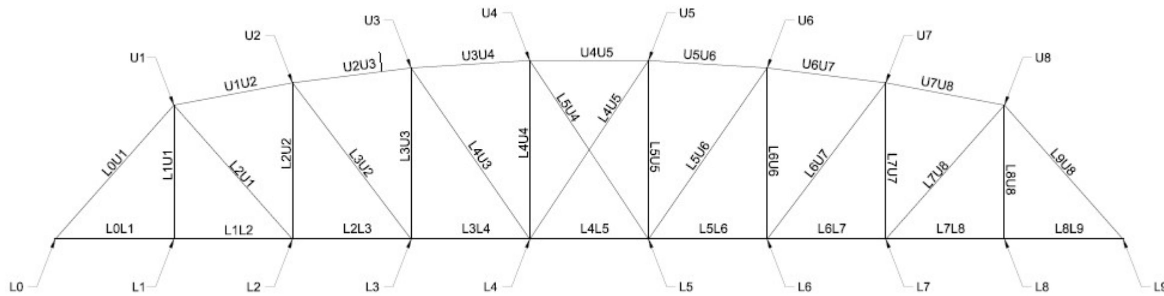

Figure 1: Typical Truss Notation

Upper Chord	L0-U1	U1-U2	U2-U3	U3-U4	U4-U5
	2[s 12x35	2[s 12x30	2[s 12x30	2[s 12x35	2[s 12x35
	PL 16x1/2	PL 16 x5/16	PL 16x1/2	PL 16x7/16	PL 16x7/16
Lower Chord	L0-L1	L1-L2	L2-L3	L3-L4	L4-L5
	4Ls 7x3-1/2x3/8	4Ls 7x3-1/2x3/8	4Ls 8x3-1/2x9/16	4Ls 8x3-1/2x9/16	4Ls 8x3-1/2x5/8
Vertical	L1-U1	L2-U2	L3-U3	L4-U4	
	2[s 8x11.25	2[s 8x11.25	2[s 8x11.25	2[s 8x11.25	
Diagonal	U1-L2	U2-L3	U3-L4	U4-L5	
	2Ls 7x3-1/2x9/16	2Ls 6x3-1/2x7/16	2Ls 4x3x3/8	2Ls 3-1/2x3x5/16	
Floorbeams	L0, L9	L1 through L8			
	24 I 80	4Ls 6x3-1/2x3/8			
		PL 27x1/2			
Stringers	All panels				
	15 I 37.5				
Upper Diagonals	All panels				
	1 Ls 4-1/2x3x5/16				
Lower Diagonals	Panesl 1,9	Panels 2,8	Panels 3,7	Panels 4-6	
	1-1/8 in.	1 in.	7/8 in.	3/4 in.	
	diameter rod	diameter rod	diameter rod	diameter rod	

Table 1: Member Sections

Details on the structural steel grade type used in the trusses, cross frames, rods, floor beams and stringers are not available through record drawings or historical data. The original steel was supplied by Illinois Steel Company.

Unless otherwise noted, reference numbering for the field evaluation and this report is in accordance with the original bridge layout shown in the as-built plans, with bents and transverse members numbered from East to West, and the trusses numbered from South to North.

4.0 Structural Inspection Methods

A hands-on inspection was performed on the structural components of the bridge. Inspection methods included visual inspection, photo documentation, measurement and in-situ data collection, non-destructive testing such as sounding and scraping, and underwater diving for the pier foundations in the river channel. Access to the structure included on foot (above and below), by boom lift (upper truss), rope access (floor system below deck), and underwater (pier foundations).

4.1. Approach Spans: Spans 1-3, 7-21

Access to the deck, superstructure and substructure in the approach spans was gained from the deck and from both banks under the bridge.

4.2. Main Spans: Spans 4-6

Access to the upper chord, gussets, and upper sections of the verticals and diagonals was gained from the deck through a trailer mounted boom lift. The bottom chord, gussets, lower sections of the verticals and diagonals, bearing assemblages, and exterior stringers were accessed from the bottom chords. The floor beams were accessed by rope access.

5.0 Structural Inspection Findings

The following sections describe the field inspection results and identify corresponding (assigned) National Bridge Inventory (NBI) rating. See Table 2 below for a description of the ratings for reference.

See Appendix 12.2 for a list of tables representing the amount of deterioration of the structural elements based on the results of the inspection. Reinforced concrete element deterioration (deck, T-beam, Diaphragm, pier caps, columns/piers) is quantified based on linear feet of cracking and area of spalls or delamination. Structural steel element deterioration (gusset plates, filler plates) is quantified based on percentage of section loss and the size of section loss.

9	Excellent condition
8	Very good condition – no problems noted
7	Good condition – some minor problems
6	Satisfactory condition – structural elements show some minor deterioration
5	Fair condition – all primary structural elements are sound but may have minor section loss, cracking, spalling, or scour
4	Poor condition – advanced section loss, deterioration, spalling, or scour

3	Serious condition – 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
2	Critical condition – 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
1	“Imminent” failure condition – 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 back in light service
0	Failed condition – out of service – beyond correction action

Table 2: National Bridge Inventory Rating Information

5.1. Approach Spans: Spans 1-3, 7-21

Deck [NBI rating 6]

The asphalt overlay (+/- 2 inches) wearing surface prevents access to the concrete deck surface. However, the wearing surface is affected by widespread cracking, large delaminated patches, and spalling along the curbs and joints. The expansion joints are paved over (photos 9 - 11). The deck soffit is affected by isolated cracking.

The curbs and railing are affected by moderate scaling. The south railing post over Bent 13 has a moderate cover spalling at its base (photo 12).

Several drains are partially clogged by debris.

Superstructure Beams [NBI rating 5]

Several T-beams are affected by unconsolidated concrete at the bottom of the stems over bearing areas. At these locations the concrete has spalled exposing the steel reinforcement. Several exterior T-beams are affected by transverse cracking, delaminations and cover spalling on the stem. There is moderate cover spalling on the inside face of the T-beam 1 stem at Span 13.

Substructure [NBI rating 6]

The caps at abutments and interior bents are affected by delaminations and superficial spalling at bearing areas (Photos 18, 19). Several columns at interior bents have isolated areas affected by delaminations and unconsolidated concrete with exposed steel reinforcement (Photo 20).

Several interior bents are partially covered by heavy vegetation.

5.2. Main Spans: Spans 4-6

Deck [NBI rating 6]

The asphalt overlay wearing surface prevents access to the concrete deck surface. The condition of the wearing surface is similar to that in the approach spans. The deck soffit away from the exterior stringers is affected by cracking. There is moderate delaminations and cover spalling along the floor beams and on the overhangs along the exterior stringers. In addition, there is moderate cover spalling on the deck soffit along the joint at Bent 6.

The expansion joints have been paved over and the joints leak.

Superstructure [Overall NBI rating 3]

Upper Chord: Satisfactory Condition [NBI 6]

There is moderate pack rust, about ½ in. thick, corrosion and section loss on the filler plate at joints U1 and U8 in all trusses. The upper chord cover plate at these joints has up to ¼ in. section loss (Photo 25).

There is isolated localized impact damage on members U0-L1 and L8-U9 at all spans (Photo 26). There is moderate pack rust, corrosion and localized section loss on the horizontal gusset plates connecting the upper chord to the diagonal bracing members and cross frames at joints U2-U7 at all spans (Photo 27). The vertical gusset plates at joint U5 in Truss 2 at Span 4 have up to ¼ in. deep gouges; the exterior gusset plate has a 1 in. diameter torch-cut hole (Photo 28).

The rivets connecting the gusset plate to the vertical and diagonal members at joint U6 in Truss 2 at Spans 4 and 5 have been replaced by welds and bolts, respectively (Photos 29, 30).

The lower chords in the cross frames bracing the trusses at joints U1 and U8 in all spans are affected by moderate out of plane impact deformation. This deformation is about 8 in. at the cross frame joining joints U8 at Span 6 (Photo 31).

Several V-lacing plates are lightly bent without significant damage (Photo 32). This condition is observed throughout all built-up members in the trusses, about two plates per member in average.

The steel protective coating paint system has failed at several locations, but the exposed steel surface is in good condition (Photo 33).

Verticals: Fair Condition [NBI 5]

A section of the channel webs at the lower chord joints has been removed from all vertical members. In addition, at these joints there is moderate pack rust, up to ¼ in. deep pitting and up to 100% localized section loss (Photo 34). Several V-lacing plates, about two per member, are lightly bent without significant damage.

The L2-U2 member in Truss 1 at Span 6 has a deflection of about 2-¼ in. at mid height (Photos 35, 36).

The lower half of members L4-U4 and L5-U5 in Truss 2 at Span 4 has been replaced (Photo 37). The full length of members L6-U6 in Truss 2 at Spans 4 and 5 has been replaced (Photo 38). The replaced sections are butt welded to the original section of the repaired member and the riveted V-lacing was replaced with welded batten plates (Photos 37 - 39).

The steel protective coating paint system has failed at several locations, but the exposed steel surface is in good condition.

Diagonals: Fair Condition [NBI 5]

Impact damage on member L5-U4 in Truss 1 at Span 6 has caused permanent section torsion and a localized deflection of up to 2-¼ in. on its interior angle (Photo 40).

The lower half of members L4-U5 and L5-U4 in Truss 2 at Span 4 has been replaced (Photo 37). The full length of members L5-U6 and L6-U7 in Truss 2 at Span 5 and member L6-U7 in Truss 1 at Span 6 has been replaced (Photo 41). The replaced sections are butt welded to the original section of the repaired member and the riveted batten plates were replaced with welded batten plates.

The steel protective coating paint system has failed at several locations, but the exposed steel surface is in good condition.

Lower chord: Serious Condition [NBI 3]

The lower chord members in all spans are mainly affected by pitting corrosion, up to ¼ in. deep, pack rust and section loss between gusset plates at joints L1-L8 (Photo 34). There are isolated locations with corrosion and section loss in areas near the floor beams (Photo 42).

There is severe pack rust, corrosion and widespread section loss, up to 100%, on the vertical gusset plates connecting the vertical and diagonal members to the lower chord at locations L1-L8 (Photos 43 - 46). Photos 43 and 46 show that at some point, the gusset plates were cleaned and repainted. However, active corrosion appears to have continued to a point where large sections of the gusset plates have thoroughly deteriorated. A summary of the gusset plates with 100% section loss is shown in Table 3. Both gusset plates at joint L4 in Truss 1 at Span 4 have 1 in. diameter torched holes.

The steel protective coating paint system has failed and is in poor condition.

			Interior Plate 100% Section Loss			Exterior Plate 100% Section Loss			
Joint	Truss	Span	Width (in)	Length (in)	Area (in2)	Width (in)	Length (in)	Area (in2)	Photo
L4	1	4	1	1	1	1	1	1	55
L5	1	4	1	1	1	1	1	1	56
L3	1	6	1	1	1	-	-	-	68
L4	1	6	-	-	-	1	1	1	69
L5	1	6	1	1	1	1	1	1	-
L7	1	6	-	-	-	1	2	2	-
L1	2	4	1	2	2	-	-	-	-
L3	2	4	2	4	8	-	-	-	58
L4	2	4	1	1	1	-	-	-	59
L5	2	4	-	-	-	1	2	2	60
L6	2	4	3	1	3	2	1	2	61
L5	2	5	-	-	-	3	1	3	44
L6	2	5	-	-	-	1	2	2	66
L7	2	5	-	-	-	2	1	2	67
			-	-	-	2	1	2	
L3	2	6	6	4	24	3	2	6	45,46
			2	2	4	2	4	8	
L4	2	6	-	-	-	3	4	12	73
L5	2	6	2	1	2	2	1	2	74
L6	2	6	-	-	-	6	2	12	75

Table 3: Gusset Plates with 100% Section Loss and Associated Hole Size

Floor Beams: Fair Condition [NBI 5]

The floor beams have moderate corrosion along the top flange at the deck interface. The interior floor beams are mainly affected by localized pack rust and corrosion on the top and bottom angles of the built-up section (Photo 47). In addition, there is moderate pack rust and section loss at the interfaces of the various connection components at floor beam ends (Photo 48). A more accurate estimate of the extent of section loss on the top flanges cannot be made without removing the deck.

The steel protective coating paint system is in poor condition.

Stringers: Fair Condition [NBI 5]

The top flanges of the exterior stringers, and some interior stringers, are affected by moderate section loss along the interface with the concrete deck (Photos 23, 49). A more accurate estimate of the extent of section loss on the top flanges cannot be made without removing the deck. In addition, there is moderate pack rust, up to 1 in. thick, at the bearing seats attached to the floorbeam webs. This condition has caused permanent deformation of the bearing seats (Photo 50).

The steel protective coating paint system on the exterior stringers and diagonal rods is in poor condition.

Bearing Assemblies: Poor Condition [NBI 4]

An anchor bolt at joint L9 in Truss 2 at Span 5 has a severe section loss (Photo 51). Similar condition is observed in anchor bolt at joint L9 in Truss 1 at Span 5. The anchor bolt at joint L9 in Truss 2 at Span 6 has failed (Photo 52). Several other anchor bolts are affected by corrosion and section loss on the fastener.

Substructure [NBI rating 7]

The piers are affected by isolated cracking, delaminations, cover spalling and efflorescence (Photos 53, 54).

Piers 2 and 3 below the water surface are covered with light marine growth from the ordinary waterline (Mean Low Waterline – 317.07 ft.) to the mudline. Marine growth was easily removed by hand. Submerged portions of Piers 2 and 3 have abrasion of the concrete surfaces throughout, exposing coarse aggregate with up to 1/4 in. penetration (Photo 116). Pier 2 has an approximately 3 SF area of honeycombed concrete with 3” penetration at the top of the caisson on the East face near the upstream side of the pier. No exposed reinforcing steel was detected. Pier 3 has an approximately 1 SF spall with 3” penetration at the top of the caisson on the West face near the upstream side of the pier. There is an approximately 2 SF area of honeycombed concrete with up to 3” penetration adjacent to and below the spall. No exposed reinforcing steel was detected in the spall or honeycombed area. The Pier 3 caisson has an approximately 1 SF area of honeycombed concrete with 3” penetration on the West face near the downstream end of the pier. No exposed reinforcing steel was detected. The Colorado River has migrated to the West since original construction. The bridge layout from the original plans shows only Pier 2 in the water at normal pool elevation. Piers 2 and 3 are now in the water at normal flow. The caisson at Pier 2 is exposed 1 ft. to 2 ft. throughout except at the Southeast corner and the downstream end where there is up to 8 ft. of exposure. Original plans show the caisson at Pier 2 exposed approximately 5 ft. by design at the time of construction. The caisson at Pier 3 is exposed 1 ft. to 2 ft. throughout. Original plans show no exposure of the caisson at Pier 3 at the time of construction. Stone riprap has been placed adjacent to the upstream (North) end and wraps around adjacent to the East and West faces of the Pier 2 caisson. Riprap extends 2/3 the length of the caisson on both sides of Pier 2. There is no indication that stone riprap was ever placed or has washed out on the downstream end of Pier 2. Stone riprap has been placed along the East face of the Pier 3 caisson. There is no indication that stone riprap was ever placed or has washed out around the other faces of the Pier 3 caisson. The East channel bank is eroded in the bridge vicinity, with sloughing and undercutting of trees and vegetation (Photos 117 & 118).

The minor material and structural defects such as spalls, honeycomb, and exposure of coarse aggregate due to abrasion have no effect on the structural capacity of the foundations or piers at Piers 2 and 3. While there is evidence of some long-term lateral stream instability, migration of the channel and minor exposure of the caissons currently have negligible effect on the stability of the foundations and piers at Piers 2 and 3.

Additional Findings

Tree branches at both approaches of the bridge on its south end extend over the roadway (Photo 1).

A compilation of damaged areas and repair quantities is included in the Appendix.

Structural Inspection Findings Summary

The overall NBI condition for the bridge is rated as Serious [NBI 3].

The main contributing factor for this rating is the current condition of the gusset plates along the lower chord. Eighteen gusset plates out of 60, about 30%, have localized areas with 100% section loss. The size of these areas ranges from 1 in^2 up to 24 in^2.

6.0 Coating Inspection Method and Results

A coating inspection was performed to determine the heavy metals content, including lead of the coating on the bridge. The evaluation was conducted using a high resolution SLR camera and a drone. Eight paint samples were collected from the structure on 11/7/18, and ten soil samples were taken below the bridge to test for lead on 12/5/18. The results of the soil testing determined that all lead content was within the allowable levels outlined in TCEQ's 2017 Protective Concentration Levels. The results of the paint testing from the structure determined that three of the eight locations exceeded the concentrations for lead as defined by the United States Federal Government. The limit for lead in paint is 5,000 parts per million (PPM). One sample contained 130,000 PPM lead paint, 12.5% higher than permissible. The other two locations with lead concentrations in excess of the federal limit contained 12,400 PPM lead and 11,200 PPM lead. All other locations contained between 2,250 PPM lead and 4,490 PPM lead (below the 5,000 PPM federal limit).

The coating system on the bridge is in poor condition and has outlived it useful life. The coating is cracked at multiple locations and in other locations is peeling off. There was section loss and severe corrosion with layering of the steel observed on the upper cord of the bridge at connection plates. The coating has failed on most of the gusset plates and connection angles where the columns and diagonal braces are connected to the horizontal structural member at the bottom the bridge. Severe section loss, some layering of the steel and holes corroded through the plates and angles were observed as described in previous sections. Corrosion layering was observed between the connection angles and the vertical truss members where the guardrail is attached to the bridge. The coating has also failed on the top flanges of the horizontal deck support members and stringers under the road deck. Section loss, severe corrosion, and layering of the steel on the top flanges were observed. The connection bolts at each of the bridge were observed to be severely corroded. Some of the nuts were missing, and some of the bolts were severely bent or sheared off.

7.0 Analysis Methodology

The evaluation consisted of analyzing the following conditions:

- Scenario 1 – Original Condition (H15) – as it was designed
- Scenario 2 – Deteriorated Condition – current condition
    - Scenario 2a – Analyzed based on original condition design loads (H15)
    - Scenario 2b – Analyzed based on pedestrian live load only
- Scenario 3 – Future Condition
    - Scenario 3a – Analyzed assuming repaired members allow structure to serve as a pedestrian walkway / viewing platform
    - Scenario 3b – Analyzed assumed repaired members allow structure to serve as a deck park (as well as pedestrian walkway / viewing platform)

For scenarios 1 and 3, the AASHTO Standard Specifications for Highway Bridges, 17th Edition (2002) using Load Factor Design (LFD) was used as the basis for design for Scenarios 1 and 2, and the current AASHTO Load and Resistance Factor Rating (LRFR) design was used as the basis for design of Scenario 3. AASHTOWare Bridge Rating (v. 6.8.2.3001) software was used to determine the Inventory and Operational Rating Factors for all bridge members. Microsoft Excel and Mathcad were used to organize inputs and perform general calculations.

All Scenarios consider one “typical” worst case bridge section that represents the deteriorated condition of all three spans on one typical bridge. During the rehabilitation phase of the project, each element to be strengthened will be identified on each bridge span in the construction documents. The evaluation uses one typical section to better understand the entire scope of the three spans in one location.

7.1. Scenario 1 – Original Condition

The purpose of the “Original Condition” evaluation is to provide a baseline capacity of each member assuming that the bridge has zero deficiencies. The bridge was originally designed for the bridge dead load and vehicular live load. According to the record drawings (“Plans for Proposed Bridge Over the Colorado River” – 1922), the dead load of the bridge is assumed to be 3200 lb/ft, which includes the concrete slab weight and structural steel members. The vehicular live load is 1216 lb/ft over the length of bridge (or 64 psf over the entire bridge) to design the truss and a 15-ton truck (H15) over one span to design the floor system. An associated Rating Factor for this condition was then generated. See appendix 12.3 for the Load Ratings associated with the original condition.

7.2. Scenario 2a – Current Condition (H15 truck)

The purpose of the “Current Condition” is to provide Load Ratings for the bridge in its deteriorated condition to provide a straight-line capacity reduction based on the “Original Condition”. Scenario 2a uses the same loading criteria as Scenario 1 (H15 truck) to represent how much capacity has been lost over time. As the *Bridge Repair Requirements* section describes, many of the bridge elements are in serious structural condition and require replacement or repair. The evaluation of the current condition considers the section loss in the gusset plates along with the section loss or damage in other structural truss members to provide a load rating for each member. The comparison of each member from the original condition to the current condition and the associated percentage in reduced capacity is provided in Appendix 12.3.

Note: The lower chord is continuous through each panel point; therefore, the Load Rating the program provides for the panel point at the lower chord is not applicable and has been removed from Appendix 12.3.

7.3. Scenario 2b – Current Condition (Pedestrian Bridge)

The purpose of the "Current Condition" is to provide Load Ratings for the bridge in its deteriorated condition to provide a straight-line capacity reduction based on the "Original Condition". Scenario 2b uses pedestrian live load criteria and models the bridge as a pedestrian only bridge, but no members have been replaced. The main conclusion to be drawn from this scenario is which members need to be repaired/replaced for the City of Bastrop to continue using the bridge as a pedestrian bridge.

The bridge has been repurposed as a pedestrian only bridge following the construction of the adjacent Texas 150 highway. The bridge has been used in the past for both pedestrian thru traffic as well as a viewing platform for city events. Based on the initial inspection findings, the bridge was closed to all traffic until the evaluation provided in this report could be completed.

The Load Rating results for a pedestrian bridge with no pedestrian access restrictions are contained in Appendix 12.3. The results determine that some of the truss elements have load ratings that are less than 1.0 and require repair or replacement. These members can be strengthened to increase their capacity above 1.0. The gusset plates identified in Table 3 (Section 5.0) as having 100% section loss do not result in load ratings below 1.0 for these elements. However, because of the significant reduction in capacity at these locations on each truss, it is recommended to strengthen the elements identified to restore the gusset plates to near original capacity. Based on the results, the bridge can remain as a pedestrian bridge with no access restrictions with the following required repairs:

- Truss Member L2U2 (Load Rating 0.674) – This vertical member sustained damage and is out of plane.
- Truss Member L5U4 (Load Rating 0.877) – This diagonal and the center of the span does not have a large enough section to provide the necessary strength for the proposed pedestrian loading.
- Truss Member L4U5 (Load Rating 0.877) – This diagonal at the center of the span does not have a large enough section to provide the necessary strength for the proposed pedestrian loading.
- Panel Point L2 (59% max capacity reduction) – Large reduction in capacity but acceptable as the load rating is above 1.0
- Panel Point L3 (80% max capacity reduction) – Large reduction in capacity but acceptable as the load rating is above 1.0

- Panel Point L4 (88% max capacity reduction) – Large reduction in capacity but acceptable as the load rating is above 1.0
- Panel Point L5 (88% max capacity reduction) – Large reduction in capacity but acceptable as the load rating is above 1.0
- Panel Point L6 (80% max capacity reduction) – Large reduction in capacity but acceptable as the load rating is above 1.0
- Panel Point L7 (59% max capacity reduction) – Large reduction in capacity but acceptable as the load rating is above 1.0

Note: The lower chord is continuous through each panel point; therefore, the Load Rating the program provides for the panel point at the lower chord is not applicable and has been removed from Appendix 12.3.

It is assumed for the purposes of this report that the deck supporting structure (floor beams, stringers, etc.) will be replaced. However, they must be further evaluated following removal of the existing deck. It is possible that sections or all the deck framing structure might be salvaged rather than strengthened or replaced. A full inspection cannot be performed without removal of the concrete deck surface so will be done as a part of design.

7.4. Scenario 3a – Future Condition (Pedestrian Bridge with Repairs)

The purpose of the “Future Condition” is to provide Load Ratings for the bridge assuming all members have zero section loss or damage. Scenario 3a uses pedestrian live load criteria and models the bridge as a pedestrian only bridge. This scenario represents the maximum Load Rating each member could have if providing the needed repairs to return all elements to 100% capacity. This scenario evaluates the bridge using the current AASHTO Load and Resistance Factor Rating (LRFR) to ensure it will meet all current AASHTO standards for bridges inspection. Considering the bridge is nearly 100 years old, caution should be taken using additional steps to ensure some load is restricted to obtain additional member capacity. This can be achieved by using light weight concrete when re-pouring the deck and reducing the overall width of the deck. Both will significantly reduce the overall dead load and the width of the live load applied to the structure. In this scenario, the deck is assumed to use 110 lb/ft^3 lightweight concrete, and the width has been reduced by 4’. The Load Rating results for a future pedestrian bridge with all elements strengthened (zero section loss or damage) are contained in Appendix 12.3.

Note: The diagonal truss members identified for repair based on Scenario 2b are still inadequate in this scenario, because the original cross section of the member is insufficient for this loading. These members will need to be strengthened beyond their original design, which will be straightforward to accomplish.

7.5. Scenario 3b – Future Condition (Pedestrian Bridge Including Deck Park)

Based on the results of the pedestrian bridge analysis, repurposing the Colorado River Bridge as a deck park is a possibility. However, this option will need to be coordinated with the designer of the deck park to manage load capacity. There will need to be a balanced selection process between dead load and the full pedestrian live load. During the design, the deck park designer should be in the position to review all options to control the proposed dead load and check allowable pedestrian live load. The assumptions for this condition are an additional 30 lb/ft^2 dead load for planters/garden areas, art features, lighting features, etc and a reduction in the pedestrian live load to 45 lb/ft^2. The live load reduction would be achieved based on strategic placement of deck park features to limit pedestrian access to portions of the bridge occupied by deck park features.

The Load Rating results contained in Appendix 12.3 should be used to see the potential load rating of the bridge using the assumptions stated above but will be variable based on the final configuration of the deck park.

8.0 Structural Bridge Repair Recommendations

Based on the results given in Appendix 12.3 and physical inspections, repairs are described below:

8.1. Repairs

Deck

- Remove and Replace the deck in Spans 4 - 6 using lightweight concrete rather than normal
- Clean and repair joint seals in approach and main spans.
- Clean debris from clogged drains on approach spans.
- Repair cover spalling on railing.

Superstructure

- Strengthen truss members and gusset plate panel points that were subject to large reductions in capacity due to plates with up to 100% section loss as identified in Section 7.3.
- Remove damaged concrete, clean steel reinforcement and patch deteriorated areas on T-beam stems in the approach spans.
- Replace the butt welds on members previously repaired: L4-U4, L5-U5, L4-U5, and L5-U4 in Truss 2 at Span 4; L6-U6 in Truss 2 at Span 4; L6-U6, L5-U6 and L6-U5 in Truss 2 at Span 5; and L6-U7 in Truss 1 at Span 6 that are not up to current standards.
- Replace cross frames bracing Truss 1 and Truss 2 at joints U1.
- Replace exterior stringer bearing seats attached to the floor beam webs.
- Replace damaged anchor bolts on bearing assemblages.

Substructure

- Remove damaged concrete and patch spalling on caps at interior bents.
- Remove damaged concrete and patch areas of unconsolidated concrete on columns at interior bents.

Coatings

- Prior to any structural repairs being made, the exiting coating will need to be removed with a shrouded abrasive blast process. The abrasive blasting process must remove lead coating on the bridge. After abrasive blasting is completed the bridge will need to be coated with a zinc primer. After removal of gusset plates and any other structural members with faying surfaces, the gusset plates and faying member surfaces will need to be abrasive blasted and coated with an American Institute of Steel Construction (AISC) approved coating for slip critical service.

  After all structural repairs and modifications are completed, damaged coating on the bridge will need to be repaired with an appropriate surface preparation process and re-primer coated. Once all primer coating repairs are completed and the primer on the bridge is properly prepared, the entire bridge will need to be coated with an epoxy intermediate coating. Once the bridge is epoxy coated and the coating properly prepared, the entire bridge should be coated with a polysiloxane final coating.

Tree limbs that are in contact with the Bridge will need to be cut back far enough that there is adequate distance between the tree limbs and bridge for abrasive blasting and coating operations.

9.0 Historic Bridge Modification Impacts

As discussed earlier in the report, the bridge is on the National Register for Historic Places and any modifications to the bridge will require prior submittal to the Texas Historical Commission (THC). Additional requirements exist if the project will use state or federal funding but will not be discussed here as this type of funding is not anticipated. The submittal to the THC must occur at least 30 days prior to any construction activities on the bridge. Two reviews are required by TCH: one to determine that the rehabilitation is in line with the Department of the Interiors' *Standards for Rehabilitation,* and the other is to ensure that no artifacts are disturbed at this historic site per the requirements of the State Antiquities Act.

The *Standards for Rehabilitation* are meant to preserve the seven aspects of a historic property's historic integrity: location, design, setting, materials, workmanship, feeling, and association. For a bridge rehabilitation project, the THC is typically most concerned about the bridge's design, materials, and workmanship. For truss members, in accordance with the Standards, the THC typically recommend that deteriorated members be repaired rather than replaced. However, where severe deterioration requires it, individual members can be replaced in-kind with new materials matching the historic. To retain eligibility for listing in the National Register, the general rule is that not more than 50% of the members can be replaced. There are some vertical and diagonal members that have already been replaced on the bridge, which will need to be considered. The current registration form provides the important features of a Parker through truss for the bridge to remain on the Register:

- Parker truss web configuration (verticals in compression, diagonals in tension)
- Polygonal top chord with more than five slopes
- Inclined endposts

- Through truss configuration (struts, sway bracing, and lateral bracing above roadway)
- Diagonal counters on some examples (character-defining if part of original design)
- Portal bracing or struts on some examples (character-defining if part of original design)
- Bottom lateral bracing (not character-defining)
- Floor beams (not character-defining)
- Stringers (not character-defining)

Demolition can be performed if determined to be the best option by the City; however, the construction plans will still need to be submitted to the THC for approval.

In addition to THC requirements for rehabilitation, additional requirements for disturbing the ground near a historic site must also be met. There is the potential for staging of equipment near the waterbed or the installation of temporary bents in the floodplain of the Colorado River. The THC will need to review to ensure there are not any known artifacts at this historic site and will need to approve any potential excavations for the proposed construction.

The 90% design submittal to the city of Bastrop will need to include a submittal to the THC to ensure that all proposed solutions are within the acceptable criteria outlined in the Standards for the bridge to remain on the Register and that no historical artifacts will be impacted. This will also allow for the THC to provide comments and direction prior to final approval of any construction plans. If demolition is chosen as the preferred option, then the submittal will detail the demolition plans to remove the bridge from the National Register.

10.0 Recommended Options

Based on the new Load Rating of the bridge and the lead content in the steel coating, the following options are recommended as described below.

OPTION 1: Pedestrian Bridge (unrestricted but potential narrower width)

- Remove concrete deck.
- Remove existing coating.
- Repair required structural members as described in Sections 7 and 8.
- Recoat the steel structure.
- Replace concrete deck with lightweight concrete based on reducing the pedestrian width by 2 feet on each side. Further analysis may allow for full width replacement.
- Rehabilitate the approach spans (spalls, cracks, and exposed reinforcing only).
- Continue operating the bridge as a pedestrian bridge with no access restrictions.

OPTION 2: Deck Park (unrestricted but potential narrower width)

- Remove concrete deck.
- Remove existing coating.

- Repair required structural members as described in Section 7 and 8. Depending on desired deck park features, additional strengthening and more extensive member replacement may be necessary.
- Recoat the steel structure.
- Replace concrete deck with lightweight concrete based on reducing the pedestrian width by 2 feet on each side. Further analysis may allow for full width replacement.
- Rehabilitate the approach spans (spalls, cracks, and exposed reinforcing only) Operate the bridge as a deck park after coordination with deck park designer to ensure deck park features are within tolerable load limits of the bridge as rehabilitated.

OPTION 3: Demolition

- Demolish truss spans and approach spans. This option assumes that costs and/or effort to rehabilitate the bridge do not meet the economical or functional goals of the City. For this option, the truss spans as well as the concrete approach spans would be demolished. As an alternative, the approach spans can be left in place and rehabilitated and repurposed into a deck park or viewing area and possibly connected to the newer pedestrian bridge at the river end of each side. It is assumed spans will be demolished by explosive measures or lowered and disassembled. Debris would be manifested and hauled off for disposal. If the bridge is demolished, the construction process will still need to be controlled to ensure no lead escapes into the riverbed or the surrounding soil but removing the coating is likely not necessary.

11.0 Costs

OPTION 1: Assumptions for rehabilitation for use as a pedestrian bridge

This option assumes full containment of the bridge for blasting and recoating. It is anticipated that the containment will be in place for a significant portion of the construction duration. The containment system will be critical to aid in collection of lead paint and to prevent painting materials from being released into the environment. The concrete bridge deck and existing pedestrian railing will be removed allowing access to the lower structural members that will require repair or replacement. Structural steel repairs will be performed including heat-straightening, over-plating, or replacement of members as required. It is assumed gusset plate strengthening can be performed in-place with minimal shoring or bracing and will done one at a time. It is assumed gusset plate replacement can be performed in-place with shoring/bracing in place to facilitate load transfer as needed across the joint(s). Structural concrete repairs will be performed to address spalling and exposed reinforcing steel in approach slabs as well as the bridge sub-structure. A light-weight concrete deck will be formed and placed to either the full width of the bridge or at an agreed upon reduced width and will include required bridge jointing. A new pedestrian rail will be installed. Site cleanup and restoration will be performed before final project completion. As an alternative, reducing the deck width can be explored to determine potential cost savings.

OPTION 2: Assumptions for rehabilitation for use as a deck park

This option includes the same assumptions listed for the pedestrian bridge option above. It is not known at this time if additional structural modifications will be required to support additional loading for possible park amenities placed on the bridge. It is assumed that the bridge deck will remain at the full bridge width for cost purposes. It is likely that the total load for additional deck park amenities will need to be limited to the difference in load effects between the bridge as it sits today and as proposed with a lighter deck.

OPTION 3: Assumptions for demolition

This option assumes that costs and/or effort to rehabilitate the bridge do not meet the economical or functional goals of the City. For this scenario, the structure would be completely demolished. It is very difficult to disassemble truss type structures in-place due to the nature of the structural design. The center span over the Colorado River would likely require explosive demolition. This would involve strategic cutting of bridge members, placing of explosives, affixing floatation devices as needed on bridge members, detonating the explosive devices, and allowing the bridge structure to fall to the surface below. This method would require coordination with governmental authorities. The exterior truss spans would likely be disconnected and lowered to the ground with mechanical methods using cranes placed along the banks of the Colorado River. The steel components would then be cut into pieces, collected, manifested and hauled to an approved recycling facility licensed to handle steel materials with lead paint. The approach spans would likely be demolished using a combination of explosive and conventional methods. As an alternative, the approach spans can be left in place and rehabilitated and repurposed into a deck park or viewing area and possibly connected to the newer pedestrian bridge at the river end of each side.

Opinion of Probable Construction Cost for the listed options*

Option Description	Conceptual Opinion of Probable Construction Cost Range
1. Pedestrian Bridge (Assumes Full Width Deck)	$8,500,000.00 - $10,500,000.00
2. Deck Park (Assumes Full Width Deck)	$8,750,000.00 - $11,000,000.00
3A Demolition of Full Bridge Structure	$3,750,000.00 – $4,750,000.00
3B Demolition of Truss Spans Only	$3,500,000.00 - $4,000,000.00

*Notes

1. The consultant has no control over the cost of labor, materials, equipment, or over the contractor's methods of determining prices or over competitive bidding or market conditions. Opinions of probable costs provided herein are based on the information known.

2. Costs are conceptual based on design assumptions. No design has been performed for the specific work required for each rehabilitation or demolition option.
3. Costs for rehabilitation options assume that the final design will allow construction of structural repairs and replacements with the use of isolated structural support and do not include costs associated with support of the full structure during construction.
4. Rehabilitation costs for the deck park option include the assumption that additional structural modifications will be required to provide the capacity to support park amenities.
5. Costs for deck park option are only for structural improvements and do not include the cost for park amenities.
6. Costs include opinion of both engineering and construction costs.

12.0 Appendices

12.1. Photo Log with Descriptions

12.2. Section Loss Estimate Tables

12.3. Load Rating Result Tables

12.4. Opinion of Probable Construction Cost for Repair Options

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Appendix 12.1

Photo Log with Descriptions

Photo 1 - East Approach Bridge Deck (Spans 1-3)

Photo 2 - East Approach Bridge Elevation

Photo 3 - Typical Approach Bridge Substructure

Photo 4 - Typical Main Span Substructure

Photo 5 - Span 5 Looking Upstream

Photo 6 - Main Spans Looking Downstream

Photo 7 - Typical Parker Truss Cross Frames

Photo 8 - Typical Floor System Layout

Photo 9 - Typical Approach Bridge Deck Joint

Photo 10 - Typical Joint at Approach Span and Main Span Interface

Photo 11 - Typical Deck Joint at Main Spans

Photo 12 - Delamination and Cover Spalling at South Rail Post over Bent 13

Photo 13 - Cover Spalling on T-beam 1 over Bent 14

Photo 14 - Cover spalling at T-beam 4 over Bent 8

Photo 15 - Delamination and Cover Spalling on Stem of T-beam 4 at Span 21

Photo 16 - Transverse Cracking on Stem of T-beam 4 at Span 21

Photo 17 - Transverse Cracking and Cover Spalling on Inside Face of T-beam 1 at Span 13 Drain

Photo 18 - Cover Spalling on West Abutment Cap

Photo 19 - Delamination and Spalling at Bearing Areas on Bent 13 Cap

Photo 20 - Unconsolidated Concrete Spalling on Column at Bent 14

Photo 21 - Typical Cracking on Deck Soffit at Main Spans

Photo 22 - Typical Spalling on Deck over Floorbeams

Photo 23 - Typical Spalling on Deck Overhang Near Joint

Photo 24 - Cover Spalling on Deck Soffit along Joint at Bent 6

Photo 25 - Typical Condition of Filler Plate and Upper Chord Cover Plate at Joints U1 and U8

Photo 26 - Typical Local Impact Damage on Upper Chord Member L0U1

Photo 27 - Typical Section Loss on Horizontal Gusset Plate

Photo 28 - Torch Hole and ¼" Deep Gouge on Exterior Gusset Plate at Joint U5, Truss 2, Span 4

Photo 29 - Repaired Connection to Diagonal and Vertical Members at Joint U6, Truss 2, Span 4

Photo 30 - Repaired Connection to Diagonal and Vertical Members at Joint U6, Truss 2, Span 5

Photo 31 - Impact Deformation up to 8 inches at Lower Chord of Cross Frame at U8 in Span 6

Photo 32 - Typical V-lacing Plate Connection In Built-Up Members

Photo 33 - Typical Paint Failure on Truss Members

Photo 34 - Typical Condition of Lower Chord Channels

Photo 35 - Condition of Vertical Member L2U2, Truss 1, Span 6

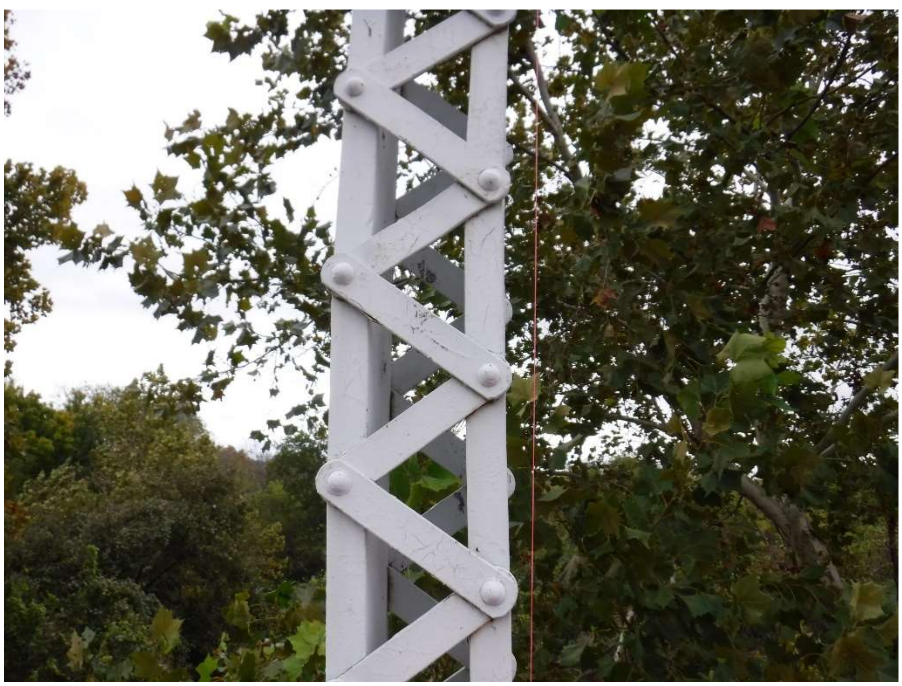

Photo 36 - 2-½” Deflection of Vertical Member L2U2, Truss 1, Span 6

Photo 37 - Lower Half Replacement of Members L4U4 and L5U5 in Truss 2, Span 4

Photo 38 - Full Length Replacement of Member L6U6, Truss 2, Span 4

Photo 39 - Butt Weld Joint at Repaired Member L4U4, Truss 2, Span 4

Photo 40 - Torsional Deformation on Member L5U4, Truss 1, Span 6

Photo 41 - Full Length Replacement of Diagonal Members L5U6 and L6U7, Truss 2, Span 5

Photo 42 - Typical Pack Rust, Corrosion, and Section Loss on Lower Chord

Photo 43 - Typical Gusset Plate Section Loss at Joints L2 through L7 in all Trusses

Photo 44 - Full Thickness 1”x4” Section Loss on Exterior Gusset Plate at Joint L5, Truss 2, Span 5

Photo 45 - Full Thickness 6”x3” Section Loss on Interior Gusset Plate at Joint L3, Truss 2, Span 6

Photo 46 - Full Thickness 3”x2” Section Loss on Exterior Gusset Plate at Joint L3, Truss 2, Span 6

Photo 47 - Typical Interior Floorbeam Condition

Photo 48 - Typical Localized Pack Rust at Ends of Interior Floorbeam

Photo 49 - Typical Section Loss along Stringer Top Flanges

Photo 50 - Typical Pack Rust Condition at Exterior Stringer Bearing Seat

Photo 51 - Section Loss on Exterior Anchor Bolt at Joint L9, Truss 2, Span 5

Photo 52 - Failed Anchor Bolt at Joint L9, Truss 2, Span 6

Photo 53 - Horizontal Cracking and Efflorescence on East Face of Pier 5

Photo 54 - Localized Delamination and Spalling at Construction Joint on West Face of Pier 6

Photo 55 - Lower chord gusset plate condition at Joint L4, Truss 1, Span 4

Photo 56 - Lower chord gusset plate condition at Joint L5, Truss 1, Span 4

Photo 57 - Horizontal Cracking and Efflorescence on East Face of Pier 5

Photo 58 - Lower chord gusset plate condition at Joint L3, Truss 2, Span 4

Photo 59 - Lower chord gusset plate condition at Joint L4, Truss 2, Span 4

Photo 60 - Lower chord gusset plate condition at Joint L5, Truss 2, Span 4

Photo 61 - Lower chord gusset plate condition at Joint L6, Truss 2, Span 4

Photo 62 - Lower chord gusset plate condition at Joint L7, Truss 2, Span 4

Photo 63 - Lower chord gusset plate condition at Joint L6, Truss 1, Span 5

Photo 64 - Lower chord gusset plate condition at Joint L2, Truss 2, Span 5

Photo 65 - Lower chord gusset plate condition at Joint L5, Truss 2, Span 5

Photo 66 - Lower chord gusset plate condition at Joint L6, Truss 2, Span 5

Photo 67 - Lower chord gusset plate condition at Joint L7, Truss 2, Span 5

Photo 68 - Lower chord gusset plate condition at Joint L3, Truss 1, Span 6

Photo 69 - Lower chord gusset plate condition at Joint L4, Truss 1, Span 6

Photo 70 - Lower chord gusset plate condition at Joint L7, Truss 1, Span 6

Photo 71 - Lower chord gusset plate condition at Joint L3, Truss 2, Span 6

Photo 72 - Lower chord gusset plate condition at Joint L3, Truss 2, Span 6

Photo 73 - Lower chord gusset plate condition at Joint L4, Truss 2, Span 6

Photo 74 - Lower chord gusset plate condition at Joint L5, Truss 2, Span 6

Photo 75 - Lower chord gusset plate condition at Joint L6, Truss 2, Span 6

Photo 76 - Lower chord gusset plate condition at Joint L7, Truss 2, Span 6

Photo 77 - Upper chord gusset plate condition at Joint U2, Truss 1, Span 4

Photo 78 - Upper chord gusset plate condition at Joint U5, Truss 1, Span 4

Photo 79 - Upper chord gusset plate condition at Joint U7, Truss 2, Span 4

Photo 80 - Upper chord gusset plate condition at Joint U5, Truss 2, Span 6

Photo 81 - Upper chord filler plate condition at Joint U8, Truss 1, Span 4

Photo 82 - Upper chord filler plate condition at Joint U1, Truss 2, Span 4

Photo 83 - Upper chord filler plate condition at Joint U7, Truss 1, Span 5

Photo 84 - Upper chord filler plate condition at Joint U1, Truss 2, Span 5

Photo 85 - Upper chord filler plate condition at Joint U1, Truss 1, Span 6

Photo 86 - Upper chord filler plate condition at Joint U8, Truss 2, Span 6

Photo 87 - Floorbeam to vertical connection angle condition at Joint L5, Truss 2, Span 4

Photo 88 - Floorbeam to vertical connection angle condition at Joint L6, Truss 2, Span 4

Photo 89 - Floorbeam to vertical connection angle condition at Joint L7, Truss 2, Span 4

Photo 90 - Floorbeam to vertical connection angle condition at Joint L8, Truss 2, Span 4

Photo 91 - Floorbeam to vertical connection angle condition at Joint L3, Truss 2, Span 5

Photo 92 - Floorbeam to vertical connection angle condition at Joint L6, Truss 2, Span 5

Photo 93 - Floorbeam to vertical connection angle condition at Joint L2, Truss 2, Span 6

Photo 94 - Floorbeam to vertical connection angle condition at Joint L5, Truss 2, Span 6

Photo 95 – Cross frame lower chord deformation at Joint U1, Span 4

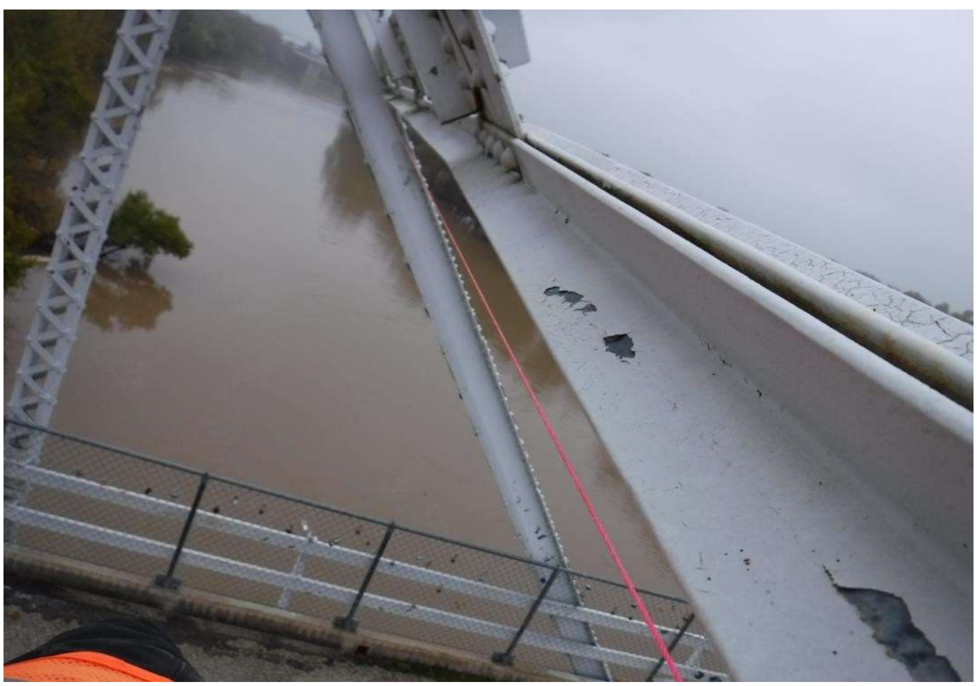

Photo 96 - Cross frame lower chord deformation at Joint U8, Span 4

Photo 97 - Cross frame lower chord deformation at Joint U1, Span 5

Photo 98 - Cross frame lower chord deformation at Joint U8, Span 5

Photo 99 - Cross frame lower chord deformation at Joint U1, Span 6

Photo 100 - Cross frame lower chord deformation at Joint U8, Span 6

Photo 101 - Exterior anchor bolt condition at Joint L0, Truss 1, Span 4

Photo 102 – Interior anchor bolt condition at Joint L0, Truss 1, Span 4

Photo 103 - Exterior anchor bolt condition at Joint L9, Truss 1, Span 5

Photo 104 - Interior anchor bolt condition at Joint L9, Truss 1, Span 5

Photo 105 - Exterior anchor bolt condition at Joint L0, Truss 2, Span 5

Photo 106 - Exterior anchor bolt condition at Joint L9, Truss 2, Span 5

Photo 107 - Interior anchor bolt condition at Joint L9, Truss 2, Span 5

Photo 108 - Interior anchor bolt condition at Joint L9, Truss 1, Span 6

Photo 109 - Exterior anchor bolt condition, missing fastener, missing bolt, at Joint L9, Truss 2, Span 6

Photo 110 - Interior anchor bolt condition at Joint L9, Truss 2, Span 6

Photo 111 – Roadway over bridge

Photo 112 – Elevation

Photo 113 - Feature crossed - Colorado River

Photo 114 – Main substructure configuration, Bent 2

Photo 115 – Main substructure configuration, Bent 3

Photo 116 – General condition of concrete bents above water. Heavier abrasion below water level.

Photo 117 – Erosion of east channel bank through Chestnut St. bridges

Photo 118 – Erosion of east channel bank upstream of Chestnut St. bridges

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Appendix 12.2

Estimate of Structural Deterioration

The following tables represent the amount of deterioration of the structural elements based on the results of the inspection. Reinforced concrete element deterioration (deck, T-beam, Diaphragm, pier caps, columns/piers) is quantified based on linear feet of cracking and area of spalls or delamination. Structural steel element deterioration (gusset plates, filler plates) is quantified based on percentage of section loss and the size of section loss.

Table 1 – Reinforced Concrete Deck Spalling/Delamination

	Wearing Surface		Soffit		Railing	
Span	Cracking (ft)	Spall/Delam (ft2)	Cracking (ft)	Spall/Delam (ft2)	Cracking (ft)	Spall/Delam (ft2)
1	80	110	8	12	10	
2	60	110	20	10		
3	90	110		5	5	
4	680	490	1180	59		
5	810	630	1180	141		
6	685	536	790	102		
7	80	150	40			
8	80	150	5			
9	75	44	5	5	5	8
10	130	150	40			
11	140	150	40			
12	140	150	45			
13	100	120	100	20		
14	160	130	80			
15	165	160	45	5		
16	175	160				
17	110	160		5		
18	140	100				
19	150	125	5			
20	150	200				
21	140	120	20	15		
Total	**4340**	**4055**	**3603**	**379**	**20**	**8**

Table 2 – Reinforced Concrete Superstructure Spalling/Delamination

	T-beam		Diaphragm	
Span	Cracking (LF)	Spall/Delam (SF)	Cracking (LF)	Spall/Delam (SF)
1		1	1	1
2	2	10	1	
3		2	1	
7			1	1
9	1		1	
13		26	1	
16	2	4	1	
18		2	1	
19	5		1	
21	10	10	1	3
Total	**20**	**55**	**10**	**5**

Table 3 – Reinforced Concrete Substructure Spalling/Delamination

	Cap		Columns/Pier	
Span	Cracking (LF)	Spall/Delam (SF)	Cracking (LF)	Spall/Delam (SF)
4			4	2
5			10	5
6			20	5
7			5	2
8		2		
9		10		
11		4		
12		2		
13		8		
14	3			1
15		1		2
19		2		
20		4		1
22		6		
Total	**3**	**39**	**39**	**18**

Table 4 – Structural Steel Truss Vertical Gusset Plates (Span 4 Truss 1) Deterioration

Truss 1	Exterior Gusset				Interior Gusset				
	Length (in)	Width (in)	Area (in2)	Section Loss (%)	Length (in)	Width (in)	Area (in2)	Section Loss (%)	Photo
L2	2	12	24	25	2	12	24	25	-
L3	2	12	24	50	3	12	36	50	-
L4	1	12	12	50	1	12	12	25	55
	1	1	1	100	1	1	1	100	
L5	2	12	24	25	2	12	24	25	56
	1	1	1	100	1	1	1	100	
L6	3	12	36	50	3	10	30	50	57
L7	2	4	8	25	1	10	10	25	
L8	1	4	4	10	-	-	-	-	-
U6	1	1	1	75	-	-	-	-	-
U7	1	2	2	50	-	-	-	-	-

Table 5 – Structural Steel Truss Vertical Gusset Plates (Span 4 Truss 2) Deterioration

Truss 2	Exterior Gusset				Interior Gusset				
	Length (in)	Width (in)	Area (in2)	Section Loss (%)	Length (in)	Width (in)	Area (in2)	Section Loss (%)	Photo
L1	1	4	4	25	1	4	4	25	-
	-	-	-	-	1	2	2	100	
L2	2	10	20	25	2	10	20	50	-
L3	2	10	20	50	3	10	30	50	58
	-	-	-	-	2	4	8	100	
L4	2	12	24	50	2	12	24	50	59
	-	-	-	-	1	1	1	100	
	1	1	1	100	1	1	1	100	
L5	2	12	24	50	2	10	20	50	60
	1	2	2	100	-	-	-	-	
	1	1	1	100	1	1	1	100	
L6	2	12	24	75	2	12	24	50	61
	1	2	2	100	1	3	3	100	
L7	3	12	36	50	3	12	36	50	62

Table 6 – Structural Steel Truss Vertical Gusset Plates (Span 5 Truss 1) Deterioration

Truss 1	Exterior Gusset				Interior Gusset				
	Length (in)	Width (in)	Area (in2)	Section Loss (%)	Length (in)	Width (in)	Area (in2)	Section Loss (%)	Photo
L1	2	4	8	25	2	4	8	25	-
L2	2	10	20	10	2	8	16	25	-
L3	1	12	12	25	2	12	24	25	-
L4	2	12	24	50	2	12	24	50	-
L5	4	10	40	25	2	10	20	25	-
L6	2	12	24	50	2	12	24	50	63
L7	2	10	20	50	2	10	20	50	-
L8	1	4	4	10	1	4	4	10	-

Table 7 – Structural Steel Truss Vertical Gusset Plates (Span 5 Truss 2) Deterioration

Truss 2	Exterior Gusset				Interior Gusset				
	Length (in)	Width (in)	Area (in2)	Section Loss (%)	Length (in)	Width (in)	Area (in2)	Section Loss (%)	Photo
L2	2	12	24	50	2	12	24	50	10
L3	2	10	20	25	2	10	20	25	-
	1	1	1	100	1	1	1	100	
L4	2	10	20	50	2	10	20	50	-
	-	-	-	-	1	1	1	100	
L5	2	10	20	25	2	10	20	25	65
	1	3	3	100	-	-	-	-	
L6	2	10	20	25	2	10	20	25	66
	1	2	2	100	-	-	-	-	
L7	2	10	20	50	2	10	20	50	67
	1	2	2	100	-	-	-	-	
	1	2	2	100	-	-	-	-	
L8	1	4	4	10	1	4	4	10	-

Table 8 – Structural Steel Truss Vertical Gusset Plates (Span 6 Truss 1) Deterioration

Truss 1	Exterior Gusset				Interior Gusset				
	Length (in)	Width (in)	Area (in2)	Section Loss (%)	Length (in)	Width (in)	Area (in2)	Section Loss (%)	Photo
L1	1	4	4	25	1	4	4	25	-
L2	2	6	12	50	2	8	16	50	-
L3	2	12	24	75	2	12	24	75	68
	-	-	-	-	1	1	1	100	
L4	2	12	24	50	2	12	24	50	69
	1	1	1	100	-	-	-	-	
L5	2	8	16	50	2	12	24	50	-
	1	1	1	100	1	1	1	100	
L6	2	12	24	50	3	12	36	50	-
L7	3	12	36	50	2	8	16	50	70
	1	2	2	100	-	-	-	-	
L8	1	4	4	25	1	4	4	10	-

Table 9 – Structural Steel Truss Vertical Gusset Plates (Span 6 Truss 2) Deterioration

Truss 2	Exterior Gusset				Interior Gusset				
	Length (in)	Width (in)	Area (in2)	Section Loss (%)	Length (in)	Width (in)	Area (in2)	Section Loss (%)	Photo
L2	2	10	20	25	1	10	10	25	-
L3	2	10	20	75	2	10	20	75	71,72
	2	4	8	100	2	2	4	100	
	2	3	6	100	4	6	24	100	
L4	2	12	24	75	2	10	20	50	73
	3	4	12	100	-	-	-	-	
L5	2	12	24	50	2	12	24	50	74
	1	2	2	100	-	-	-	-	
L6	4	10	40	25	2	12	24	50	75
	2	6	12	100	-	-	-	-	
L7	2	10	20	50	2	12	24	75	76
L8	2	8	16	50	1	6	6	50	-

Table 10 – Structural Steel Truss Horizontal Gusset Plates Deterioration

Span	Truss	Location	Width (in)	Length (in)	Area (in2)	Section Loss (%)	Photo
4	1	U2	1	6	6	50	77
4	1	U2	1	6	6	50	
4	1	U3	1	1	1	25	
4	1	U5	1	2	2	50	78
4	1	U6	1	6	6	25	
4	1	U7	1	2	2	25	
4	2	U2	1	4	4	25	
4	2	U4	1	6	6	25	
4	2	U7	1	4	4	75	79
5	2	U2	1	4	4	25	
6	1	U3	1	4	4	25	
6	1	U6	1	2	2	25	
6	1	U7	1	3	3	50	
6	2	U3	1	8	8	25	80
6	2	U5	1	3	3	25	

Table 11 – Structural Steel Truss Filler Plates Deterioration

Span	Truss	Location	Width (in)	Length (in)	Area (in2)	Section Loss (%)	Photo
4	1	U2	1	5	5	100	
4	1	U8	2	12	24	100	81
4	2	U1	2	12	24	100	82
4	2	U2	2	8	16	100	
4	2	U8	2	12	24	100	
5	1	U1	2	12	24	100	
5	1	U2	2	12	24	100	
5	1	U7	2	8	16	100	83
5	1	U8	2	12	24	100	
5	2	U1	2	10	20	100	84
5	2	U8	2	12	24	100	
6	1	U1	2	14	28	100	85
6	1	U8	2	12	24	100	
6	2	U1	3	14	42	100	
6	2	U2	2	10	20	100	
6	2	U8	2	14	28	100	86

Table 12 – Structural Steel Truss Cross Frame Lower Chord Deformation

Span	Location	Deformation (in)	Photo
4	U1	1.25	95
4	U8	2.5	96
5	U1	2.75	97
5	U8	1.5	98
6	U1	8	99
6	U8	0.75	100

Table 13 – Structural Steel Truss Bearing Assembly Anchor Bolts Corrosion/Section Loss

Span	Truss	Location	Outside Bolts	Inside Bolts	Photo
4	1	L0	Moderate Corrosion	Moderate Corrosion	101,102
5	1	L9	Severe Section Loss	Moderate Corrosion	103,104
5	2	L0	Moderate Corrosion	-	105
5	2	L9	Severe Section Loss	Severe Section Loss	106,107
6	1	L9	-	Moderate Corrosion	108
6	2	L9	Moderate Section Loss, Missing Fastener, Missing Bolt	Moderate Corrosion	109,110

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Appendix 12.3

Load Rating Tables

	Scenario and Load Rating													
	Scenario 1 - Original Condition (H15 Truck)		Scenario 2a - Deteriorated Condition (H15 Truck)		Capacity Reduction (%)		Scenario 2b - Deteriorated Condition (Ped LL)		Capacity Reduction (%)		Scenario 3a - Repaired Condition (Ped LL)		Scenario 3b - Repaired Condition (Deck Park)	
	H15-MOD - Lane (Design Lane)													
	Truss Members													
	INV RF	OPR RF	INV RF	OPR RF	INV RF	OPR RF	INV RF	OPR RF	INV RF	OPR RF	INV RF	OPR RF	INV RF	OPR RF
L0L1	3.507	5.857	3.507	5.857	0	0	3.301	5.512	6	6	4.131	5.355	5.937	7.697
L1L2	3.507	5.857	3.507	5.857	0	0	3.301	5.512	6	6	4.131	5.355	9.937	7.697
L2L3	3.916	6.540	3.916	6.540	0	0	3.686	6.155	6	6	4.583	5.941	6.651	8.622
L3L4	3.097	5.172	3.097	5.172	0	0	2.915	4.868	6	6	3.677	4.767	5.221	6.768
L4L5	3.315	5.536	3.315	5.536	0	0	3.120	5.211	6	6	3.918	5.079	5.602	7.262
L5L6	3.097	5.172	3.097	5.172	0	0	2.915	4.868	6	6	3.677	4.767	5.221	6.768
L6L7	3.916	6.540	3.916	6.540	0	0	3.686	6.155	6	6	4.583	5.941	6.651	8.622
L7L8	3.507	5.857	3.507	5.857	0	0	3.301	5.512	6	6	4.131	5.355	5.937	7.697
L8L9	3.507	5.857	3.507	5.857	0	0	3.301	5.512	6	6	4.131	5.355	5.937	7.697
U1U2	2.428	4.054	1.566	2.616	36	35	1.474	2.462	39	39	3.289	4.275	4.622	5.992
U2U3	2.329	3.889	2.329	3.889	0	0	2.192	3.660	6	6	3.184	4.127	4.442	5.758
U3U4	2.410	4.024	2.410	4.024	0	0	2.268	3.787	6	6	3.294	4.270	4.616	5.984
U4U5	2.380	3.974	2.380	3.974	0	0	2.240	3.741	6	6	3.259	4.224	4.560	5.911
U5U6	2.410	4.024	2.410	4.024	0	0	2.268	3.787	6	6	3.294	4.270	4.616	5.984
U6U7	2.329	3.889	2.329	3.889	0	0	2.192	3.660	6	6	3.184	4.127	4.442	5.758
U7U8	2.428	4.054	2.428	4.054	0	0	2.285	3.816	6	6	3.298	4.275	4.622	5.992
L1U1	6.500	10.856	6.500	10.856	0	0	6.118	10.217	6	6	7.443	9.649	11.167	14.476
L2U2	2.445	4.083	0.716	1.196	71	71	0.674	1.126	72	72	2.901	3.760	4.203	5.449
L3U3	4.359	7.280	4.359	7.280	0	0	4.103	6.852	6	6	4.832	6.263	7.520	9.748
L4U4	11.156	18.630	11.156	18.630	0	0	10.500	17.535	6	6	12.452	16.141	19.398	25.145
L5U5	11.156	18.630	11.156	18.630	0	0	10.499	17.534	6	6	12.451	16.140	19.397	25.144
L6U6	4.359	7.280	4.359	7.280	0	0	4.103	6.852	6	6	4.832	6.264	7.520	9.748
L7U7	2.445	4.084	2.445	4.084	0	0	2.301	3.843	6	6	2.901	3.760	4.203	5.449
L8U8	6.500	10.856	6.500	10.856	0	0	6.118	10.217	6	6	7.443	9.649	11.167	14.476
L0U1	3.313	5.532	3.015	5.035	9	9	2.838	4.739	14	14	4.191	5.432	6.032	7.819
U8L9	3.313	5.532	3.313	5.532	0	0	3.118	5.207	6	6	4.191	5.432	6.032	7.819
U1L2	3.160	5.277	3.160	5.277	0	0	2.974	4.966	6	6	3.730	4.835	5.343	6.926
U2L3	3.457	5.774	3.457	5.774	0	0	3.254	5.434	6	6	4.012	5.200	5.897	7.645
U3L4	1.896	3.166	1.896	3.166	0	0	1.784	2.980	6	6	1.952	2.530	3.482	4.514
U4L5	0.932	1.556	0.932	1.556	0	0	0.877	1.464	6	6	0.976	1.265	1.648	2.136
L4U5	0.931	1.555	0.931	1.555	0	0	0.877	1.464	6	6	0.976	1.265	1.647	2.135
L5U6	1.896	3.166	1.896	3.166	0	0	1.784	2.980	6	6	1.952	2.530	3.482	4.513
L6U7	3.457	5.774	3.457	5.774	0	0	3.254	5.434	6	6	4.012	5.200	5.897	7.644
L7U8	3.160	5.277	3.160	5.277	0	0	2.974	4.966	6	6	3.730	4.835	5.343	6.926

		Scenario and Load Rating															
		Scenario 1 - Original Condition		Scenario 2a - Deteriorated Condition (H15 Truck)		Capacity Reduction (%)		Scenario 2b - Deteriorated Condition (Ped LL)		Capacity Reduction (%)		Scenario 3a - Repaired Condition (Ped LL)		Scenario 3b - Repaired Condition (Deck Park)			
		H15-MOD - Lane (Design Lane)															
		Panel Points (Gussets)															
L0	L0L1	Lower Chord is continuous at gusset plates - see member report															
	L0U1	1.896	3.166	1.896	3.166	0	0	1.784	2.980	6	6	2.477	3.212	3.057	3.963		
L1	L0L1	Lower Chord is continuous at gusset plates - see member report															
	L1L2																
	L1U1	5.857	9.781	5.857	9.781	0	0	5.513	9.206	6	6	6.175	8.005	8.707	11.287		
L2	L1L2	Lower Chord is continuous at gusset plates - see member report															
	L2L3																
	U1L2	3.455	5.769	1.418	2.367	59	59	1.334	2.228	61	61	3.658	4.742	4.909	6.364		
	L2U2	2.490	4.158	2.214	3.697	11	11	2.084	3.480	16	16	2.500	3.241	4.795	6.216		
L3	L2L3	Lower Chord is continuous at gusset plates - see member report															
	L3L4																
	U2L3	3.190	5.328	2.523	4.214	21	21	2.375	3.966	26	26	3.400	4.407	4.703	6.096		
	L3U3	10.041	16.768	2.044	3.414	80	80	1.924	3.213	81	81	10.539	13.661	16.486	21.370		
L4	L3L4	Lower Chord is continuous at gusset plates - see member report															
	L4L5																
	U3L4	3.322	5.548	2.100	3.507	37	37	1.976	3.301	41	41	3.551	4.603	5.206	6.749		
	L4U4	17.534	29.282	2.062	3.444	88	88	1.941	3.241	89	89	18.450	23.917	28.548	37.007		
	L4U5	3.940	6.580	3.940	6.580	0	0	3.708	6.193	6	6	4.161	5.394	6.462	8.376		
L5	L4L5	Lower Chord is continuous at gusset plates - see member report															
	L5L6																
	L4L5	3.940	6.580	3.940	6.580	0	0	3.708	6.193	6	6	4.161	5.394	6.461	8.376		
	L5U5	17.534	29.281	2.062	3.444	88	88	1.941	3.241	89	89	18.450	23.916	28.547	37.005		
	L5U6	3.323	5.549	2.100	3.507	37	37	1.977	3.301	41	41	3.551	4.603	5.206	6.749		
L6	L5L6	Lower Chord is continuous at gusset plates - see member report															
	L6L7																
	L6U6	10.041	16.769	2.044	3.414	80	80	1.924	3.213	81	81	10.539	13.661	16.486	21.371		
	L6U7	3.190	5.328	2.523	4.214	21	21	2.375	3.966	26	26	3.400	4.407	4.703	6.096		
L7	L6L7	Lower Chord is continuous at gusset plates - see member report															
	L7L8																
	L7U7	2.490	4.158	2.214	3.697	11	11	2.084	3.480	16	16	2.500	3.240	4.795	6.216		
	L7U8	3.455	5.769	1.418	2.367	59	59	1.334	2.228	61	61	3.658	4.742	4.909	6.364		
L8	L7L8	Lower Chord is continuous at gusset plates - see member report															
	L8L9																
	L8U8	5.857	9.781	5.857	9.781	0	0	5.513	9.206	6	6	6.175	8.005	8.707	11.287		
L9	L8L9	Lower Chord is continuous at gusset plates - see member report															
	U8L9	1.896	3.166	1.896	3.166	0	0	1.784	2.980	6	6	2.477	3.212	3.057	3.963		

	Scenario and Load Rating													
	Scenario 1 - Original Condition		Scenario 2a - Deteriorated Condition (H15 Truck)		Capacity Reduction (%)		Scenario 2b - Deteriorated Condition (Ped LL)		Capacity Reduction (%)		Scenario 3a - Repaired Condition (Ped LL)		Scenario 3b - Repaired Condition (Deck Park)	
	H15-MOD - Lane (Design Lane)													
	Floorbeams													
Floorbeam1	5.271	8.802	5.271	8.802	-	-	3.968	6.627	-	-	7.730	10.020	11.483	14.886
Floorbeam2	2.107	3.519	2.107	3.519	-	-	1.586	2.649	-	-	3.309	4.289	5.225	6.773
Floorbeam3	2.107	3.519	2.107	3.519	-	-	1.586	2.649	-	-	3.309	4.289	5.225	6.773
Floorbeam4	2.107	3.519	2.107	3.519	-	-	1.586	2.649	-	-	3.309	4.289	5.225	6.773
Floorbeam5	2.107	3.519	2.107	3.519	-	-	1.586	2.649	-	-	3.309	4.289	5.225	6.773
Floorbeam6	2.107	3.519	2.107	3.519	-	-	1.586	2.649	-	-	3.309	4.289	5.225	6.773
Floorbeam7	2.107	3.519	2.107	3.519	-	-	1.586	2.649	-	-	3.309	4.289	5.225	6.773
Floorbeam8	2.107	3.519	2.107	3.519	-	-	1.586	2.649	-	-	3.309	4.289	5.225	6.773
Floorbeam9	2.107	3.519	2.107	3.519	-	-	1.586	2.649	-	-	3.309	4.289	5.225	6.773
Floorbeam10	5.271	8.802	5.271	8.802	-	-	3.968	6.627	-	-	7.730	10.020	11.483	14.886
	Stringers													
Stringer1	3.506	5.856	3.506	5.856	-	-	2.075	3.465	-	-	2.591	4.327	3.559	4.613
Stringer2	3.291	5.495	3.291	5.495	-	-	2.477	4.137	-	-	1.273	1.650	1.705	2.210
Stringer3	3.291	5.495	3.291	5.495	-	-	2.477	4.137	-	-	1.273	1.650	1.705	2.210
Stringer4	3.291	5.495	3.291	5.495	-	-	2.477	4.137	-	-	1.273	1.650	1.705	2.210
Stringer5	3.291	5.495	3.291	5.495	-	-	2.477	4.137	-	-	1.273	1.650	1.705	2.210
Stringer6	3.506	5.856	3.506	5.856	-	-	2.075	3.465	-	-	2.591	4.327	3.559	4.613

	Scenario and Load Rating															
	Scenario 1 - Original Condition		Scenario 2a - Deteriorated Condition (H15 Truck)		Capacity Reduction (%)		Scenario 2b - Deteriorated Condition (Ped LL)		Capacity Reduction (%)		Scenario 3a - Repaired Condition (Ped LL)		Scenario 3b - Repaired Condition (Deck Park)			
	H15-MOD - Truck (Design Truck)															
	Truss Members															
	INV RF	OPR RF	INV RF	OPR RF	INV RF	OPR RF	INV RF	OPR RF	INV RF	OPR RF	INV RF	OPR RF	INV RF	OPR RF		
L0L1	6.937	11.585	6.937	11.585	0	0	-	-	-	-	-	-	-	-		
L1L2	6.937	11.585	6.937	11.585	0	0	-	-	-	-	-	-	-	-		
L2L3	7.764	12.967	7.764	12.967	0	0	-	-	-	-	-	-	-	-		
L3L4	6.160	10.288	6.160	10.288	0	0	-	-	-	-	-	-	-	-		
L4L5	7.167	11.968	7.167	11.968	0	0	-	-	-	-	-	-	-	-		
L5L6	6.160	10.287	6.160	10.287	0	0	-	-	-	-	-	-	-	-		
L6L7	7.764	12.967	7.764	12.967	0	0	-	-	-	-	-	-	-	-		
L7L8	6.937	11.585	6.937	11.585	0	0	-	-	-	-	-	-	-	-		
L8L9	6.937	11.585	6.937	11.585	0	0	-	-	-	-	-	-	-	-		
U1U2	4.814	8.039	3.106	5.186	35	35	-	-	-	-	-	-	-	-		
U2U3	4.632	7.736	4.632	7.736	0	0	-	-	-	-	-	-	-	-		
U3U4	4.814	8.040	4.814	8.040	0	0	-	-	-	-	-	-	-	-		
U4U5	5.145	8.592	5.145	8.592	0	0	-	-	-	-	-	-	-	-		
U5U6	4.814	8.040	4.814	8.040	0	0	-	-	-	-	-	-	-	-		
U6U7	4.632	7.735	4.632	7.735	0	0	-	-	-	-	-	-	-	-		
U7U8	4.814	8.039	4.814	8.039	0	0	-	-	-	-	-	-	-	-		
L1U1	3.235	5.403	3.235	5.403	0	0	-	-	-	-	-	-	-	-		
L2U2	3.522	5.882	1.032	1.723	71	71	-	-	-	-	-	-	-	-		
L3U3	5.270	8.801	5.270	8.801	0	0	-	-	-	-	-	-	-	-		
L4U4	11.548	19.285	11.548	19.285	0	0	-	-	-	-	-	-	-	-		
L5U5	11.548	19.285	11.548	19.285	0	0	-	-	-	-	-	-	-	-		
L6U6	5.270	8.801	5.270	8.801	0	0	-	-	-	-	-	-	-	-		
L7U7	3.522	5.882	3.522	5.882	0	0	-	-	-	-	-	-	-	-		
L8U8	3.235	5.403	3.235	5.403	0	0	-	-	-	-	-	-	-	-		
L0U1	6.552	10.943	6.552	10.943	0	0	-	-	-	-	-	-	-	-		
U8L9	6.552	10.943	6.552	10.943	0	0	-	-	-	-	-	-	-	-		
U1L2	5.370	8.968	5.370	8.968	0	0	-	-	-	-	-	-	-	-		
U2L3	3.827	6.391	3.827	6.391	0	0	-	-	-	-	-	-	-	-		
U3L4	1.484	2.479	1.484	2.479	0	0	-	-	-	-	-	-	-	-		
U4L5	0.927	1.549	0.927	1.549	0	0	-	-	-	-	-	-	-	-		
L4U5	0.927	1.548	0.927	1.548	0	0	-	-	-	-	-	-	-	-		
L5U6	1.484	2.478	1.484	2.478	0	0	-	-	-	-	-	-	-	-		
L6U7	3.827	6.391	3.827	6.391	0	0	-	-	-	-	-	-	-	-		
L7U8	5.370	8.968	5.370	8.968	0	0	-	-	-	-	-	-	-	-		

		Scenario and Load Rating													
		Scenario 1 - Original Condition		Scenario 2a - Deteriorated Condition (H15 Truck)		Capacity Reduction (%)		Scenario 2b - Deteriorated Condition (Ped LL)		Capacity Reduction (%)		Scenario 3a - Repaired Condition (Ped LL)		Scenario 3b - Repaired Condition (Deck Park)	
		H15-MOD - Truck (Design Truck)													
		Panel Points (Gussets)													
L0	L0L1	Lower Chord is continuous - no load rating - see report body													
	L0U1	3.750	6.263	3.750	6.263	0	0	-	-	-	-	-	-	-	-
L1	L0L1	Lower Chord is continuous - no load rating - see report body													
	L1L2														
	L1U1	2.915	4.868	2.915	4.868	0	0	-	-	-	-	-	-	-	-
L2	L1L2	Lower Chord is continuous - no load rating - see report body													
	L2L3														
	U1L2	5.871	9.805	2.409	4.024	59	59	-	-	-	-	-	-	-	-
	L2U2	1.430	2.388	1.430	2.388	0	0	-	-	-	-	-	-	-	-
L3	L2L3	Lower Chord is continuous - no load rating - see report body													
	L3L4														
	U2L3	4.648	7.761	3.676	6.139	21	21	-	-	-	-	-	-	-	-
	L3U3	10.697	17.865	2.471	4.127	77	77	-	-	-	-	-	-	-	-
L4	L3L4	Lower Chord is continuous - no load rating - see report body													
	L4L5														
	U3L4	4.084	6.821	2.582	4.311	37	37	-	-	-	-	-	-	-	-
	L4U4	18.151	30.312	2.135	3.565	88	88	-	-	-	-	-	-	-	-
	L4U5	4.004	6.686	4.004	6.686	0	0	-	-	-	-	-	-	-	-
L5	L4L5	Lower Chord is continuous - no load rating - see report body													
	L5L6														
	L4L5	4.003	6.686	4.003	6.686	0	0	-	-	-	-	-	-	-	-
	L5U5	18.150	30.311	2.135	3.565	88	88	-	-	-	-	-	-	-	-
	L5U6	4.084	6.821	2.582	4.311	37	37	-	-	-	-	-	-	-	-
L6	L5L6	Lower Chord is continuous - no load rating - see report body													
	L6L7														
	L6U6	10.697	17.865	2.472	4.128	77	77	-	-	-	-	-	-	-	-
	L6U7	4.647	7.761	3.676	6.139	21	21	-	-	-	-	-	-	-	-
L7	L6L7	Lower Chord is continuous - no load rating - see report body													
	L7L8														
	L7U7	1.430	2.388	1.430	2.388	0	0	-	-	-	-	-	-	-	-
	L7U8	5.871	9.805	2.409	4.024	59	59	-	-	-	-	-	-	-	-
L8	L7L8	Lower Chord is continuous - no load rating - see report body													
	L8L9														
	L8U8	2.915	4.868	2.915	4.868	0	0	-	-	-	-	-	-	-	-
L9	L8L9	Lower Chord is continuous - no load rating - see report body													
	U8L9	3.750	6.263	3.750	6.263	0	0	-	-	-	-	-	-	-	-

	Scenario and Load Rating															
	Scenario 1 - Original Condition		Scenario 2a - Deteriorated Condition (H15 Truck)		Capacity Reduction (%)		Scenario 2b - Deteriorated Condition (Ped LL)		Capacity Reduction (%)		Scenario 3a - Repaired Condition (Ped LL)		Scenario 3b - Repaired Condition (Deck Park)			
	H15-MOD - Truck (Design Truck)															
	Floorbeams															
Floorbeam1	1.312	2.19	1.312	2.19	0	0	-	-	-	-	-	-	-	-		
Floorbeam2	1.049	1.751	1.049	1.751	0	0	-	-	-	-	-	-	-	-		
Floorbeam3	1.049	1.751	1.049	1.751	0	0	-	-	-	-	-	-	-	-		
Floorbeam4	1.049	1.751	1.049	1.751	0	0	-	-	-	-	-	-	-	-		
Floorbeam5	1.049	1.751	1.049	1.751	0	0	-	-	-	-	-	-	-	-		
Floorbeam6	1.049	1.751	1.049	1.751	0	0	-	-	-	-	-	-	-	-		
Floorbeam7	1.049	1.751	1.049	1.751	0	0	-	-	-	-	-	-	-	-		
Floorbeam8	1.049	1.751	1.049	1.751	0	0	-	-	-	-	-	-	-	-		
Floorbeam9	1.049	1.751	1.049	1.751	0	0	-	-	-	-	-	-	-	-		
Floorbeam10	1.312	2.19	1.312	2.19	0	0	-	-	-	-	-	-	-	-		
	Stringers															
Stringer1	0.947	1.582	0.947	1.582	0	0	-	-	-	-	-	-	-	-		
Stringer2	0.889	1.485	0.889	1.485	0	0	-	-	-	-	-	-	-	-		
Stringer3	0.889	1.485	0.889	1.485	0	0	-	-	-	-	-	-	-	-		
Stringer4	0.889	1.485	0.889	1.485	0	0	-	-	-	-	-	-	-	-		
Stringer5	0.889	1.485	0.889	1.485	0	0	-	-	-	-	-	-	-	-		
Stringer6	0.947	1.582	0.947	1.582	0	0	-	-	-	-	-	-	-	-		

Appendix 12.4

Opinion of Probable Construction Cost

Colorado River Bridge Rehabilitation
City of Bastrop, Texas
Option 1 Pedestrian Bridge - Full Width Deck
Conceptual Opinion of Probable Construction Cost

Kimley»Horn
2201 West Royal Lane, Irving, Texas 75063
kimley-horn.com 214.420.5600
Texas Firm Registration No. F-928

Date: 02/15/19

Item	Qnty	Description	Unit	Unit Price	Total Price
1	1	Mobilization, Bonds and Insurance	LS	$450,000.00	$450,000.00
2	1	Temporary Traffic Control	LS	$10,000.00	$10,000.00
3	1	Erosion Control Measures	LS	$10,000.00	$10,000.00
4	1	Site Preparation	LS	$50,000.00	$50,000.00
5	1	Containment, Blasting and Painting (Includes special measures to address lead paint)	LS	$3,000,000.00	$3,000,000.00
6	1	Concrete Bridge Deck Removal	LS	$300,000.00	$300,000.00
7	1	Structural Steel Repairs (Includes straightening some members, strengthening some members, and replacing floor beams and stringers. Assumes some isolated bracing of structure required.)	LS	$2,500,000.00	$2,500,000.00
8	1	Structural Concrete Repairs	LS	$100,000.00	$100,000.00
9	1	Bridge Deck Joints	LS	$30,000.00	$30,000.00
10	220	Lightweight Structural Concrete (Reduced width deck replacement)	CY	$1,800.00	$396,000.00
11	1200	Pedestrian Rail	LF	$75.00	$90,000.00
12	1	Site Restoration	LS	$25,000.00	$25,000.00
		Subtotal			$6,961,000.00
		Engineering Fees			$700,000.00
		Contingency		30%	$2,088,300.00
TOTAL CONSTRUCTION COST					**$9,749,300.00**

NOTES:

1 The Consultant has no control over the cost of labor, materials, equipment, or over the Contractor's methods of determining prices or over competitive bidding or market conditions. Opinions of probable costs provided herein are based on the information known to Consultant at this time and represent only the Consultant's judgment as a design professional familiar with the construction industry. The Consultant cannot and does not guarantee that proposals, bids, or actual construction costs will not vary from its opinions of probable costs.

2 All quantities are conceptual only based on design assumptions. Additional design is required for the proposed improvements.

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Colorado River Bridge Rehabilitation
City of Bastrop, Texas
Option 2 Deck Park
Conceptual Opinion of Probable Construction Cost

Kimley»Horn
2201 West Royal Lane, Irving, Texas 75063
kimley-horn.com 214.420.5600
Texas Firm Registration No. F-928

Date: 02/15/19

Item	Qnty	Description	Unit	Unit Price	Total Price
1	1	Mobilization, Bonds and Insurance	LS	$450,000.00	$450,000.00
2	1	Temporary Traffic Control	LS	$10,000.00	$10,000.00
3	1	Erosion Control Measures	LS	$10,000.00	$10,000.00
4	1	Site Preparation	LS	$50,000.00	$50,000.00
5	1	Containment, Blasting and Painting (Includes special measures to address lead paint)	LS	$3,000,000.00	$3,000,000.00
6	1	Concrete Bridge Deck Removal	LS	$300,000.00	$300,000.00
7	1	Structural Steel Repairs (Includes straightening some members, strengthening some members, and replacing floor beams and stringers. Assumes some isolated bracing of structure required. Assumes some additional strengthening of structure for deck park improvements.)	LS	$2,750,000.00	$2,750,000.00
8	1	Structural Concrete Repairs	LS	$100,000.00	$100,000.00
9	1	Bridge Deck Joints	LS	$30,000.00	$30,000.00
10	220	Lightweight Structural Concrete (Full width deck replacement)	CY	$1,800.00	$396,000.00
11	1200	Pedestrian Rail	LF	$75.00	$90,000.00
12	1	Site Restoration	LS	$25,000.00	$25,000.00
		Subtotal			$7,211,000.00
		Engineering Fees			$750,000.00
		Contingency		30%	$2,163,300.00
TOTAL CONSTRUCTION COST					**$10,124,300.00**

NOTES:

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2 All quantities are conceptual only based on design assumptions. Additional design is required for the proposed improvements.

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Colorado River Bridge Rehabilitation
City of Bastrop, Texas
Option 3A Full Bridge Demolition
Conceptual Opinion of Probable Construction Cost

Kimley»Horn
2201 West Royal Lane, Irving, Texas 75063
kimley-horn.com 214.420.5600
Texas Firm Registration No. F-928

Date: 02/15/19

Item	Qnty	Description	Unit	Unit Price	Total Price
1	1	Mobilization, Bonds and Insurance	LS	$80,000.00	$80,000.00
2	1	Temporary Traffic Control	LS	$10,000.00	$10,000.00
3	1	Erosion Control Measures	LS	$10,000.00	$10,000.00
4	1	Site Preparation	LS	$25,000.00	$25,000.00
5	1	Approach Span Demolition	LS	$750,000.00	$750,000.00
6	1	Truss Span Demolition (Includes special measures to address lead paint)	LS	$2,000,000.00	$2,000,000.00
7	1	Site Improvements (to include railing, barricades, and signage)	LS	$50,000.00	$50,000.00
8	1	Site Restoration	LS	$25,000.00	$25,000.00
		Subtotal			$2,950,000.00
		Engineering Fees			$325,000.00
		Contingency		30%	$885,000.00
TOTAL CONSTRUCTION COST					**$4,160,000.00**

NOTES:

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2 All quantities are conceptual only based on design assumptions. Additional design is required for the proposed improvements.

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Colorado River Bridge Rehabilitation
City of Bastrop, Texas
Option 3B Partial Bridge Demolition
Conceptual Opinion of Probable Construction Cost

Kimley»Horn
2201 West Royal Lane, Irving, Texas 75063
kimley-horn.com 214.420.5600
Texas Firm Registration No. F-928

Date: 02/15/19

Item	Qnty	Description	Unit	Unit Price	Total Price
1	1	Mobilization, Bonds and Insurance	LS	$80,000.00	$80,000.00
2	1	Temporary Traffic Control	LS	$10,000.00	$10,000.00
3	1	Erosion Control Measures	LS	$10,000.00	$10,000.00
4	1	Site Preparation	LS	$25,000.00	$25,000.00
5	1	Truss Span Demolition (Includes special measures to address lead paint)	LS	$2,000,000.00	$2,000,000.00
6	1	Substructure Demolition	LS	$150,000.00	$150,000.00
7	1	Site Improvements (to include railing, barricades, and signage)	LS	$150,000.00	$150,000.00
8	1	Site Restoration	LS	$25,000.00	$25,000.00
		Subtotal			$2,450,000.00
		Engineering Fees			$375,000.00
		Contingency		30%	$735,000.00
TOTAL CONSTRUCTION COST					**$3,560,000.00**

NOTES:

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