HISTORIC LAKE PARK ARCH BRIDGE OVER RAVINE ROAD IN-DEPTH INSPECTION REPORT

Structure P-40-576

Milwaukee County Project Number P484-15619

Prepared for

MILWAUKEE COUNTY
DEPARTMENT OF ARCHITECTURE AND ENGINEERING
633 W. Wisconsin Ave., Suite 1000
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July, 2015

Prepared by

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GRAEF Project No. 2015-0032.00

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1. EXECUTIVE SUMMARY

To further investigate bridge deterioration noted in a September 2014 inspection and a follow-up December 2014 inspection, an arm's length In-Depth Inspection of Lake Park's concrete arch bridge over Ravine Road was performed between March 9 and March 27, 2015. The bridge is a 118 ft long open spandrel reinforced concrete arch structure constructed in 1905/1906. It is comprised of reinforced concrete arch ribs, open spandrels, deck, parapets, and vaulted abutments. The bridge carries pedestrian, bicycle, and occasional light maintenance vehicles, but has been closed to traffic on and below due to advanced deterioration since the December 2014 inspection.

Field findings – earlier rehabilitation efforts which had been conducted on the bridge at an unknown time include bridge rail replacement, concrete surface repair, and crack routing/caulking. Most existing concrete patches on the arch ribs and spandrels are tightly adhered to the base concrete. Concrete delaminations and spalls were most prevalent on the west arch and appear to have occurred at previous concrete patches. Caulk in the routed cracks appears cracked and loose in some places. Many concrete patches on the deck underside have spalled off and other patches are delaminated.

Local failures have occurred at the northeast and southeast wingwalls of the north and south abutments. Advanced slope erosion at the southeast wingwall has caused footing undermining with resulting 3" of settlement and rotation resulting in 8" of horizontal movement. At the northeast wingwall, 1" of settlement and rotation resulting in 5" of horizontal movement has occurred, but without evidence of erosion. The northwest and southwest wingwalls exhibit lesser degrees of movement. Extensive soil erosion inside of the south vaulted abutment is occurring.

Milwaukee County survey crews have been monitoring the edge of deck elevations. After 3 cycles of shots, it appears that over its lifetime the main span has experienced minor west side settlement resulting in a slight rotation.

Load ratings – concrete cores taken at the abutment thrust blocks were used to establish the existing concrete strength. This strength was used in load rating calculations to determine structural pedestrian and maintenance vehicle live load capacities of the arch ribs, spandrel, and deck elements. The resulting load ratings indicate that the arch ribs have adequate capacity to resist current code prescribed pedestrian and maintenance vehicle loads. The spandrels and deck do not have adequate capacity. Each has capacity to resist 44% of current code prescribed pedestrian loads. The spandrels can only resist 20%, and the deck can only resist 30% of the current code prescribed maintenance vehicle loads.

Conclusions and recommendations – Reopening Ravine Road and the pedestrian trail below the bridge should only be allowed after loose concrete delamination removal occurs on the superstructure and deck elements. The pathway on the bridge can be opened up to pedestrians and bicyclists only as long as surveys continue to monitor for abutment settlements. Excessive settlements would be cause for bridge closure to all path users.

Light reinforcement, general deterioration, local failures, and inadequate live load capacities suggest that the bridge has reached the end of its useful life. The historic nature of this structure may warrant rehabilitation of as many elements as possible. Since many members are inadequately reinforced, are deteriorated, and have experienced local failures, our first option is to rehabilitate the arch ribs, spandrels, diaphragms, struts, and thrust block elements. The remaining deck, railing, and abutment elements should be removed and replaced. If this approach is chosen, it must be understood that the rehabilitated arch rib, spandrel, and thrust block elements may only have a remaining estimated life of 15 to 25 years – future replacement will require deck, railing, and partial abutment removal and replacement.

In-kind replacement of the entire bridge using modern materials and design provisions is a second option where the architectural features of a new bridge will match those of the original design. A third option is to replace the entire bridge with a longer span steel prefabricated truss bridge. A fourth option is to replace the entire bridge with a longer span prestressed concrete girder bridge with optional decorative precast concrete panels attached to the sides. All elements of a new bridge would have an estimated life of 75 years.

Construction cost estimates – for planning purposes, estimated construction costs of the four options are:

1. Bridge rehabilitation option - $1.8 million. This cost estimate does not include future repairs or replacement in 25 years.
2. Bridge replacement in-kind - $2.6 million.
3. Bridge replacement with a steel prefabricated truss - $1.6 million.
4. Bridge replacement with prestressed concrete girders:
    a. $1.5 million with decorative panels.
    b. $1.4 million without decorative panels.

2. INTRODUCTION AND BACKGROUND

2.1. Construction History

The Lake Park arch bridge conveys recreational, pedestrian, and park service vehicle traffic between the bluffs adjacent to Ravine Road. It was designed in the early 1900s by Ferry & Clas, a Wisconsin architectural firm. George Bowman Ferry and Alfred Charles Clas were partners and designed many buildings and structures listed on the National Register of Historic Places, including this bridge. The bridge was constructed by Newton Engineering Co., and the original plans suggest construction occurred around 1905/1906. It is located in the Milwaukee County's Historic Lake Park, designed by renowned Landscape Architect Frederick Law Olmsted in the late 19th Century.

This reinforced concrete arch bridge spans 118-ft spring line to spring line. Specifically, it is an open spandrel type arch with two arch ribs spaced 13-ft apart and a rise of 18-ft. Vaulted abutments are located at the north and south ends of the arch. Each abutment incorporates a solid concrete thrust block to resist arch rib forces at the spring line, as well as strip footings supporting the vaulted abutments walls. The spandrels, full depth diaphragms between the arch ribs, and vaulted abutment walls support two-way concrete deck slabs. Solid concrete parapets are 3'-8" tall and allow for a 12'-5" clear deck width between the inside faces. Approaches to the bridge are asphalt.

The early proprietary Kahn system of reinforcing steel is used to reinforce the arch ribs, spandrels, and deck in the transverse direction. One-quarter inch diameter rods reinforce the vaulted abutment walls and deck in the longitudinal direction.

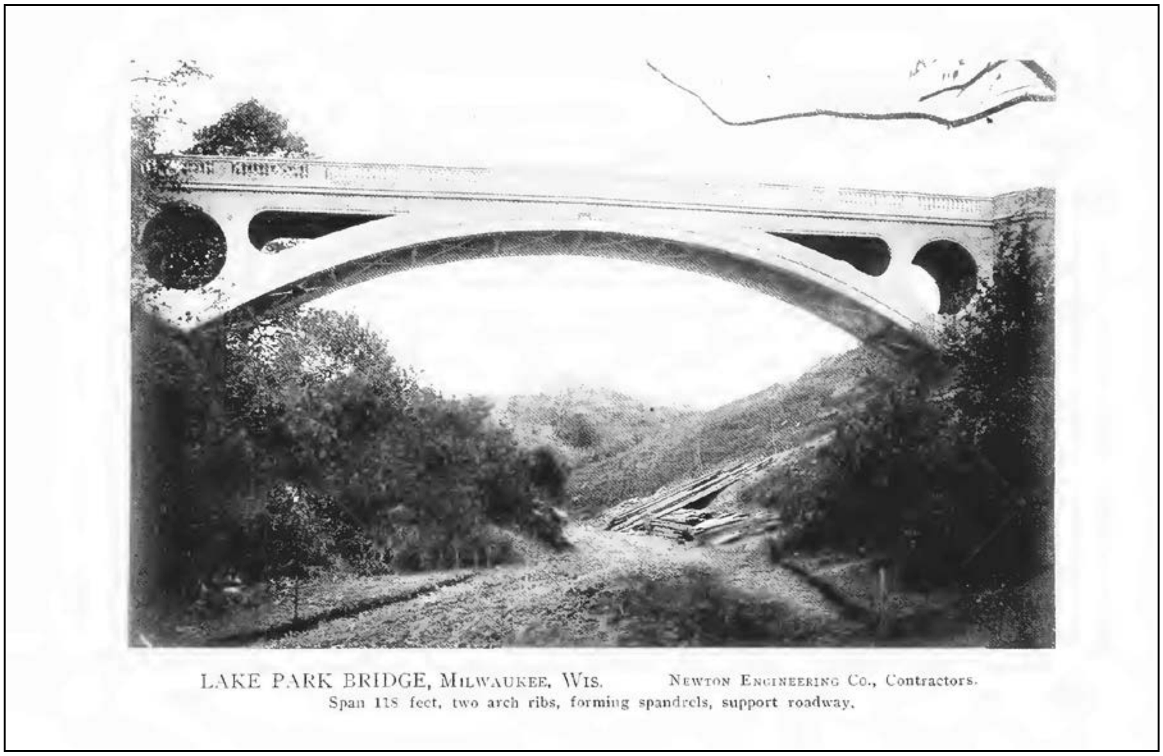


Figure 1: Apparent original construction photo (retrieved from the 1913 Kahn Building Products catalog)

Though the timeline is uncertain, reconfigurations and rehabilitations have taken place during its lifetime. This work includes:

- Replacement of the original open baluster parapets with solid concrete parapet railings
- Removal of the steel lateral bracing between the arch ribs
- Concrete patching on the arch ribs and spandrels
- Concrete surface repair at the thrust blocks
- Downspout installation
- Routing/caulk sealing of cracks on the exterior surfaces of all bridge elements
- Addition of a concrete overlay over the abutment deck slabs

2.2. Load Rating

In 2005, Milwaukee County contracted with GRAEF (dba Graef, Anhalt, Schloemer) to perform a load rating analysis of the bridge. Though the original plans indicated an 80 lb/ft^2 live load, an H-5 (10 kip) vehicle was used to establish the load ratings to mimic a maintenance vehicle moving over the structure. The load ratings were completed according to AASHTO load factor procedures. Only the arch rib and deck elements were load rated. The bridge was load rated using “as new” conditions with no material deterioration assumed.

For the arch ribs, the resulting governing inventory rating was H-8, and the governing operating rating was H-13. The arch rib ratings were controlled by element bending at the abutments/thrust blocks.

The deck inventory rating was H-1 and the operating rating was H-2. As the deck ratings were the lowest, this element controlled the vehicle live load capacity of the bridge. It was not investigated at that time if the bridge had the capacity to resist the 80 lb/ft^2 live load called for on the original drawings. It was recommended at that time to restrict live loading to light park maintenance vehicles or occasional police cars or business owner cars. Further, strengthening the deck was recommended for any future rehabilitation projects.

2.3. Previous Inspections

Review of the WisDOT Highway Structures Information site indicates that Routine Inspections have been conducted on this bridge at 2-year intervals from 1998 to 2010, and Interim Inspections took place in 2004 and 2011. The 2011 Interim Inspection report rated elements for this structure as being fair (deck) to poor (substructure and superstructure) with spalled concrete and slope erosion being the primary areas of concern.

A Structural Safety Inspection report was submitted by K. Singh & Associates, Inc. (a sub-consultant to Collins Engineering) in September 2014. Narrative from this report indicated element conditions of poor (deck), serious (superstructure) and critical (substructure). Issues raised include:

- Extensive spalling of the deck, arch ribs, and spandrel elements
- Abutment foundation settling which has caused cracks in several elements. Several of these cracks were reported to be wide and transversely displaced
- Flexural forces in the arch being transferred into the parapet railing
- Substructure settlement causing water ponding on the deck surface
- Erosion of the embankments/side slopes around the vaulted abutments and compromising the abutment’s integrity

Recommendations included closing the bridge to traffic, installation of a netting system to prevent spalled concrete from falling onto the roadway below, clearing snow from the deck, clearing and grubbing to allow for geotextile fabric installation on the side slopes, and redirection of storm water away from the abutments. A final recommendation for structure removal was offered due to concerns of future bridge instability.

2.4. Current In-Depth Inspection

Results of the Structural Safety Inspection report by K. Singh and Associates prompted Milwaukee County to obtain a second engineering opinion by Malas Engineering in December, 2014. Following a site visit, Malas Engineering recommended that the bridge and Ravine Road be closed immediately. Reopening the bridge was dependent on conducting an In-Depth Inspection of the bridge, computing a new load rating, and obtaining recommendations for rehabilitation or replacement. In addition, it was recommended that Milwaukee County surveyors monitor the bridge for signs of vertical movement.

3. FIELD FINDINGS

The following subsections outline the material conditions of the bridge's primary load carrying members including the arch ribs, spandrels, deck, octagonal shaped vaulted abutment, wingwalls, and abutment thrust blocks. Findings of the bridge secondary elements, which include the diaphragms, struts and parapets, are also provided. See Figure 2 for a definition of these elements.

This report is based on the conditions of the structure that were readily observable at the time of assessment. Nondestructive and destructive testing was only performed at specific areas of concern. Our observations were intended to be an assessment of the visible elements of the structure from areas accessible as described throughout the report.

This report is intended to inventory existing conditions of the observed areas in the winter of 2014-2015, and to provide general recommendations for repair. Conditions observed on the date of assessment may change if noted deficiencies are not corrected.

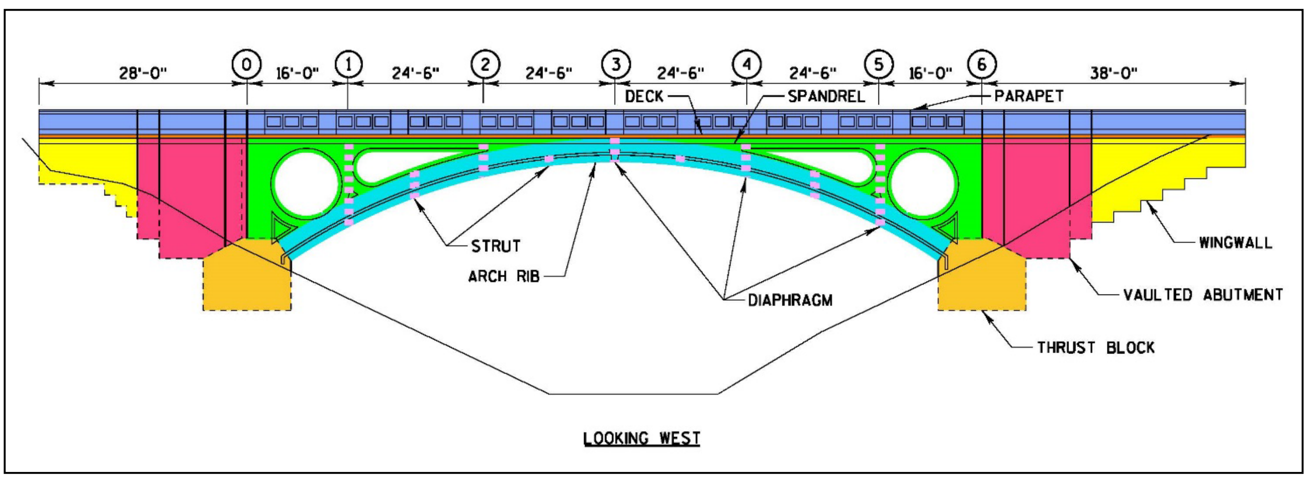


Figure 2: Bridge Elements

3.1. Inspection Methods

Both visual "from the ground" and In-Depth Inspection techniques were used for this structure. An In-Depth Inspection is defined as an arm's length, up close visual inspection of structure elements to identify deficiencies not readily detectible from the ground. This method was used for the arch ribs, spandrels, deck underside, and other elements not readily accessible without specialized equipment. Visual inspection was used for the remaining elements such as the parapet outside faces and high portions of the abutments. Present at each inspection was a two-person crew consisting of either two WisDOT certified bridge inspectors or a certified bridge inspector and an assistant. Access to the arch underside was provided with an 80-ft aerial lift, and a ladder was used to reach openings allowing entry into the vaulted abutment.

On the bridge arch ribs and spandrels, surfaces of these concrete superstructure elements were visually inspected for defects such as cracks, delaminations, spalls, and loose concrete patches. In addition, hammer tapping was performed on all surfaces of these elements to detect delaminations and check the soundness of concrete patches. Concrete impact echo nondestructive testing was also performed to estimate the thickness of delaminations and loose concrete patches, as well as to spot check surfaces appearing sound for deeper flaws.

The remaining concrete surfaces were primarily visually inspected with limited hammer tapping used to spot check suspect areas for delaminations. Hammer tapping was also used to remove loose delaminations that were observed on the underside of the deck over the north hiking path. Some delaminations were also removed over the roadway. Not all delaminations were removed, as many were tightly adhered and would be best removed by a contractor with the proper equipment.

Concrete deterioration was documented using sketches and photos. Several crack widths were measured, marked onto the concrete, and recorded in our field notes. Quantities of spalls and delaminations were visually estimated. An overall view of these findings can be seen in Appendix A.

3.2. Arch Ribs

The arch ribs are in fair to poor condition. Existing concrete patches indicate earlier rehabilitation work, and many of these patches are still sound. Existing spalls and moderate steel section loss of the exposed rebars have slightly reduced the elements' structural capacity.

Review of the original design drawings, and verification in the field, reveals that each arch rib is reinforced with four 1" x 3" Kahn bars (one in each corner). In addition, two 1" diameter Trusscon bars (one each top and bottom) are placed between the spring line and panel points 1 and 5, changing to two ¾" diameter Trusscon bars between panel points 1 to 2 and 4 to 5. Previous rehabilitation work includes patching spalls with concrete and applying a skim coat of a cementitious material over many surfaces to seal narrow cracks and improve the aesthetics.

3.2.1. Concrete Patches, Delaminations and Spalls

Both arch ribs contain many concrete patches. These earlier repairs are present along approximately 75% of the length of the rib soffits (bottom surfaces, Figure 3). Patches are also common on the rib soffit at all four spring lines (Figure 4). The soffit patches were apparently needed to fix spalls that had occurred at the lower corners. The rib backs (top surfaces) are exposed only along the circular and teardrop shaped spandrel openings, and patches exist along about 50% of the length of these surfaces. Fewer concrete patches were found on the rib vertical surfaces. Most were located near the arch crown between panel points 2 to 4 on the inside faces (Figure 5).

Figure 3: Concrete patches on east arch rib soffit

Figure 4: Concrete patches on arch rib soffits at the north spring line.

Figure 5: Concrete patches on east arch rib inside vertical face near the crown

Delaminations and spalls were most prevalent on the west rib (Figures 6 and 7):

- 3 soffit delaminations totaling 10 ft^2
- 10 soffit spalls totaling 48 ft^2
- 2 back spalls on the rib back totaling 16 ft^2 at the north teardrop spandrel opening
- 1 spall on the inside vertical face totaling 4 ft^2

Along the east rib there were:

- 5 soffit delaminations totaling 19 ft^2

- 4 soffit spalls totaling 5 ft^2
- 1 delamination on the outside vertical face totaling 4 ft^2
- 2 spalls on the outside vertical face totaling 5 ft^2
- 3 delamination on the inside vertical face totaling 13 ft^2
- 2 spalls on the inside vertical face totaling 5 ft^2

Figure 6: West arch rib soffit with spalls and exposed reinforcing steel

Figure 7: West arch rib spall with exposed Kahn bar reinforcement (note the bent up shear reinforcement)

When hammer tapped, most of the patches sounded well adhered to the base concrete, suggesting thick patches that encapsulate the reinforcing steel. Approximately 10% to 20% of the patches sounded hollow when tapped, indicating debonding from the base concrete and possibly no encapsulation of the reinforcing steel.

3.2.2. Cracks

The arch ribs contain a few routed/caulked cracks, generally concentrated near the spring line (Figure 8). It is believed most of the routed/caulked cracks are at concrete cold joints. The cold joints were formed at concrete lift boundaries when new concrete was placed against cured concrete during the original construction. Most of the caulk is cracked, loose or missing and is ineffective in sealing out water.

Figure 8: East arch rib north spring line with routed/caulked cracks

Several unsealed longitudinal hairline (<0.012”) to narrow (0.012” to 0.05”) to medium (0.05” to 0.1”) cracks occur along the arch ribs. The cracks generally occur randomly along the length of each arch. Cracks on the rib soffits typically occur at the existing concrete patches (Figure 9). Though most were scattered, there was some consistency of the longitudinal crack location on the rib vertical surfaces between panel points 1 and 2. These occurred on both vertical faces of the west rib, and on the inside vertical face of the east rib (Figure 10). Very few transverse cracks were noted. Three were located on the east rib and two on the west rib.

Figure 9: West arch rib soffit with unsealed 0.040" longitudinal crack in concrete patch

Figure 10: East arch rib inside face with unsealed 0.080” longitudinal crack near the top

Map cracks occur near the spring lines of all arch ribs. These map cracks coincide with concrete patches found on the rib soffits.

Four unsealed cracks were measured on the east arch. These ranged from 0.02” to 0.08” wide. Four unsealed cracks were measured on the west arch which ranged from 0.01” to 0.07” wide. On the west arch rib between panel points 1 and 2, a narrow longitudinal crack is present approximately mid-height on both the east and west faces. This could be indicative of a through thickness crack. It was measured at 0.01” thick on the west face. See Appendix A for the crack locations, and where applicable, the width measurement.

3.3. Spandrels

The spandrels are in fair to poor condition. Existing concrete patches indicate earlier rehabilitation work, and many of these patches are still sound. Existing spalls and steel section loss of the exposed rebars have reduced the southeast spandrel’s structural capacity.

The top longitudinal member of each spandrel is reinforced with one ¾” x 2” Kahn bar along the bottom surface above the circular spandrel openings (panel points 0 to 1 and 5 to 6). Above the teardrop shaped openings (panel points 1 to 2 and 4 to 5), two 1” x 3” Kahn bars are placed along the bottom surface to reinforce the longitudinal member. Previous rehabilitation work includes patching spalls with concrete and applying a skim coat of a cementitious material over many surfaces to seal narrow cracks and improve the aesthetics.

3.3.1. Concrete Patches, Delaminations and Spalls

The spandrels above both arch ribs contain several concrete patches. On the exterior surfaces, patches are common at the tip of the teardrop shaped openings. Patches also occur along the concrete cold joints between the spandrel openings (Figure 11). On the east spandrel, patches exist just underneath the deck overhang between panel points 2 and 4 for about 50% of this length.

Figure 11: Concrete patch at east spandrel outside face

On the spandrel interior surfaces, patches tend to be concentrated near the deck between panel points 2 and 4. A full length patch runs along the bottom of the longitudinal member above the teardrop opening between panel points 1 and 2 of the west arch.

Along the east spandrel there were:

- 2 delaminations on the outside vertical face totaling 8 ft^2
- 1 spall on the outside vertical face totaling 3 ft^2
- 1 spall on the inside vertical face totaling 4 ft^2

In addition, the east spandrel between panel points1 and 2 has a long spall with exposed reinforcing steel (Figure 12). It is approximately 10' long and located on the longitudinal member underside above the teardrop shaped spandrel opening. Several pieces of loose concrete were removed during the inspection. There were 6 delaminations and 3 spalls noted on the remainder of the spandrels. Four of these defects occur at concrete patches.

Along the west spandrel there were:

- 3 delaminations on the outside vertical face totaling 19 ft^2

See Appendix A for locations of the delaminations and spalls.

Figure 12: Large spall at east spandrel opening with exposed Kahn bar reinforcing steel

When hammer tapped, most of the patches sounded well adhered to the base concrete, suggesting thick patches that encapsulate the reinforcing steel. Approximately 10% of the patches sounded hollow when tapped, indicating debonding from the base concrete and possibly no encapsulation of the reinforcing steel.

3.3.2. Cracks

The spandrels contain many routed/caulked cracks. It is believed the majority of these routed/caulked cracks occur at concrete cold joints. Most of the caulk is cracked, loose or missing and is ineffective in sealing out water. These cracks are common around the circular spandrel openings between panel points 0 to 1 and 5 to 6 (Figure 13). The cracks typically reflect completely through the spandrel wall thickness. Other typical crack locations are near the top of the spandrel inside and outside faces where the spandrel/deck slab cold joint exists (Figure 14). Between panel points 2 and 5 of the east spandrel, about 33% of the spandrel/deck slab interface is caulked. Near the arch spring lines, routed/caulked cracks follow the rib back/spandrel interface on the east and west rib exterior vertical surfaces.

Figure 13: East spandrel routed/caulked cracks

Figure 14: East spandrel routed/caulked crack between top of spandrel and deck

Several unsealed hairline (<0.012”) to wide (> 0.1”) cracks occur on the spandrels. These cracks occur around the circular spandrel openings between panel points 0 to 1 and 5 to 6, and typically reflect completely through the spandrel wall thickness (Figures 15 and 16). Near the arch spring lines, cracks coinciding with concrete patches follow the east and west rib back/spandrel interface on the inside surface between panel points 0 and 1. On the outside faces of the east and west spandrel andbetween panel points 2 and 3, a 1/8” wide crack exists between the spandrel top and deck underside. On the spandrel inside faces, unsealed cracks are sporadically located near the spandrel/deck slab interface.

Figure 15: West spandrel with unsealed 0.070" longitudinal crack between spandrel and arch rib

Figure 16: West spandrel south end with unsealed 0.030" crack in concrete patch between openings

Four unsealed cracks were measured on the east arch. These ranged from 0.05" to 3\16" wide. The 3/16" wide crack is located between the spandrel openings at panel point 1. Six unsealed cracks were measured on the west arch which ranged from 0.01" to 1/8" wide. Two 1/8" wide cracks were present at the circular spandrel opening between panel points 0 and 1. See Appendix A for the crack locations, and where applicable, the width measurement.

3.4. Superstructure Diaphragms and Struts

The diaphragms and struts are in good to fair condition. Full depth concrete diaphragms are present between the east and west arch ribs/ spandrels at panel points 1, 2, 3, 4, and 5. Concrete struts between only the east and west arch ribs are present at panel points 1.5, 2.5, 3.5, and 4.5. Both of these secondary superstructure elements serve to provide lateral stability to the arch ribs and spandrels, and lateral wind load resistance to the bridge. As barely seen in the photograph of Figure 1, evidence of the original lower lateral steel bracing angles were present after the structure was built. These steel angles, connected directly to the bottom ledge of the arch, were used to provide lateral stability directly to the arch ribs. Since their removal, the concrete diaphragms, struts, and deck provide the structure's lateral bracing.

The diaphragms contain many routed/caulked cracks. It is believed the majority of these routed/caulked cracks occur at concrete cold joints (Figure 17). Earlier rehabilitation work also includes several concrete patches to both the diaphragms and struts. The patches generally occur at the bottom corners of these elements, and in some cases along the entire bottom edge. Patches were noted on all diaphragms and all struts except for strut 3.5.

Figure 17: Diaphragm at panel point 5 with routed/caulked cracks at concrete cold joints

Spalls are present on several of the secondary members, typically along the bottom edges or bottom corners (Figure 18). Members containing spalls include diaphragms 3 and 4, and struts 1.5, 2.5, and 3.5.

Figure 18: Concrete patches and spalls on the crown diaphragm at panel point 3

3.5. Deck

The deck elements over the superstructure and substructure units are in poor condition. Existing concrete surface patches on the deck underside indicate earlier rehabilitation work, though many of the patches have debonded or spalled off. Newer delaminations and spalls also exist. Steel section loss of the exposed reinforcing steel has reduced the deck's structural capacity.

The original 6" thick concrete deck is supported on 4 edges at each bay of the superstructure. Specifically, the spandrels support the deck's east and west edges while diaphragms at each panel point support the north and south edges. Review of the original design drawings reveals that the deck is reinforced with ½" x 1-½" Kahn bars spaced at 18" on center in the east-west direction, and ¼" diameter rods spaced at 18" on center in the north-south direction.

Over the abutments and wingwalls (collectively with the arch thrust block the "substructure"), the deck is supported along two edges only. At the abutments, 12" x 17" beams in line with the arch ribs support the east and west edges, whereas the wingwalls themselves support the deck. Reinforcement is the same as that on the superstructure.

Previous deck rehabilitation work includes placing a concrete overlay over the substructure. A 6-ft long concrete transition ramp was placed on the first bay between panel points 0 to 1 and 5 to 6. It is unknown if an overlay had ever been placed on the superstructure deck.

3.5.1. Top Side

Regular transverse cracks are present along the bridge length between panel points 0 to 6, spaced at about 4' on center. Approximately half of these cracks have been previously routed/caulked, with the remainder being unsealed narrow cracks. Only four spalls are present on the top surface, and three areas of scaling were noted (Figure 19).

Figure 19: Small spalls on top of deck and typical cracks

Over the substructure units, the deck top side is in good condition. Between the south end wingwalls, two transverse routed/caulked cracks and one unsealed transverse crack are present. Between the north end wingwalls, four unsealed transverse cracks are present along with one spall on the top surface.

3.5.2. Soffit

Along the bridge length between panel points 0 to 6, concrete patches indicate earlier rehabilitation work. Some of these patches have spalled or delaminated from the base concrete (Figures 20 and 21). In addition, newer delaminations and spalls also exist (Figure 22). The spalls have exposed the deck's bottom mat of reinforcing steel which exhibits laminate rust.

Total quantity of spalls over the superstructure is approximately 225 ft^2. Spalls are extensive between panel points 1 to 2, 3 to 4, and 5 to 6, or 50% of the deck panels. The panel between points 5 to 6 is located over a pedestrian hiking path and contains a large region of delaminated concrete as well. Loose delaminations in this bay were removed where possible during the inspection but other more tightly adhered delaminations remain. About 105 ft^2 of spalls were noted between panel points 2 and 4. See Appendix A for the crack and defect locations on the deck top side.

Figure 20: Deck soffit concrete patch and spalls with exposed Kahn bar reinforcement between panel points 1 and 2

Figure 21: Deck soffit with spalled concrete patch and exposed Kahn bar reinforcement

Figure 22: Deck soffit with spalled concrete patch and exposed Kahn bar reinforcement between panel points 3 and 4

Over the substructure units, as inspected from the inside the vaulted abutments, the deck underside has spalls with exposed reinforcing steel (Figures 23 to 26). With one exception, there does not seem to have been prior rehabilitation work performed, likely due to access difficulties. The one exception occurs at the north octagonal vaulted abutment where about a 3' x 3' area of leave-in-place form work was found, possibly to repair full depth deterioration.

Figure 23: Deck soffit spall with exposed Kahn bar reinforcement between the wingwalls of the south abutment

Figure 24: Deck soffit condition within the octagonal region of the south vaulted abutment

Figure 25: Deck soffit spalls with exposed Kahn bar reinforcement between the wingwalls of the north abutment

Figure 26: Deck soffit spalls with exposed Kahn bar reinforcement within the octagonal region of the north vaulted abutment

Total quantity of spalls over the substructure units is approximately 200 ft^2. About 120 ft^2 occurs at the south vaulted abutment and wingwalls. There is about an 8' x 10' spall at the deck's extreme south end between the wingwall tips. At the north vaulted abutment and wingwalls, about 80 ft^2 of spalls exist. There is also a region of exposed reinforcing steel at the north half of the octagonal shaped vaulted abutment, likely due to insufficient concrete cover. See Appendix A for the crack and defect locations on the deck soffit.

3.6. Abutment Thrust Blocks

The abutment thrust blocks are in fair condition. The thrust block is a solid mass of concrete used as the shallow foundation for the superstructure. It delivers the vertical gravity loads and horizontal arch thrust loads into the surrounding soil, controlling the superstructure vertical settlement and arch rib spreading. Each thrust block is approximately 10' H x 16' L x 24' W and was originally designed without reinforcing steel.

Existing concrete surface patches indicate earlier rehabilitation work. As measured through concrete cores, patch thicknesses of the north thrust block are approximately 6" between the arch ribs, 2" on the west side and 3" on the east side. Patch thicknesses of the north thrust block are approximately 12" between the arch ribs, 9" on the west side and 6" on the east side.

The surface patches appear to be reinforced with welded wire fabric. The surfaces have extensive shrinkage cracks and are debonding from the base concrete. The north thrust block concrete facing has wide and extensive map cracks (Figure 27). The south thrust block concrete facing has a 1/8” wide vertical crack and an approximate 1/8” to ¼” wide horizontal crack (Figure 28). The east and west sides of the thrust blocks are also exhibiting deterioration in the form of cracks. On the north thrust block, map cracks are present (Figure 29), while on the south thrust block, 3 longitudinal cracks exist on the east face and 2 longitudinal cracks exist on the west face (Figure 30). The deterioration does not, however, affect the function of the thrust blocks. See Appendix A for the crack locations.

Figure 27: Extensive map cracks in the surface repair concrete at the north thrust block

Figure 28: Wide 1/8" + cracks in the surface repair concrete at the south thrust block

Figure 29: Map cracks in the concrete patch at the north thrust block west face

Figure 30: Longitudinal cracks in the concrete patch at the south thrust block west face

3.7. Vaulted Abutment Walls and Wingwalls

The vaulted abutment walls and wingwalls are in serious to critical condition. Existing concrete patches and routed/caulked cracks indicate earlier rehabilitation work, and many new unsealed cracks have developed since. Local failures have occurred at four locations. See Figure 2 for limits of the vaulted abutment walls and wingwalls.

3.7.1. Exterior

All abutment walls and wingwalls contain numerous routed/caulked and unsealed cracks (Figures 31 and 32). Many of the horizontal routed/caulked cracks occur at concrete cold joints. Earlier rehabilitation work also includes several concrete patches to both the abutment walls and wingwalls.

Figure 31: Extensive routed/caulked cracks on the east face of the north vaulted abutment wall

Figure 32: Extensive routed/caulked cracks on the west face of the north vaulted abutment wall

There are four local failures listed beginning with the worst conditions:

1. Southeast wingwall and south side of vaulted abutment wall – approximately 10’ of erosion along the side slope has caused a localized washout hole (Figure 33). Both the erosion and washout are along the wingwall. At the washout, undermining has occurred below the wingwall access opening. The loss of vertical support has allowed a large section of the wingwall to crack and drop vertically. Active soil pressure on the exterior and lack of passive soil pressure on the interior has pushed on the wall, caused the wingwall to rotate towards the vault’s interior. The top of the failed wall has moved approximately 8” horizontally and 3” vertically.

Figure 33: Washout and wall failure at the southeast wingwall (looking northwest)

2. Northeast wingwall and east side of vaulted abutment wall – soil consolidation or erosion on the interior of the vaulted abutment interior has allowed the wingwall to settle after development of a horizontal and vertical crack (Figure 34). Though the wingwall strip footing was not exposed during the inspection to confirm this hypothesis, active soil pressure from the exterior has pushed on the wall, causing it to rotate towards the vault’s interior. At the north end of the octagonal abutment, the wingwall top has moved approximately 5” horizontally and 1” vertically (Figure 35).

Figure 34: Wall and abutment wall failure at the northeast wingwall (looking south)

Figure 35: 1" gap between top of wall and deck at northeast wingwall

3. North abutment wall, northwest quadrant - soil consolidation or erosion on the interior of the vaulted abutment interior has allowed a portion of the abutment wall to settle after development of horizontal and vertical cracks (Figure 36). Though the wingwall strip footing was not exposed during the inspection to confirm this hypothesis, active soil pressure from the

exterior has pushed on the wall, causing it to rotate towards the vault's interior. Caulk covers the width of the horizontal crack so an accurate measurement of vertical settlement could not be made. However, at the north side of the octagonal abutment, the fractured panel of wall top has moved approximately 1-½" horizontally (Figure 37).

Figure 36: North abutment wall, northwest quadrant (looking southeast)

Figure 37: 1 ½" of horizontal movement at north abutment wall northwest quadrant (looking southwest)

4. South abutment wall, northwest quadrant – there is a large concrete patch adjacent to the access opening that is part of an earlier repair effort (Figure 38). This area has wide unsealed vertical and horizontal cracks, and the patched region is bulged outwards. Soil aggradation inside of the vaulted abutment was evident at the grated opening. This aggradation is a result of scour along the wingwall inside faces farther upslope. Soils washed away from the wingwalls and were deposited inside of the octagonal shaped vaulted abutment. The deposited soil is imparting active pressure from the inside of the vault and causing the observed wall movement/bulging (Figure 39).

Figure 38: South abutment wall, northwest quadrant (looking southeast)

Figure 39: South abutment wall northwest quadrant with outward bulging (looking southwest)

3.7.2. Interior of the Vaulted Abutments

All abutment and wingwalls contain unsealed cracks. Many of the horizontal cracks occur at concrete cold joints.

Inside of the south vaulted abutment, extensive erosion has taken place along the wingwalls, leaving a central mound of soil between and soil aggradation within the octagonal vaulted abutment downslope (Figure 40). The erosion is especially severe along the east wingwall where an approximately 15' length of the wingwall's stepped strip footing has been exposed and undermined (Figure 41). As was seen from the exterior, the loss of vertical support has allowed a large section of the wingwall to crack, settle, and rotate (Figures 42 and 43).

Along the west wingwall, an approximately 1-½" wide horizontal crack exists at the north end of the wingwall (Figure 44). This crack continues along the west walls of the octagonal abutment walls. Other horizontal cracks were noted near the top of the wingwall and near the bottom of the west abutment walls. Vertical cracks exist at the north and west abutment walls. Less severe undermining is present along the west wingwall, although it appears that what appears that a concrete repair was used to fill a previous area of undermining adjacent to the abutment wall (Figure 45).

At the southeast and southwest quadrants of the south abutment's octagonal region, an apparent rehabilitation was performed by casting a block of concrete along the wall (Figure 46). This may have been done to fill erosion holes or provide lateral support to control wall movement caused by exterior soil active pressures. An approximate 1” gap exists between the wall and the west side cast-in-place block.

Figure 40: South abutment interior with major erosion and failure of east wingwall (looking south)

Figure 41: Erosion and undermining of exposed strip footing of the south abutment east wingwall (looking south)

Figure 42: Top of failed portion of south abutment east wingwall with 3" drop provides no support for deck above (looking southeast)

Figure 43: 8" shift in failed portion of south abutment lower east wingwall (looking north)

Figure 44: Wide cracks in south abutment west wingwall (looking northwest)

Figure 45: Apparent cast-in-place repair of west wingwall undermining at south abutment (looking southwest)

Figure 46: Apparent previous cast-in-place rehabilitation at southwest quadrant of octagonal region of the south vaulted abutment (looking south)

Inside of the north vaulted abutment, several cracks are present on the wingwalls and octagonal abutment walls.

Strip footings were not exposed on the inside of the north vaulted abutment. As was already seen from the exterior, the east wingwall and abutment walls contain wide vertical and horizontal cracks accompanied with wall movement. After horizontal and vertical wall cracks developed, portions of the wall were allowed to settle (Figure 47). As the wingwall section settled, the top was allowed to rotate/move 3” to 5” towards the inside of the vaulted abutment (Figure 48). On the east wall of the octagonal abutment, a section of wall near the bottom moved 6” horizontally. On the west octagonal abutment walls, vertical cracks occur generally at the corners of the wall. A single vertical crack is present on the south abutment wall. See Appendix A for the crack and other defect locations on the vaulted abutment and wingwall interiors.

Figure 47: 1 ½" settlement of failed portion of north abutment east wingwall provides no support for deck above (looking east)

Figure 48: Interior of north vaulted abutment with 5" horizontal gap between failed east wingwall and abutment wall (looking northwest)

3.8. Parapets

The existing solid concrete parapets are in fair condition. Plan and photographic evidence show that they are replacements to the open concrete or carved stone rails that existed in the early life of the bridge. One of the concrete balusters, as shown on the original drawing, was retrieved from inside the vaulted abutment and was brought back to GRAEF's office.

Both the east and west parapets contain routed/caulked cracks and unsealed cracks, with the heaviest concentrations occurring above the vaulted abutments and wingwalls (Figures 49 and 50). Several of the unsealed cracks contain light leaching. Cracks are primarily oriented vertically and occur on the inside and exterior surfaces.

Figure 49: Northwest wingwall parapet showing typical routed/caulked cracks on inside face

Figure 50: North abutment east parapet showing typical routed/caulked cracks

Randomly located spalls were noticed on the parapets. They were generally located on the inside surfaces at the base, and it is suspected that freeze/thaw action of ponded water is the contributing factor (Figure 51). Approximately 75-ft of spalls were recorded along the curb line at both parapets. Random spalls were also found on the exterior surfaces.

Figure 51: Northeast wingwall parapet showing typical spalls along curb line

3.9. Open Expansion Joints

The superstructure is tied to the substructure by way of continuous longitudinal Kahn bars between the arch rib spring lines and thrust blocks. Though filled with caulk, there is an open joint between the spandrel and abutment wall at panel points 0 and 6. An open gap exists between the superstructure parapet and abutment parapet (Figure 52). The gap width and vertical distance difference was measured at the ends of each joint:

- Northeast
  - Horizontal gap parapet bottom = 1”
  - Horizontal gap parapet top = 1-¼”
  - Vertical difference parapet top = 3/32” (superstructure higher than abutment)
- Northwest
  - Horizontal gap parapet bottom = 1”
  - Horizontal gap parapet top = 1-¾”
  - Vertical difference parapet top = 5/8”
- Southeast
  - Horizontal gap parapet bottom = 1-5/8”
  - Horizontal gap parapet top = 1-11/16”
  - Vertical difference parapet top = ½”

- Southwest
    - Horizontal gap parapet bottom = 2-¼”
    - Horizontal gap parapet top = 2-13/16”
    - Vertical difference parapet top = ¾”

Figure 52: Open expansion joint at parapet and deck

3.10. Downspouts

To drain the walking surface, 2” diameter floor drains are located along both edges of the deck at the curb line. These lead to PVC downspouts that empty onto the ground below. The downspouts themselves are in good condition; however, several steel brackets used to secure the downspout to the superstructure diaphragms have completely rusted through. (Figure 53).

Figure 53: Failed downspout steel bracket

3.11. Site Drainage

Both the north and south approaches slope down towards the structure. Although a storm drain exists at the south approach, severe erosion has occurred along the southeast wingwall (Figures 54). Southeast slope runoff directed towards the southeast wingwall has created a large washout hole (Figure 55). Water runs into this hole, under the wingwall strip footing, and into the vaulted abutment. This has caused erosion of the soil along the wingwall's inside face and undermining of the footing. Similar erosion has occurred along the southwest wingwall's inside face, though the source of the water could not be determined and the footing has not been undermined. Sandbags have recently been placed upslope of the southeast wingwall to help redirect rain runoff away from the washout hole.

Less severe erosion was noted at the north abutment's south face. Approximately 8" of erosion has occurred since the concrete facing was placed on the thrust block (Figure 56).

Figure 54: Drainage inlet at south approach

Figure 55: Sandbags to redirect runoff around the southeast wingwall

Figure 56: Slope settlement in front of north abutment thrust block

4. MATERIAL TESTING

Destructive and nondestructive material testing was performed as part of this In-Depth Inspection. Giles Engineering Associates, Inc. performed concrete compressive strength testing, concrete imaging radar scanning, and concrete impact echo testing. Their report is included in Appendix B.

Sampling and testing for asbestos within the existing caulk was performed by Jackson/McCludden, Inc. The report is included in Appendix C.

4.1. Concrete Compression Testing

To obtain concrete strengths needed to perform the load ratings, 6 concrete core samples were taken from the thrust blocks. At each thrust block, a 4” diameter core was taken at the front face and one at each side face. Core sampling was advanced to a depth of 24”.

Surface patch material was recovered at each core. Patch thicknesses ranged from approximately 2” to 6” at the north abutment, and 9” to 12” at the south abutment. The original concrete was found to be generally sound. Some fracturing and moderate honeycombing was present in all of the cores.

Unconfined compression testing using ASTM D39 procedures was performed on all 6 specimens. Four of the tests were performed on original concrete obtained from cores taken from the thrust block east and west sides, while 2 of the tests were performed on the surface patch material taken from the north and south faces.

Concrete strengths of the original concrete ranged from 1,595 psi to 3,166 psi, averaging 2,255 psi. South thrust block concrete was about 60% weaker (1,679 psi average) than the north thrust block concrete (2,830 psi average). More extensive honeycombing within the south thrust block core samples may account for the lower average strength. Concrete strengths of the patch material concrete were very high, ranging from 7,543 psi at the south abutment to 9,822 psi at the north abutment, averaging 8,683 psi.

4.2. Concrete Imaging Radar Scanning

Concrete Imaging Radar (CIR) Scanning was performed to confirm reinforcement placement that is shown on the original construction drawings. CIR was performed from the ground on the arch rib spring lines, thrust blocks, and at spot locations of the vaulted abutment walls.

At the arch rib spring line and east/west sides of the thrust blocks, CIR indicated the longitudinal and shear Kahn bar reinforcing steel was placed in general conformance with quantity and locations shown on the original construction drawings. Three longitudinal bars were indicated on the arch rib soffit at all spring lines. The shear reinforcing was found to be more random than as indicated on the original drawings, with spacing varying from about 6" to 14". The orientation was generally transverse to the longitudinal bars as opposed to bent at a 45° angle. Some of the shear reinforcement was not detected though this could be attributed to bar depth or shielding by shallow bars.

Concrete resurfacing of the thrust blocks was performed as part of earlier bridge repair efforts. On surfaces between the arch ribs, CIR revealed vertical reinforcement at approximately 10" on center was placed at the north thrust block. A reinforcing grid, possibly welded wire fabric, was placed at the south thrust block. Reinforcing spacing at the south thrust block is 18" to 20" on center in each direction.

Horizontal steel bars were indicated at the vaulted abutment walls. Spacing ranged from approximately 10" to 20" on center, with a minimum spacing of 7" indicated on the south abutment's west side and a maximum spacing of 47" on the north abutment's east side. A 12" spacing was specific on the original construction drawings, though it was not stated if this reinforcement was to be placed vertically, horizontally, or both. Vertical reinforcing steel was not detected with the CIR.

4.3. Impact Echo Concrete Integrity Testing

Impact Echo (IE) tests were performed to verify the soundness of the base concrete and to estimate the thicknesses of delaminations found using hammer tapping. A total of 17 tests were performed, with 3 of these conducted on sound concrete for equipment calibration and spot checking the concrete's internal soundness. Of the remaining 14 tests, 2 indicated no delaminations were present. Quantifiable delamination thicknesses between 1½" and 3" were obtained at 7 test locations, while the remaining 5 tests detected the delamination but were unable to focus on a specific thickness due to multiple peak frequency responses.

4.4. Asbestos

Several hundred feet of caulk was used to fill routed cracks on this bridge. All samples tested negative for detectable asbestos.

5. SURVEY

Milwaukee County survey crews established benchmarks and points to monitor the bridge for vertical and horizontal movements. A baseline survey was taken on March 9, 2015, and follow up surveys were taken on April 2 and April 23, 2015. All survey data can be reviewed in Appendix D.

Points established for the survey monitoring are the deck elevations on the outsides of the east and west parapets. Specifically these points are located:

- South and north ends of the abutment (wingwall tips)
- South and north corners of the vaulted abutment octagons
- Midpoint of the vaulted abutment octagons
- 1/10 points along the main span

Early findings show that the east side of the bridge is generally higher in elevation than the west side. Range of the elevation variations show that along the superstructure, the east side is 0.02 ft lower (one location only) to 0.09 ft higher than the west side. At the north abutment, the east side is 0.30 ft lower (maximum at the wing tips) to 0.07 ft higher than the west side, and at the south abutment the east side is 0.26 ft higher maximum than the west side.

Elevation difference of the same point taken at different dates exhibited some variations. On the superstructure, the maximum elevation difference was 0.08 ft located at the east parapet/north end of the deck. Maximum elevation difference was 0.04 ft at the north abutment and 0.06 ft at the south abutment.

6. LOAD RATINGS

The load ratings were completed according to AASHTO Load Resistance Factor Rating (LRFR) procedures which are the current state-of-the-art. The 2005 load ratings used Load Factor

Rating (LFR) procedures. The computer model used for the 2005 load ratings was updated to adjust for LRFR load factors, and an H-10 maintenance vehicle live load as dictated by the AASHTO LRFD Guide Specification for the Design of Pedestrian Bridges, 2nd ed. (Guide Specification), for pedestrian bridge clear widths over 10-ft. A 90 psf pedestrian live load was added. This load is prescribed within the Guide Specification and pattern loading was used to maximize the axial and bending effects along different parts of the arch.

6.1. Materials

As part of the 2005 load rating analysis, several assumptions were made with regards to the material. For consistency, these assumptions are used for the LRFR analysis. Reinforcing steel in the original drawing specifications indicate the Kahn bars and round Truscon bars have an allowable design stress of 16,000 psi. Based on a telephone conversation with CRSI, it was suggested that a yield stress of 33,000 psi be used for this reinforcement.

The drawings also indicate an allowable concrete compressive stress of 400 psi. Review of literature around the time of construction, as well as early ACI Code values suggest that allowable concrete compressive stresses in columns used for designs was $0.25f'_c$ (factor of safety = 4). This would give an ultimate concrete compressive strength used for the original design of f'_c = 1,600 psi. However, part of the 2015 GRAEF inspection scope included taking concrete cores from both thrust blocks. Compressive testing of the original concrete resulted in strengths of 2,494 psi and 3,166 psi at the north abutment and 1,595 psi and 1,763 psi at the south abutment, resulting in a total average of 2,254 psi. Lower strengths at the south thrust block were likely due to the honeycombing observed within the samples.

Large honeycombs were not observed on the arch rib surfaces. If large honeycombs are contained within the arch rib volume, they would likely be localized and not distributed across the entire rib cross section. The concrete strength was therefore assumed to be f'_c = 2,000 psi for the rating analysis which is 25% higher than what we presume was used for the original design.

6.2. Modeling

A 2-D structural computer model was built using Visual Analysis design software to analyze the arch rib, and a finite element model was used to analyze the deck for vehicle loading.

The 2-D structural model used beam elements to create the arch ribs and diaphragms. Since only the arch rib results were of interest for this modeling, the parapets were also modeled as beam elements though it is understood they are not structural elements. Modeling the parapets in this fashion allowed their self-weight to be accounted for automatically, and allowed loading to the arches to take place at the diaphragm locations. Self-weights of the spandrel elements, arch rib struts, and deck were accounted for by modeling these as superimposed loads.

For the finite element deck model, one way transverse action was assumed and support was provided along the spandrels only. Wheel point loads were placed at various locations in order to maximize the live load transverse bending effects.

6.3. Rating Analysis

Three bridge elements were load rated: the arch ribs, the spandrel longitudinal member above the teardrop shaped opening, and the deck. Further, each of these elements were load rated twice, once for a maintenance vehicle load and once for pedestrian loading.

6.3.1. Arch Ribs

A load rating procedure from the AASHTO Manual for Condition Evaluation and Load and Resistance Factor Rating (LRFR) of Highway Bridges was used for concrete components with compression and bending. In summary, the member's interaction diagram is used to establish the factored dead load moment and thrust. From this point on the diagram, a slope is drawn equal to the live load eccentricity (factored live load moment over factored live load thrust). Member ultimate capacity is where this slope intersects the interaction diagram. Two load ratings are then calculated: one for moment and one for axial load.

To account for deterioration, 1/16" section loss on all surfaces of the bottom 1"x1" Kahn reinforcing bars was used. This was based on field measurements at one spall and assumed to occur throughout the arch length. Spalls were accounted for by ignoring the 9" x 11" horizontal "ledge" of the cross section, leaving a remaining rectangular section of 1'-0" wide x 4'-6" deep.

6.3.2. Spandrel

The longitudinal spandrel element is reinforced for positive bending only. Scaling off of the original plans between the tip of the opening (towards midspan) and tangent point of the circular end (towards the abutment), a simple span of 20-ft was assumed. Because the axle spacing of the H-10 maintenance vehicle is 14-ft, one wheel point load was used to obtain the maximum bending moment, whereas two axles were used to determine maximum live load shear. A uniform load was used for the pedestrian loading. Uniform loading was also used for the dead loads.

To account for deterioration, 1/16" section loss on all surfaces of the bottom 1"x1" Kahn reinforcing bars was assumed based on field observations at the SE spandrel. In-house spreadsheets were used to determine the member capacity.

6.3.3. Deck

One way action was assumed for the deck because the longitudinal reinforcement consisting of ¼" diameter rods spaced at 18" in the direction provided low bending capacity. Though spalls and exposed transverse reinforcing steel exist, section loss was conservatively ignored for the ½" x ½" Kahn bars spaced at 18" on center.

6.3.4. Load Rating Summary

The governing load rating elements include the spandrel longitudinal members and deck. For the spandrel member, all ratings are controlled by bending. Transverse bending controls the deck ratings.

The arch ribs have adequate excess load capacity. The arch rib pedestrian load ratings are controlled by arch positive bending within the middle arch segment under maximum positive moment effects at the rib ¼ point. Arch rib vehicle load ratings are controlled by arch negative bending at the abutments.

Rated Element	Pedestrian Loading		H-10 Vehicle Loading	
	Inventory Rating	Operating Rating	Inventory Rating	Operating Rating
Arch Rib	90 psf	115 psf	H-17	H-22
Spandrel	30 psf	40 psf	H-2	H-2
Deck	30 psf	40 psf	H-2	H-3

7. CONCLUSIONS

7.1. Superstructure

Earlier concrete patching rehabilitation work on the superstructure (arch ribs, spandrels, diaphragms, and struts) remains generally effective. Though shrinkage cracks typically exist at the patches, most sounded tightly adhered to the base concrete when tapped with a hammer. Most of the existing large spalls have occurred at previous patches. Very few new spalls to the original base concrete were found. The one exception is along the top of the southeast teardrop shaped spandrel opening. Exposed reinforcing steel at these spalls exhibit laminate rust and an associated loss in cross sectional area.

7.1.1. Arch Ribs

Though a loss in the arch rib structural capacities can be measured, **load ratings indicate the losses have not significantly affected the member's capacity**. The rib load ratings are adequate to resist live loads required for a new pedestrian bridge of the same size. Rehabilitation methods include removal of loose spalls, concrete patching, removal of deteriorated caulk, crack injection of old sealed and new unsealed cracks, and recaulking.

7.1.2. Spandrels

On the southeast spandrel longitudinal member, **spalling and rebar corrosion have also led to a reduction of the already low member capacity**. A structural analysis indicates that the capacity results in operating load rating values of 20% (vehicle loading) and 44% (pedestrian loading) of those required for a new pedestrian bridge of the same size. Methods include concrete patching, crack injection, recaulking, and spandrel strengthening using externally applied glass or carbon fiber reinforced plastic (FRP) strips. Alternately, replacing the understrength spandrel longitudinal members could be performed.

7.1.3. Deck

The concrete deck is showing advanced deterioration. Though earlier concrete patching is evident, many of these patches have spalled off and exposed the transverse reinforcing steel to the elements. New delaminations of the original base concrete were also noted, indicating continued deterioration and potential spall hazards to vehicular traffic on Ravine Road and pedestrians on the hiking trail at the north abutment. A structural analysis indicates that the capacity results in operating load rating values of 30% (vehicle loading) and 44% (pedestrian loading) of those required for a new pedestrian bridge of the same size. Given that the deck has already received one series of major repairs, **continued concrete deterioration and rebar corrosion suggests that it has surpassed the end of its useful life**.

7.1.4. Diaphragms and Struts

The superstructure diaphragms and struts exhibit minor deterioration which does not affect the structural performance of the bridge. Concrete patching of these members is feasible.

7.1.5. Parapets and Downspouts

The parapets are moderately cracked with concentrations at the vaulted abutments and wingwalls. Localized spalling exists along the curb line on both the east and west sides. It is unknown how the parapet is attached to the deck or what its lateral strength is. For pedestrian use, current codes dictate that a 50 lb/ft linear load or 200 lb point load be applied in any direction at the top of the parapet. Parapet self-weight alone is not adequate to resist the code prescribed forces. Parapet deterioration does not pose a structural risk at this time.

Downspouts are in good condition but are not adequately anchored to the bridge.

7.2. Substructure

Caulk at previously routed cracks and cold joints are cracked due to deterioration and in localized areas falling out of the rout. They are generally ineffective at sealing out water. Newer unsealed cracks also exist, along with advanced concrete deterioration at isolated locations.

7.2.1. Thrust Blocks

Though extensive cracks and some spalls were found on the abutment thrust blocks, these occur within the surface repair concrete. The surface repair concrete is being held together with what is believed to be welded wire fabric located between the arch ribs. **The surface deterioration does not significantly affect the thrust blocks' function** to act as mass concrete resisting arch rib horizontal thrust and vertical gravity loading. Aside from what can be deduced from the honeycombing and cracks found in some of the concrete core samples, the extent of deterioration within the original thrust block concrete is unknown.

7.2.2. Abutment and Wingwalls

The octagonal abutment walls and wingwalls are a major structural concern for this 110 year old bridge. Original drawing notes indicate that ¼" diameter round bars spaced at 12" on center were to be used near the wall exterior surfaces to guard against "checking" and cracking. The drawings also indicate that the octagonal abutment walls are 12" thick and founded on 2'-0" wide strip footings, whereas the wingwalls are 8" thick and founded on 1'-6" wide strip footings. Original drawing notes do not indicate whether these strip footings were to be reinforced, but local failures that were noted suggest the footings are plain unreinforced concrete. It is believed that the original design intent for these elements was for soil on the exterior and interior to be placed at the same elevations, creating a balance of soil lateral pressures forces on each side.

As the walls were originally designed to resist only vertical loading, the narrow strip footings were designed to support gravity loading from the walls, deck, and live loads. Since wall bending was not anticipated during the original design, minimal crack control reinforcing steel is used within the wall elements. In the intervening years, water infiltration inside of the vaulted abutments caused erosion, washing soil towards the low end of the slope. The resulting imbalance of soil horizontal forces on the walls and apparent inadequate reinforcing have created numerous wall cracks. Localized footing undermining has removed vertical support, allowing horizontal tension and vertical/diagonal shear cracks to develop within the wall, allowing the wall to settle. An imbalance of soil lateral pressures and narrow unreinforced footings (not designed to resist overturning moments), have combined to allow for wall horizontal shifting and rotation. These **local wall failures have resulted in missing support for the concrete deck at the southeast and northeast wingwalls**. Though deck settlement/failure has not been observed, we anticipate that it is only a matter of time before this occurs. The advanced state of deterioration, effort required to rehabilitate, and limited effectiveness of existing repairs confirm that **the vaulted abutments and wingwalls have surpassed the ends of their useful lives**.

7.3. Survey

Given the bridge's age of approximately 110 years, some settlement should be expected. The three cycles of survey measurements suggest that west side of the bridge superstructure is on average about 0.03 ft lower than the east side. This translates to a rotation of approximately 0.14 degrees, suggesting the thrust blocks at both abutments have settled in the same direction. At the north abutment, the west side of the bridge superstructure is on average about 0.12 ft higher than the east side with an associated rotation of 0.49 degrees. East wingwall deterioration and rotation is a likely cause of this variation. At the south abutment, the east side is on average 0.07 ft higher than the west side with an associated rotation of 0.29 degrees. This result is unusual in that the east wingwall is experiencing significant undermining and deterioration, and the east edge of deck settlement would have been expected.

The magnitude of elevation variations of the same point taken at different dates was larger than expected. Magnitude differences of up to 0.02 ft would seem reasonable in that the leveling rod may not have been placed in exactly the same location during successive surveys. The maximum elevation difference between shots taken on different dates is 0.06 ft along the west edge and 0.08 ft along the east edge. These larger differences may be due to ground thawing or bridge movement due to differences in the concrete temperature at each survey. Future survey data can be used to generate long term trends in the differences.

8. RECOMMENDATIONS

To address the current state of bridge deterioration, immediate maintenance actions can be taken to reopen the bridge to pedestrian/bicycle traffic as well as to reopen Ravine Road below. Given the bridge's age and state of deterioration, we see two options for future major construction efforts: rehabilitation with major component replacement or complete structure replacement. Cost estimates for the major construction efforts are provided for the two options.

8.1. Reopening the Bridge and Short Term Maintenance

With a nominal amount of effort, we recommend that Ravine Road and the north foot path adjacent to the north abutment be reopened to traffic after the loose delaminations are removed by a contractor from the deck underside, arch ribs, spandrels, diaphragms, and struts. All bays between the abutments should be addressed. After 100% of the surfaces have been checked and loose concrete removed and any structural concerns addressed, then Ravine Road could be reopened.

Until temporary shoring supports the deck over the failed east wingwalls or the abutments are replaced, we recommend that the path on the bridge remain closed to all motorized vehicles. The temporary concrete barriers at the north and south approaches should be maintained, but shifted slightly apart, to allow only pedestrians and bicyclists access (Figure 54). Because deck support is slowly being compromised by failing walls at the NE and SE corners of the bridge, we recommend that Milwaukee County continue to monitor via survey for vertical settlement or horizontal sliding at these points. If any of these movements are detected, we recommend closing the bridge to all pedestrians and bicycles and reanalyzing the situation.

A short term recommendation to address drainage issues is to conduct an investigation of the south approach inlet. In particular, the washout/erosion occurring at the southeast wingwall may be due to a broken drainage pipe. The investigation should include checking to see if the drainage pipe is broken and where it discharges. Dye could be used to assist in this effort. Swales could be constructed to help divert runoff away from the wingwalls and abutments

8.2. Rehabilitation

8.2.1. Structure Rehabilitation

Given the historic nature of this unique bridge, there will likely be a desire to rehabilitate it and keep as many of the original components as possible. It is our opinion that arch ribs, spandrels, diaphragms, struts, and thrust blocks could be rehabilitated. Due to advanced deterioration, low load rating, and limited effectiveness of rehabilitation efforts on minimally reinforced concrete, we recommend replacing the deck, parapets, vaulted abutments, and wingwalls if the rehabilitation option is pursued. Replaced elements would require design strengths to resist current code prescribed live loads.

For the existing bridge components to be rehabilitated, all delaminated concrete and loose patches of concrete should be removed. If the concrete is reinforced, the removal should extend beyond the reinforcing steel so that new concrete patches will be able to mechanically bond to the original structure. Existing caulk from previously routed cracks and cold joints should be removed to maximize the rehabilitation life. These routed cracks, cold joints, and unsealed cracks should then be injected with an epoxy compound. The routs should then be recaulked.

Load ratings indicate the spandrel longitudinal members between panel points 1 to 2 and 4 to 5 do not have the capacity to resist current code prescribed loading. One potential option to improve the load carrying capacity of the spandrels to meet current standards include applying FRP strips to the underside after concrete patching. The FRP strips act as externally applied concrete reinforcement. A second strengthening option is to remove the spandrel longitudinal members over the teardrop shaped opening and replace it with new reinforcing steel and concrete. This method would address questions that would arise about applying structural FRP strips to a concrete patch.

Replacement elements will need to be tied to the members being rehabilitated. Masonry anchors will need to be drilled through the arch rib crown, spandrels, and diaphragms so that the new deck can be securely attached to the superstructure. A similar approach will be required to connect the new abutment walls to the existing thrust block concrete. The new deck and parapets should be reinforced with epoxy coated reinforcing steel in the event the bridge would be salted in the winter. New parapets could be of the Wisconsin DOT standard Vertical Face

Parapet “TX”, formally known as a “Texas Rail” type in order to mimic the original baluster railing used for the original construction. Alternately, a concrete or cut stone railing similar to the original and capable of resisting current code prescribed loads could be used. To further improve the load carrying capabilities of the rehabilitated elements, lightweight concrete could be used for the deck and parapets.

To maintain the look of the original bridge, substructure replacement elements could be replaced in-kind with a few modifications to improve performance. To address the erosion problems currently experienced inside of the vaulted abutments, a solid slab could be constructed between the wingwalls and abutment walls to cap off the soil. An end wall between the wingwall tips would seal out major water infiltrations. This slab will also act as a shallow footing for the walls. Since the concrete deck will provide restraint at the tops of the walls, the walls should be reinforced with vertical steel to resist at-rest soil pressures from the embankment. Temperature and shrinkage reinforcement should be provided as horizontal reinforcing steel. Black reinforcing steel is recommended to be used for the substructure wall and foundation elements, but epoxy coated reinforcing steel may be used for a minimal cost.

As a final aesthetic treatment to tie the old and new concrete together, we recommend that all exposed surfaces receive a coating of concrete stain.

After structure rehabilitation, the only remaining original portion of the bridge would be the two arch ribs, spandrels, thrust blocks, diaphragms and struts (Figure 57). The remaining life of the original arch is estimated at 15-25 years without additional major rehabilitation. **It must be understood that future replacement of these elements will require removal and replacement of the newer deck and railings above. It will also require removal of some parts of the newer abutment elements to replace the thrust blocks.**

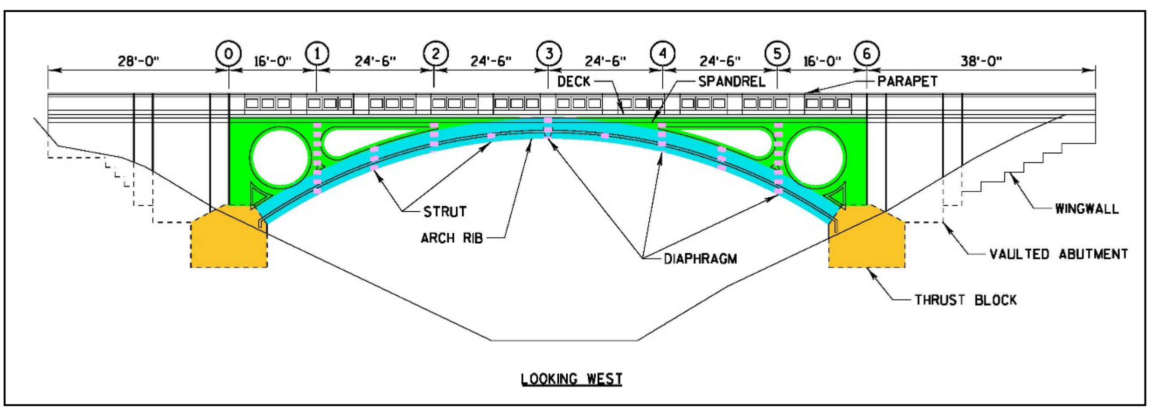


Figure 57: Original bridge elements (colored) remaining after rehabilitation

8.2.2. Site Rehabilitation

As part of future structure rehabilitation work, we suggest that the approaches be re-graded to slope up to the bridge. The low point should be located a minimum of 20 ft away from the bridge, and swales should be graded to direct flow down the slope away from the bridge. To protect the side slopes from concentrated runoff, provide curbs at the approaches and provide an inlet structure at the low point or each approach. Construct a storm drain and direct the pipe for discharge along the bank/slope away from the bridge.

8.3. Replacement

Total replacement is a second option. There are several types of superstructures available for total replacement. Three possibilities for the type of superstructure are discussed below.

8.3.1. Reinforced Concrete Arch

Dimensions of the existing structure can be used to match the historic aesthetics/architectural features with the added benefit of using modern materials. Deficiencies in the existing design can be addressed with improved detailing (such as at the vaulted abutment walls and wingwalls), and modern reinforcement standards. The new design will be more durable than the existing and will likely have a longer lifespan expected to be a minimum of 75 years with minimal maintenance.

Of the 5 bridges in Lake Park, 3 have been rehabilitated within the past 10 years (Lake Park Road bridge and the 2 Lions bridges). Although it is desired to keep the original construction of historic structures, complete bridge replacement may be the better engineering solution for this situation. Milwaukee County has demonstrated its desire to rehabilitate historic structures when such efforts make engineering and economic sense. For the Ravine Road bridge, a long term economical rehabilitation does not seem feasible. A new replacement structure receiving primarily pedestrian and bicycle loading will not require major rehabilitation for perhaps 40 to 50 years and would prove to be an economical solution for this public asset.

8.3.2. Prefabricated Steel Truss

Prefabricated steel trusses are common superstructure types used for recreation trails. They are normally supported by reinforced concrete abutments and piers. The truss superstructure, along with the railings and concrete deck form, is shop fabricated in field sections which are then bolted together on site. Cranes then lift the superstructure onto the substructure units. Temporary shoring may be required to support the end of the truss field sections if the entire truss is too long to be lifted in one piece.

Three different styles of trusses were investigated to cross Ravine Road in one single span. The first style is a half-through H-section system. Half-through H-sections are constant depth pony truss systems (no horizontal lateral bracing between the top chords) and have single span capabilities up to 220-ft long (Figure 58). The second style is a full-through H-section system. This system uses constant depth trusses with both top chord lateral bracing to achieve maximum span lengths up to 250-ft (Figure 59). The third truss style is a bowstring system. Bowstring trusses use a vertically curved top chord and have single span capabilities up to 180-feet long (Figure 60). All three truss styles have single span capabilities close to or exceeding the existing bridge's wingwall tip to wingwall tip length of 196-ft.

Aesthetically, bowstring trusses are used when an architectural statement is desired, however, they are more expensive than a half-through H-section pony truss or full-through H-section truss. The superstructure may be fabricated with unpainted weathering steel or steel with a three coat paint system. Safety fencing to help prevent objects from being thrown onto the roadway below (as shown in Figures 58 and 60) is normally only required on bridges spanning state or interstate highways.

Figure 58: Half-through H-section trail bridge with 170-ft middle span Ozaukee County Trail over I-43 (courtesy Google Earth)

Figure 59: Full-through H-section trail bridge with 145-ft span
Badger State Trail over 8th Street in Monroe (courtesy Google Earth)

Figure 60: Bowstring truss trail bridge with 200-ft span
Ice Age Trail over USH 12 near Waunakee (courtesy Google Earth)

8.3.3. Prestressed Concrete Girders

Prestressed concrete girders have been used as the superstructure type for pedestrian bridges (Figure 61). The girders are shop fabricated and transported to the construction site for erection onto the substructure units. Because prestressed concrete girders can not be field spliced, girder lengths may be limited by the transportation route from the fabrication yard to the bridge site, as well as the crane's lifting capacity. A concrete deck and railings are then cast on top of the girders.

Two possible girder depths may be considered for Ravine Road site. The first is 54 inches deep which has a span capability of 135-ft which closely matches the existing bridge's span length between arch springlines of 130-ft. The second girder depth is 72 inches which has a span capability of 160-ft. The deeper 72 inch girders will allow for smaller and shorter abutments, but delivery and crane capacity may be a concern.

Aesthetically, prestressed girder bridges may be enhanced by using decorative rails such as the Wisconsin DOT standard Vertical Face Parapet "TX". Variable depth precast panels could be hung from the sides of the bridge to mimic the arch shape of the existing structure (Figure 62). The façade panels would hide the girders. Concrete formliner and/or concrete stain could be used on the precast panels to further enhance the aesthetics.

Figure 61: Prestressed concrete girder trail bridge with 90-ft spans Path over USH 12/18 in Madison (courtesy Google Earth)

Figure 62: Architectural panels hung from prestressed concrete bridge (Potawatomi Casino)

8.4. Estimated Construction Costs

Details for the estimated construction cost estimates can be reviewed in Appendix E.

8.4.1. Rehabilitation

For planning purposes, the estimated construction cost for rehabilitation is **$1.8 million**. This figure includes estimates for construction of $1,152,000, design services at 20% of construction ($230,400), and 15% each for construction contingency and construction management (2 x $172,800 = $345,600), and 10% for owner services ($115,200).

The estimated life for arch and spandrel rehabilitation efforts is 15 to 25 years. The 110 year old concrete will continue to crack, delaminate, and spall, and it is not expected that these elements would be able to be rehabilitated again. Future replacement of the arch rib and spandrels would require removal and replacement of the newer deck and railings above.

8.4.2. Replacement

Reinforced Concrete Arch - For planning purposes, the estimated construction cost for complete bridge replacement in-kind is **$2.6 million**. This figure includes estimates for construction of $1,708,000, design services and construction contingency each at 15% of construction (2 x $256,200 = $512,400), 12% for construction management ($204,960), and 10% for owner services ($170,800).

The estimated life for a new bridge constructed in-kind with modern materials and current design provisions is at least 75 years. All elements would be designed for the required live loads, required reinforcing standards to control crack widths, and concrete strengths greater than existing.

Prefabricated Steel Truss - For planning purposes, the estimated construction cost for replacement with prefabricated steel truss is **$1.6 million**. This figure includes estimates for construction of $1,058,000, design services and construction contingency each at 15% of construction (2 x $159,000 = $318,000), 12% for construction management ($127,000), and 10% for owner services ($106,000).

A 195-ft single span half-through H-section system with concrete sill abutments (supported on 4 piles at each abutment) was assumed. Truss depth for a span of this length is approximately 11 to 12 feet. The estimated life for a new prefabricated truss bridge is at least 75 years with proper maintenance.

Prestressed Concrete Girders - For planning purposes, the estimated construction cost for replacement with prestressed concrete girder bridge is **$1.5 million**. This figure includes estimates for construction of $1,012,000, design services and construction contingency each at 15% of construction (2 x $152,000 = $304,000), 12% for construction management ($121,000), and 10% for owner services ($101,000). The above estimates include costs of approximately $125,000 for decorative precast panels attached to the sides of the bridge. Total cost without the decorative panels is approximately $1.4 million.

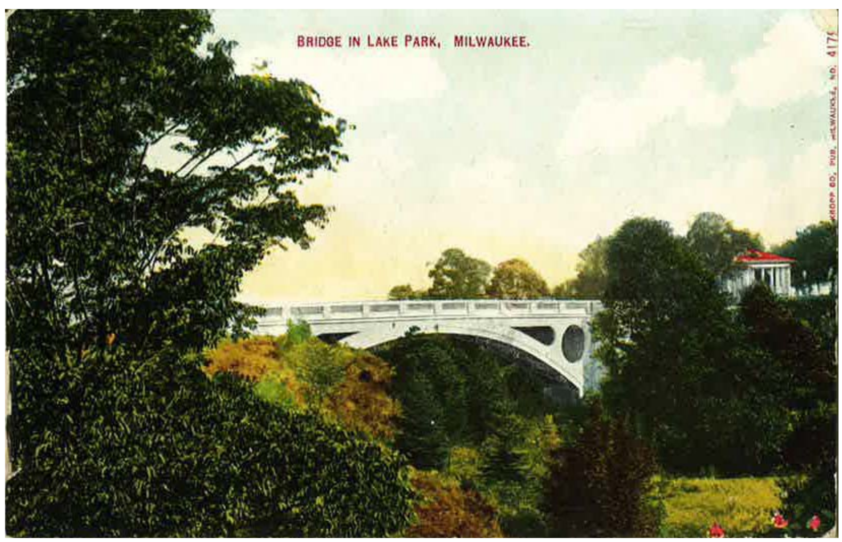
A 160-ft single span bridge using 3 - 72” deep prestressed girders with full retaining concrete A4 abutments (supported on 10 piles per abutment) was assumed. The estimated life for a new prestressed concrete girder bridge is at least 75 years with proper maintenance.

8.4.3. Construction Cost Estimate Summary

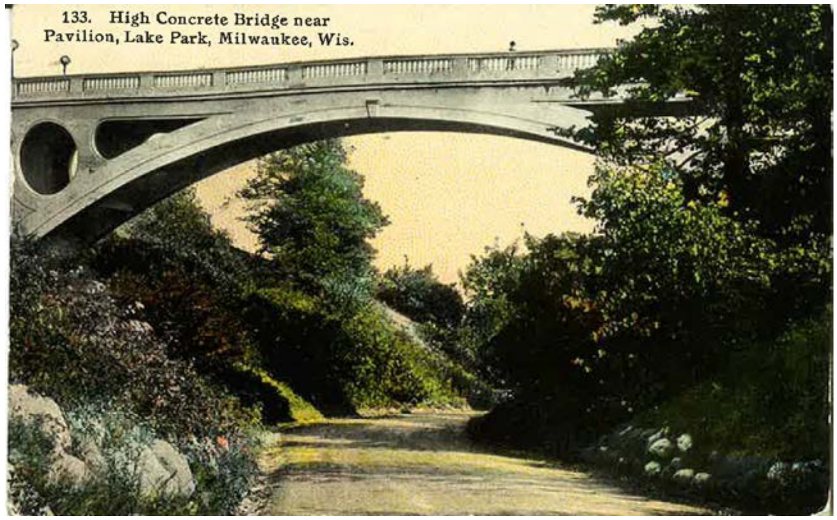
1. Rehabilitation - $1.8 million
2. Replacement with a reinforced concrete arch - $2.6 million
3. Replacement with a prefabricated steel half-through truss – $1.6 million
4. Replacement with prestressed concrete girders
    a. $1.5 million with decorative concrete panels
    b. $1.4 million without decorative concrete panels

Lake Park Bridge Historic Post Cards

Postmark 1908 – looking east



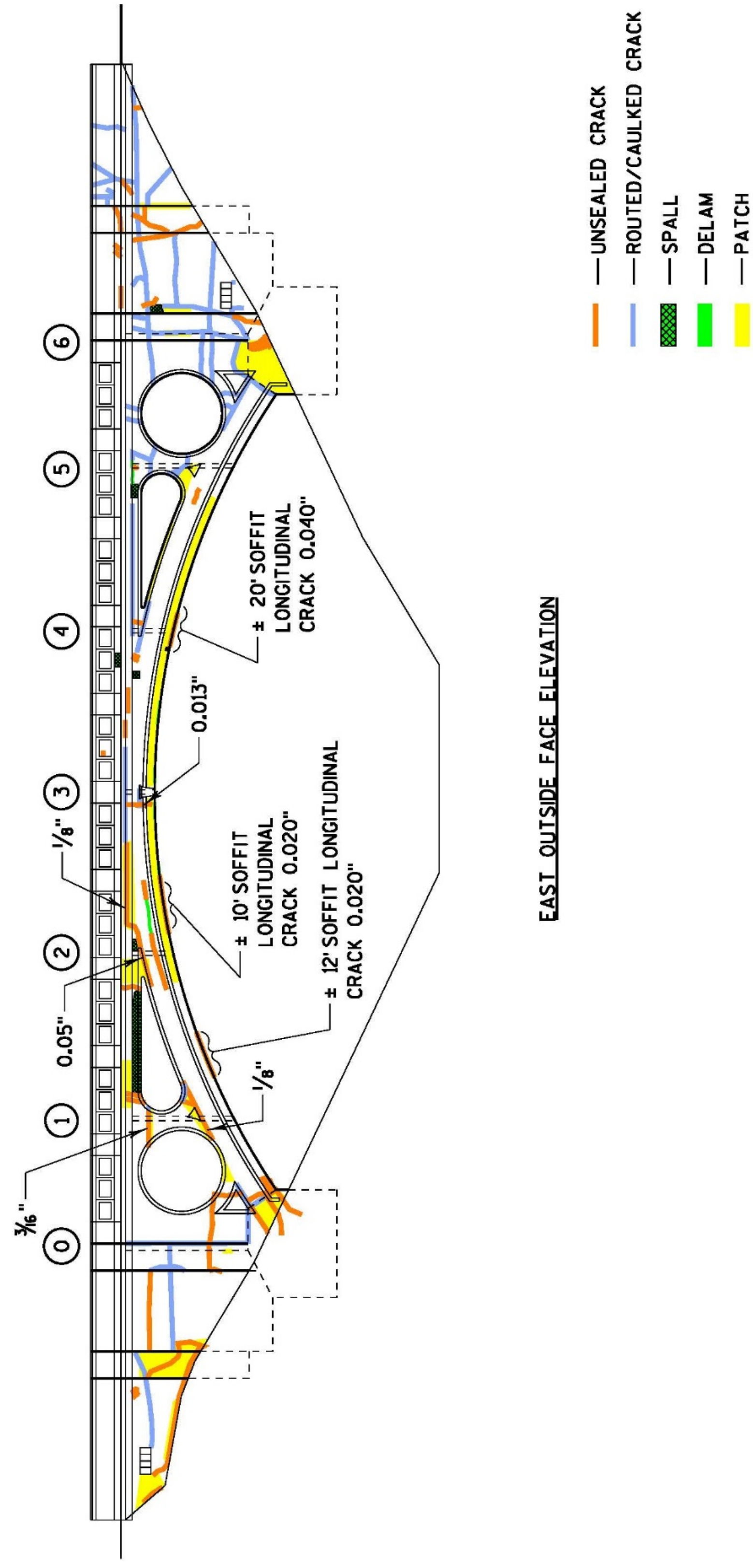
Postmark 1 1910 – looking southeast

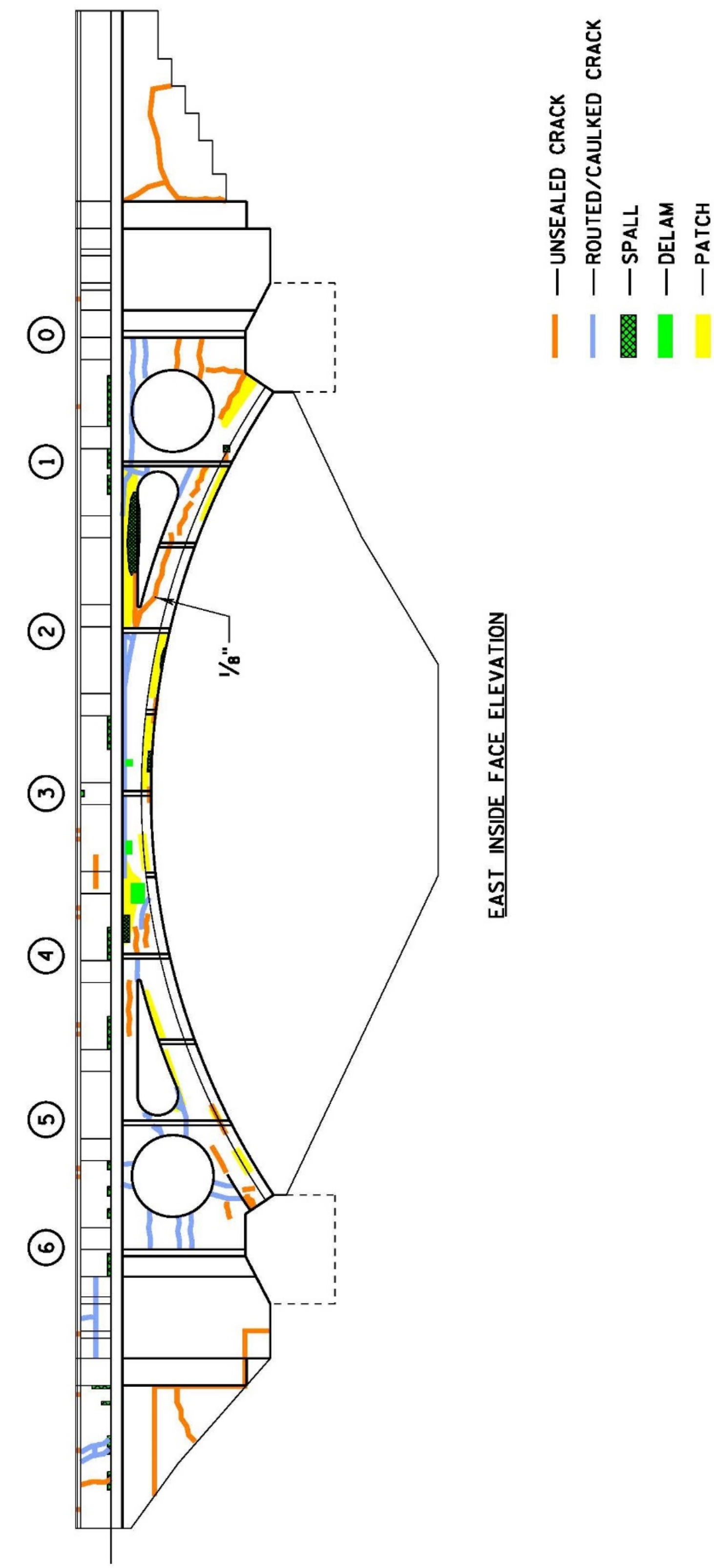


Postmark 1911 – looking west

Postmark unknown – looking southwest

APPENDIX A – FIELD FINDINGS


0
1
2
3
4
5
6
3/16"
0.05"
1/8"
1/8"
0.013"
± 10' SOFFIT LONGITUDINAL CRACK 0.020"
± 12' SOFFIT LONGITUDINAL CRACK 0.020"
± 20' SOFFIT LONGITUDINAL CRACK 0.040"
EAST OUTSIDE FACE ELEVATION
UNSEALED CRACK
ROUTED/CAULKED CRACK
SPALL
DELAM
PATCH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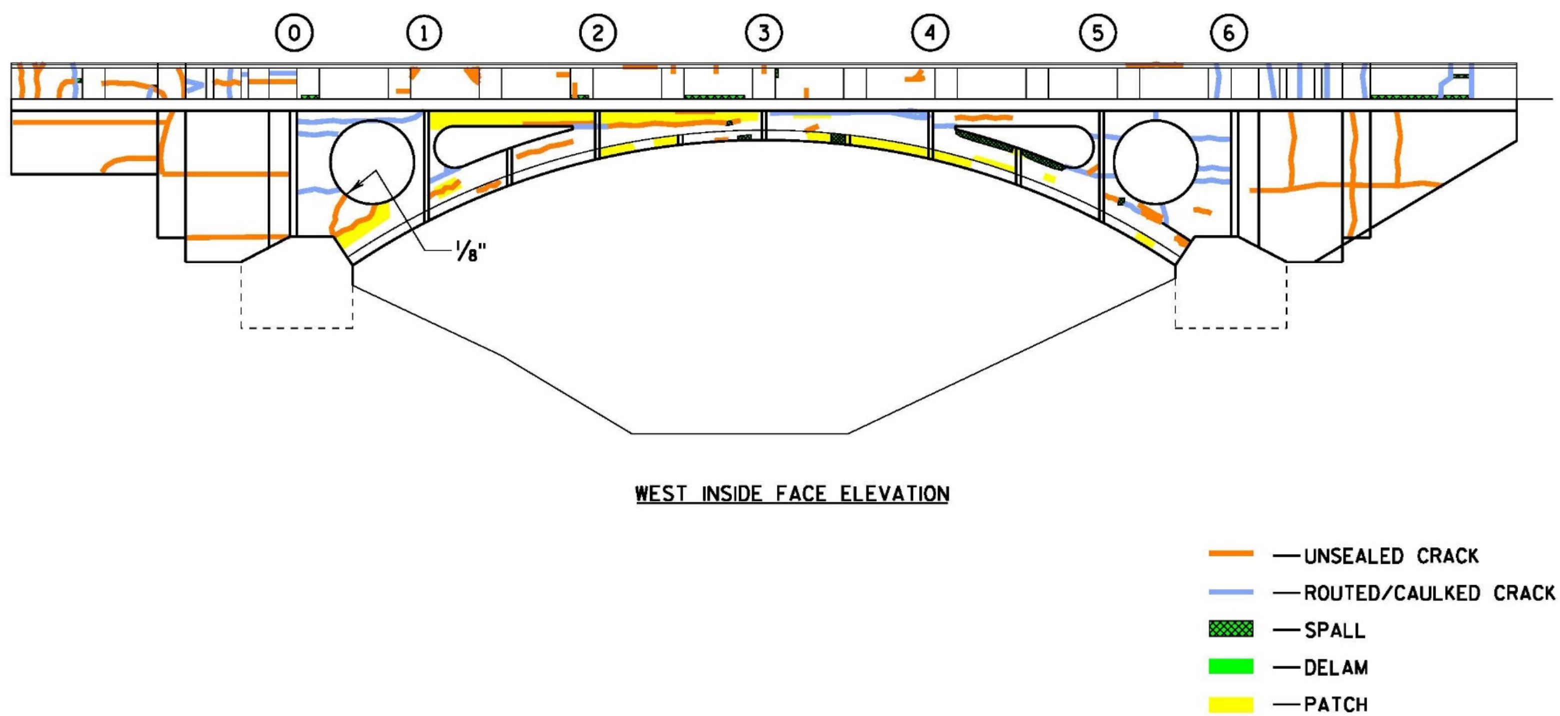
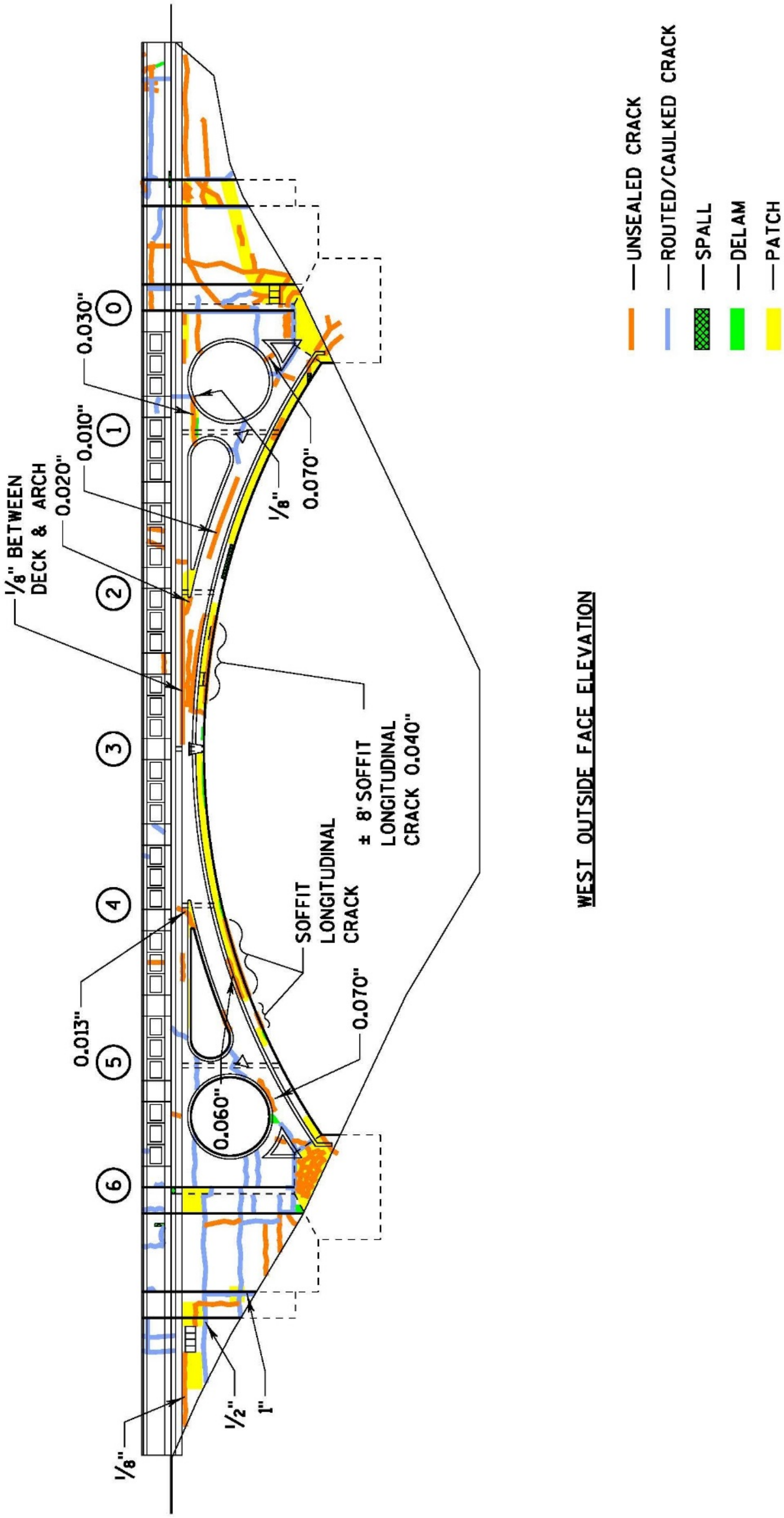
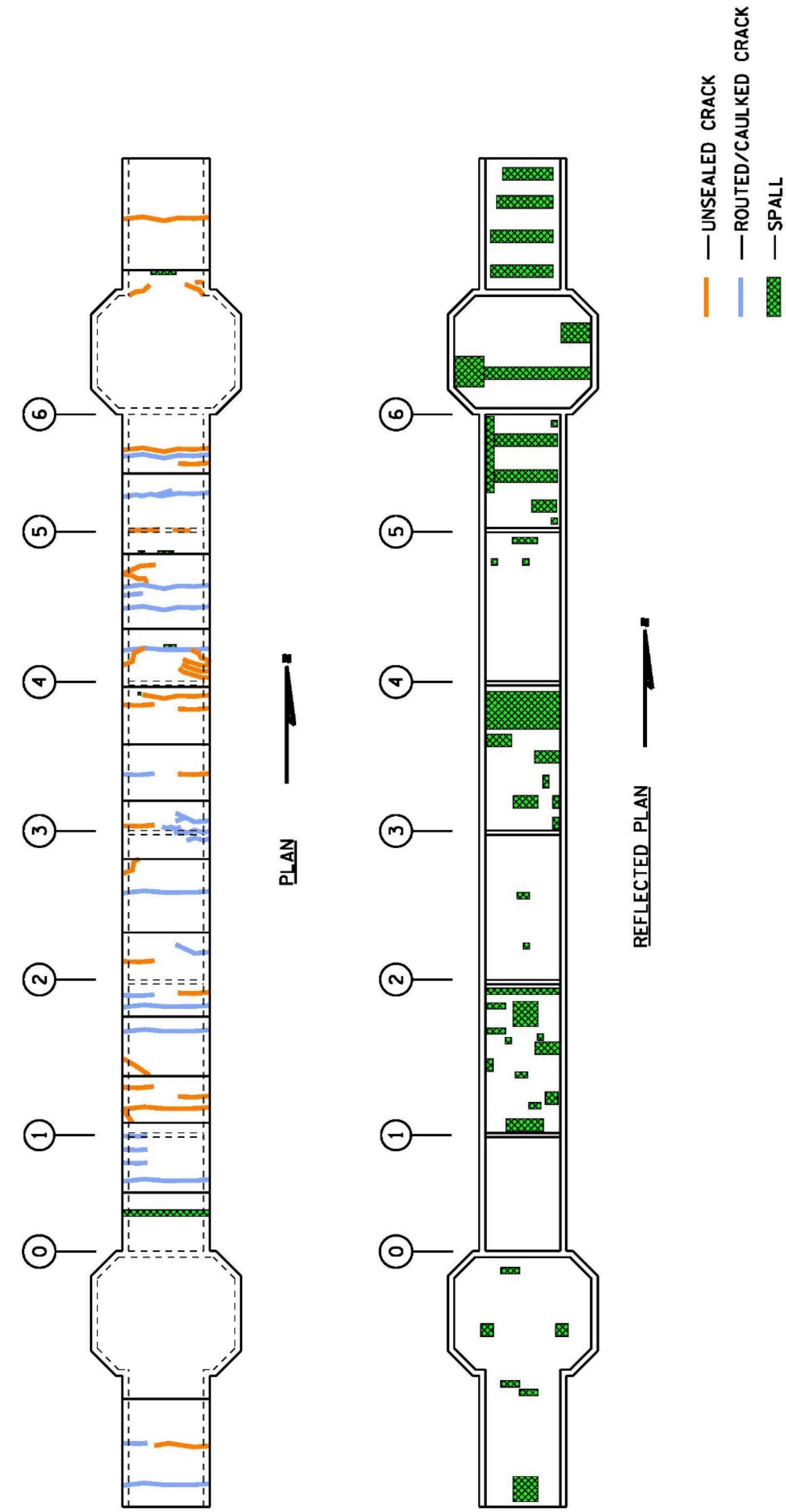
1/8"
EAST INSIDE FACE ELEVATION
UNSEALED CRACK
ROUTED/CAULKED CRACK
SPALL
DELAM
PATCH



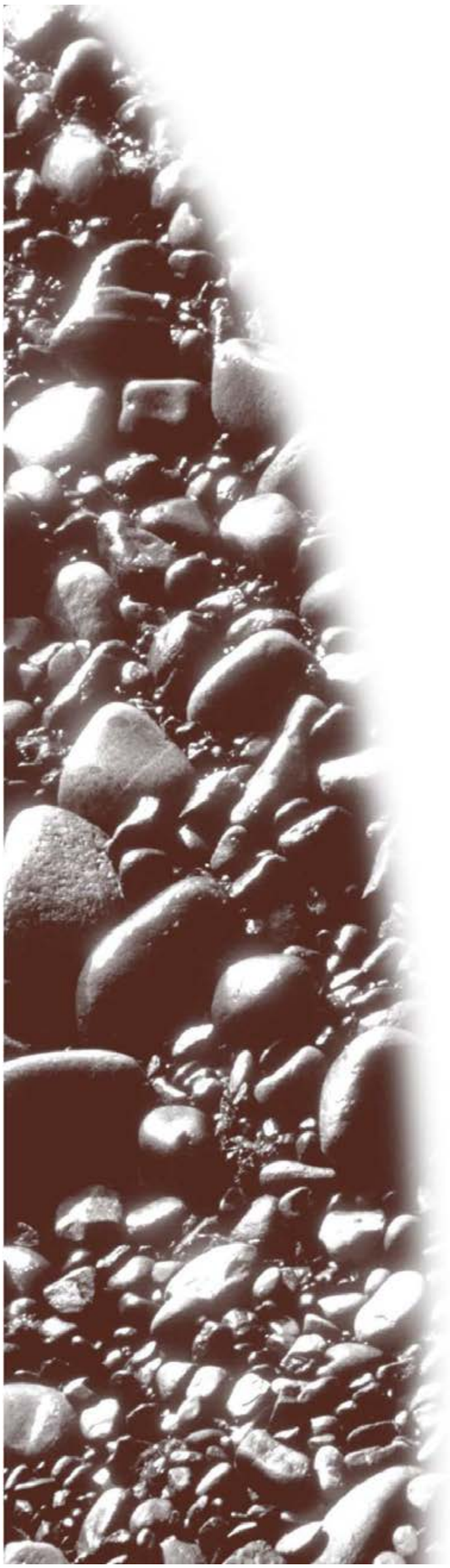
WEST INSIDE FACE ELEVATION



WEST OUTSIDE FACE ELEVATION


0
1
2
3
4
5
6
PLAN
REFLECTED PLAN
UNSEALED CRACK
ROUTED/CAULKED CRACK
SPALL

I. APPENDIX B – CONCRETE TESTING REPORT



CONCRETE CORING AND TESTING

Ravine Road Bridge
Lake Park
Milwaukee, Wisconsin

Prepared for:

GRAEF
Milwaukee, Wisconsin

April 24, 2015
Project No. 1G-1503018

GILES
ENGINEERING ASSOCIATES, INC.

GILES
ENGINEERING ASSOCIATES, INC.
GEOTECHNICAL, ENVIRONMENTAL & CONSTRUCTION MATERIALS CONSULTANTS

- Atlanta, GA
- Baltimore/Wash. DC
- Dallas, TX
- Los Angeles, CA
- Milwaukee, WI
- Orlando, FL

April 24, 2014

GRAEF
125 S. 84th Street, Suite 401
Milwaukee, WI 53214

Attention: Mr. Al Lindner

Project: Concrete Coring and Testing
Ravine Road Bridge
Lake Park
Milwaukee, Wisconsin
Proposal No. 1G-1503018

Dear Mr. Lindner:

As requested, Giles Engineering Associates, Inc. conducted Concrete Coring and Testing for the proposed project. The accompanying report and additional enclosures describes the services that were conducted for the project and it provides the results of those services.

We sincerely appreciate the opportunity to provide materials testing services for the Ravine Drive Bridge evaluation project. Please contact the undersigned if there are questions concerning the report or if we may be of further service.

Very truly yours,

GILES ENGINEERING ASSOCIATES, INC.

WISCONSIN
4-24-15
DAVID M CORNALE
43336-6
WAUKESHA
WIS.
PROFESSIONAL ENGINEER

David M. Cornale, P.E.
Project Professional II

Paul J. Giese, P.E.
Geotechnical Division Manager

ENCLOSURES:
- Concrete Coring and Testing Report (5 pgs.)
- Concrete Imaging Radar Results and Concrete Core Location (Figures 1 through 5)
- Impact Echo (IE) Test Locations (Figures 6 through 8)
- IE Test Location Photographs (Figures 9 through 17)
- Concrete Core Photographs (Figures 18 and 19)
- Report of Test on Concrete Cores (1 pg.)

Distribution: GRAEF
Attn: Mr. Al Lindner (2 via USPS; 1 via email: al.lindner@graef-usa.com)
Attn: Mr. Kevin Wood (1 via email: kevin.wood@graef-usa.com)

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CONCRETE CORING AND TESTING

RAVINE ROAD BRIDGE
LAKE PARK
MILWAUKEE, WISCONSIN
PROJECT NO. 1G-1503018

1.0 SCOPE OF SERVICES

This report provides a description of the concrete imaging radar (CIR) scanning, impact echo (IE) concrete testing and concrete coring that Giles Engineering Associates, Inc. ("Giles") performed for the project, along with a summary of the results of the concrete coring and testing. The concrete coring and testing was performed to assist GRAEF in their structural evaluation of the Ravine Drive Bridge. Evaluation of the bridge structure or foundations was beyond Giles scope of services for the project.

The services requested for this project included:

- Performing a concrete imaging radar (CIR) scanning of each of the three faces of the north and south abutment thrust blocks and both faces of the abutment vaults
- Performing impact echo (IE) testing at select locations identified at the site by GRAEF on the bridge structure concrete
- Obtaining concrete cores from each of the three faces of the north and south abutment thrust blocks
- Unconfined compression testing (ASTM D39) on concrete specimens obtained from the coring

2.0 SITE AND PROJECT DESCRIPTION

The site of the concrete coring and testing consists of the Ravine Drive Bridge, located within Lake Park, in the City of Milwaukee (Milwaukee County), Wisconsin. Ravine Drive Bridge is a single span concrete structure that spans Ravine Drive approximately 100 yards west of Lincoln Memorial Drive. It is understood that the bridge is on the order of approximately 110 years old. It is understood that the bridge concrete arch and thrust block contains longitudinal and shear reinforcing bars consisting of Kahn trussed bars, according to the provided information, and as shown on the provided original bridge plan. Additional information regarding the bridge was not currently available to Giles.

3.0 CONCRETE IMAGING RADAR (CIR) SCANNING

CIR scanning was performed on each of the three faces of the abutment thrust blocks and both faces of the abutment vaults, at both the north and south bridge abutments. The CIR scanning was performed using a Conquest SL radar imaging device. The areas subjected to CIR scanning were limited to those areas that were accessible from the ground without the use of lifts, scaffolding or ladders.

Thrust Blocks

In summary, the CIR scanning indicated longitudinal steel reinforcement in the locations generally depicted on the original building plans. Additionally, the CIR scanning indicated the presence of three reinforcing bars, spaced at approximately 7 inches on center, within the 22± inch wide flange at the bottom of the arch. Additional steel reinforcement, likely indicating shear reinforcement, was also detected during the CIR scanning. The apparent shear reinforcement detected during CIR scanning was, in general, somewhat consistent with that shown on the original plan; however, the reinforcement indicated by the CIR scanning was generally more random than that shown on the plan. Additionally, shear reinforcing bars were generally not observed by CIR scanning in both directions, as indicated on the original plans. However, it should be noted that reinforcing may be present and not detected by the CIR scanning, due to its depth, shielding by other steel bars, or other factors. The results of the CIR scanning performed on the east and west sides of the thrust blocks are shown on the attached Figures 1 through 4.

CIR scanning on the south face of the north abutment thrust block indicated vertical reinforcing steel at a spacing of approximately 10 inches on center. At the north face of the south abutment thrust block, vertical steel reinforcing at spacings between 18± and 20± inches was indicated by the scanning. Additionally, four horizontal steel bars were indicated by the CIR scanning at the north face of the south abutment thrust block. Wire fabric reinforcement was interpreted to be present at the north abutment thrust block face. The wire fabric appears to be related to surface patching previously performed on the south face of the north abutment thrust block. The results of the CIR scanning performed on the north and south faces of the thrust blocks are shown on the attached Figure 5.

Abutment Vaults

CIR scanning of the abutment vaults did not indicate the presence of vertical steel reinforcement. The scanning did indicate the presence of horizontal steel bars, at variable spacings that most typically ranged between approximately 10 and 20 inches on center. However, some estimated bar spacings were lesser and greater than the indicated typical range. A depiction of the approximate bar spacings and results of the CIR scanning performed on the abutment vaults is shown on the attached Figures 1 through 4.

4.0 IMPACT ECHO (IE) CONCRETE INTEGRITY TESTING

Integrity testing of select locations of the bridge concrete was performed by Giles using Impact Echo (IE) non-destructive methods. The IE testing was performed using the portable impact echo system (PIES) by Qualitest. The locations at which IE testing was performed were selected by GRAEF. The approximate locations of the IE testing are shown on the attached Figures 6 through 8. Additionally, photographs of each of the IE test locations are shown on the attached Figures 9 through 17. Access to the IE test locations was provided by an aerial lift operated by a GRAEF engineer.

Three of the IE test locations (Tests 1, 2 and 9) were performed in areas of apparently sound concrete and where the thickness of the concrete could be measured. These tests were performed to assist calibration of the IE equipment and adjustment of the material wave velocity. A summary of the results of the IE testing is provided in the following Table 1.

TABLE 1 – IMPACT ECHO (IE) TEST RESULTS	
Test Location	**IE Test Result Summary Comments**
1	Sound concrete – location used for calibration and settings adjustment
2	Sound concrete – location used for calibration and settings adjustment
3	Shallow delamination indicated - multiple peak frequency responses
4	Shallow delamination indicated - multiple peak frequency responses
5	Shallow delamination indicated - multiple peak frequency responses
6	Shallow delamination indicated - multiple peak frequency responses
7	No delamination indicated
8	Shallow delamination indicated at 3 inches
9	Sound concrete – location used for calibration and settings adjustment
10	Shallow delamination indicated - multiple peak frequency responses
11	No delamination indicated
12	Shallow delamination indicated at 1½ inches
13	Shallow delamination indicated at 3 inches
14	Shallow delamination indicated at 2½ inches
15	Shallow delamination indicated at 2 inches
16	Shallow delamination indicated at 3 inches
17	Shallow delamination indicated at 2 inches

5.0 CONCRETE CORING

Six concrete cores were obtained from the thrust blocks. Further, one core was obtained from each face of the thrust blocks at both the north and the south abutments, as requested. The specific core locations were adjusted after performing the CIR scanning, to avoid encountering reinforcing steel. The approximate core locations are shown on Figures 1 through 5. The cores were obtained using a diamond toothed wet-cut 4-inch diameter core barrel. The core sampling was advanced to a depth of 24 inches at each location, as requested.

At each of the core locations, concrete, which appeared to be a surface patch material placed at some time after the original bridge construction, was encountered at the surface. At the north abutment, the surface patch material was between approximately 1.8 and 5.8 inches thick. This patch concrete consisted of a material with fine aggregate (3/8± in. max aggregate). Wire fabric was observed approximately 1 inch from the core surface at the center face core obtained from the south thrust block. At the south abutment cores, the surface patch concrete thickness ranged between 8.7± and 12.2± inches. In contrast to the patch concrete at the north abutment, the south thrust block patch concrete contained relatively coarse aggregate (1½± in max aggregate). The interior original concrete was generally in sound condition, but did contain some fracturing, and a moderate level of honey-combing was generally present throughout the concrete cores recovered. The lower portions of Cores 2 and 5 were not recovered due to deeper fractures. A summary of the concrete core specimens recovered from the coring is provided in Table 2 below. Photographs of the core specimens recovered from the site are also provided in Figures 18 and 19.

TABLE 2 – CONCRETE CORE SUMMARY

Thrust Block	Location	Core ID	Total Core Recovery (inches)	Surface Patch Concrete		Original Concrete
				Thickness (inches)	Condition	Condition
North	West Side	1	23.3±	1.8±	Sound; unbonded with original concrete	Sound; honey-combing present in outer 7± inches, fracture at 3± inches
	Center	2	16.5±	5.8± (1)	Sound; bonded to original concrete	Partially sound, fractures at 1± and 5± inches; honey-combing present; lower portion of core not recovered
	East Side	3	23.9±	2.5±	Sound; bonded to original concrete	Sound; extensive honey-combing present, some possible loss of aggregate bond
South	Center	4	23.2±	12.2±	Sound; unbonded with original concrete	Sound; honey-combing present
	West Side	5	19.8±	8.8±	Sound; unbonded with original concrete	Sound; honey-combing present; fracture at 1½± inches
	East Side	6	23.5±	10.2±	Sound; unbonded with original concrete	Sound; honey-combing present

(1) Wire fabric (mesh) reinforcement at 1± inch from outside surface of core

Unconfined compression testing (ASTM D39) was performed on six concrete specimens obtained from the coring. Two of the specimens (Cores 2 and 4) consisted of the concrete that was used to patch the surface of thrust blocks subsequent to their original construction. These specimens were obtained from the south face of the north thrust block and the north face of the south thrust block. The compressive strength of these cores was relatively higher than the compressive strength of the original thrust block specimens, with the test results indicating a compressive strength of 7,543 psi and 9882 psi for Cores 4 and 2, respectively. The compressive strength test results obtained from the other cores consisting of what appeared to be the original thrust block concrete ranged between 1,595 psi and 3,166 psi. Compressive strength testing was not performed on the original concrete from the center face of the thrust blocks due to the poor core recovery and the poor condition of the recovered cores, as it was determined that the condition of the cores may provide results that are not representative of the concrete due to specimen defects. The compressive strength test results are listed on the attached *Report of Test on Concrete Cores*, along with other information regarding the compressive test specimens prepared from the concrete cores.

6.0 BASIS OF REPORT

This report is based on *Giles*' proposal, which is dated January 8, 2015 (Revised January 15, 2015) and is referenced by *Giles*' proposal number 1GP-1501008. The actual services for the project varied somewhat from those described in the proposal because of the conditions that were encountered while performing the services and in consideration of the proposed project.

The test results presented in this report have been promulgated in accordance with generally accepted professional engineering practices in the field of geotechnical engineering and construction materials testing. No other warranty is either expressed or implied.

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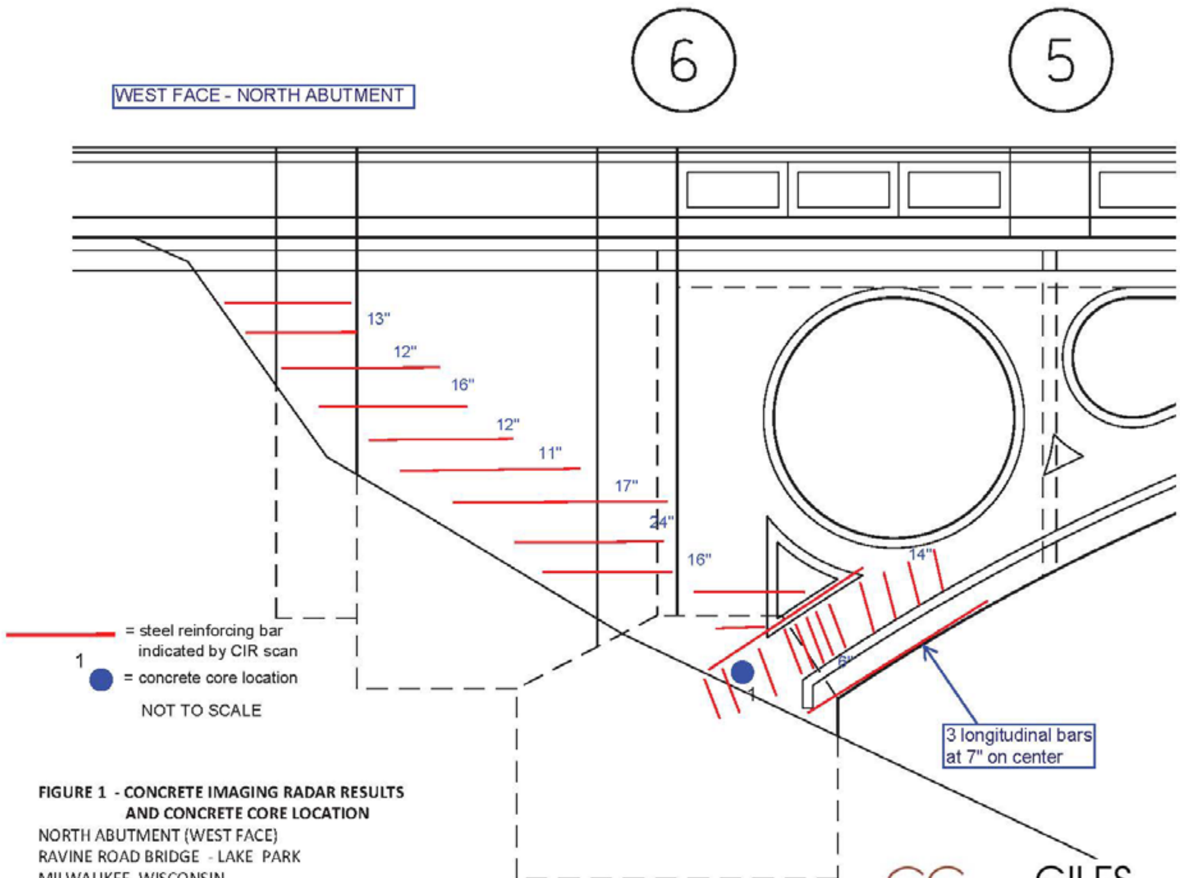


FIGURE 1 - CONCRETE IMAGING RADAR RESULTS AND CONCRETE CORE LOCATION
NORTH ABUTMENT (WEST FACE)
RAVINE ROAD BRIDGE - LAKE PARK
MILWAUKEE, WISCONSIN
PROJECT NO: 1G-1503018

GILES ENGINEERING ASSOCIATES, INC.

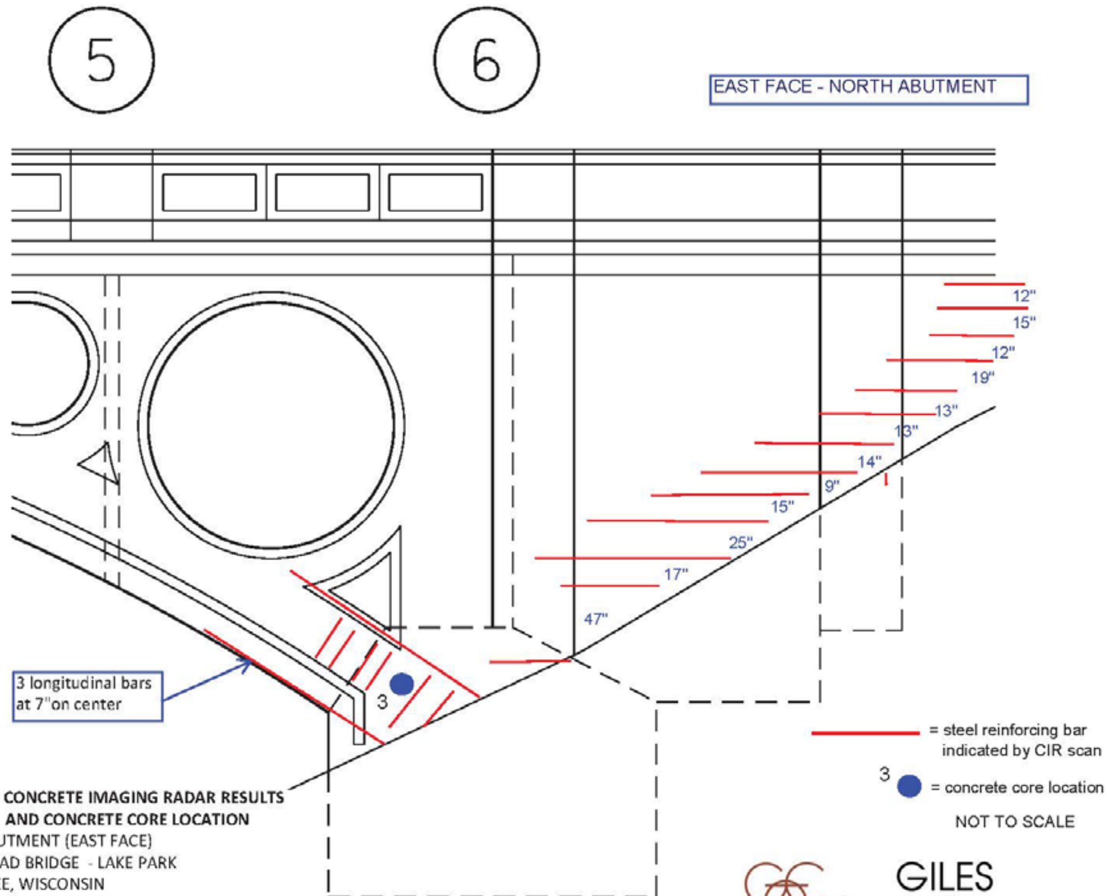


FIGURE 2 - CONCRETE IMAGING RADAR RESULTS AND CONCRETE CORE LOCATION
NORTH ABUTMENT (EAST FACE)
RAVINE ROAD BRIDGE - LAKE PARK
MILWAUKEE, WISCONSIN
PROJECT NO: 1G-1503018

GILES ENGINEERING ASSOCIATES, INC.

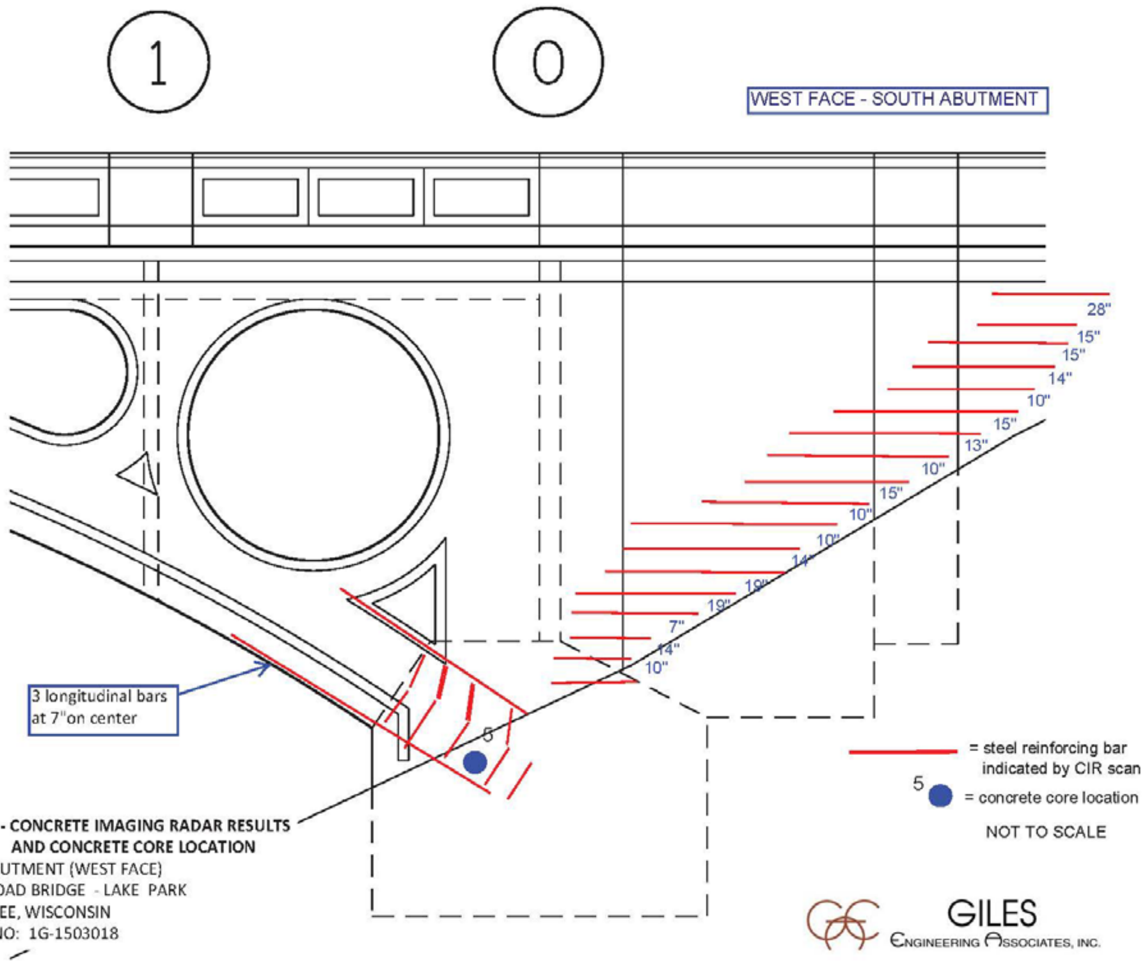


FIGURE 3 - CONCRETE IMAGING RADAR RESULTS AND CONCRETE CORE LOCATION
SOUTH ABUTMENT (WEST FACE)
RAVINE ROAD BRIDGE - LAKE PARK
MILWAUKEE, WISCONSIN
PROJECT NO: 1G-1503018

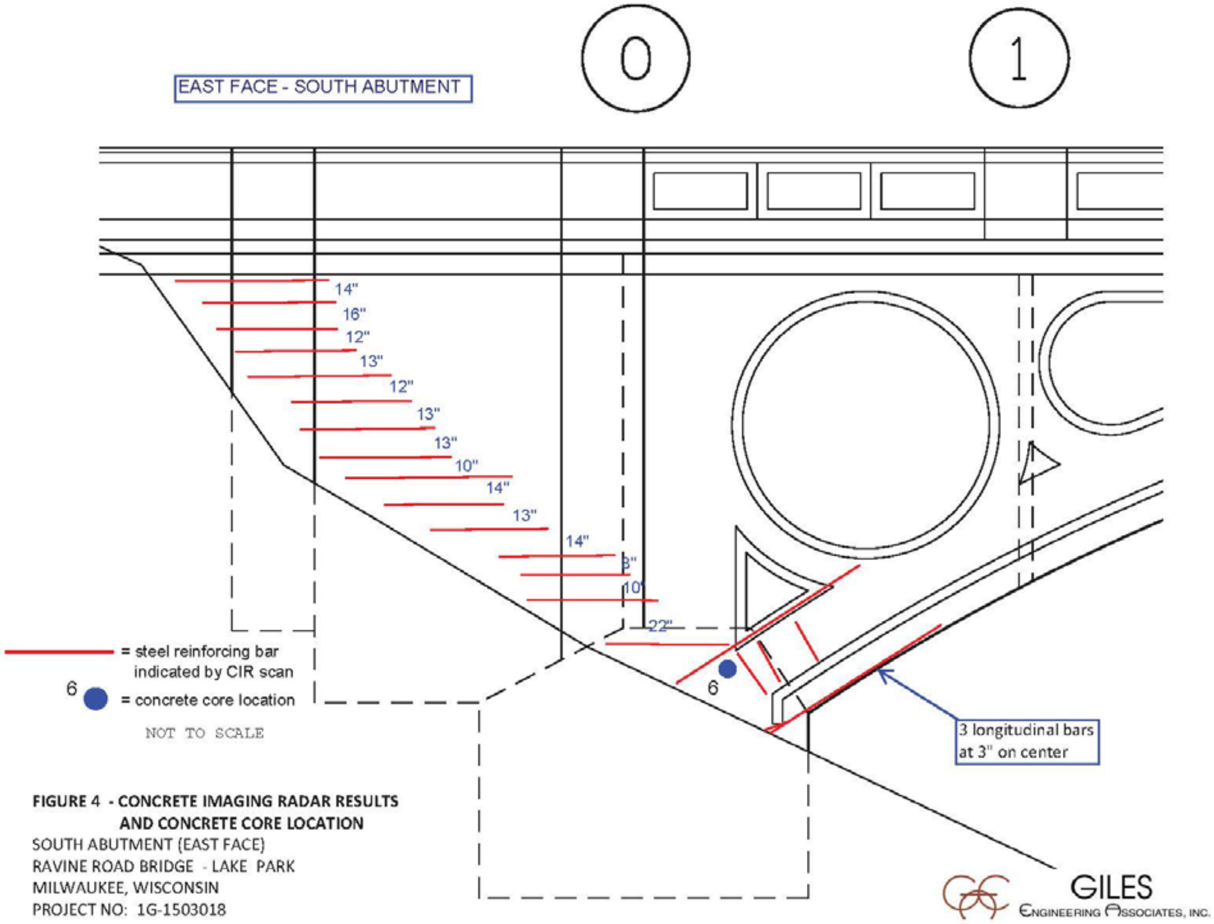


FIGURE 4 - CONCRETE IMAGING RADAR RESULTS AND CONCRETE CORE LOCATION
SOUTH ABUTMENT (EAST FACE)
RAVINE ROAD BRIDGE - LAKE PARK
MILWAUKEE, WISCONSIN
PROJECT NO: 1G-1503018

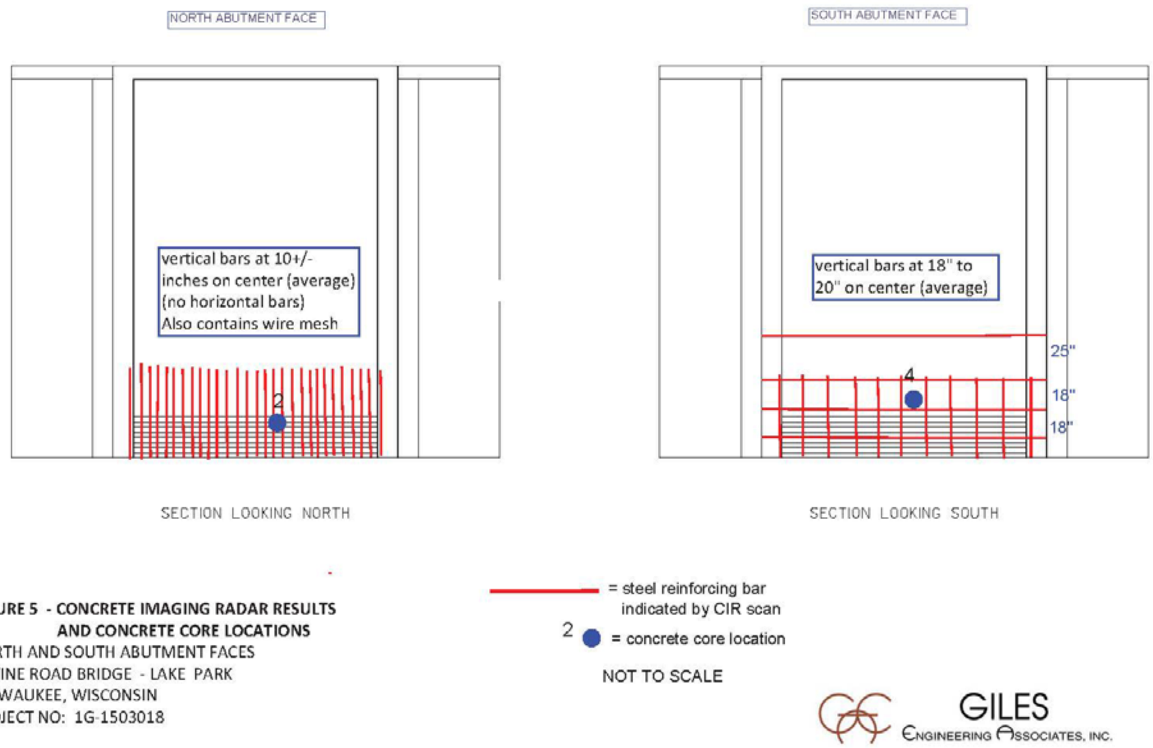


FIGURE 5 - CONCRETE IMAGING RADAR RESULTS AND CONCRETE CORE LOCATIONS
NORTH AND SOUTH ABUTMENT FACES
RAVINE ROAD BRIDGE - LAKE PARK
MILWAUKEE, WISCONSIN
PROJECT NO: 1G-1503018

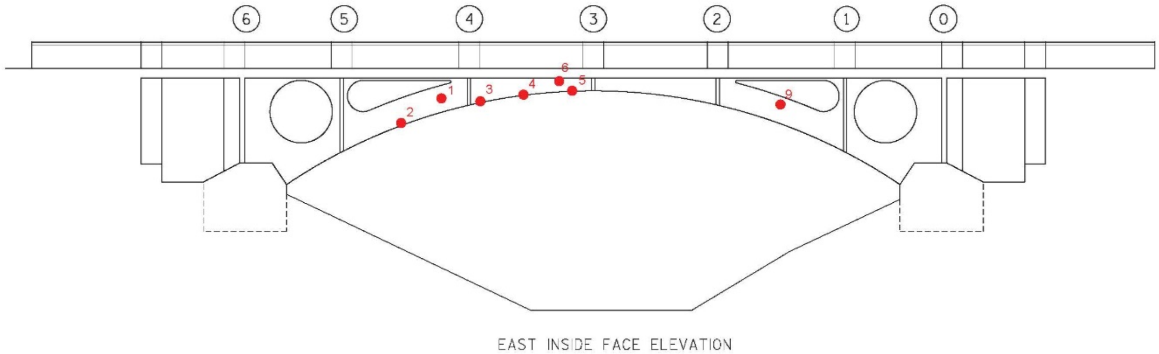


FIGURE 6 - IMPACT ECHO TEST LOCATIONS
BRIDGE EAST INSIDE FACE
RAVINE ROAD BRIDGE - LAKE PARK
MILWAUKEE, WISCONSIN
PROJECT NO. 1G-1503018

GILES ENGINEERING ASSOCIATES, INC.

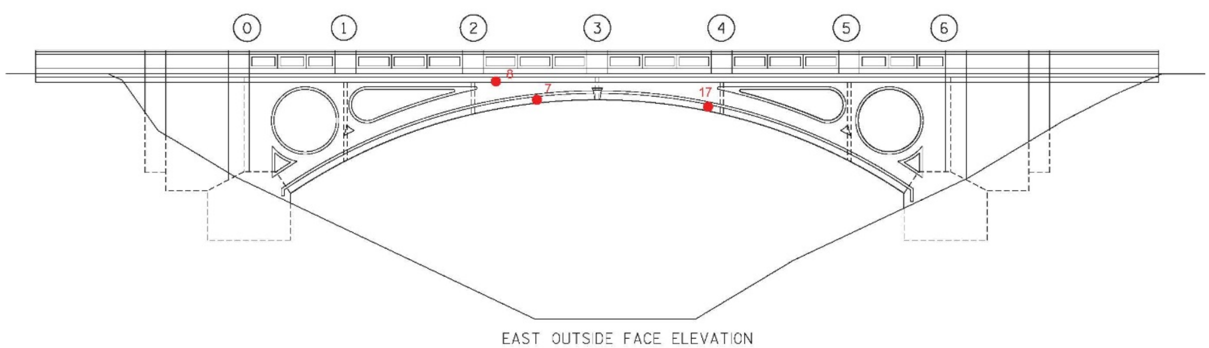
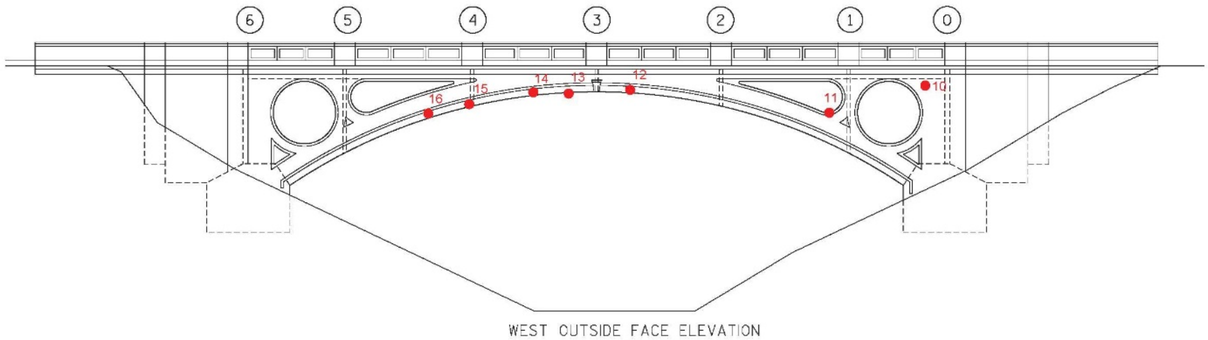


FIGURE 7 - IMPACT ECHO TEST LOCATIONS
BRIDGE OUTSIDE EAST FACE
RAVINE ROAD BRIDGE - LAKE PARK
MILWAUKEE, WISCONSIN
PROJECT NO. 1G-1503018

GILES ENGINEERING ASSOCIATES, INC.



●10 = IMPACT ECHO TEST APPROXIMATE LOCATION

FIGURE 8 - IMPACT ECHO TEST LOCATIONS
BRIDGE WEST FACE
RAVINE ROAD BRIDGE - LAKE PARK
MILWAUKEE, WISCONSIN
PROJECT NO. 1G-1503018

GILES
ENGINEERING ASSOCIATES, INC.

TEST LOCATION 1

TEST LOCATION 2

FIGURE 9
IE TEST LOCATION PHOTOGRAPHS

Ravine Road Bridge
Lake Park
Milwaukee, Wisconsin
Project No. 1G-1306105

GILES ENGINEERING ASSOCIATES, INC.

TEST LOCATION 3

TEST LOCATION 4

FIGURE 10
IE TEST LOCATION PHOTOGRAPHS

Ravine Road Bridge
Lake Park
Milwaukee, Wisconsin
Project No. 1G-1503018

GILES
ENGINEERING ASSOCIATES, INC.

TEST LOCATION 5

TEST LOCATION 6

FIGURE 11
IE TEST LOCATION PHOTOGRAPHS

Ravine Road Bridge
Lake Park
Milwaukee, Wisconsin
Project No. 1G-1503018

GILES ENGINEERING ASSOCIATES, INC.

TEST LOCATION 7

TEST LOCATION 8

FIGURE 12
IE TEST LOCATION PHOTOGRAPHS

Ravine Road Bridge
Lake Park
Milwaukee, Wisconsin
Project No. 1G-1503018

GILES
ENGINEERING ASSOCIATES, INC.

TEST LOCATION 9

TEST LOCATION 10

FIGURE 13
IE TEST LOCATION PHOTOGRAPHS

Ravine Road Bridge
Lake Park
Milwaukee, Wisconsin
Project No. 1G-1503018

GILES ENGINEERING ASSOCIATES, INC.

TEST LOCATION 11

TEST LOCATION 12

FIGURE 14
IE TEST LOCATION PHOTOGRAPHS

Ravine Road Bridge
Lake Park
Milwaukee, Wisconsin
Project No. 1G-1503018

GILES
ENGINEERING ASSOCIATES, INC.

TEST LOCATION 13

TEST LOCATION 14

FIGURE 15
IE TEST LOCATION PHOTOGRAPHS

Ravine Road Bridge
Lake Park
Milwaukee, Wisconsin
Project No. 1G-1503018

GILES
ENGINEERING ASSOCIATES, INC.

TEST LOCATION 15

TEST LOCATION 16

FIGURE 16
IE TEST LOCATION PHOTOGRAPHS

Ravine Road Bridge
Lake Park
Milwaukee, Wisconsin
Project No. 1G-1503018

GILES
ENGINEERING ASSOCIATES, INC.

TEST LOCATION 17

FIGURE 17
IE TEST LOCATION PHOTOGRAPHS

Ravine Road Bridge
Lake Park
Milwaukee, Wisconsin
Project No. 1G-1503018

GILES
ENGINEERING ASSOCIATES, INC.

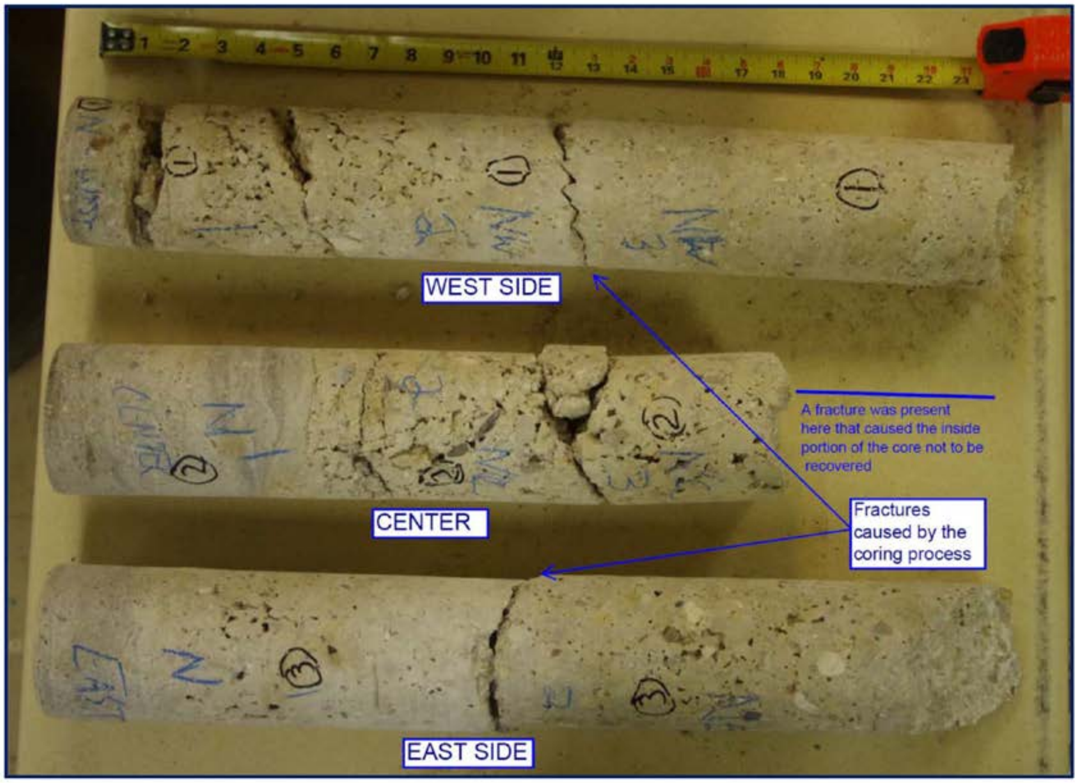


FIGURE 18 – CONCRETE CORE PHOTOGRAPHS

Ravine Road Bridge - Lake Park
North Abutment
Milwaukee, Wisconsin
Project No. 1G-1503018

GILES Engineering Associates, Inc.

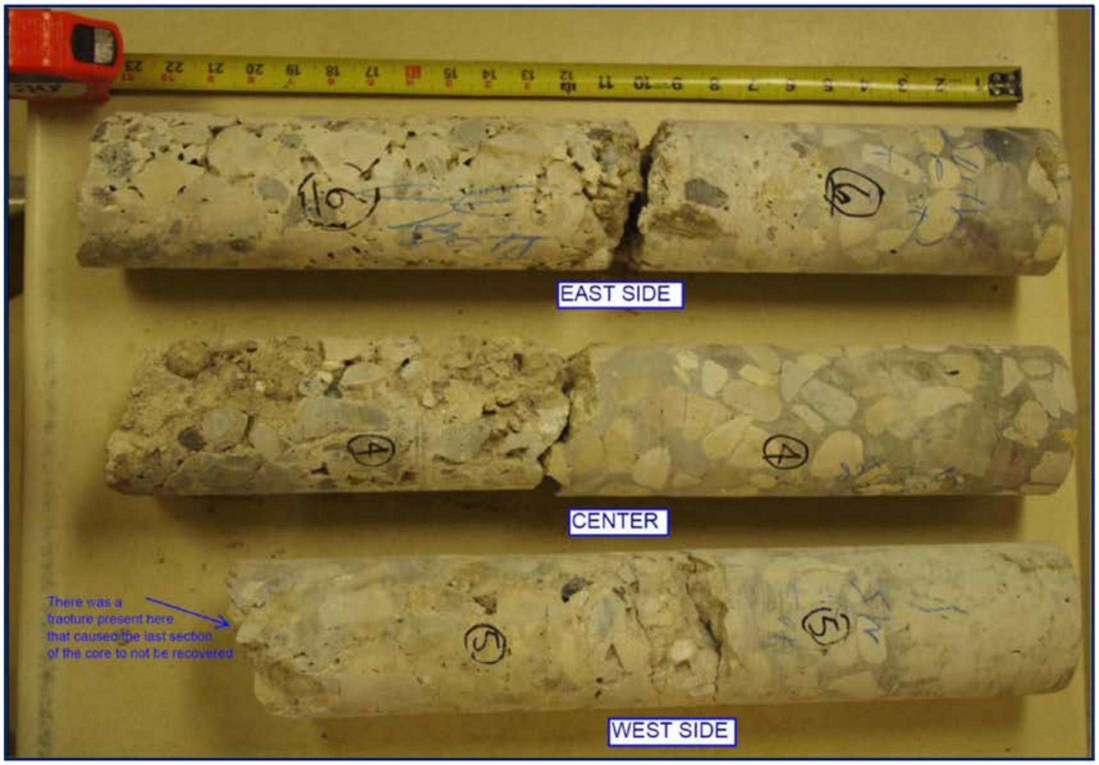


FIGURE 19 – CONCRETE CORE PHOTOGRAPHS

Ravine Road Bridge - Lake Park
South Abutment
Milwaukee, Wisconsin
Project No. 1G-1503018

GILES ENGINEERING ASSOCIATES, INC.

GILES ENGINEERING ASSOCIATES, INC.

-GEOTECHNICAL, ENVIRONMENTAL AND CONSTRUCTION MATERIALS CONSULTANTS-

N8 W22350 JOHNSON ROAD, SUITE A1/WAUKESHA, WI 53186/(262) 544-0118/FAX: (262) 549-5868

REPORT OF TEST ON CONCRETE CORES; ASTM C39

CLIENT: Graeff

PROJECT: Lake Park Bridge Concrete Cores

DATE: April 1, 2015

PROJ. NO.: 1G-1503018

Client Sample No. : Cores #1 to 6

Lab No.: R150361

Material Description: Cores from Bridge Deck
Cast date(s) unknown

Core Dimensions

Core ID	Date Rec'd	Date Tested	Capped L (in)	Diameter (in)	L / D Ratio	Other Information
1	3/17/15	4/1/15	7.81	3.74	2.09	NW
2	3/17/15	4/1/15	5.79	3.74	1.55	NC
3	3/17/15	4/1/15	7.16	3.74	2.22	NE
4	3/17/15	4/1/15	7.71	3.74	2.06	SC
5	3/17/15	4/1/15	7.60	3.74	2.03	SW
6	3/17/15	4/1/15	7.75	3.74	2.07	SE

Strength Test Data

Core ID	Type of Curing	Date Rec;d	Date Tested	Age (Days)	Area (sq.in.)	Total Load	Compress. Strength	Type of Fracture
1	Air	3/17/15	4/1/15	N/A	10.99	27,410	2494	Conical
2	Air	3/17/15	4/1/15	N/A	10.99	113,130	9882	Conical
3	Air	3/17/15	4/1/15	N/A	10.99	34,790	3166	Conical
4	Air	3/17/15	4/1/15	N/A	10.99	82,900	7543	Conical
5	Air	3/17/15	4/1/15	N/A	10.99	17,530	1595	Conical
6	Air	3/17/15	4/1/15	N/A	10.99	19370	1763	Conical

All cores were sawcut for planeness and capped using sulphur mortar

Reviewed By: David Cornale, P.E.

Y:\excel\MATERIALS LAB\ConCore

II. APPENDIX C – AESBESTOS REPORT

Fax: (414) 771-6370

BULK ASBESTOS ANALYTICAL REPORT
Utilizing PLM and Dispersion Stain Technique

Customer: Jackson-MacCudden, Inc.
9870 Elmleaf Lane
Franklin , WI 53132

Report #: 137199
Received: 10-Apr-2015
Analyzed: 13-Apr-2015

Job ID: FB-104105

Sample ID	% Asbestos	Non-Asbestos Fibrous Components	Non-Fibrous Components	Color	Texture
101A	None Detected		100%	Gray	Resinous
101B	None Detected		100%	Gray	Resinous
102A	None Detected		100%	Gray	Resinous
102B	None Detected		100%	Gray	Resinous

Analyzed By: Kevin Hachey

Test method: EPA/600/R-93/116 and EPA/600/M4-82-020. Quantitation is done by Calibrated Visual Estimation which has an accepted Relative Percent Difference of 35. This report may not be used to claim product endorsement by NVLAP or any agency of the U.S. Government. This test report relates only to the items tested and shall not be reproduced except in full, without the written approval of MICRO ANALYTICAL, INC.

NVLAP Lab Code 101247-0

APPENDIX A
BULK MATERIAL SAMPLE SUMMARY

Client Address:
Milwaukee County Environmental Services
633 West Wisconsin Ave.; Suite 1000
Milwaukee, Wisconsin 53203

Site Location:
Lake Park Foot Bridge
Bulk Sampling

Laboratory: MicroAnalytical
Analytical Method: Polarized Light Microscopy EPA/600/R-93/116
Job Number: FB-104105

Date Sampled: 4/10/2015
Date Sent to Lab: 4/10/2015
Results Received: 4/13/2015- E-mail

Sample #	Sample Description	Sample Location	Asbestos Content
101A	Soft Gray Caulk	West Side; North End	None Detected
101B	Woven Elbow Jacket	West Side; South End	None Detected
102A	Capstone Caulk - Gray/white	West	None Detected
102B	Capstone Caulk - Gray/white	East	None Detected

Thomas R. Jackson (Certification #: AII-572)

III. APPENDIX D – SURVEY DATA

Lake Park pedestrian Bridge over Ravine Road

SURVEY information

Benchmarks

1 benchmark on south side of bridge	SW corner concrete wall @ grand staircase
1(A) benchmark on south side of bridge	#10 pk ion southside of bridge
2 benchmark on the north side of bridg	#11 pk on north side of bridge

Outside edge of deck elevations, outside of parapet

Survey date	9-Mar-15				2-Apr-15		23-Apr-15	
CONCRETE ELEVATION	Outside WEST parapet	Inside West parapet	Inside East parapet	Outside EAST parapet	Outside WEST parapet	Outside EAST parapet	Outside WEST parapet	Outside EAST parapet
South end of abutment	660.06	660.57	660.55		660.08	659.73	660.06	660.06
South corner of octagon	660.39	661.18	661.22	660.65	660.41	660.65	660.39	660.65
Midpoint of octagon	660.66	661.36	661.49	660.87	660.72	660.87	660.69	660.86
North corner of octagon	660.93	661.52	661.55	660.97	660.93	660.99	660.93	660.96
0/10 point South end of deck	660.90	661.54	661.53	660.93	660.90	660.93	660.89	660.91
1/10 point at the pilaster	660.98	660.96	661.03	661.04	660.98	661.04	660.97	661.02
2/10 point at the pilaster	661.05	661.05	661.08	661.08	661.06	661.07	661.04	661.06
3/10 point at the pilaster	661.16	661.18	661.10	661.21	661.18	661.20	661.14	661.18
4/10 point at the pilaster	661.29	661.32	661.37	661.38	661.30	661.35	661.27	661.35
5/10 mid-point	661.43	661.43	661.46	661.47	661.42	661.45	661.41	661.44
6/10 point at the pilaster	661.33	661.31	661.30	661.31	661.33	661.31	661.30	661.29
7/10 point at the pilaster	661.21	661.19	661.20	661.25	661.20	661.22	661.18	661.21
8/10 point at the pilaster	661.09	661.07	661.10	661.18	661.10	661.13	661.07	661.11
9/10 point at the pilaster	661.05	661.06	661.06	661.10	661.06	661.07	661.04	661.06
10/10 point North end of deck	660.94	660.99	660.99	661.02	660.94	660.96	660.91	660.94
South corner of octagon	660.91	661.55	661.54	660.98	660.94	660.95	660.93	660.94
Midpoint of octagon	660.99	661.65	661.63	660.92	661.03	660.91	660.97	660.91
North corner of octagon	660.90	661.51	661.49	660.81	660.90	660.81	660.89	660.80
North end of the abutment	660.66	661.21	661.01	660.39	660.69	660.39	660.69	660.39

IV. APPENDIX E – ESTIMATED CONSTRUCTION COSTS

Malas Engineering LLC

Integrated Innovative Solutions and Excellence in Engineering

Project I.D.:	PUC-1004-MCDT	Date:	5/4/2015
Project Description:	Rehabilitation-Lake Park Drive Bridge over Ravine Road	Client:	Milwaukee County
Computation By:	MNM	Structure No.:	P-40-0576
File Name:	C:\Users\Mahmoud\Desktop\Malas Engineering LLC\PUC1004MCPK Lake Park Ravine Road Bridge\Cost Estimating\[R1_Prelim Estimate_Rehabilitation of Ravine Road Bridge 05042015.xls]Sheet1		

BID ITEM NO.	BID ITEM DESCRIPTION	UNIT	QUANTITY	UNIT COST	TOTAL COST
	Demolition	LS	1	$80,000.00	$80,000.00
	Excavation for structure/Site Grading/Clearing & Grubbing	LS	1	$50,000.00	$50,000.00
	Preserving Historic Elements/Replace with original cut stone railing	LS	1	$150,000.00	$150,000.00
	New Deck	SF	3,400	$25.00	$85,000.00
	Crack Filling	LF	1,500	$30.00	$45,000.00
	Concrete Surface Repairs	SF	1,000	$50.00	$50,000.00
	Spandrel Replacement and/or Repair & Strengthening	LS	1	$150,000.00	$150,000.00
	Aubutment Vaulted Structure Replacement	CY	170	$1,200.00	$204,000.00
	Mobilization	LS	1	$75,000.00	$75,000.00
	Traffic Control	LS	1	$25,000.00	$25,000.00
	Staging & Site/slope Re-grading Restoration/Protection	LS	2	$50,000.00	$100,000.00
	Concreter Staining/Anti-Graffiti	SF	6,500	$12.00	$78,000.00
	Approach Trail & Entrance Reconstruction	LS	2.00	$17,500.00	$35,000.00
	Street Lighting	LS	1	$25,000.00	$25,000.00
					$1,152,000.00
	Total Estimated Design Services:				$230,400.00
	15% Estimated Construction Contingencies/Allowance:				$172,800.00
	15% Construction Management:				$172,800.00
	10% Owner Services:				$115,200.00
	Total Project Cost Estimate:				$1,843,200.00

Malas Engineering LLC

Integrated Innovative Solutions and Excellence in Engineering

Project I.D.: PUC-1004-MCDT
Project Description: Replacement-Lake Park Drive Bridge over Ravine Road
Computation By: MNM
File Name: C:\Users\Mahmoud\Desktop\Malas Engineering LLC\PUC1004MCPK Lake Park Ravine Road Bridge\Cost Estimating\[Prelim Estimate_Replacement of Ravine Road Bridge 04 29 2015_CY Cost.xls]Sheet1

Date: 4/29/2015
Client: Milwaukee County
Structure No.: P-40-0576

BID ITEM NO.	BID ITEM DESCRIPTION	UNIT	QUANTITY	UNIT COST	TOTAL COST
	Demolition	LS	1	$120,000.00	$120,000.00
	Excavation for structure/Site Grading/Clearing & Grubbing	LS	1	$80,000.00	$80,000.00
	Preserving Historic Elements/Replace with original cut stone railing	LS	1	$150,000.00	$150,000.00
	New Arch/Bridge Structure	CY	650	$1,500.00	$975,000.00
	Mobilization	LS	1	$120,000.00	$120,000.00
	Traffic Control	LS	1	$25,000.00	$25,000.00
	Staging & Site/slope Re-grading Restoration/Protection	LS	2	$50,000.00	$100,000.00
	Concrete Staining/Anti-Graffiti	SF	6,500	$12.00	$78,000.00
	Approach Trail & Entrance reconstruction	LS	2.00	$17,500.00	$35,000.00
	Street Lighting	LS	1	$25,000.00	$25,000.00
					$1,708,000.00
	Total Estimated Design Services:				$256,200.00
	15% Estimated Construction Contingencies/Allowance:				$256,200.00
	12% Construction Management:				$204,960.00
	10% Owner Services:				$170,800.00
	Total Project Cost Estimate:				$2,596,160.00

<u>Prefabricated Pedestrian Truss Bridge Replacement</u>

Lake Park Drive Bridge over Ravine Road

Structure P-40-576

Item	Unit	Estimated Quantity	Estimated Unit Cost	Total Estimated Cost
Demolition	LS	1	$ 120,000	$ 120,000
Excavation for structure/Site Grading/Clearing & Grubbing	LS	1	$ 80,000	$ 80,000
New Prefabricated Pedestrian Bridge Structure	LS	1	$ 550,000	$ 550,000
Mobilization	LS	1	$ 120,000	$ 120,000
Traffic Control	LS	1	$ 25,000	$ 25,000
Staging & Site/slope Re-grading Restoration/Protection	LS	2	$ 50,000	$ 100,000
Concrete Staining/anti Graffitti	SF	256	$ 12	$ 3,000
Approach Trail & Entrance reconstruction	LS	2	$ 17,500	$ 35,000
Street Lighting	LS	1	$ 25,000	$ 25,000
				$ 1,058,000
Total Estimated Design Services:				$ 159,000
15% Estimated Construction Contingencies/Allowance:				$ 159,000
12% Construction Management:				$ 127,000
10% Owner Services:				$ 106,000
Total Project Cost Estimate:				$ 1,609,000

<u>Prestressed Concrete Girder Bridge Replacement</u>

Lake Park Drive Bridge over Ravine Road

Structure P-40-576

Item	Unit	Estimated Quantity	Estimated Unit Cost	Total Estimated Cost
Demolition	LS	1	$ 120,000	$ 120,000
Excavation for structure/Site Grading/Clearing & Grubbing	LS	1	$ 80,000	$ 80,000
New Prestressed Concrete Grider Bridge Structure	LS	1	$ 375,000	$ 375,000
Decorative Precast Panels	SF	3800	$ 32	$ 122,000
Mobilization	LS	1	$ 120,000	$ 120,000
Traffic Control	LS	1	$ 25,000	$ 25,000
Staging & Site/slope Re-grading Restoration/Protection	LS	2	$ 50,000	$ 100,000
Concrete Staining/anti Graffitti	SF	874	$ 12	$ 10,000
Approach Trail & Entrance reconstruction	LS	2	$ 17,500	$ 35,000
Street Lighting	LS	1	$ 25,000	$ 25,000
				$ 1,012,000
Total Estimated Design Services:				$ 152,000
15% Estimated Construction Contingencies/Allowance:				$ 152,000
12% Construction Management:				$ 121,000
10% Owner Services:				$ 101,000
Total Project Cost Estimate:				$ 1,538,000