# Lake Park Foot Bridge – Peer Assessment
**Lake Park**
**Milwaukee, Wisconsin**

**June 28, 2016**

**Prepared by**


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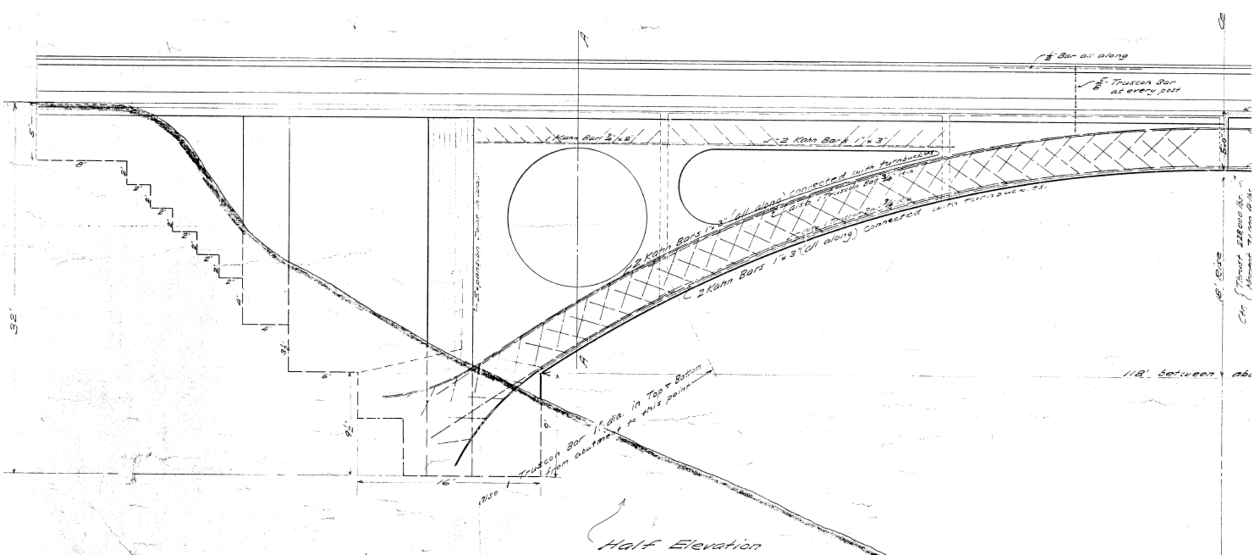
At the request of Milwaukee County Parks, Olson & Nesvold Engineers, PSC performed a limited peer assessment of preliminary project development of the Lake Park Footbridge. The peer assessment included a short site visit, a review of recent project documentation, and the assembly of this short letter report.

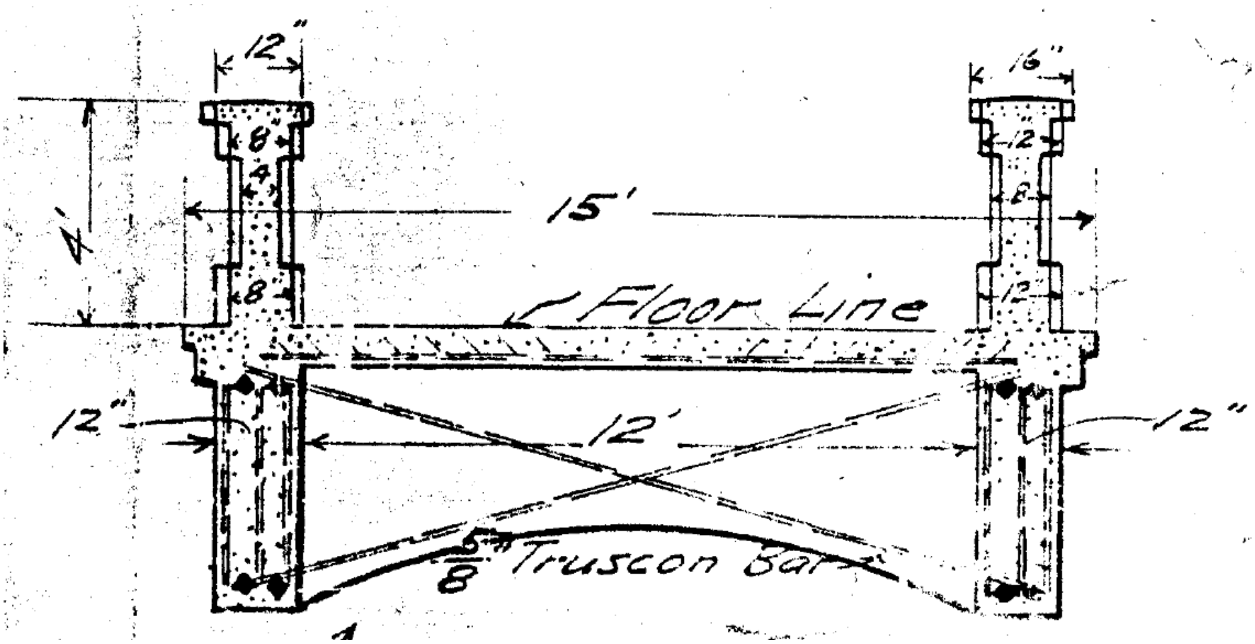
Looking west at the bridge (Photo from the Graef Report)

## Background

The Lake Park Footbridge is a reinforced concrete rib-arch bridge. The ribs support spandrel walls with unique openings (one is tear-drop shaped, the other is circular). Atop the spandrel walls is a reinforced concrete deck and solid concrete parapet railings. Constructed in 1906, parts of the structure contain historic reinforcing steel elements (Kahn System). A detail from the original plans is provided below:



In cross section, the bridge has two arch ribs and a clear distance between railings and arch ribs of 12 feet.




Recently Graef conducted fieldwork and assembled a report in 2015 titled "HISTORIC LAKE PARK ARCH BRIDGE OVER RAVINE ROAD IN-DEPTH INSPECTION REPORT". Inside the report they identified bridge deficiencies, summarized load rating findings, and described alternatives to address the deficiencies. In summary they found the arch ribs and concrete diaphragms to be in fair to poor condition. They found the arch ribs to have adequate load capacity for current design loads. They also found the deck and abutment elements to be in poor to critical condition. Extensive spalling and fractures were the primary deficiencies associated with the poor condition of the deck and abutment elements.

Graef assembled four alternatives for consideration to address the deficiencies. Alternative 1 would rehabilitate deteriorated elements above the arch ribs and would have an estimated service life of 15-25 years. It was estimated to cost $1.8 million. The second alternative would reconstruct the bridge from scratch using new materials. The expected service life for Alternative 2 was $2.6 million. It had an expected service life of 75 years.

Alternative 3 would replace the bridge with single span prefabricated metal superstructures. Alternative 3 was estimated to cost $1.6 million.

Lastly, prestressed concrete beam alternatives were considered. Alternative 4a was simple straight prestressed concrete beams with fascia panels to provide an arched appearance. Alternative 4a was estimated to cost $1.5 million. Alternative 4b did not include the fascia panels and was estimated to cost $1.4 million.

All of the new structures were assumed to have a service life of 75 years.

The purpose of the peer assessment was to review the bridge and available information and examine whether or not a cost-effective rehabilitation alternative could be assembled that would extend the service life of the rehabilitation option.

Olson & Nesvold Engineers approached the task with the following assumptions:

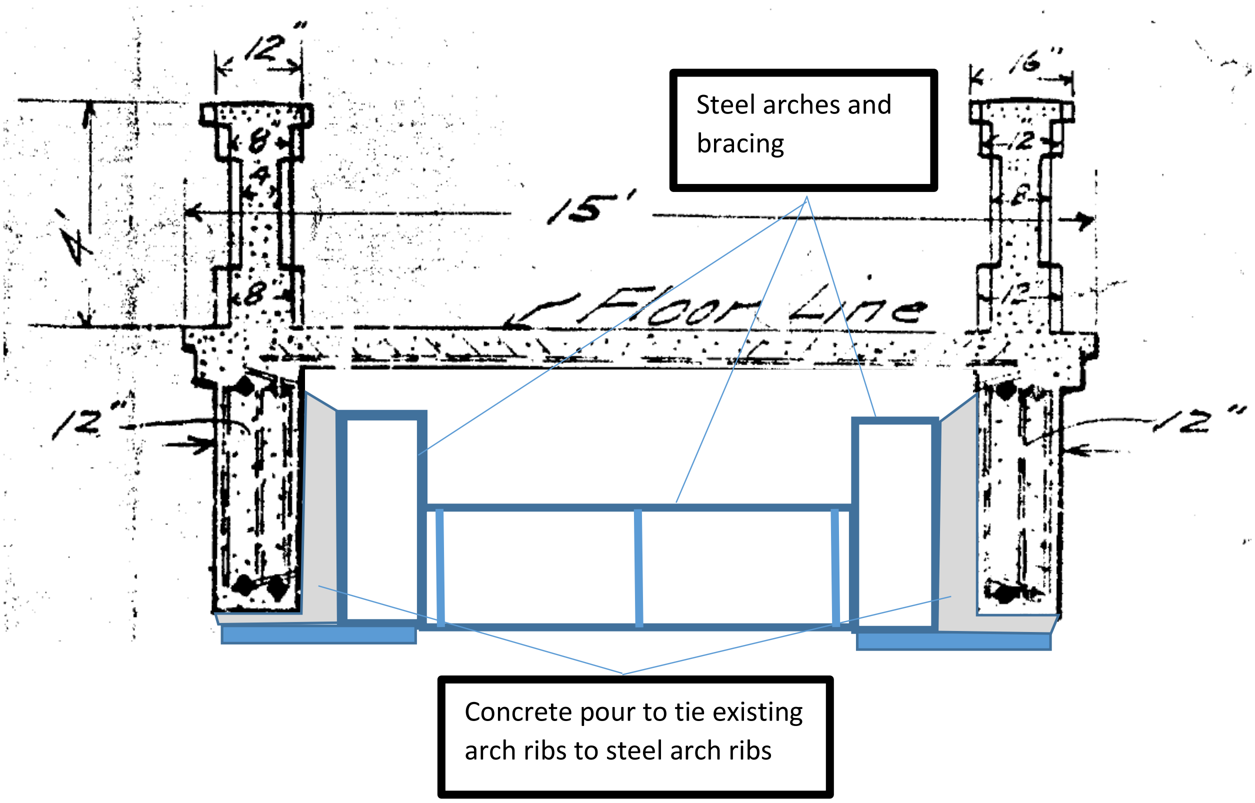
1. The rehabilitation option would need to be performed in compliance with the Secretary of the Interiors Standards for the Rehabilitation of Historic Properties. This includes the National Park Service Preservation Brief 15 "Preservation of Historic Concrete".
2. A 40 to 50-year service life of the bridge should be the design target for rehabilitation. We would expect that the County would continue to perform minor maintenance activities on the bridge as they have in the past.
3. The rehabilitation should recognize the funding limitations available to owners such as Milwaukee County. Essentially, the rehabilitation should be comparable in price to the replacement options assembled by Graef.
4. We understand that it is desirable to retain historic fabric of the bridge. We also understand that it is consistent with the Secretary of the Interior Standards, to reconstruct, deteriorated elements.
5. We understand that the embankments near both abutments are steep and that it would be expensive to mobilize large equipment to work at the site near the abutments.
6. We often look for ways where the existing structure can facilitate construction activities for the rehabilitation of a structure. For the Lake Park Footbridge, we understand that the arch ribs and the existing diaphragms have sufficient structural capacity to carry modern pedestrian bridge live loads (pedestrian and vehicle). Consequently, modest construction loads could be supported from these elements.
7. We understand that at least one early photograph of the bridge illustrated it with steel bracing between the arch ribs.

Using information observed at our site visit along with past project documentation and the above assumptions we believe there is a reasonable rehabilitation option with a service life of 40-50 years. It is our understanding that an independent cost estimator will assign a construction cost to the alternative described below.

ONE rehabilitation option characteristics.

One of the concerns Graef highlighted in their report was that the arch ribs would reach the end of their service life after another 15-25 years. We concur with Graef that the existing are ribs would likely be compromised to a significant degree, if simply patched as they have been in the past. The corrosion on the Kahn bars would continue and the spalling would continue.

To address this service life concern, we believe a new structural steel framework (including steel arch ribs) could be constructed just inside of the existing arch ribs. The steel arch ribs would be galvanized to provide some passive cathodic protection to the Kahn bars in the existing arch ribs. In addition, a support plate would be detailed under the existing arch ribs to prevent any future spalls from dropping below. The support plate would function much as a lintel does in masonry construction. The steel framework could be designed to carry all the loads between the abutments or some smaller fraction. Resulting in a reduced load demand for the existing arch ribs.




The steel frame work would also be detailed to accept construction walkway loads.

Prior to steel framework installation, we envision a modest amount of work done from manlifts to prepare the arch ribs for the framework. This would include temporary bracing above the locations of the existing small transverse struts. After which the existing small struts would be cut out to provide room for the steel framework. At the location of the tall diaphragms, the lower 2 to 4 feet would be cut out to provide room for the framework.

Once the diaphragms and struts have been prepared, loose spalls would be removed from the existing inside and bottom faces of the existing arch ribs. This would be followed by drilling and doweling in anchors to engage the closure pour with the steel framework. The steel framework would have shear studs welded to the outside face of the section to engage the closure pour.

The steel framework would be supported at the existing thrust blocks.

The current thrust blocks have compromised material between the existing arch ribs. As part of the reconstruction of these areas we would embed galvanized steel components to act as a hinge for the arch rib and to facilitate the erection of the steel framework.

We envision that the steel framework would be shipped to the site in three pieces.

We also envision that winches could be used to lift the steel framework segments into position. Segments would be unloaded from the fabricators trucks using telehandlers or similar equipment.

Holes would be drilled through the existing bridge deck in the vicinity of the full depth diaphragms near the small end of the tear shaped openings in the spandrel walls. Winch reactions would be supported by the full depth diaphragms and carried by the existing arch ribs. Each side segment would be installed first and lastly the center segment would be lifted into position.

We envision that removals of the existing railings, deck, and spandrel walls would take place next. This would most likely be performed by sawcutting components and lifting pieces off the bridge. We anticipate that the spandrel walls would be removed with a “bottom” cut just above the top of the existing arch ribs.

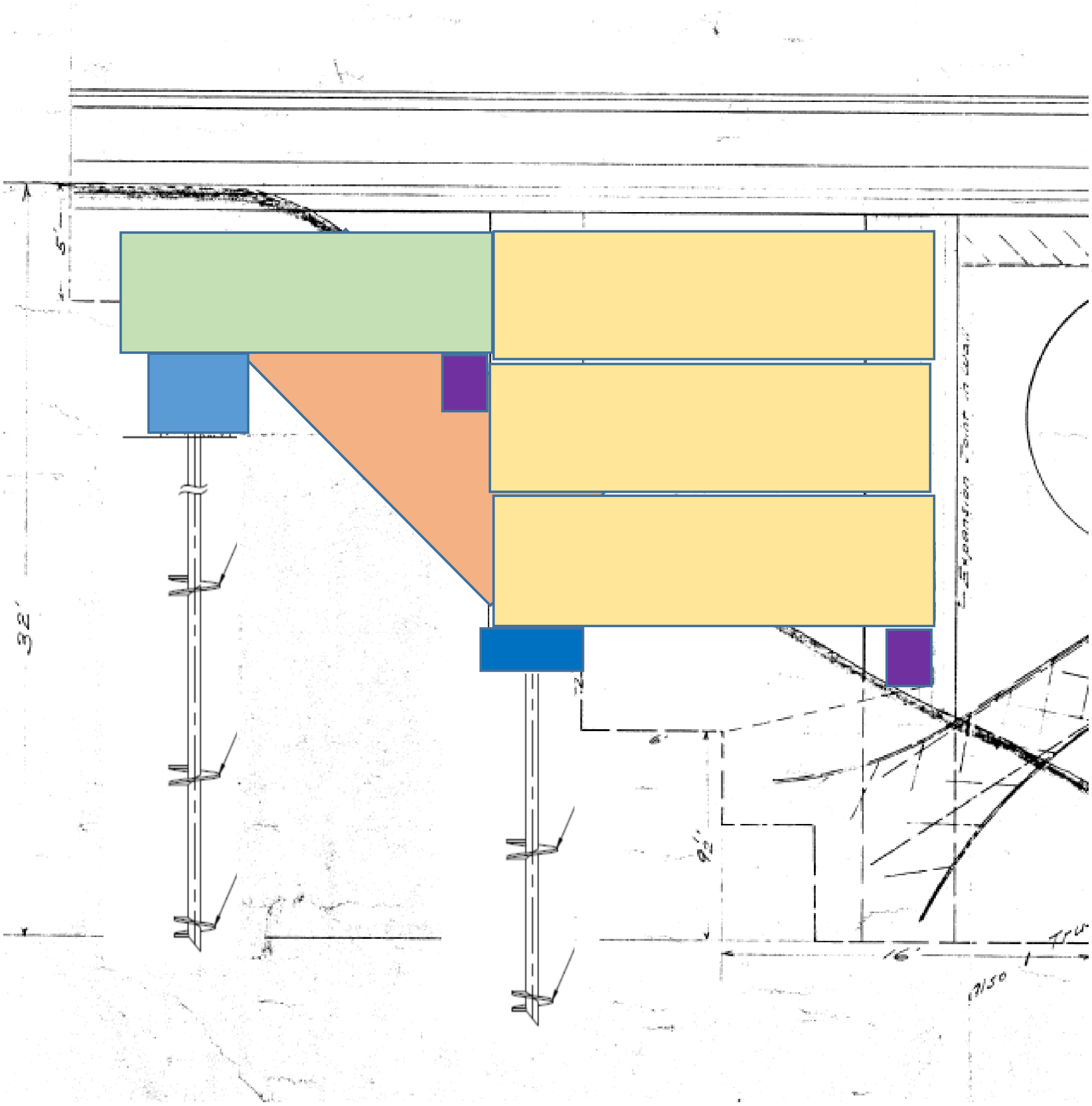
Reconstruction of components above the arch ribs would begin by drilling and epoxying vertical dowel bars into the top face of the existing arch ribs or connecting to the new steel arch ribs. The spandrel walls would be reconstructed following the original geometry. The walls would be reinforced with minimum steel according to current design standards.

Above the spandrel walls, precast deck sections would be installed. The deck section would be 4-5 inches deep and topped with a 1-2 inch overlay. The precast panels would include the lower rail elements and function as a curb for drainage. We envision spandrel wall vertical reinforcement extending into pockets in the deck sections. These pockets would be grouted to tie the walls and deck section together. Upper railing components would cover the pockets.

The abutments are in extremely poor condition and in a difficult to access location. There is extensive deterioration in both abutments that includes shearing of the vertical walls due to unbalanced earth pressures.

County staff indicated that the geotechnical conditions vary considerably in this area of the park. With the relatively modest loads needed to support the bridge it is anticipated that helical anchors could be used to support the abutment components.

A handful of box culvert sections oriented vertically would be used as the “backbone” to support wall elements. These could be stepped to mimic the existing wall foundation steps. The sections would be “stitched” together with galvanized threadbars to create a “jumbo” beam that spans from the vertical wall over the thrust block to the supports with the helical anchors.



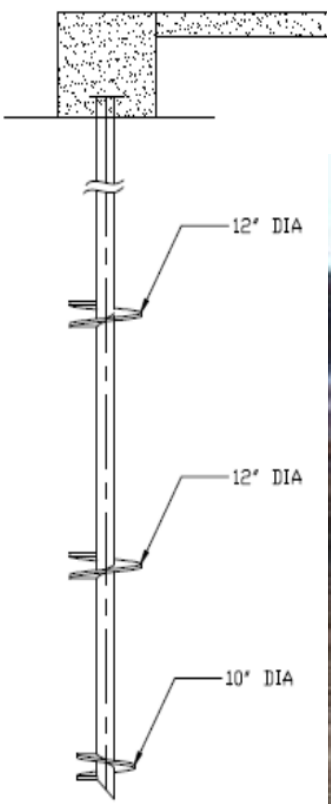
The abutment beam consists of a purple reaction pad constructed on the top of the thrust block. It includes a dark blue footing to support the yellow box culvert sections. After the yellow box culvert sections are in place a purple support block would be cast against the yellow box culvert sections. The vertical helical anchors are envisioned to reach a competent geotech level at the same level as the bottom of the thrust block. Atop the helical anchors a pile cap is cast. Upon the pile cap and the second purple block the green box culvert section is installed. The last step in the construction of the "mega" beam is the infill triangle wall (light orange).

Once the abutment beam is constructed it could be temporarily capped with timber mats to support construction equipment such as small truck cranes. With lifting equipment positioned over the thrustblocks of the bridge, demolition and reconstruction activities would be expedited for both the main span and the abutments.

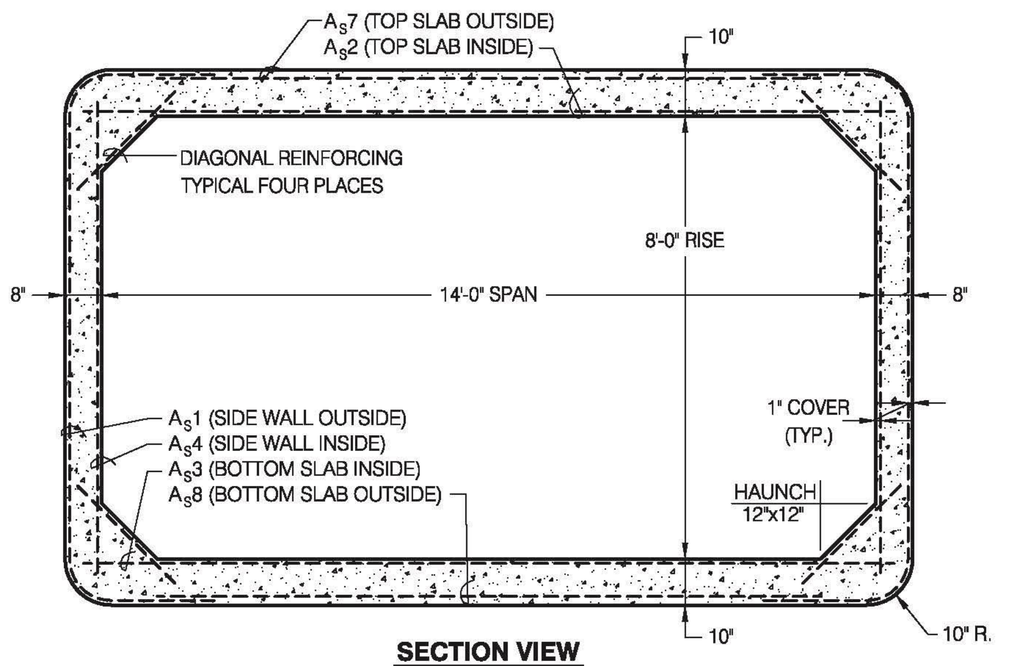
The following pages contain capacity information for helical piles, images of equipment to install helical piles, and a typical box culvert section that would fit between the abutment walls.

**Table A1: Helical Anchors Product Rating**

| Helical Anchors Products | Shaft Size (in) | Wall Thickness (in) | Ultimate Tension Strength (lbs) | Compression Load Limit (lbs) | Ultimate Torsional Strength (ft-lb) | Installation Torque Factor (k) | Capacity Based on Torsional Strength (lbs) |
|---|---|---|---|---|---|---|---|
| **TS238190** | 2.375 OD | 0.190 | 125,000 | 100,000 | 6,000 | 9 - 10 | 60,000 |
| **TS238254** | 2.375 OD | 0.254 | 125,000 | 135,000 | 9,000 | 9 - 10 | 90,000 |
| **TS278217** | 2.875 OD | 0.217 | 180,000 | 140,000 | 13,000 | 8 - 9 | 117,000 |
| **TS278276** | 2.875 OD | 0.276 | 180,000 | 180,000 | 16,000 | 8 - 9 | 144,000 |
| **TS312254** | 3.50 OD | 0.254 | 250,000 | 210,000 | 18,000 | 6.5-8 | 144,000 |
| **TS312368** | 3.50 OD | 0.368 | 250,000 | 290,000 | 27,000 | 6.5-8 | 216,000 |
| **TS412250** | 4.50 OD | 0.250 | 275,000 | 260,000 | 30,000 | 5-6.5 | 195,000 |
| **TS412337** | 4.50 OD | 0.337 | 360,000 | 350,000 | 48,000 | 5-6.5 | 312,000 |
| **TS500362** | 5.00 OD | 0.362 | 413,000 | 413,000 | 74000* | 4.5-6 | 413,000 |
| **TS512361** | 5.50 OD | 0.361 | 510,000 | 466,000 | 90700* | 4-5.5 | 466,000 |
| **TS700498** | 7.00 OD | 0.498 | 999,000 | 814,000 | 200000* | 3-4.5 | 814,000 |

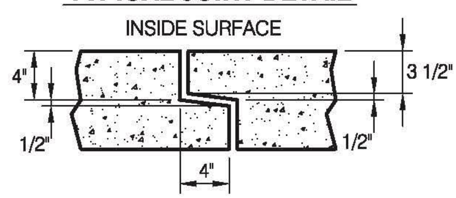



# 14' x 8' SINGLE CELL BOX CULVERT DESIGN DETAILS




**SECTION VIEW**

**TYPICAL JOINT DETAIL**




WEIGHT / FT. = 5,650 LBS.

LOADING, DESIGN METHODS AND
MATERIALS COMPLY WITH ASTM C789 or C850.
Standard laying length = 6'-0"
WWF ASTM A185, fy = 65 KSI
Concrete Strength, f'c = 5 KSI
Box culvert design and analysis is performed on
the BOX CAR computer program.

Cretex Concrete Products
the shape of solutions

# APPENDIX

## Wood, Kevin

**From:** Steve Olson <steve.olson@one-mn.com>
**Sent:** Thursday, June 30, 2016 8:26 AM
**To:** Stave, Karl
**Cc:** Wood, Kevin; Haley, Kevin; Mahmoud Malas
**Subject:** RE: ONE Draft Letter Report

Karl,

We would anticipate that the spandrel walls would be replaced with the same shaped walls and the same openings as the original spandrel walls.

The steel arch ribs would not be visible through the spandrel openings. On site I think I mentioned using tube sections. After thinking about it a bit more, I think that built up sections would be a better choice. The plate steel could simply be cut on the radius of the arch and there would be no need to reshape a tube to a curved profile. Heat curving or cold bending the section would probably be pricy. The steel arch or a steel arch with concrete fill would be able to have more capacity than the existing arch ribs in the same profile.

You are correct, on site I was thinking about using the existing walls as forms for new walls. After noodling about it a bit more, I thought three things, one, the horizontal offsets would get tricky to deal with at 6 or more inches in offset, and second, more importantly, being able to get construction equipment to the thrust block area from above would probably be more important to a contractor. Having the ability to readily lift out old elements and lift in equipment and forms, etc. will be important. And lastly, I think the benefit of using the walls diminished a bit when the clayey soils finally locked into my noggin. The box culvert appealed to me because of the construction access it offers and it also is a good section to resist lateral earth pressures.
So, in summary, I probably should have clarified my transition in approach to the abutments a bit more in the report. New walls make sense to me.
Working on that 1 on 1 slope to install wall footings will be tricky. If new walls are constructed on "pads" like Kevin W. suggested, they may not need to be located below the "frost" line if they are "hung" from the box culvert sections. The idea of some precast walls for portions of the abutments has some merit to me.

Steve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**From:** Stave, Karl [mailto:Karl.Stave@milwaukeecountywi.gov]
**Sent:** Wednesday, June 29, 2016 5:05 PM
**To:** Steve Olson <steve.olson@one-mn.com>
**Cc:** Wood, Kevin <kevin.wood@graef-usa.com>; Haley, Kevin <Kevin.Haley@milwaukeecountywi.gov>; Mahmoud Malas <mmalas@malasengineering.com>
**Subject:** RE: ONE Draft Letter Report

Steve,

Thanks for the quick turn around with your letter report. Here are a couple questions:

- The report indicates the spandrels will be removed by sawcutting directly above the arches. Are the spandrels being replaced or left out? If left out, will the steel arches/bracing be visible behind the arches?
- If the spandrels are replaced, will the steel arches/bracing be visible through the circle and teardrop?
- On site you talked about about stabilizing the existing abutment walls and using them as forms for an outer cast concrete wall. The report indicated the box culverts would be used as a working platform for demolition and reconstruction for both the main span and abutments. This seems to indicate the abutment walls would be removed and new walls cast in place or pre-cast walls installed. Is that correct?

Given this is just at a concept stage, you may not have answers to some of the above. I'm anticipating questions from the Work Group so anything you can clarify would be appreciated.

Thanks,

**Karl Stave, P.E.**
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DAS - Facilities Management Division
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Suite 1000
Milwaukee, WI 53203
(414) 278-4863

**From:** Steve Olson [mailto:steve.olson@one-mn.com]
**Sent:** Tuesday, June 28, 2016 4:56 PM
**To:** Stave, Karl <Karl.Stave@milwaukeecountywi.gov>
**Subject:** ONE Draft Letter Report

Karl,

Please find attached a PDF file. The file contains our letter report on the Lake Park Footbridge.

If there are questions or concerns, please do not hesitate to contact me.

Sincerely,

Steve Olson

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