Municipality of
Huron Shores

Dean Lake Bridge

Dean Lake Road, 110m South of Hwy 17

Load Evaluation and Condition Assessment

19-1414
October 2020

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ENGINEERING

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October 8, 2020
19-1414

Municipality of Huron Shores
7 Bridge Street, P.O. Box 460
Iron Bridge, ON, P0R 1H0

Attn: Debbie Tonelli – AMCT, Clerk/Administrator

Re: Dean Lake Bridge Load Evaluation and Condition Assessment

Dear Mrs. Tonelli:

TULLOCH Engineering Inc. (TULLOCH) has completed our load evaluation and condition assessment for the Dean Lake Bridge.

Please find enclosed a report outlining our general observations noted during our detailed inspection and the results of our load evaluation completed for the Dean Lake Bridge in accordance with Section 14 – Evaluation of the Canadian Highway Bridge Design Code CSA-S6-14.

The report outlines some recommendations in terms of repair items which should be completed to maintain the service level of the bridge at the identified load posting. Also included are the calculations for the required load posting for the bridge for your information. To avoid any confusion, the bridge should be posted with a single load posting of 10 tonnes and signs erected depicting “No Trucks” to prevent heavy trucks from crossing.

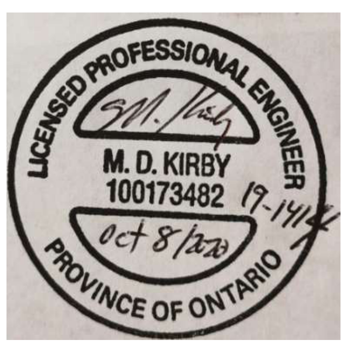
General maintenance items outlined in the 2019 OSIM inspection report should be also completed if they have not been completed since our inspection in the summer of 2019. The bridge shall continue to be inspected through biennial bridge inspections in accordance with the respective standards and we anticipate that this bridge will require major rehabilitation or replacement within the next 2 years.

We trust the enclosed is adequate for your needs at this time. If there is anything further we can provide or should you have questions regarding the information provided herein, please contact us at your convenience.

Sincerely,

TULLOCH Engineering Inc.

Matt Kirby



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Revision Log

Revision #	Revised By	Date	Issue / Revision Description
0	M.K.	October 8, 2020	Final Report

EXECUTIVE SUMMARY

TULLOCH Engineering (TULLOCH) was retained by the Corporation of the Municipality of Huron Shores (Client) to conduct and supervise a condition assessment with the intent of performing a load evaluation of the Dean Lake Bridge located approximately 20km southeast of the Town of Iron Bridge, Ontario adjacent to Highway 17E.

The following is a summary of work completed during the visual inspection:

- TULLOCH field personnel attended the site on August 12th, 13th, and 14th, 2019 to complete the condition assessment and document conditions throughout the bridge.
- Inspection of the bridge superstructure and concrete abutments was completed by Rope Access Maintenance personnel with video cameras attached to their helmets which allowed the TULLOCH inspectors to monitor the inspection and bridge conditions.
- Inspection of the bridge substructure elements below the water surface was completed by Porpealia Repairs personnel who also had a video camera attached to their helmet which allowed the TULLOCH inspectors to monitor the inspection and document conditions of the piers and embankments.
- The conditions were recorded in accordance with the Ontario Structural Inspection Manual – Field Inspection Guide, April 2008 (OSIM).

As shown in Figure 1 below, the bridge consists of a triple span through truss bridge measuring approximately 111.0m in total length (37.0m each span). The concrete abutments and piers are severely weathered and will require extensive concrete rehabilitation in order to maintain the function of the bridge.

Figure 1 - Elevation of Middle and South Spans

The asphalt wearing surface is generally in fair condition with light to medium wear throughout with some light potholes. The deck has previously been suspected of retaining water and was cored in all 3 bays. Upon coring, the northernmost span was found to be holding water throughout the entire span. The other 2 spans showed no signs of retaining water.

The exposed steel through trusses above the bridge deck are in fair to poor condition with light localized surface corrosion and minor impact damage. The bottom chord is severely corroded in localized areas due to winter sand/gravel buildup as shown in Figure 2 below. Several loose and missing rivets were noted on the top chords, diagonal members and top brace bays which are suspected to have been missing since original construction.

Figure 2 - Severe Section Loss in Bottom Chord

Minor settlement of the approaches has occurred and there are light depressions, light potholes, wheel rutting and raveling. The wooden guiderail posts on both approaches have moderate to severe decay, medium to wide splits and checks throughout. The terminal end treatments have moderate to severe impact damage and light to moderate corrosion at ground level.

The steel floor beams & stringers are in good to fair condition with localized moderate corrosion of the gusset plates and rivets. The stringers have localized moderate corrosion at the connections and are the governing factor for the load posting.

Significant repairs are expected in the next 5 years to maintain use of the bridge. The total project cost estimate for repairs involves; deck replacement, new joints, localized steel repairs, concrete repairs and refacing of abutments and piers, replacement and upgrades to approaches/guiderail is estimated at approximately $4,000,000+HST including engineering and construction administration fees. The estimated replacement cost of the structure is $8,175,000+HST.

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1. INTRODUCTION

TULLOCH Engineering Inc. was retained by Corporation of the Municipality of Huron Shores (Client) to conduct and supervise a condition assessment with the intent of performing a load evaluation of the Dean Lake Bridge. The bridge is located approximately 20km southeast of the Town of Iron Bridge, Ontario and spans the Mississaugi River approximately 50m from Hwy 17E and is a detour route for a provincial highway. In accordance with Ontario Regulation 104/97 – Standards for Bridges, the structural integrity, safety and condition of every bridge shall be determined through the performance of at least one inspection in every second calendar year under the direction of a professional engineer (O. Reg. 472/10, s. 2). Bridges within the municipality are considered to be within Provincial lands and the inspections were completed in accordance with the Ontario Structure Inspection Manual (OSIM).

The objectives of the condition assessment are as follows:

- To identify critical maintenance, rehabilitation, and/or replacement needs of the bridge;
- Assess corrosion levels throughout various regions of the bridge that were not readily accessible from the abutment embankments or bridge deck and;
- Perform an underwater inspection to verify the integrity of certain bridge elements that are not visible below the water surface.

This report contains descriptive summaries of each bridge element, relevant photographs, load evaluation results and a bridge replacement cost estimate.

2. BACKGROUND

The Dean Lake Bridge is located approximately 20km southeast of the Town of Iron Bridge, Ontario and spans over the Mississaugi River just 50m from Hwy 17E. The Dean Lake Bridge is a three (3) span through truss bridge which is approximately 111.0m long (37.0m each span).

The bridge is supported by two concrete abutments and two intermediate concrete piers. The abutments and pier foundations consist of timber piles driven into the riverbed to an unknown depth. The piles are capped with concrete and protected from erosion/ice damage by steel sheet piling that was installed after original construction. The original construction drawings from 1908 depict rock filled timber cribs at the base of the two abutments. The rock filled timber cribs were removed and rehabilitated/repaired with sheet piling and are no longer visible.

The bridge has undergone several rehabilitations in its history, mainly in 1963,1988, and 2008. A rehab drawing supplied by "Department of Highways Ontario" dated 1963 show the original timber deck being replace with a new 3" thick creosote timber deck. The drawing also notes replacement of several original stringers from 9" channels and beams to 8" channels and beams.

A drawing set provided by "Kresin Engineering & Planning" dated 1988 show the bridge had undergone a significant rehabilitation involving the replacement of; the old 3" timber creosote deck with a new 5.5" thick laminated creosote timber deck, elastomeric bearing pads, new concrete ballast walls, flex beam guiderail on wooden posts on the approaches and new flex beam guiderail on the structure, replacement/modifications to existing stringer beams and reinforcing/ modifications to the gusset plates at the base of the connection nodes of specified panel points.

TULLOCH was provided with information from the Client relating to the last rehabilitation which occurred in 2008 by the Newton Group. The old 5.5" thick laminated creosote timber deck was replaced with a new laminated timber deck wrapped in fiberglass and embedded with an epoxy resin. The laminated timber deck system was then topped with a layer of asphalt as a wearing surface. The deck sections on the north span had to be pushed in place "forcibly" by an excavator bucket. This could explain why the wearing surface on the north span is more damaged than the south and center spans. Another possible scenario is that the northernmost deck panels were the first panels manufactured and they may have been the panels selected to be load tested at the plant for strength verification. Correspondence from the time of testing states that "the deck panels satisfy the design requirements and can be installed at the bridge site".

The timber deck panels were a proprietary system prepared/fabricated by Guardian Bridges or Newton Group. The panels were constructed in a shop around Guelph, Ontario and township personnel visited the facility during the fabrication process. Two panels were load tested to prove capacity and verify structural capacities. It is unknown if the tested panels were eventually installed on the structure. The asphalt topping on the 2 northern deck panels (first two sections installed during the 2008 rehabilitation) have had issues since being installed. Sections of the asphalt were cut out and replaced with cold mix patching material from Elliot Lake, Ontario.

Figure 3 below provides a visual representation of the location of the bridge site relative to the town of Iron Bridge, Ontario as well as the town of Blind River, Ontario.

Figure 3 - Dean Lake Bridge Location Map (Google Earth, 2019)

Figure 4 below provides a general depiction of the bridge spans/layout and the locations of where the intermediate pier supports have been installed.

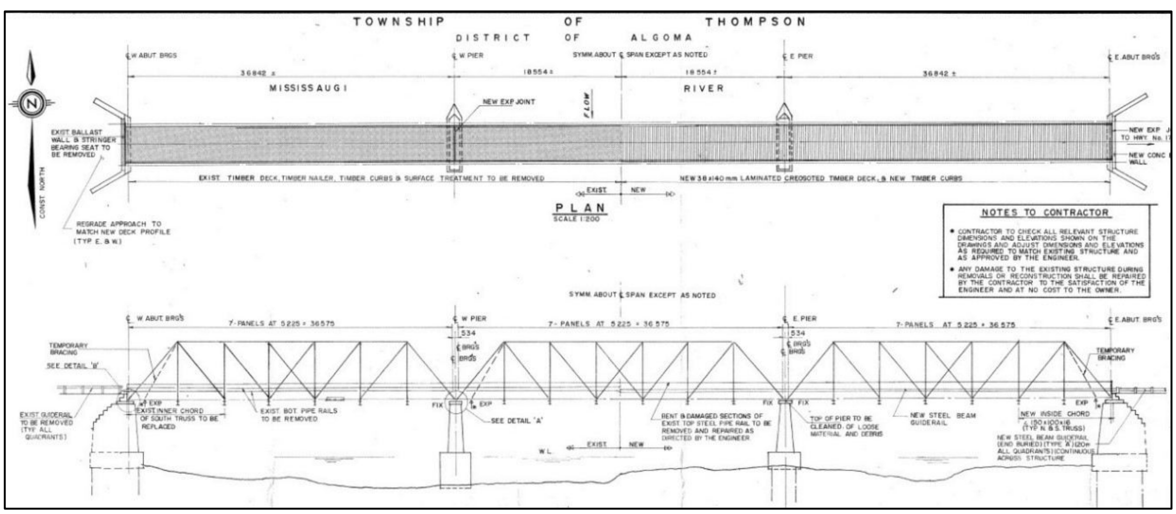

Figure 4 - Dean Lake Bridge Rehabilitation Drawing circa 1988

3. INSPECTION METHODOLOGY

TULLOCH field personnel attended the site on August 12th, 13th and 14th, 2019 to complete a detailed visual inspection within an arm's reach of each element (or as close as practical). Reviewing the structural elements above the water surface was completed with the use of Rope Access Maintenance (RAM) inspection personnel. The steel was scraped throughout and the thicknesses of the steel members were measured to understand if and how much section loss was occurring at each of the various locations. The review of the structural elements below the water surface was completed with Porpealia Repairs (underwater dive team) personnel. A diver inspected the abutment & pier foundations for any evidence of corrosion, undermining, scouring or structural damage.

The structural elements are identified by primary groupings and sub-groupings of each element. The condition of each element is quantified and assessed with a rating of 'excellent', 'good', 'fair', or 'poor'. A glossary of definitions is given in Appendix A.

4. ASSUMPTIONS

Below is a list of assumptions TULLOCH used while completing the load evaluation and condition assessment:

- The stringers are placed as per the drawings provided by the municipality from I and F Engineering Corp dated October 10, 2007 with the exception of the interior stringers in place are actually W250x33 and not W250x45 as stated on dwg S01. There may have been a revised set of drawings that TULLOCH is unaware of.
- The laminated wood deck was designed for a max 16/24/28 tonnes (single unit vehicle, two-unit vehicle, vehicle train) and is comprised of 20f-E Bending Grade glue-laminated Douglas fir wood.
- The laminated wood deck is bolted/attached to the girders at a minimum interval of 3 times per span between floor beams. The deck provides lateral stability to the stringers. The wooden deck only provides transverse bending across the stringers and adds no additional bending capacity to the stringer due to any composite action between the stringers/deck.
- The center of any truck wheel loads cannot be any closer to the centerline of the truss than 0.7m.
- All steel members are original steel from 1908 with the exception of the interior girders which were replaced in 2008 and some areas of the bottom chord which were replaced in 1988.
- Rivet shank diameter is 19mm (3/4”).
- Steel grade for bridge members from original construction (1908) is fy = 210 MPa, 1988 construction is fy=250 MPa and 2008 construction is fy=345 MPa as per the Canadian Highway Bridge Design Code.

5. OBSERVATIONS

During the visual inspection of the substructure and superstructure components of the bridge a number of observations were noted. A summary of these observations is as follows:

5.1 Deck Top & Wearing Surface

The bridge deck is comprised of a laminated timber deck that is wrapped in fiberglass and reinforced with an epoxy resin. The deck system was then topped with an asphalt wearing surface.

The asphalt wearing surface is generally in fair condition with light wear throughout, light to medium transverse cracking, patching at intermediate deck joints, wheel rutting on the north span, and localized potholes. Figure 5 below shows the typical deck wearing surface.

Figure 5 - Typical Wearing Surface on North Span (Patching of the 1st installed deck panel)

Based on previous inspections, it was suspected that the deck on the north span of the bridge may be retaining water within the deck system. The north span deck was cored with a 1/2" hole approximately 4" in depth and at various locations throughout its length. Upon completion of coring the side of the deck on the most northern section, water proceeded to "shoot out" horizontally from internal pressure/buildup. After 2 minutes, the water continued at a steady drip for approximately another 2.5 hours. The deck on the northern span was cored again in various locations with a similar drip occurring at each location. However, the deck on the center span and south span were also cored in various locations and no excessive water/moisture was found to residing within the deck panels.

5.2 Joints

At the end of each span there is a protective armoring plate. The armoring plates have light to moderate wear and light corrosion, which is typical at all joint locations along the bridge length. The joints between deck panels within each span consist of a cementitious material. These joints have narrow to wide cracking, separation or debonding, missing sections, and sections which have been previously patched. The joints are in poor condition overall.

Figure 6 - Typical Deck Joint

5.3 Curbs and Railings

There were no curbs present on the structure at the time of inspection. The steel flex beam guardrail on the bridge is bolted to the truss vertical members and has localized impact damaged, scrape damage, and light corrosion throughout as shown in Figure 7 below. A loose guiderail bolt was also noted in center span on the east truss.

Figure 7 - Typical Guiderail and Railing on Structure

5.4 Steel Stringers & Floor Beams

The steel stringers which support the deck are in good condition with light to moderate corrosion on the top flange directly below the deck panel joint. The floor beams are generally in fair condition with medium to severe corrosion/section loss at connection locations. The corrosion/section loss is being accelerated by the buildup of gravel and debris that migrates from the top of the bridge deck down onto the connections. Figure 8 below shows the typical stringer and floor beam layout. Rust jacking of the gusset plates at connection locations was also noted throughout the bridge during the inspection. This rust jacking is typical across the bridge structure at connection locations and is shown in Figure 9 on the following page. The floor beam connections have fairly significant breakdown of the protective coating as shown in Figure 9 on the following page.

Figure 8 – Typical Deck Soffit Layout

Figure 9 - Typical Floor Beam Connection/Flaking of Protective Coating

5.5 Rivets/Connections

During the inspection, several loose and missing rivets were noted on the top chords, diagonal members and top brace bays. The majority of the loose rivets were in the top chord and diagonal bracing of the south span. Figure 10 below shows typical rust jacking at upper truss connections. Figure 11 below shows a missing rivet and a number of loose rivets in one connection in the south span.

Figure 10 - Typical Rust Jacking at Upper Truss Connections

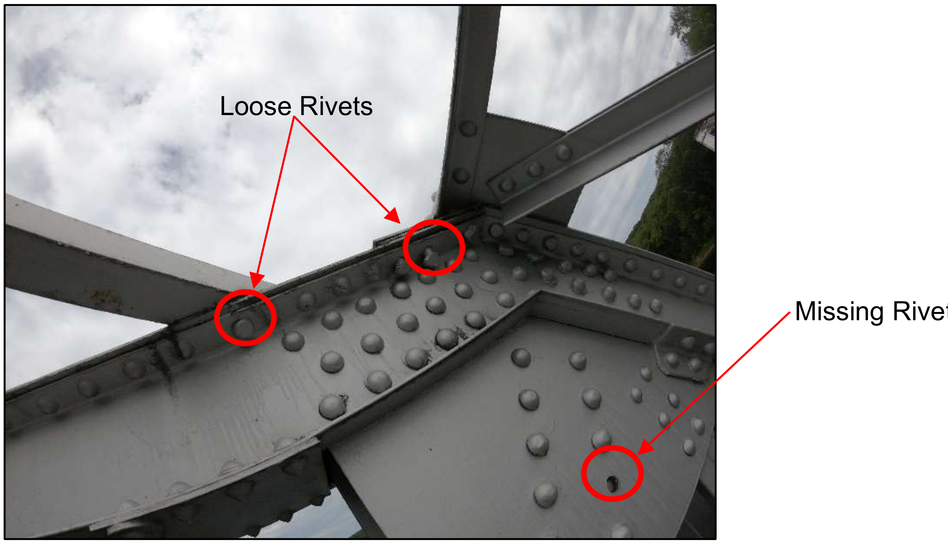

Figure 11 - Missing/Loose Rivets in South Span

5.6 Steel Trusses

The steel trusses primarily have light localized surface corrosion where the protective coating has blistered and flaked off. The failure of the coat may be attributed to poor surface preparation or contaminates below the latest coating. The bottom chord has severe to very severe section loss (see Appendix C) in multiple members due to gravel/sand/debris constantly residing on the steel as shown below in Figure 12. Figure 13 shows a deflection in the bottom chord. The cause and date of when the deflection began surfacing is unknown. The truss elements are in fair condition with localized sections that are in poor condition.

Figure 12 - Severe Section Loss in Bottom Chord at Connection Gusset Plate

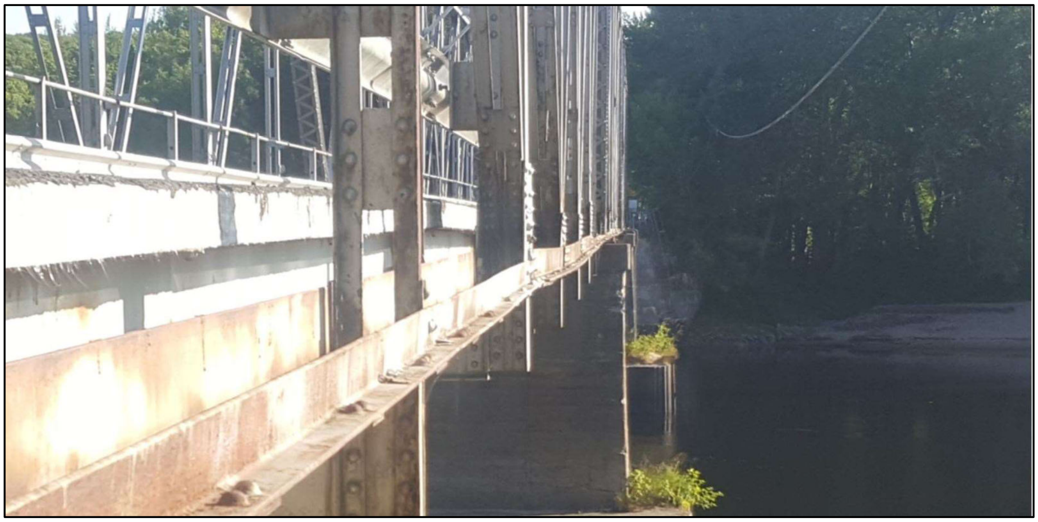

Figure 13 - Deflection in Bottom Chord (South Span)

5.7 Bearings

The bearing pads for each span consist of steel plates at the fixed ends and elastomeric bearing pads at the movement ends. The steel bearing pads exhibited light to moderate corrosion and pitting while the elastomeric bearing pads exhibited light cracking and weathering. The steel and elastomeric bearing pads were covered in gravel and debris at the time of inspection as shown below in Figure 14. The bearings are considered to be in good to fair condition.

Figure 14 - Gravel Buildup on Bearing Pad

5.8 Intermediate Support Piers

The concrete piers were generally in fair to poor condition. The face of the piers are highly weathered with light to severe spalls, scaling, wide cracking & efflorescence, and localized delamination throughout as shown below in Figure 15 and Figure 16. The top portion of the piers directly around the bearing seats have light to moderate spalls and have wide localized cracks. The armouring angles on the upstream faces have light surface corrosion and pitting. The piers appear to be mass concrete with no visible signs of rebar reinforcing.

Figure 15 - Spalls, Cracking and Efflorescence at South Intermediate Pier

Figure 16 - Wide Crack and Severe Delamination at Bottom of South Intermediate Pier

5.9 Concrete Abutments

The concrete abutments are generally in poor condition. Similar to the intermediate support piers, the abutments are highly weathered with widespread spalls, scaling, wide horizontal and vertical cracks, delaminations, efflorescence and disintegration throughout as shown on the following page.

Figure 17 - Wide Cracks, Spalls, Delamination and Efflorescence and Map Cracking in Concrete Abutment/Wingwall

5.10 Approaches and Guiderails

The surface treated approaches were in fair to poor condition with some light to medium cracking, wheel rutting and some light gravel build up. The steel beam guiderail is in good condition with light corrosion in areas with to impact damage which have removed the galvanized protective coating. The supporting wood posts for the guiderail system are severely decayed with wide splits, checks and moderate to severe weathering. The flex beam has buried end treatments at both ends of the bridge with impact damage.

5.11 Signs

A 10-16-20 tonne load posting, and single lane bridge sign were present at both ends of the structure while a 15km/h speed sign was only present at the north approach. No hazard markers were present at the structure to identify the corners of the bridge and no delineators were present to show the start and stop locations of the guiderail for snowplow vehicles.

5.12 Embankments, Streams and Waterways

The exposed vegetated & rock protected slopes on the embankments surrounding the bridge were generally in good condition with no evidence of scouring or erosion. Rock protection is currently installed at the northwest quadrant along the riverbank which was planned previously to remediate scour issues. The rock protection in the northwest quadrant is stable and in good condition, shown below in Figure 18.

Figure 18 - Rock Protection in NW Quadrant Only

6. ANALYSIS

6.1 Load Evaluation Results

Using the information obtained from the enhanced biennial inspection condition assessment, the bridge was evaluated for a load posting in accordance with the Canadian Highway Bridge Design Code CSA S6-14 (CHBDC) – Section 14, Evaluation. The evaluation section of the CHBDC outlines a procedure for establishing load postings for structures using f-factors, for elements of the bridge under loadings caused by three Evaluation trucks (i.e. Evaluation Level 1, 2 and 3). The Evaluation Level trucks represent vehicle trains, two-unit vehicles and single unit vehicles respectively. An analysis of the bridge was completed with the aid of a computer analysis software known as STAAD. The resulting outputs (bending moments, shears, axial loads) of the analysis software were used to calculate the f-factors for each element of the structure in accordance with Section 14 of the CHBDC. The lowest resultant f-factor for the bridge will govern and determine whether a load posting is required. In accordance with the CHBDC, if the governing resultant f-factor is greater than 1.0 then no load posting is required for the bridge. If the f-factor is greater than 0.3 but less than 1.0 a load posting can be developed for the bridge, however, if the f-factor is less than 0.3 the bridge should be considered for immediate repairs, upgrades or replacement. The resultant f-factor for the Dean Lake Bridge was calculated in two scenarios. The results of the bridge load evaluation are provided in Appendix B of this report.

6.2 Loading

6.2.1 Wind

Wind loading was calculated for each bridge element, and on a moving live load as per the CHBDC Section 3. The horizontal and vertical wind loads were calculated for each bridge element and are provided in Appendix B. As per the CHBDC Section 3, a 1/50 design wind load was used because the bridge span is less than 125m. The horizontal drag load was applied on the windward truss and leeward truss simultaneously as per the CHBDC. Once the wind loads were calculated, they were applied to each element within the STAAD model to create the appropriate load cases.

6.2.2 Vertical Loading

A CL-625-ONT truck loading was used for the vertical loading conditions as per the CHBDC Section 3. The truck loading was recreated in STAAD and placed at 100 different locations throughout one span of the bridge to check for the worst-case scenario.

6.2.3 Scenario 1 – Worst Case

Scenario 1 is the worst-case scenario where the truck could be located anywhere on the bridge, including being placed adjacent to the guiderail. This scenario causes a greater load being applied to the exterior truss due to the truck being offset of the roadway centerline. In scenario 1, the lowest f-factor found was 0.40 and governed by the bending moment resistance of the exterior stringers. These exterior stringers (W200x31) were installed in 1988 and therefore a lower strength of steel (fy=250 MPa) was used in developing the overall member resistance as per the CHBDC. This f-factor corresponds to a triple load posting on the bridge of 10, 17, 24 tonnes for single unit vehicles, two-unit vehicles and vehicle trains respectively.

6.2.4 Scenario 2 – Normal Case

Scenario 2 places the truck centered on the lane directly in the middle of the bridge (normal case) causing the interior stringers to take most of the loading and alleviating the stress from the exterior stringers. The interior stringers were replaced in 2008 (as per I & F Engineering Corp drawings dated October 10, 2007) from W200x31 to W250x45 but were field measured to be W250x33. Given the higher strength of steel in the newer stringers (fy=345MPa) and the larger size, the interior stringers can take a larger load than the exterior stringers allowing a higher f-value. In scenario 2, the lowest f-factor found was 0.56 and governed by applied bending moment to the interior stringers. An f-value of 0.56 corresponds to a triple load posting of 14, 25, 34 tonnes.

6.3 Bridge Deck Considerations

The bridge deck is comprised of a laminated timber deck that is wrapped in fiberglass and injected with an epoxy resin. The deck design was tested for bending moment capacity and deflection by I and F Engineer Corp in January of 2008 per correspondence provided by the client.

6.4 Summary of Loads, Resistances and F-Values

The maximum transverse bending moment capacity allowed in the deck was calculated as 19.65 kN m with the assumption that the deck is comprised of 20f-E bending grade, glue laminated douglas fir timber deck and was never designed to take a load greater than a 16 tonne, single unit vehicle. The maximum bending moment calculated based on these assumptions is 24.0 kN-m which is greater than the moment resistance. At the time of construction, the inspecting engineer from M.R. Wright recommended the load limit on the bridge remain at 10/16/20 tonnes instead of the newly designed 16/24/28 tonnes due to the unsatisfactory condition/installation of the new deck. Although the bending moment is greater than the bending resistance, TULLOCH assumes that the deck remains in a satisfactory state for a 10-tonne load limit based on the design of the deck from 2008. A single unit posting will prevent trucks from overloading the structure. The results of the above calculations are provided in Appendix B.

6.5 Upgrading Options

TULLOCH believes the municipality has 3 options for upgrading the bridge:

Option 1 – Replace sections of localized severe steel section loss and corroded rivets. Sand blast and re-coat all structural steel. Replace/fix other deficiencies (bridge deck seals, guiderail, south wearing surface, hazard markers, etc.). Keep 10 tonne single load posting.

Option 2 – Partial bridge rehab, replace stringers (with an additional number of stringers), replace deck, complete coupon testing on existing steel to verify in0situ strengths. Complete all repairs in option 1. Possibly increase load posting pending coupon test results.

Option 3 – Major rehab, partial depth concrete repairs, reinforce truss elements, potentially remove load posting.

7. CONCLUSIONS AND RECOMMENDATIONS

The following is a summary of the bridge conditions and our conclusions:

- The bridge should have a single 10 tonne load limit sign and a "No Trucks" sign installed immediately to prevent heavy trucks from crossing.
- The wearing surface is in fair condition with some minor potholes and cracking.
- The north span bridge deck is absorbing/retaining water and accelerating the deterioration. This will continue and eventually need replacement.
- The deck joints/cracks require sealing to avoid further deterioration
- The guiderail ends should be replaced with MTO approved energy attenuators.
- Rust jacking is present at almost all of the connection locations and some connections have loose or missing rivets. The truss system has several loose/missing rivets however no single connection has more than 10% loose/missing.
- The steel trusses are in good to fair shape overall, however, are considered to be in poor condition due to the bottom chord which is severely corroded at locations (section loss up to 70% on one leg of the angles).
- The bottom chord of the east truss has permanently deflected in the middle of the south span.
- The intermediate concrete piers and the concrete abutments are in fair to very poor condition and show spalling, delaminations, efflorescence, medium to wide map cracking and disintegration. The abutments are in worse condition than the piers.
- The south approach is missing a speed limit sign and there are no hazard markers present at corners of the bridge.

The following is a list of the recommendations that shall be completed at the bridge and the associated timelines:

Recommendation	Timeframe
• Load limit signage should be changed to 10 tonnes and "No Trucks" signs installed to prevent transports from attempting to cross	<1 year
• Decayed guiderail posts should be replaced immediately and monitored during annual maintenance inspections	Now
• Concrete repairs (substantial on abutments) and localized repairs on piers is required.	Now
• Isolated steel repairs to the truss	Now
• Wearing surface shall be sealed with a flush coat of asphalt sealer to prevent water infiltration. Sand shall be broadcast into the sealer for traction on the bridge deck	<1 year
• Deck joints and seals need to be replaced and armouring angles be reset	<1 year
• Upgrade approach guiderail end treatments	<1 year
• Install delineator signs at ends of guiderail and install speed limit sign on south approach	<1 year
• Change out severely corroded rivets with equal size structural bolts	<1 year
• Replace wood deck panels (if sealing and joints are not replaced/repaired the deck will deteriorate faster and lower the replacement timeline)	1-5 years
• Clean and recoat structural steel if planned long term use at reduced load rating	5-10 years

If a repair plan is not scheduled/completed by the end of 2021, the bridge will be closed.

TULLOCH recommends changing the current load posting of 10/16/20 tonnes to a single load of 10 tonnes and to not allow transports to cross the bridge. Although the calculations in Appendix C show a possible load limit of 10/17/24 tonnes, TULLOCH believes it is in the best public interest to set the load posting to a single load as a triple load posting still allows light transport vehicles to cross the bridge and poses and issue for travelling public meeting trucks at the intersection just south of the bridge.

TULLOCH recommends that in order to prolong the life of the bridge that it undergoes a major rehabilitation. The estimated cost for the rehabilitation of this bridge, including a 10% contingency & 15% Environmental Assessments, Engineering Design and Contract Administration throughout the project would be $4,000,000+ HST. This rehab would not increase the load limit recommended above, but instead prolong the life of the bridge. If the municipality wished to increase this load limit it must conduct a major rebuild to the bridge including replacing or reinforcing some of the key truss members.

8. REPLACEMENT OPTIONS & COST ESTIMATES

Table 1 below outlines the rehabilitation cost estimate.

Table 1 – Preliminary Cost Estimate

Item/Description	Estimated Costs
Sand Blasting & Re-coat Structural Steel	$2,500,000
Partial Depth Concrete Repairs	$500,000
Miscellaneous Steel Repairs During Sand Blasting	$75,000
Replace Bridge Deck Seals	$40,000
Replace Guiderail & Upgrade End Treatments	$35,000
Replace North Wearing Surface	$10,000
Localized Steel Repairs (Top Chord) at North Pier at Connections	$2,500
Replace Decayed Wooden Posts	$5,000
Install Hazard Markers & Single Lane Bridge Sign	$1,000
Install New Rock Protection on 3 Embankments	$15,000
Project Subtotal	$3,173,500
Contingency (10%)	$317,400
Environmental Assessment, Engineering Design & Contract Administration (~15%)	$476,000
Total Project Costs	$3,966,900

The estimated replacement cost of this structure is $8,175,000.00+HST.

9. STATEMENT OF QUALIFICATIONS AND LIMITATIONS

The attached Report (the "Report") has been prepared by TULLOCH Engineering ("Consultant") for the benefit of the client ("Client") in accordance with the agreement between Consultant and Client, including the scope of work detailed therein (the "Agreement").

The information, data, recommendations and conclusions contained in the Report (collectively, the "Information"):

- is subject to the scope, schedule, and other constraints and limitations in the Agreement and the qualifications contained in the Report (the "Limitations")
- represents Consultant's professional judgement in light of the Limitations and industry standards for the preparation of similar reports
- may be based on information provided to Consultant which has not been independently verified
- has not been updated since the date of issuance of the Report and its accuracy is limited to the time period and circumstances in which it was collected, processed, made or issued
- must be read as a whole and sections thereof should not be read out of such context
- was prepared for the specific purposes described in the Report and the Agreement
- in the case of subsurface, environmental or geotechnical conditions, may be based on limited testing and on the assumption that such conditions are uniform and not variable either geographically or over time

Consultant shall be entitled to rely upon the accuracy and completeness of information that was provided to it and has no obligation to update such information. Consultant accepts no responsibility for any events or circumstances that may have occurred since the date on which the Report was prepared and, in the case of subsurface, environmental or geotechnical conditions, is not responsible for any variability in such conditions, geographically or over time.

Consultant agrees that the Report represents its professional judgement as described above and that the Information has been prepared for the specific purpose and use described in the Report and the Agreement, but Consultant makes no other representations, or any guarantees or warranties whatsoever, whether express or implied, with respect to the Report, the Information or any part thereof.

The Report is to be treated as confidential and may not be used or relied upon by third parties, except: as agreed in writing by Consultant and Client

- as required by law
- for use by governmental reviewing agencies

Consultant accepts no responsibility, and denies any liability whatsoever, to parties other than Client who may obtain access to the Report or the Information for any injury, loss or damage suffered by such parties arising from their use of, reliance upon, or decisions or actions based on the Report or any of the Information ("improper use of the Report"), except to the extent those parties have obtained the prior written consent of Consultant to use and rely upon the Report and the Information. Any damages arising from improper use of the Report or parts thereof shall be borne by the party making such use.

This Statement of Qualifications and Limitations is attached to and forms part of the Report and any use of the Report is subject to the terms hereof.

10. CLOSURE

We trust you will find the information presented acceptable and meets your requirements at this time. If you have any questions or wish to discuss any of the information presented herein, do not hesitate to contact the undersigned at your convenience.

Sincerely,
TULLOCH Engineering Inc.

Project Manager/Structural Engineer
Matt Kirby, P. Eng.

MK/sm

APPENDIX A

Glossary of Definitions

Abutment - A substructure unit which supports the end of the structure and retains the approach fill.

Auxiliary Components - Any component which does not share in the load carrying capacity of the structure.

Biennial Structure Inspection - An inspection performed in every second calendar year to assess the condition of the structure, in accordance with the methodology described in OSIM.

Bridge - A structure which provides a roadway or walkway for the passage of vehicles, pedestrians or cyclists across an obstruction, gap or facility and is greater than or equal to 3 m in span.

Bridge Condition Index (BCI) - The BCI rating is a planning tool developed by the Ontario Ministry of Transportation that helps schedule maintenance and rehabilitation work. The BCI is not used to rate or indicate the safety of a bridge. The BCI result is organized into ranges from 0 to 100. To calculate the BCI rating, the current dollar value of the bridge is divided by the replacement cost of the bridge. The replacement value is based on the cost to reconstruct a new bridge. Using this formula enables the Owner to make an informed decision about the amount of work a bridge requires and whether or not to pursue replacement over repair in the near future.

Rating	Maintenance Schedule
Good: BCI Range 70 -100	Maintenance is not usually required within the next five years
Fair: BCI Range 60 -70	Maintenance work is usually scheduled within the next five years. This is the ideal time to schedule major bridge repairs to get the most out of bridge spending.
Poor: BCI Less than 60	Maintenance work is usually scheduled within one year.

Bridge Sufficiency Index (BSI) – The BSI rating is a planning tool developed by the Ontario Ministry of Transportation. The BSI is calculated using the BCI rating less ratings for Importance Factors including Traffic, Economic Implications, Bridge Width and Bridge Profile or Alignment. It is a planning tool with a range of 0 to 100 and helps prioritize maintenance and rehabilitation work, and replacement, with bridges of equal BCI but lower BSI having importance over bridges with higher BSI.

Chord - The upper and lower main longitudinal component in trusses or arches extending the full length of the structure.

Coating - The generic term for paint, lacquer, enamel, sealers, galvanizing, metallizing, etc.

Concrete Deck Condition Survey - A detailed inspection of a concrete deck in accordance with The Structure Rehabilitation Manual.

Culvert (Structural) - A Structure that forms an opening through soil and has a span of 3 metres or more

Defect - An identifiable, unwanted condition that was not part of the original intent of design.

- **Scaling** - Scaling is the local flaking, or loss of the surface portion of concrete or mortar as a result of the freeze-thaw deterioration of concrete. Scaling is common in non air-entrained concrete, but can also occur in air-entrained concrete in the fully saturated condition. Scaling is prone to occur in poorly finished or overworked concrete where too many fines and not enough entrained air is found near the surface.

- **Disintegration** - Disintegration is the physical deterioration or breaking down of the concrete into small fragments or particles. The deterioration usually starts in the form of scaling and, if allowed to progress beyond the level of very severe scaling is considered as disintegration. Disintegration may be caused by de-icing chemicals, sulphates, chlorides or by frost action.

- **Erosion** - Erosion is the deterioration of concrete brought about by water-borne sand and gravel particles scrubbing against concrete surfaces. Similar, damage may be caused by flowing ice. Erosion is sometimes combined with the chemical action of air and water-borne pollutants which accelerate the breakdown of the concrete. Erosion is generally an indication that the concrete is not durable enough for the environment in which it has been placed.

- **Corrosion of Reinforcement** - Corrosion is the deterioration of reinforcement by electrolysis. The alkali content in concrete protects the reinforcement from corrosion. However, when chloride ions above a certain concentration are dissolved in water and penetrate through the concrete to the reinforcement this protection breaks down and corrosion starts. In the initial stages, corrosion may appear as a rust-stain on the concrete surface. In the advanced stages, the surface concrete above the reinforcement cracks, delaminates and spalls off exposing heavily rusted reinforcement.

- **Delamination** - Delamination is defined as a discontinuity of the surface concrete which is substantially separated but not completely detached from concrete below or above it. Visibly, it may appear as a solid surface but can be identified as a hollow sound by tapping or chain dragging. Delamination begins with the corrosion of reinforcement and subsequent cracking of the concrete. Delamination or debonding may also occur in concrete that has been patched or overlaid due to the continued deterioration of the older concrete. This may happen even in the absence of any rusting of reinforcing steel.

- **Spalling** - A spall is a fragment, which has been detached from a larger concrete mass. Spalling is a continuation of the delamination process whereby the actions of external loads, pressure exerted by the corrosion of reinforcement or by the formation of ice in the delaminated area results in the breaking off of the delaminated concrete.

- **Cracking -** A crack is a linear fracture in concrete which extends partly or completely through the member. Cracks in concrete occur as a result of tensile stresses introduced in the concrete. Tensile stresses are initially carried by the concrete and reinforcement until the level of the tensile stresses exceeds the tensile capacity of the concrete. After this point the concrete cracks and the tensile force is transferred completely to the steel reinforcement. The crack widths and distribution is controlled by the reinforcement in reinforced and prestressed concrete, whereas in plain concrete there is no such control.

- **Alkali-Aggregate Reaction -** In Ontario, there exists several sources of aggregates that react adversely with the alkalis in cement to produce a highly expansive gel. Currently, these sources of reactive aggregates are generally avoided, but they do exist in many existing structures and still may occur in newer structures. The two general types of reactions in Ontario are alkali-carbonate and alkali-silica reaction. The expansion of the gel and aggregates occurs due to hydroxyl ions in the concrete pore solution, which under moist conditions, leads to cracking and deterioration of the concrete.

- **Surface Defects -** Surface defects are not necessarily serious in themselves; however, they are indicative of a potential weakness in the concrete, and their presence should be noted but not classified as to severity, except for honeycombing and pop-outs.

  - STRATIFICATION is the separation of the concrete components into horizontal layers in over-wetted or over vibrated concrete. Water, laitance, mortar and coarse aggregates occupy successively lower positions. A layered structure in concrete will also result from the placing of successive batches that differ in appearance.

  - SEGREGATION is the differential concentration of the components of mixed concrete resulting in nonuniform proportions in the mass. Segregation is caused by concrete falling from a height, with the coarse aggregates settling to the bottom and the fines on top. Another form of segregation occurs where reinforcing bars prevent the uniform flow of concrete between them.

  - COLD JOINTS are produced if there is a delay between the placement of successive pours of concrete, and if an incomplete bond develops at the joint due to the partial setting of the concrete in the first pour.

  - DEPOSITS are often left behind where water percolates through the concrete and dissolves or leaches chemicals from it and deposits them on the surface.

  - HONEYCOMBING is produced due to the improper or incomplete vibration of the concrete which results in voids being left in the concrete where the mortar failed to completely fill the spaces between the coarse aggregate particles.

  - POP-OUTS are shallow, typically conical depressions, resulting from the breaking away of small portions of the concrete surface, due to the expansion of some aggregates or due to frost action. The shattered aggregate particle may be found at the bottom of the depression, with a part of the aggregate still adhering to the pop-out cone.

- ABRASION is the deterioration of concrete brought about by vehicles or snow-plough blades scraping against concrete surfaces, such as, decks, curbs, barrier walls or piers.

- WEAR is usually the result of dynamic and/or frictional forces generated by vehicular traffic, coupled with the abrasive influx of sand, dirt and debris. It can also result from the friction of ice or water-borne particles against partly or completely submerged members. The surface of the concrete appears polished.

- SLIPPERY CONCRETE SURFACES may result from the polishing of the concrete deck surface by the action of repetitive vehicular traffic.

Detailed Visual Inspection - An element by element visual assessment of material defects, performance deficiencies and maintenance needs of a structure.

Deterioration - A defect that has occurred over a period of time.

Distress - A defect produced by loading.

Elements - The individual parts of a structure defined for inspection purposes. Several bridge components may be grouped together to form one bridge element for inspection purposes

Environment - An element's exposure to salt spray:

- Benign - Not exposed (e.g. River Pier)
- Moderate - Exposed but element somewhat protected (e.g. Asphalt covered and waterproofed deck)
- Severe - Exposed and element not protected (e.g. Exposed concrete deck, Barrier Wall)

Evaluation - The determination of the load carrying capacity of structures in accordance with the requirements of the Canadian Highway Bridge Design Code.

Maintenance - Any action which is aimed at preventing the development of defects or preventing deterioration of a structure or its components.

Primary Components - The main load carrying components of the structure.

Rehabilitation - Any modification, alteration, retrofitting or improvement to a structure subsystem or to the structure which is aimed at correcting existing defects or deficiencies.

Remaining Service Life - Remaining Service Life is an estimate of the useful remaining life of the structure and is based on the year of construction or major rehabilitation and a service life of 50 years for culverts that are not plastic, polymer coated or concrete and a service life of 70 years for other structures.

Repair - Any modification, alteration, retrofitting or improvement to a component of the structure which is aimed at correcting existing defects or deficiencies.

Retaining Wall - Any structure that holds back fill and is not connected to a bridge.

Secondary Components - Any component which helps to distribute loads to primary components, or carries wind loads, or stabilizes primary components.

Sign Support - A metal, concrete or timber structure, including supporting brackets, service walks and mechanical devices where present, which support a luminaire, sign or traffic signal and which span or extend over a highway.

Span - The horizontal distance between adjacent supports of the superstructure of a bridge, or the longest horizontal dimension of the cross-section of a culvert or tunnel taken perpendicular to the walls.

Stringers - Stringers span between floor beams and provide the support for the deck above.

Structure - Bridge, culvert, tunnel, retaining wall or sign support.

Suspected Performance Deficiency - A Suspected Performance Deficiency should be recorded during an inspection, if an element's ability to perform its intended function is in question, and one or more performance defects exist.

APPENDIX B

Calculations

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Project: Dean Lake Bridge				Project # 19-1414	
Description: Dead Load Calculations				Sheet: 1 of 1 Rev 0	
Calc By: K. Lajambe	Date: Jan. 17, 2020	Checked By: M. Kirby	Date: Feb. 24, 2020	Approved By: M. Kirby	Date: Feb. 24, 2020

Dead Loads

*Assume each stringer supports 2m (1/2 driving surface), neglect outside stringers

*Assume 1" asphalt wearing surface

Asphalt Wearing Surface:

23.5 kN/m3 x 0.025m x 2m = 1.175 kN/m

Wooden Deck (Douglas Fir, glue laminated):

5.34 kN/m3 x 0.14m x 2m = 1.495 kN/m

Total Deck and Wearing Surface:

1.175 kN/m + 1.495 kN/m = 2.67 kN/m

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Project: Dean Lake Bridge				Project # 19-1414	
Description: Wind Load Calculations				Sheet: 1 of 2 Rev 0	
Calc By: K. Lajambe	Date: Jan. 17, 2020	Checked By: M. Kirby	Date: Feb. 24, 2020	Approved By: M. Kirby	Date: Feb. 24, 2020

Wind Loads

Hourly Wind Pressures (1/50yr) = q = 0.4 kPa (HBDC)

*CHBDC Clause 3.10.1.2 specifies use a 1/50yr if bridge span is less than 125m

Gust Coefficent = Cg = 2.0 (CHBDC 3.10.1.3)

Wind Exposure Coefficient = Ce = 1.1 (CHBDC 3.10.1.4)

Ch = 2.0 (CHBDC 3.10.2.2)

Cv = 1.0 (CHBDC 3.10.2.3)

Horizontal Drag Load: CHBDC 3.10.2.2

Fh = q Ce Ch Ch = 0.4 x 1.1 x 2 x 2 = 1.76 kPa

*Applied on windward truss and leeward truss simultaneously

Vertical Load: CHBDC 3.10.2.3

Fv = q Ce Cg Cv = 0.4 x 1.1 x 2 x 1 = 0.88 kPa

*Acts either upwards or downwards

Wind Load on Live Load: CHBDC 3.10.2.4

Fhl = q Ce Cg (1.2) = 0.4 x 1.1 x 2 x 1.2 = 1.056 kPa

*Applied 3m above roadway on exposed frontal area

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Project: Dean Lake Bridge				Project # 19-1414	
Description: Wind Load Calculations				Sheet: 2 of 2 Rev 0	
Calc By: K. Lajambe	Date: Jan. 17, 2020	Checked By: M. Kirby	Date: Feb. 24, 2020	Approved By: M. Kirby	Date: Feb. 24, 2020

Wind Loads on Each Element (Unfactored)

Fh = 1.76 kPa
Fv = 0.88 kPa

1) Top Chord: Wh = depth x load = 0.208m x 1.76kPa = 0.366 kN/m
   Wv = 0.356m x 0.88kPa = 0.313 kN/m
2) Diagonals: Wh = 0.076m x 1.76kPa = 0.134 kN/m
   Wv = 0.076m x 0.88kPa = 0.067 kN/m
3) Verticals: Wh = 0.1m x 1.76kPa = 0.176 kN/m
   Wv = neglected due to top chord
4) Bottom Chord: Wh = 0.15m x 1.76kPa = 0.264 kN/m
   Wv = 0.15m x 0.88kPa = 0.088 kN/m
5) Deck+Stringers+Guiderail = Wh = (0.152m+0.025m+0.2m+0.311m) x 1.76kPa = 1.211 kN/m (total)
   Wh = 1.211 / 2 = 0.61kN/m per exterior stringer
   Wv = 4.00m x 0.88kPa = 3.52 kN/m / 2 stringers = 3.52 kN/m (total)
   Wv = 1.76 / 2 = 1.76 kN/m per exterior stringer
6) Live Load on Truck: 1.056 kPa x 3m = 3.18 kN/m acting on the truck
   Equates to 10.6 kN/m acting downwards / 2 girders = 5.3 kN/m
   And 3.18 kN/m acting outwards (x direction) / 4 girders = 0.795 kN/m

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Project: Dean Lake Bridge				Project # 19-1414	
Description: Wooden Deck Deflection Calculations				Sheet: 1 of 1 Rev 0	
Calc By: K. Lajambe	Date: Feb. 7, 2020	Checked By: M. Kirby	Date: Feb. 24, 2020	Approved By: M. Kirby	Date: Feb. 24, 2020

Wooden Deck Deflection Calculations

Max Deflection Allowed (CHBDC 9.4.2): Δall = (1/400) x (Length of span in m)

= (1/400) x (1,522mm)

Δall = 3.805mm

Max Deflection = Δmax = (P x L^3) / (48 x E x I)

= [(78, 750N o/c x $(1{,}522mm)^3$] / (48 x 11,800 N/mm^2 x 197,539,200mm^4)

Δmax = 2.48mm

P = 87.5kN x 0.9 = 78, 750N (*See Note 1 below)

L = 1, 522mm

E = 11, 800 N/mm^2 (See CHBDC Table 9.15)

I = (bd^3) / 3 = (400×114^3) / 3 = 197,539,200 mm^4

Δall = 3.805mm > Δmax = 2.48mm, therefore deflection is OK

Notes:

1) As per CHBDC Clause 9.4.2, only the live load is considered for SLS Combination 1 of Table 3.1, therefore the live load factor = 0.9

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Project: Dean Lake Bridge				Project # 19-1414	
Description: Wooden Deck Bending Moment Calculations				Sheet: 1 of 1 Rev 0	
Calc By: K. Lajambe	Date: Feb. 7, 2020	Checked By: M. Kirby	Date: Feb. 24, 2020	Approved By: M. Kirby	Date: Feb. 24, 2020

Wooden Deck Bending Moment Calculations

Assumptions:

1) Deck was originally designed for a max 16 tonne, 3 axle truck as per M.R. Wright Plans dated 2007 with a triple load posting of 16, 24, 28 tonnes.
2) Composite deck comprised of 20f-E Bending Grade glue-laminated douglas fir timber deck (lowest bending grade provided by the CHBDC) as no specification has been provided to TULLOCH as per the timber grade.
3) Fibreglass strength is neglected in bending moment calculations due to strength/property information of fibreglass is unknown to TULLOCH.

Bending Moment Resistance (CHBDC 9.6.1): $M_r = \emptyset_a\, k_a\, k_{ls}\, k_m\, k_{sb}\, f_{bu}\, S$

$= 0.9 \times 1 \times 1 \times 1 \times 1 \times 12.6 \times 1{,}732{,}800$

Mr = 19.65 kN m

$\emptyset_a = 0.9$ (CHBDC Table 9.1)

$k_a = 1.0$ (CHBDC 9.5.3)

$k_{ls} = 1.0$ (CHBDC 9.6.3)

$k_m = 1.0$ (CHBDC 9.5.6)

$k_{sb} = 1.0$ (CHBDC 9.6.2)

$f_{bu} = 12.6$ N/mm2 (CHBDC Table 9.15)

$S = (bd^2) / 3 = (400\times114^2) / 3 = 1{,}732{,}800\ mm^3$

Note: Tire load supported by 400mm width/plank of deck between stringers.

Max Moment = $M_{max} = \emptyset_L \times I \times \emptyset_M \times P \times L$

$= 1.7 \times 1.4 \times (-0.175) \times 32 \times 1.8$

Mmax = 24.0 kN m

$\emptyset_L$ = Live Load Factor of Safety = 1.7

I = Dynamic Load Allowance = 1.4

$\emptyset_M$ = Moment Coefficient = -0.175 (HSC Moment, Reactions Beam Tables, Section 5)

P = Point Load from Tires = 32kN (Based on 16 tonne truck, 16tonnex0.2x

P = 16 tonne truck x 0.2 (equivalent force per ½ axle) x 9.81 kN/tonne = 32 kN

L = Length of Span = 1.8m

Mr = 19.65 kN m < Mmax = 24.0 kN m, therefore bending moment FAILS

F =

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Project: Dean Lake Bridge				Project # 19-1414	
Description: Sample Load Posting Calculation				Sheet: 1 of 2	Rev 0
Calc By: K. Lajambe	Date: Feb. 21, 2020	Checked By: M. Kirby	Date: Feb. 24, 2020	Approved By: M. Kirby	Date: Feb. 24, 2020

Sample Load Posting Calculation

Below is an example of the load posting calculation (F calc) for the exterior stringers. For all other load posting calculations please see attached spreadsheets.

Exterior Girders (W200x31)

F_y = 210 MPa (CHBDC)

F_u = 420 MPa (CHBDC

Area = Ag = 3970mm^2

Effective Length = 1.306m

ϕs = 0.95 (CHBDC 10.5.7)

Aw = d x w = 200mm x 6.4mm = 1280mm^2

Fs = Fcr + Ft = 0.577Fy + 0 = 0.577(210) + 0 = 121.17 MPa

Axial Resistance = ϕs x Ag x Fy = 0.95 x 3970mm^2 x 250 MPa / 1000 = **943 kN** (CHBDC 10.8.2)

Shear Resistance = ϕs x Aw x Fs = 0.95 x 1280mm^2 x 144.25 MPa /1000 = 175 **kN** (CHBDC 10.10.5.1)

Moment Resistance = 75.4 kN m (HSC Beam Load Tables factored down to 250 MPa)

Applied Loads taken from STAAD software.

F calculation (CHBDC 14.15.2.1):

$$F = [U\,R_r - \sum\alpha_D D - \sum\alpha_A A] / [\alpha_L L (1 + I)]$$

$$= [1 \times 75.4 - 1.09\,(1.0+0) - 0.5\,(8.8+8.6)] / [1.63\,(79.7)\,(1 + 0.25)]$$

F = 0.404

U = Resistance Adjustment Factor = 1 (CHBDC Table 14.15)

Rr = Factored Resistance = 75.4 kN m (HSC Beam Load Tables)

α_D = Dead Load Factor = 1.09 (CHBDC Table 14.7)

D = Unfactored Dead Loads = taken from STAAD

$\sum\alpha_A$ = Alternative Load Factors (Wind) = 0.5 (CHBDC Load Combinations)

A = Unfactored Wind Load = taken from STAAD

α_L = Live Load Factors = 1.63 (CHBDC Table 14.8)

L = Unfactored Live Loads = taken from STAAD

I = Dynamic Load Allowance = 0.25 (CHBDC 3.8.4.5.3)

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Project: Dean Lake Bridge				Project # 19-1414	
Description: Sample Load Posting Calculation				Sheet: 2 of 2	Rev 0
Calc By: K. Lajambe	Date: Feb. 21, 2020	Checked By: M. Kirby	Date: Feb. 24, 2020	Approved By: M. Kirby	Date: Feb. 24, 2020

Sample Load Posting Calculation

F = 0.404

Using Figure 14.8 and Clauses 14.17.3.1 – 14.17.3.4 from the CHBDC:
Evaluation Level 3: P = 0.0153
Evaluation Level 2: P = 0.0277
Evaluation Level 1: P = 0.0387

As per Clause 14.17.3.1: Load Posting = PW where W = 625kN
Evaluation Level 3: 10 tonnes
Evaluation Level 2: 17 tonnes
Evaluation Level 1: 24 tonnes

As per Clause 14.17.3.3:
Evaluation Level 3 refers to a single-unit vehicle
Evaluation Level 2: refers to a two-unit vehicle
Evaluation Level 1: refers to a vehicle train

Therefore, load posting shall be a triple load posting of 10, 17, 24 tonnes.

Dean Lake Bridge Summary

			Unfactored Loads				Factored Load Case #1 (1.25DL + 1.7LL + 0.5W)				Factored Load Case #2 (1.1DL + 0LL + 1.65W)										
	Element	Beam Type	Unfactored Dead Load (kN)	Unfactored Live Load (kN)	Unfactored Wind Load (kN)	Total Unfactored Load (kN)	1.25DL	1.7LL	0.5W	Total Factored Load Case #1	1.1DL	0LL	1.65W	Total Factored Load Case #2	Governing Factored Load (Worst Case) (kN)	Governing Factored Load (Normal Case) (kN)	Max Factored Resistance (kN)	Max Factored Resistance with Section Loss (kN)	F Value (Worst Case)	F Value (Truck in Middle of Bridge) (Normal Case)	Element
TRUSS	**Top Chord**	2 - C8x13.25 + PL 14 x 1/4	131.7	415.5	237.0	784.2	164.6	706.4	118.5	989.5	181.1	0.0	195.5	376.6	989.5	989.5	1404.9	1404.9	1.48	1.48	**Top Chord**
TRUSS	**Verticals (ends)**	2 - 2.5x2x5/16	23.4	183.0	15.2	221.6	29.3	311.1	7.6	348.0	32.2	0.0	12.5	44.7	348.0	348.0	337.2	337.2	0.78	0.78	**Verticals (ends)**
TRUSS	**Verticals (interior)**	4 - 4x3x5/16	33.0	169.7	80.4	283.1	41.3	288.5	40.2	369.9	45.4	0.0	66.3	111.7	369.9	369.9	1020.6	1020.6	2.99	2.99	**Verticals (interior)**
TRUSS	**Bottom Chord**	2 - 150x100x7.9	107.7	356.9	263.8	728.4	134.6	606.7	131.9	873.3	148.1	0.0	217.6	365.7	873.3	873.3	929.7	849.3	0.79	0.79	**Bottom Chord**
TRUSS	**Diagonals (ends)**	2 - 4x3x3/8	71.9	288.1	146.5	506.5	89.9	489.8	73.3	652.9	98.9	0.0	120.9	219.7	652.9	652.9	638.4	638.4	0.79	0.79	**Diagonals (ends)**
TRUSS	**Diagonals (interior)**	2 - 3x2.5x5/16	35.7	210.4	101.8	347.9	44.6	357.7	50.9	453.2	49.1	0.0	84.0	133.1	453.2	453.2	419.0	419.0	0.74	0.74	**Diagonals (interior)**
TRUSS	**Diagonals (middle)**	2 - 2.5x2x0.25	3.4	77.4	26.4	107.2	4.3	131.6	13.2	149.0	4.7	0.0	21.8	26.5	149.0	149.0	243.5	243.5	1.38	1.38	**Diagonals (middle)**
TRUSS	**Top Horizontals (ends)**	2 - 4x3x5/16	3.2	13.7	23.5	40.4	4.0	23.3	11.8	39.0	4.4	0.0	19.4	23.8	39.0	39.0	538.7	538.7	17.98	17.98	**Top Horizontals (ends)**
TRUSS	**Top Horizontals (interior)**	4 - 2.5x2x1/4	10.4	24.9	11.7	47.0	13.0	42.3	5.9	61.2	14.3	0.0	9.7	24.0	61.2	61.2	546.6	546.6	10.00	10.00	**Top Horizontals (interior)**
TRUSS	**Top Diagonals (ends)**	1 - 3x2.5x1/4	8.3	20.5	32.9	61.7	10.4	34.9	16.5	61.7	11.4	0.0	27.1	38.6	61.7	61.7	159.7	159.7	3.52	3.52	**Top Diagonals (ends)**
TRUSS	**Top Diagonals (interior)**	1 - 2.5x2x1/4	7.5	18.0	28.0	53.5	9.4	30.6	14.0	54.0	10.3	0.0	23.1	33.4	54.0	54.0	129.3	129.3	3.20	3.20	**Top Diagonals (interior)**
TRUSS	**Bottom Diagonals**	1 - 89x64x7.9	35.1	73.3	59.0	167.4	43.9	124.6	29.5	198.0	48.3	0.0	48.7	96.9	198.0	198.0	229.4	229.4	1.04	1.04	**Bottom Diagonals**
	Element	Beam Type	Unfactored Dead Load (kN m)	Unfactored Live Load (kN m)	Unfactored Wind Load (kN m)	Total Unfactored Load (kN m)	1.25DL	1.7LL	0.5W	Total Factored Load Case #1	1.1DL	0LL	1.65W	Total Factored Load Case #2	Governing Factored Load (Worst Case) (kN m)	Governing Factored Load (kN m)	Max Factored Resistance (kN m)	Max Factored Resistance with Section Loss (kN m)	F Value (Worst Case)	F Value (Truck in Middle of Bridge)	Element
	Floor Beams	5 - S460x81.5	27.0	196.6	51.9	275.5	33.8	334.2	26.0	393.9	37.1	0.0	42.8	79.9	393.9	393.9	323.2	290.9	0.59	0.59	**Floor Beams**
	Stringers (interior)	2 - W250x33	7.7	113.7	20.1	141.5	9.6	193.3	10.1	213.0	10.6	0.0	16.6	27.2	213.0	213.0	132.0	132.0	0.56	0.56	**Stringers (interior)**
	Stringers (exterior)	2 - W200x31	1.0	79.7	17.4	98.1	1.3	135.5	8.7	145.4	1.4	0.0	14.4	15.7	145.4	29.8	75.4	75.4	0.40	7.85	**Stringers (exterior)**

***CHBDC Specifies fy=210 MPA for all steel from 1905-1932 and fy=250MPa for all steel newer than 1975**
****Applied loads taken from STAAD**
*****Truss elements show axial loads while beams and stringers are applied moments**
******Red highlighted cell denotes lowest f value**

*Red text if greater than factored resistance

Lowest F Value	**0.40**	**0.56**	
Load Posting Based on F Value			
Level 3	**10**	**14**	**tonnes**
Level 2	**17**	**25**	**tonnes**
Level 1	**24**	**34**	**tonnes**

Dean Lake Truss Loading (Worst Case)

			Applied Axial Force (kN)									Applied Shear (kN)						Applied Moment Force (kN m)							
Element	Beam Type	Effective Length (m)	Self Weight	Deck + Wearing Surface	Wind (On Structure)	Beam (worst wind)	Wind (On Truck)	Truck (Live Load)	Load Case, beam (for truck location)	Total Axial Loading (Unfactored) (kN)	Max Combined Factored Force (kN)	Self Weight	Deck + Wearing Surface	Wind (On Structure)	Wind (On Truck)	Truck (Live Load)	Max Combined Factored Force (kN)	Self Weight	Deck + Wearing Surface	Wind (On Structure)	Wind (On Truck)	Truck (Live Load)	Max Combined Factored Force (kN m)	Tension/ Compression	Element
Top Chord	2 - C8x13.25 + PL 14 x 1/4	5.225	64.9	66.8	90.2	42.0	146.8	415.5	160,4	784.2	1404.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	compression	**Top Chord**
Verticals (ends)	2 - 2.5x2x5/16	1.313	8.3	15.1	11.1	54.0	4.1	183.0	125,14	221.6	337.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	tension	**Verticals (ends)**
Verticals (interior)	4 - 4x3x5/16	1.313	18.9	14.1	34.1	66.0	46.3	169.7	167,139	283.1	1020.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	compression	**Verticals (interior)**
Bottom Chord	2 - 150x100x7.9	1.045	53.5	54.2	127.8	8.0	136.0	356.9	158,9	728.4	929.7	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	tension	**Bottom Chord**
Diagonals (ends)	2 - 4x3x3/8	1.181	35.9	36.0	57.2	55.0	89.3	288.1	164,20	506.5	638.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	tension	**Diagonals (ends)**
Diagonals (interior)	2 - 3x2.5x5/16	1.182	17.8	17.9	42.0	56.0	59.8	210.4	154,19	347.9	419.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	tension	**Diagonals (interior)**
Diagonals (middle)	2 - 2.5x2x0.25	1.182	1.6	1.8	21.0	327.0	5.4	77.4	144,17	107.2	243.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	tension	**Diagonals (middle)**
Top Horizontals (ends)	2 - 4x3x5/16	1.541	0.4	2.8	20.2	33.0	3.3	13.7	167, 324	40.4	538.7	0.0	0.0	0.0	0.0	0.0	0.0	4.7	0.0	3.6	0.0	0.0	11.1	tension	**Top Horizontals (ends)**
Top Horizontals (interior)	4 - 2.5x2x1/4	1.541	5.2	5.2	5.0	294.0	6.7	24.9	165,317	47.0	546.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	tension	**Top Horizontals (interior)**
Top Diagonals (ends)	1 - 3x2.5x1/4	3.489	4.1	4.2	28.0	114.0	4.9	20.5	167,113	61.7	159.7	0.0	0.0	0.0	0.0	0.0	0.0	4.4	0.0	3.4	0.0	0.0	10.5	compression	**Top Diagonals (ends)**
Top Diagonals (interior)	1 - 2.5x2x1/4	3.489	3.7	3.8	22.9	109.0	5.1	18.0	159,107	53.5	129.3	0.0	0.0	0.0	0.0	0.0	0.0	0.1	0.0	0.0	0.0	0.0	0.1	compression	**Top Diagonals (middle)**
Bottom Diagonals	1 - 89x64x7.9	3.491	17.2	17.9	29.6	335.0	29.4	73.3	262,336	167.4	229.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	tension	**Bottom Diagonals**
Floor Beams	5 - S460x81.5	1.800	11.4	11.8	19.6	152.0	19.4	48.5	262,275	110.7	131.0	12.8	11.8	22.1	46.3	183.3	376.6	5.4	21.6	15.6	36.3	196.6	393.9	tension	**Floor Beams**
Stringers (interior)	2 - W250x33	1.306	4.9	5.0	21.4	286.0	24.0	20.6	262,288	75.9	85.8	0.9	8.3	5.6	14.9	124.2	232.9	1.0	6.7	10.2	9.9	113.7	213.0	compression	**Girders (interior)**
Stringers (exterior)	2 - W200x31	1.306	17.7	18.4	62.3	227.0	65.3	75.5	262,229	239.2	250.3	1.0	0.3	4.6	15.0	88.5	161.9	1.0	0.0	8.8	8.6	79.7	145.4	tension	**Girders (exterior)**

***CHBDC Specifies fy=210 MPA for all steel from 1905-1932 and fy=250MPa for all steel newer than 1975**

****Applied loads taken from STAAD**

*****Interior stringers replaced in ~2010, therefore fy=345 Mpa and fu = 450MPa**

Truck Location

Axles on Bridge	Load Case
1	107-114 and 207-214
2	115-116 and 215-216
3	117-129 and 217-229
4	130-142 and 230-242
5	143-180 and 243-280

Load Case 107-206 = Truck on edge of bridge
Load Case 207-306 = Truck in middle of bridge

Dean Lake Truss Loading (Normal Case)

			Applied Axial Force (kN)									Applied Shear (kN)						Applied Moment Force (kN m)							
Element	Beam Type	Effective Length (m)	Self Weight	Deck + Wearing Surface	Wind (On Structure)	Beam (worst wind)	Wind (On Truck)	Truck (Live Load)	Load Case, beam (for truck location)	Total Axial Loading (Unfactored) (kN)	Max Combined Factored Force (kN)	Self Weight	Deck + Wearing Surface	Wind (On Structure)	Wind (On Truck)	Truck (Live Load)	Max Combined Factored Force (kN)	Self Weight	Deck + Wearing Surface	Wind (On Structure)	Wind (On Truck)	Truck (Live Load)	Max Combined Factored Force (kN m)	Tension/ Compression	Element
Top Chord	2 - C8x13.25 + PL 14 x 1/4	5.225	64.9	66.8	90.2	42.0	146.8	415.5	160,4	784.2	1404.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	compression	**Top Chord**
Verticals (ends)	2 - 2.5x2x5/16	1.313	8.3	15.1	11.1	54.0	4.1	183.0	125,14	221.6	337.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	tension	**Verticals (ends)**
Verticals (interior)	4 - 4x3x5/16	1.313	18.9	14.1	34.1	66.0	46.3	169.7	167,139	283.1	1020.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	compression	**Verticals (interior)**
Bottom Chord	2 - 150x100x7.9	1.045	53.5	54.2	127.8	8.0	136.0	356.9	158,9	728.4	929.7	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	tension	**Bottom Chord**
Diagonals (ends)	2 - 4x3x3/8	1.181	35.9	36.0	57.2	55.0	89.3	288.1	164,20	506.5	638.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	tension	**Diagonals (ends)**
Diagonals (interior)	2 - 3x2.5x5/16	1.182	17.8	17.9	42.0	56.0	59.8	210.4	154,19	347.9	419.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	tension	**Diagonals (interior)**
Diagonals (middle)	2 - 2.5x2x0.25	1.182	1.6	1.8	21.0	327.0	5.4	77.4	144,17	107.2	243.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	tension	**Diagonals (middle)**
Top Horizontals (ends)	2 - 4x3x5/16	1.541	0.4	2.8	20.2	33.0	3.3	13.7	167, 324	40.4	538.7	0.0	0.0	0.0	0.0	0.0	0.0	4.7	0.0	3.6	0.0	0.0	11.1	tension	**Top Horizontals (ends)**
Top Horizontals (interior)	4 - 2.5x2x1/4	1.541	5.2	5.2	5.0	294.0	6.7	24.9	165,317	47.0	546.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	tension	**Top Horizontals (interior)**
Top Diagonals (ends)	1 - 3x2.5x1/4	3.489	4.1	4.2	28.0	114.0	4.9	20.5	167,113	61.7	159.7	0.0	0.0	0.0	0.0	0.0	0.0	4.4	0.0	3.4	0.0	0.0	10.5	compression	**Top Diagonals (ends)**
Top Diagonals (interior)	1 - 2.5x2x1/4	3.489	3.7	3.8	22.9	109.0	5.1	18.0	159,107	53.5	129.3	0.0	0.0	0.0	0.0	0.0	0.0	0.1	0.0	0.0	0.0	0.0	0.1	compression	**Top Diagonals (middle)**
Bottom Diagonals	1 - 89x64x7.9	3.491	17.2	17.9	29.6	335.0	29.4	73.3	262,336	167.4	229.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	tension	**Bottom Diagonals**
Floor Beams	5 - S460x81.5	1.800	11.4	11.8	19.6	152.0	19.4	48.5	262,275	110.7	131.0	12.8	11.8	22.1	46.3	183.3	376.6	5.4	21.6	15.6	36.3	196.6	0.0	tension	**Floor Beams**
Stringers (interior)	2 - W250x33	1.306	4.9	5.0	21.4	286.0	24.0	20.6	262,288	75.9	85.8	0.9	8.3	5.6	14.9	124.2	232.9	1.0	6.7	10.2	9.9	113.7	0.0	compression	**Girders (interior)**
Stringers (exterior)	2 - W200x31	1.306	17.7	18.4	62.3	227.0	65.3	75.5	262,229	239.2	250.3	1.0	0.3	4.6	15.0	88.5	161.9	1.0	0.0	8.8	8.6	4.1	0.0	tension	**Girders (exterior)**

Factored Resistance Calculations

	Beam Type							Compression			
Element	Quantity	Leg 1	Leg 2	Thickness	Area (mm^2)	Area After Section Loss (mm^2)	Effective Length (m)	r (lowest) (mm)	Delta(s) (10.5.7)	Cr (kN)	Cr (After Section Loss) (kN)
Top Chord	2 - C8x13.25 + PL 14 x 1/4						5.225		0.9	1405	
Verticals (ends)	2	64	51	7.9	1690	1690	1.313	19.7	0.9	319	319
Verticals (interior)	4	102	76	7.9	5400	5400	1.313	32.4	0.9	1021	1021
Bottom Chord	2	152	102	7.9	4660	4257	1.045	42.7	0.9	881	805
Diagonals (ends)	2	102	76	9.5	3200	3200	1.181	32.2	0.9	605	605
Diagonals (interior)	2	76	64	7.9	2100	2100	1.182	23.8	0.9	397	397
Diagonals (middle)	2	64	51	6.4	1370	1370	1.182	19.9	0.9	259	259
Top Horizontals (ends)	2	102	76	7.9	2700	2700	1.541	32.4	0.9	510	510
Top Horizontals (interior)	4	64	51	6.4	2740	2740	1.541	19.9	0.9	518	518
Top Diagonals (ends)	1	76	64	6.4	845	845	3.489	19.1	0.9	160	160
Top Diagonals (middle)	1	64	51	6.4	684	684	3.489	15	0.9	129	129
Bottom Diagonals	1	89	64	7.9	1150	1150	3.491	18.5	0.9	217	217
Floor Beams	5 - S	460	81.5		10400	10133	1.800	28.8	0.9	1966	1915
Stringers (interior)	2 - W	250	33		4190	4190	1.306	33.7	0.9	1301	792
Stringers (exterior)	2 - W	200	31		3970	3970	1.306	32	0.9	750	750

	AXIAL – Tension								AXIAL	
Element	Delta(s) (10.5.7)	Ane (mm2)	A'ne (mm2)	Fy	Delta (s) Ag Fy (kN)	Delta (s) Ag Fy (After Section Loss)(kN)	0.85 Delta(s) Ane Fu (kN)	0.85 Delta(s) A'ne Fu (kN)	Max Axial Resistance (kN)	Max Axial Resistance After Section Loss (kN)
Top Chord	0.95								1405	1405
Verticals (ends)	0.95	3955	3164	210	337	337	1341	1073	337	337
Verticals (interior)	0.95	4332	3465	210	1077	1077	1469	1175	1021	1021
Bottom Chord	0.95	6036	4829	210	930	849	2047	1638	930	849
Diagonals (ends)	0.95	2834	2267	210	638	638	961	769	638	638
Diagonals (interior)	0.95	2475	1980	210	419	419	839	671	419	419
Diagonals (middle)	0.95	898	718	210	273	273	304	244	244	244
Top Horizontals (ends)	0.95	2071	1657	210	539	539	702	562	539	539
Top Horizontals (interior)	0.95	3589	2871	210	547	547	1217	974	547	547
Top Diagonals (ends)	0.95	936	749	210	169	169	317	254	160	160
Top Diagonals (middle)	0.95	336	269	210	136	136	114	91	129	129
Bottom Diagonals	0.95	1142	913	210	229	229	387	310	229	229
Floor Beams	0.95	5710	4854	210	2075	2021	1937	1646	1646	1646
Stringers (interior)	0.95	4190	3562	345	1373	1373	1523	1294	1301	792
Stringers (exterior)	0.95	3970	3375	250	943	943	1346	1144	943	943

	SHEAR						MOMENT			
Element	Delta (s) (10.5.7)	Shear Area (Aw) (mm2)	h/w	502 (Sq rt kv/Fy)	Fs	Max Shear Resistance (kN)	Max Moment Resistance (kN m)	Max Moment Resistance After Section Loss (kN m)	Tension/ Compression	Element
Top Chord	0.95	2276	55.56	80.1	121.17	262	0.0	0.0	compression	Top Chord
Verticals (ends)	0.95	506	1.25	80.1	121.17	58	0.0	0.0	tension	Verticals (ends)
Verticals (interior)	0.95	806	1.34	80.1	121.17	93	0.0	0.0	compression	Verticals (interior)
Bottom Chord	0.95	1201	1.49	80.1	121.17	138	0.0	0.0	tension	Bottom Chord
Diagonals (ends)	0.95	969	1.34	80.1	121.17	112	0.0	0.0	tension	Diagonals (ends)
Diagonals (interior)	0.95	600	1.19	80.1	121.17	69	0.0	0.0	tension	Diagonals (interior)
Diagonals (middle)	0.95	410	1.25	80.1	121.17	47	0.0	0.0	tension	Diagonals (middle)
Top Horizontals (ends)	0.95	806	1.34	80.1	121.17	93	0.0	0.0	tension	Top Horizontals (ends)
Top Horizontals (interior)	0.95	410	1.25	80.1	121.17	47	0.0	0.0	tension	Top Horizontals (interior)
Top Diagonals (ends)	0.95	486	1.19	80.1	121.17	56	0.0	0.0	compression	Top Diagonals (ends)
Top Diagonals (middle)	0.95	410	1.25	80.1	121.17	47	0.0	0.0	compression	Top Diagonals (middle)
Bottom Diagonals	0.95	703	1.39	80.1	121.17	81	0.0	0.0	tension	Bottom Diagonals
Floor Beams	0.95	5382	39.32	80.1	121.17	620	323.2	290.9	tension	Floor Beams
Stringers (interior)	0.95	1525	40.98	62.5	199.065	288	132.0	132.0	compression	Stringers (interior)
Stringers (exterior)	0.95	1280	31.25	80.1	121.17	147	75.4	75.4	tension	Stringers (exterior)

***CHBDC Specifies fy=210 MPA for all steel from 1905-1932 and fy=250MPa for all steel newer than 1975**

*****Assume 3/4" rivet plus 2mm diameter for punch out 21.1 mm**

*****Interior stringers replaced in ~2010, therefore fy=345 Mpa and fu = 450MPa**

Axial Compressive Resistance
(10.9.3.1)
$Cr = Delta(s)\ A\ Fy\ (1+\lambda^{2n})^{-1/n}$

Axial Tensile Resistance
(10.8.2)
Least of:
Delta(s) Ag Fy
0.85 Delta(s) Ane Fu
0.85 Delta(s) A'ne Fu

Factored Shear Resistance
Beams and Girders (10.10.5.1)
Vr=Delta(s) Aw Fs

Moment Resistance
(10.10.3.2)
or from beam selection table in HSC (pg 5-26) for W and S sections

Load Posting Calculations

Element	Tension/ Compression	Beam Type	Effective Length (m)	System Behaviour (14.12.2	Element Behaviour (14.12.3)	Inspection Level (14.12.4)	Target Reliability Index (Beta) (Table 14.5)	Dead Load Category (14.8.2.1)	Dead Load Factor (Table 14.7)	Live Load Factor for Normal Traffic (Table 14.8)	Resistance Adjustment Factor (U) (Table 14.15)	Wind Factor (from load combinations)	Dynamic Load Allowance (3.8.4.5.3)	F (Worst Case) (ULS) (14.15.2.1)	F (Truck in Middle) (ULS) (14.15.2.1)	Element
Top Chord	compression	2 - C8x13.25 + PL 14 x 1/4	5.225	1	3	3	3.00	1	1.07	1.49	1	0.5	0.25	1.48	1.48	**Top Chord**
Verticals (ends)	tension	2 - 2.5x2x5/16	1.313	1	1	3	3.75	1	1.10	1.70	1	0.5	0.25	0.78	0.78	**Verticals (ends)**
Verticals (interior)	compression	4 - 4x3x5/16	1.313	1	3	3	3.00	1	1.07	1.49	1	0.5	0.25	2.99	2.99	**Verticals (interior)**
Bottom Chord	tension	2 - 150x100x7.9	1.045	1	1	3	3.75	1	1.10	1.70	1	0.5	0.25	0.79	0.79	**Bottom Chord**
Diagonals (ends)	tension	2 - 4x3x3/8	1.181	1	1	3	3.75	1	1.10	1.70	1	0.5	0.25	0.79	0.79	**Diagonals (ends)**
Diagonals (interior)	tension	2 - 3x2.5x5/16	1.182	1	1	3	3.75	1	1.10	1.70	1	0.5	0.25	0.74	0.74	**Diagonals (interior)**
Diagonals (middle)	tension	2 - 2.5x2x0.25	1.182	1	1	3	3.75	1	1.10	1.70	1	0.5	0.25	1.38	1.38	**Diagonals (middle)**
Top Horizontals (ends)	tension	2 - 4x3x5/16	1.541	1	1	3	3.75	1	1.10	1.70	1	0.5	0.25	17.98	17.98	**Top Horizontals (ends)**
Top Horizontals (interior)	tension	4 - 2.5x2x1/4	1.541	1	1	3	3.75	1	1.10	1.70	1	0.5	0.25	10.00	10.00	**Top Horizontals (interior)**
Top Diagonals (ends)	compression	1 - 3x2.5x1/4	3.489	1	3	3	3.00	1	1.07	1.49	1	0.5	0.25	3.52	3.52	**Top Diagonals (ends)**
Top Diagonals (middle)	compression	1 - 2.5x2x1/4	3.489	1	3	3	3.00	1	1.07	1.49	1	0.5	0.25	3.20	3.20	**Top Diagonals (middle)**
Bottom Diagonals	tension	1 - 89x64x7.9	3.491	1	1	3	3.75	1	1.10	1.70	1	0.5	0.25	1.04	1.04	**Bottom Diagonals**
Floor Beams	tension	5 - S460x81.5	1.800	2	1	3	3.50	1	1.09	1.63	1	0.5	0.25	0.59	0.59	**Floor Beams**
Stringers (interior)	compression	2 - W250x33	1.306	2	3	3	2.75	1	1.06	1.42	1	0.5	0.25	0.56	0.56	**Stringers (interior)**
Stringers (exterior)	tension	2 - W200x31	1.306	2	1	3	3.50	1	1.09	1.63	1	0.5	0.25	0.40	7.85	**Stringers (exterior)**

***Red highlighted cell denotes lowest f value**

	F (Worst Case)	F (Truck in Middle)	
Lowest F =	**0.40**	**0.56**	
From Figure 14.8, Posting Factor, P =			
Level 1	0.0387	0.0551	
Level 2	0.0277	0.0396	
Level 3	0.0153	0.0219	

Current Posting	**From Clause 14.17.3, Load Postings =**			
10	Level 3	**10**	**14**	**tonnes**
16	Level 2	**17**	**25**	**tonnes**
20	Level 1	**24**	**34**	**tonnes**

Dynamic Load Allowance (3.8.4.5.3)

Case	**Factor**
Deck Joints	0.5
1 axle	0.4
2 or 3 axles	0.3
4+ axles	0.25

Shear Connections Check

**Assuming driving SW across bridge

Element	Joint	# of Rivets (n)	# of Shear Planes (m)	Cross Sectional Area of Rivet Ar (mm2)	Fu (MPa) (14.7.4.6)	Factored Tensile Resistance (Tr) (kN)	Steel Member Thickeness (t) (mm)	Edge Distance (e) (mm)	Φmc t n e Fu	3Φmc t n d Fu	Factored Bearing Resistance (Br) (kN)	Factored Shear Resistance (Vr) (kN)	Factored Shear Resistance (kN)	Which Governs? Bearing or Shear?	Factored Axial Force Acting on Joint (kN)	Element
Top Chord	Ends	38	1	1140	320	9289	7.7	25	1593.4	3585.2	1593.4	6966	1593.4	Bearing	1405	**Top Chord**
Verticals (ends)	Top	12	1	1140	320	2933	7.9	25	516.3	1161.6	516.3	2200	516.3	Bearing	337	**Verticals (ends)**
	Bottom	10	2	1140	320	2444	7.9	25	430.2	968.0	430.2	3667	430.2	Bearing	337	
Verticals (interior)	Top	18	1	1140	320	4400	7.9	38	1161.6	1742.4	1161.6	3300	1161.6	Bearing	1021	**Verticals (interior)**
	Bottom	10	1	1140	320	2444	7.9	51	863.8	968.0	863.8	1833	863.8	Bearing	342	
Diagonals (ends)	Top	24	1	1140	320	5866	9.5	25	1241.6	2793.7	1241.6	4400	1241.6	Bearing	638	**Diagonals (ends)**
	Bottom	24	1	1140	320	5866	9.5	25	1241.6	2793.7	1241.6	4400	1241.6	Bearing	638	
Diagonals (interior)	Top	12	1	1140	320	2933	9.5	25	620.8	1396.8	620.8	2200	620.8	Bearing	419	**Diagonals (interior)**
	Bottom	12	1	1140	320	2933	7.9	25	516.3	1161.6	516.3	2200	516.3	Bearing	419	
Diagonals (middle)	Top	6	1	1140	320	1467	6.4	38	313.7	470.5	313.7	1100	313.7	Bearing	244	**Diagonals (middle)**
	Bottom	6	1	1140	320	1467	6.4	38	313.7	470.5	313.7	1100	313.7	Bearing	244	
Top Horizontals (ends)	Left Side	10	1	1140	320	2444	7.9	38	645.3	968.0	645.3	1833	645.3	Bearing	539	**Top Horizontals (ends)**
	Right Side	10	1	1140	320	2444	7.9	38	645.3	968.0	645.3	1833	645.3	Bearing	539	
Top Horizontals (interior)	Left Side	16	1	1140	320	3911	6.4	25	557.6	1254.7	557.6	2933	557.6	Bearing	547	**Top Horizontals (interior)**
	Right Side	16	1	1140	320	3911	6.4	25	557.6	1254.7	557.6	2933	557.6	Bearing	547	
Top Diagonals (ends)	Left Side	4	1	1140	320	978	6.4	38	209.1	313.7	209.1	733	209.1	Bearing	160	**Top Diagonals (ends)**
	Right Side	4	1	1140	320	978	6.4	38	209.1	313.7	209.1	733	209.1	Bearing	160	
Top Diagonals (interior)	Left Side	4	1	1140	320	978	6.4	25	139.4	313.7	139.4	733	139.4	Bearing	129	**Top Diagonals (interior)**
	Right Side	4	1	1140	320	978	6.4	25	139.4	313.7	139.4	733	139.4	Bearing	129	
Bottom Diagonals	Left Side (bolted)	3	1	1140	320	733	7.9	38	193.6	290.4	193.6	550	193.6	Bearing	229	**Bottom Diagonals**
	Right Side (bolted)	3	1	1140	320	733	7.9	38	193.6	290.4	193.6	550	193.6	Bearing	229	
Floor Beams	Left Side	5	2	1140	320	1222	7.9	25	215.1	484.0	215.1	1833	215.1	Bearing	131	**Floor Beams**
	Right Side	5	2	1140	320	1222	7.9	25	215.1	484.0	215.1	1833	215.1	Bearing	131	
Bottom Chord	Ends (bolted)	28	1	1140	320	6844	7.9	25	1204.6	2710.4	1204.6	5133	1204.6	Bearing	930	**Bottom Chord**
	2nd Vertical	36	2	1140	320	8800	7.9	25	1548.8	3484.7	1548.8	13200	1548.8	Bearing	930	
	3rd (1st middle) vertical	36	2	1140	320	8800	7.9	25	1548.8	3484.7	1548.8	13200	1548.8	Bearing	930	

***Assume 3/4" rivets**
****Applied loads taken from STAAD**
*****Bolted connections were checked for bolted resistance which exceeded rivet calculations**

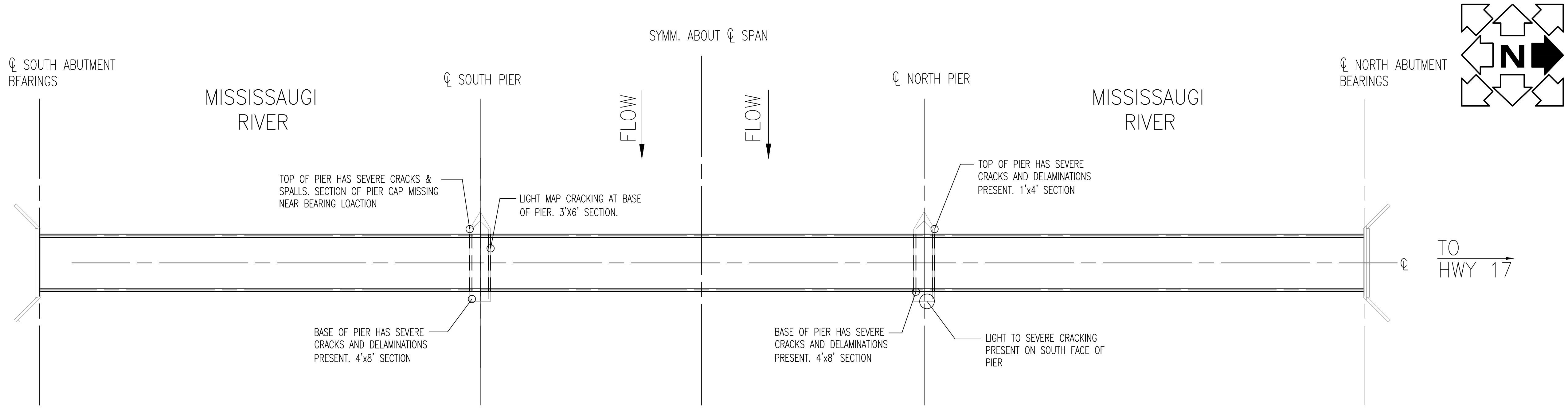
Rivets in Tension
14.14.1.4.1
$T_r = \Phi_r\, n\, A_r\, F_u$
$\Phi_r = 0.67$

Rivets in Shear
14.14.1.4.2
Lesser of
$B_r = \Phi_{mc}\, t\, n\, e\, F_u < 3\Phi_{mc}\, t\, n\, d\, F_u$
$V_r = 0.75\, \Phi_r\, n\, m\, A_r\, F_u$

**Axial Force highlighted if higher than factored shear resistance.

APPENDIX C

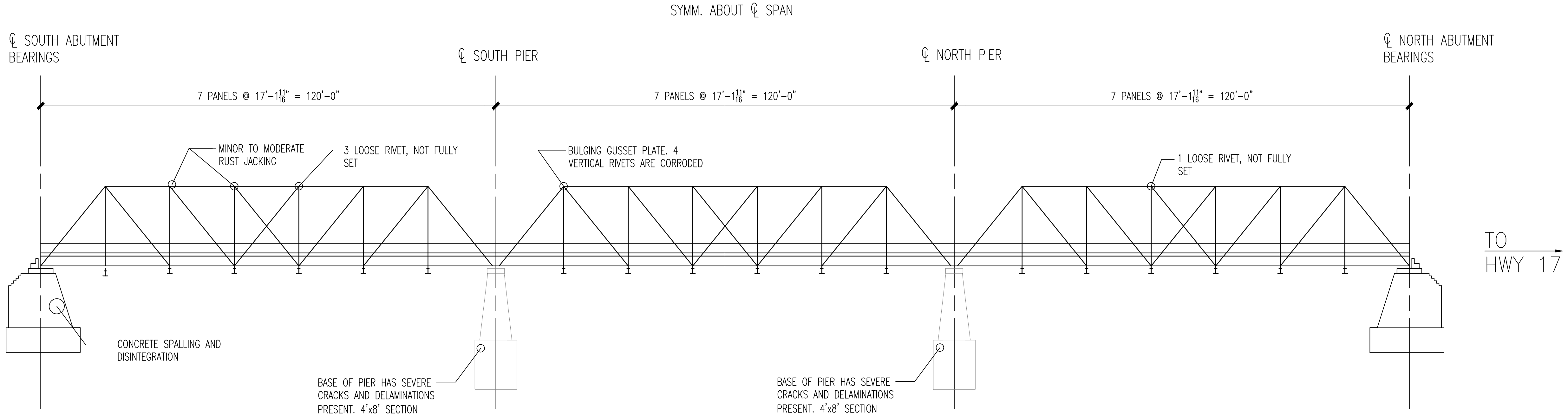
Drawings



PLAN

SCALE 1/32"=1'-0"

NOTE: THE NORTH & SOUTH ABUTMENTS HAVE SEVERE SCALING, SPALLING, CRACKING, POP-OUTS AND EROSION PRESENT ON ALL SURFACES.



EAST ELEVATION/EAST TRUSS

SCALE 1/32"=1'-0"

TULLOCH ENGINEERING
ISSUED FOR REPORT
OCTOBER 8, 2020

No.	DATE	BY	ISSUES / REVISIONS
0	OCT. 8, 2020	SM	ISSUED FOR REPORT

PROJECT No.: 19-1414
DRAWING No.: S1
REVISION No.: 0
DRAWN BY: SM
CHECKED BY: KL
DESIGNED BY: -
APPROVED BY: MK
SCALE: As Noted
DATE: JUNE, 2020

ENGINEER'S SEAL

PROJECT:
DEAN LAKE BRIDGE
CONDITION ASSESSMENT
MUNICIPALITY OF HURON SHORES

DRAWING:
BRIDGE PLAN & EAST ELEVATION

TULLOCH ENGINEERING

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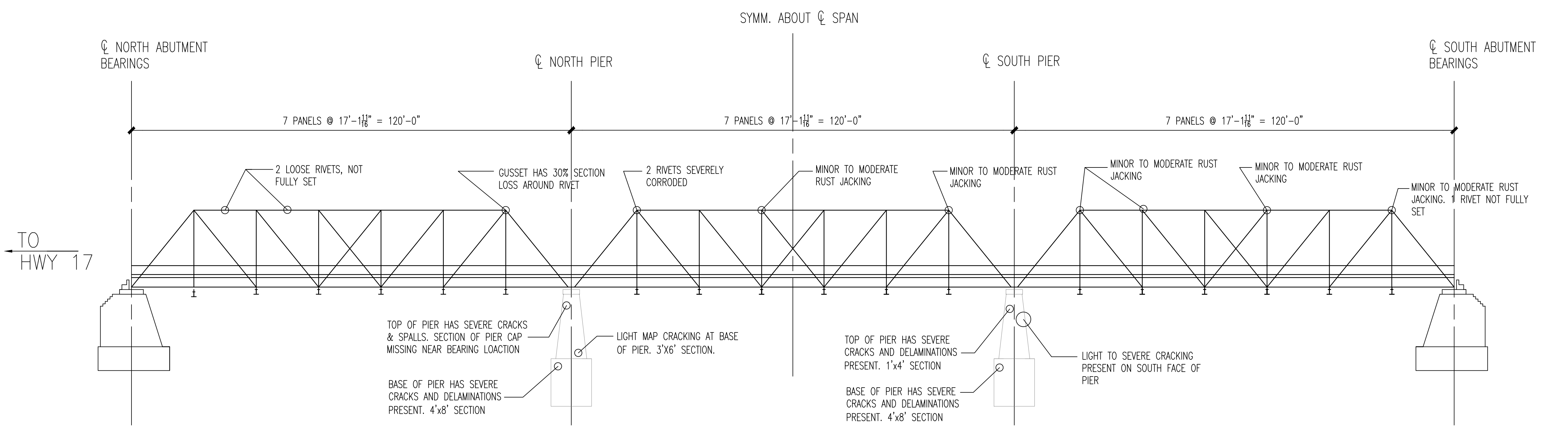
NOTE: THE NORTH & SOUTH ABUTMENTS HAVE SEVERE SCALING, SPALLING, CRACKING, POP-OUTS AND EROSION PRESENT ON ALL SURFACES.



WEST ELEVATION/WEST TRUSS

SCALE 1/32"=1'-0"

N

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OCTOBER 8, 2020

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ENGINEER'S SEAL

DRAWN BY: SM
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PROJECT No.: 19-1414
DESIGNED BY: -
APPROVED BY: MK
DRAWING No. S2
REVISION No. 0
SCALE: As Noted
DATE: JUNE, 2020

PROJECT:
DEAN LAKE BRIDGE
CONDITION ASSESSMENT
MUNICIPALITY OF HURON SHORES

DRAWING:
WEST ELEVATION

TULLOCH ENGINEERING

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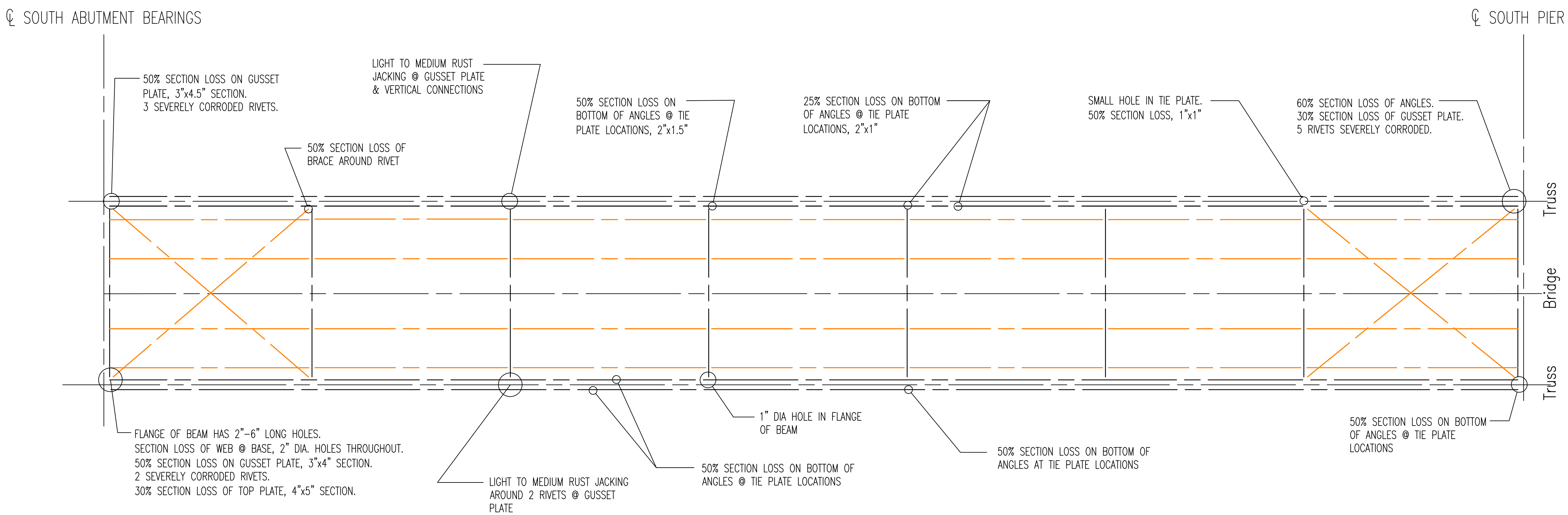
NOTE: MINOR TO MODERATE RUST JACKING PRESENT @ EVERY NODE BETWEEN TOP CHORD BRACING & GUSSETS UNLESS OTHER WISE NOTED.

NOTE: ALL TRANSOM BEAMS HAVE 5%–15% LOCALIZED SECTION LOSS ON TOP FLANGE DIRECTLY ABOVE DECK PANEL JOINTS.

NOTE: 15%–50% SECTION LOSS ON BOTTOM LEG OF ANGLES @ TIE PLATE LOCATIONS, TYP.



PARTIAL BRIDGE PLAN AT DECK ELEVATION – SOUTH SPAN

Scale 3/32"=1'-0"

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PROJECT No. : 19-1414
DRAWING No. S3
REVISION No. 0
CHECKED BY: KL
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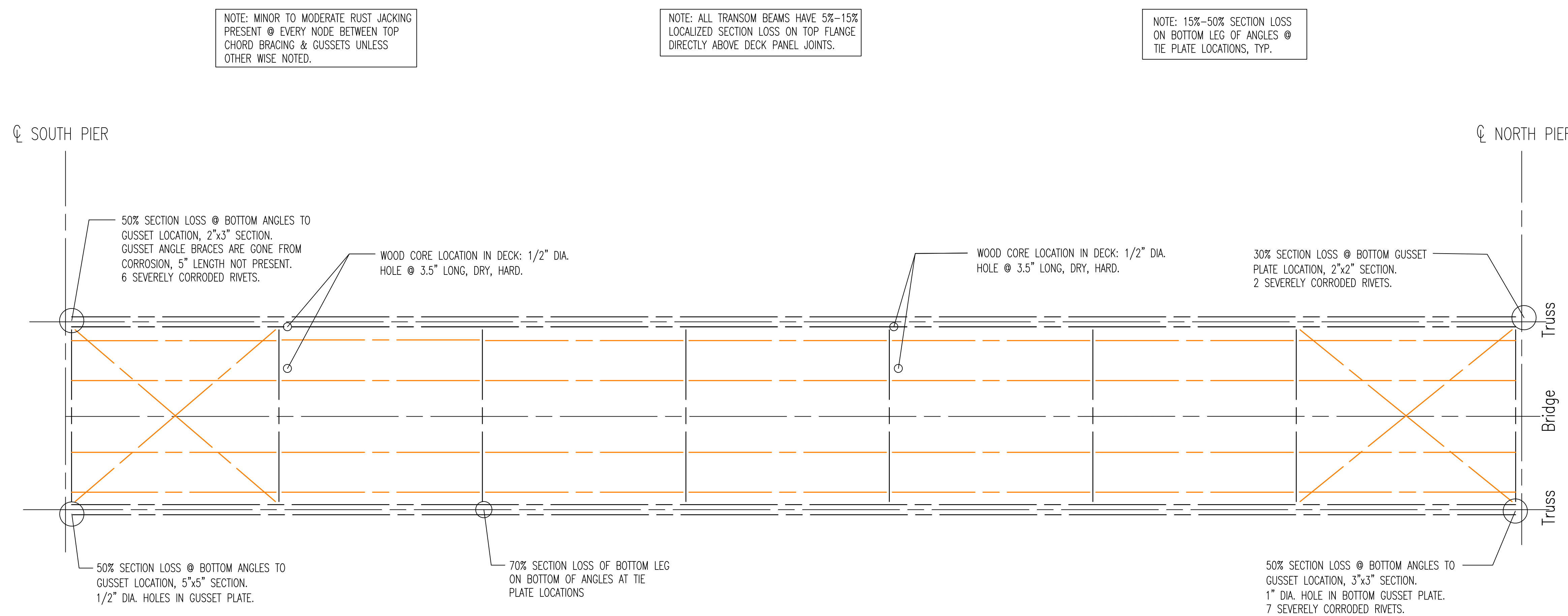
DRAWN BY: SM
DESIGNED BY: -
SCALE: As Noted

ENGINEER'S SEAL

PROJECT:
DEAN LAKE BRIDGE
CONDITION ASSESSMENT
MUNICIPALITY OF HURON SHORES

DRAWING:
PARTIAL BRIDGE PLAN
SOUTH SPAN

TULLOCH ENGINEERING



PARTIAL BRIDGE PLAN AT DECK ELEVATION – CENTER SPAN

Scale 3/32"=1'-0"

ISSUED FOR REPORT
OCTOBER 8, 2020

No.	DATE	BY	ISSUES / REVISIONS
0	OCT. 8, 2020	SM	ISSUED FOR REPORT

PROJECT No.: 19-1414
DRAWING No.: S4
REVISION No.: 0
DRAWN BY: SM
CHECKED BY: KL
DESIGNED BY: -
APPROVED BY: MK
SCALE: As Noted
DATE: JUNE, 2020

ENGINEER'S SEAL

PROJECT:
DEAN LAKE BRIDGE
CONDITION ASSESSMENT
MUNICIPALITY OF HURON SHORES

DRAWING:
PARTIAL BRIDGE PLAN
CENTER SPAN

TULLOCH ENGINEERING

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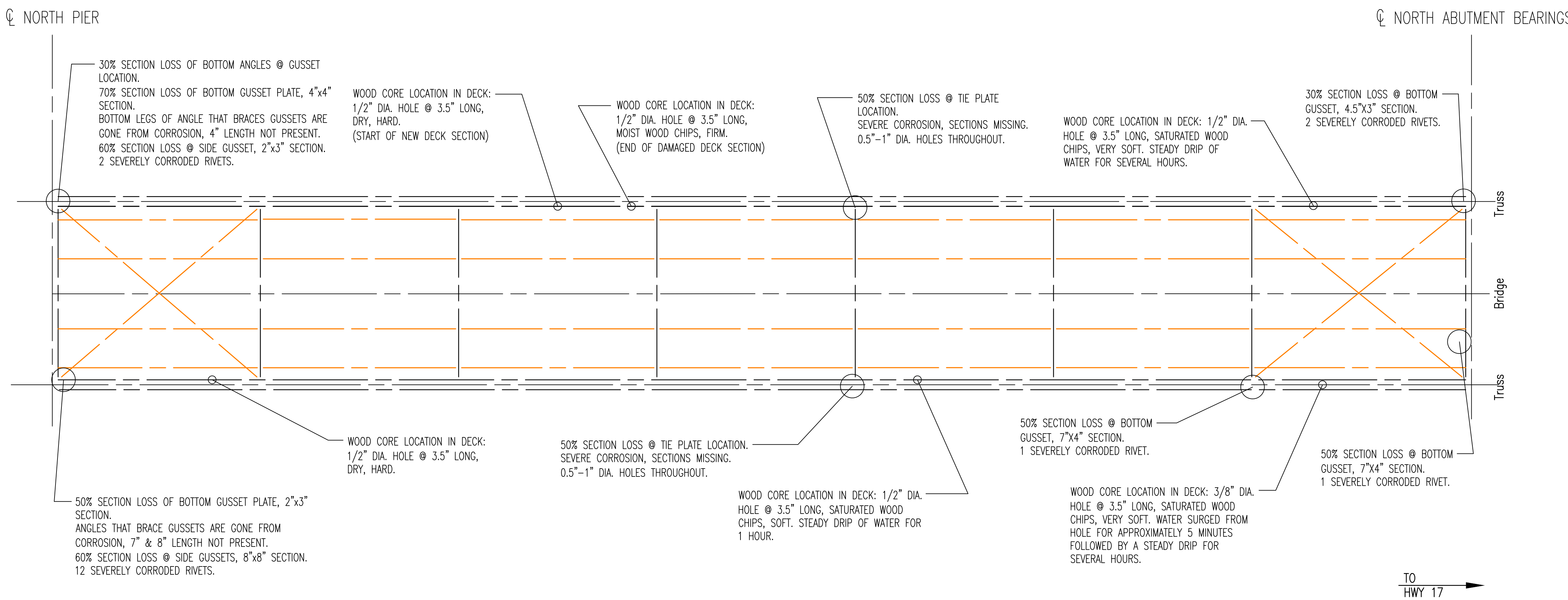
NOTE: MINOR TO MODERATE RUST JACKING PRESENT @ EVERY NODE BETWEEN TOP CHORD BRACING & GUSSETS UNLESS OTHER WISE NOTED.

NOTE: ALL TRANSOM BEAMS HAVE 5%-15% LOCALIZED SECTION LOSS ON TOP FLANGE DIRECTLY ABOVE DECK PANEL JOINTS.

NOTE: 15%-50% SECTION LOSS ON BOTTOM LEG OF ANGLES @ TIE PLATE LOCATIONS, TYP.



PARTIAL BRIDGE PLAN AT DECK ELEVATION – NORTH SPAN

Scale 3/32"=1'-0"

ISSUED FOR REPORT
OCTOBER 8, 2020

No.	DATE	BY	ISSUES / REVISIONS
0	OCT. 8, 2020	SM	ISSUED FOR REPORT

PROJECT No. : 19-1414
DRAWING No. S5
REVISION No. 0
CHECKED BY: KL
APPROVED BY: MK
DATE: JUNE, 2020
DRAWN BY: SM
DESIGNED BY: -
SCALE: As Noted

ENGINEER'S SEAL

PROJECT:
DEAN LAKE BRIDGE
CONDITION ASSESSMENT
MUNICIPALITY OF HURON SHORES

DRAWING:
PARTIAL BRIDGE PLAN
NORTH SPAN

TULLOCH ENGINEERING

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