Townline Trail Bridge (S-0005) Load Evaluation Report

The Township Of Georgian Bluffs

PEARSON ENGINEERING
PEARSONENG.COM

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Project No. 24017.01

Table of Contents

Appendices

TOWNLINE TRAIL BRIDGE (S-0005)

Load Evaluation Report

The Township of Georgian Bluffs

1. Introduction

PEARSON Engineering Ltd. (PEARSON) has been retained by the Township of Georgian Bluffs (Township) to perform a load capacity evaluation of the Townline Trail Bridge (S-0005) located on Keppel-Sarawak Townline (the Project). In 2024, PEARSON completed the Biennial OSIM Inspections of the Township's bridge and culvert inventory (roadway and trail structures), which included the inspection of S-0005. As part of the 2024 OSIM Inspection Report for S-0005, it was recommended that a detailed load evaluation be completed on the bridge as the structure was noted to be in fair to poor condition.

Members of PEARSON staff attended the site on November 11, 2024 to conduct a detailed review of the structures load bearing elements. The onsite condition assessment was completed in accordance with the procedures outlined in the Canadian Highway Bridge Design Code (CHBDC), which included obtaining record measurements of the general layout of the structure, record member sizes, and to review the state of deterioration for each of the critical members.

This report outlines our observations, the results of the detailed load analysis, and recommendations regarding the future usage / repair of the subject bridge structure.

2. Supporting Documents

The following documents have been referenced in the preparation of this report:

- 2024 OSIM Inspection Report, S-0005, prepared by Pearson Engineering Ltd. dated June 12, 2024.
- 2024 OSIM Summary Report, prepared by Pearson Engineering Ltd. and sealed by Mr. Jesse Borges, P. Eng., dated October, 2024.

3. Methodology

The detailed load evaluation of S-0005 was completed in accordance with Section 14 of the CHBDC (CAN/CSA S6-19). Calculations were completed utilizing loading conditions from the following vehicles: CL1-625-ONT (Vehicle Trains), CL2-625-ONT (Two-Unit Vehicles) and CL3-625-ONT (Single Unit Vehicles). An inspection level INSP3 was used to establish the Target Reliability Index, which requires members of our staff to attend site in-person and directly assess the condition of the structural elements of the bridge. The system behaviour categories and element behavior categories varied depending on the structural element. Refer to Section 6.2 for further details regarding the Target Liability Index.

This report describes the evaluation assumptions, criteria, methodology, and summarizes the live load capacity factor (LLCF) results for each structural element. LLCF values which are greater than 1.0 indicate the structural element has sufficient capacity to support the applied loads. If the LLCF is less than 1.0, but greater than 0.3, a load reduction is recommended. If the LLCF is less than 0.3, it is recommended to temporarily close the structure. Refer to Section 7.3 for further details regarding the LLCF.

It should be noted that no original construction or rehabilitation drawings were available for our review during the completion of this load evaluation. Therefore, detailed calculations completed within this load evaluation are based on observations and measurements obtained on site (where possible), as well as conservative assumptions regarding the construction methodology for structural components either concealed or not accessible.

4. Description Of Structure

Keppel-Sarawak Townline Bridge is a single lane, single span structure located between Lot 34, Concession 14 in Keppel, and Lot 28, Concession 1 in Sarawak. The existing load carrying superstructure of the bridge utilizes the steel frame of a flatbed trailer, which spans over an existing concrete T-beam bridge (original structure). The concrete T-beam bridge spans 9.3m and conveys water flow for the Indian creek. The concrete structure has five (5) cast-in-place T-beams which are spaced at ±1m on-center. The beams support a cast-in-place concrete deck, and bear on cast-in-place concrete abutment walls. Though not visible, it is assumed that the original structure is supported by concrete shallow foundations.

It is our understanding that due to the poor condition of the original concrete structure, the Township elected to install a flatbed trailer over the bridge to remove the live loading from the concrete deck. The steel flatbed trailer spans 14.1m with the south end of the trailer bearing on precast concrete blocks, and the north end of the trailer bearing on granular materials. As the elevation of the bridge structure was increase after the installation of the flatbed trailer without any adjustments to the roadway vertical alignment, the approaches leading up to the bridge are considered very steep.

The steel flatbed trailer is supported by two W-shape, steel plate girders which are spaced at 960mm on-center and span the full length of the structure. The girders are 536mm in depth from the north end until 4.4m from the south support, where they taper down to 276mm. The girders support 152mm deep floor beams, which are spaced at 305mm on-center and dropped below the top flange of the steel girders. The floor beams are W-shape and are 3.2m in length which run continuously through the supporting girders (penetrating web of girders). The floor beams support timber deck boards which run longitudinally along the structure.

During the installation of the flatbed trailer, it appears that additional timber planks were installed over the deck to reinforce the riding surface of the bridge. The reinforced timber deck is composed of 2” thick longitudinal decking which span over transverse boards that are spaced at approximately 450mm on center. The longitudinal decking varies in width from 150mm to 250mm. The transverse deck boards located below the longitudinal decking are used to support both the timber deck, and the steel beam barrier system.

The steel beam barrier system is supported on 185mm x 185mm posts that are 980mm tall. The post spacing varies from 1.1m to 1.9m on-center. The posts bear on the transverse deck boards and are fastened with wood gussets on either side of the post, as well as a wood bracket on the backside of the post.

The critical load bearing elements which were reviewed for this load evaluation included the longitudinal steel plate girders, the transverse W-shape floor beams, and the exposed timber deck boards. Figure 3.1 outlines the general cross section of the existing bridge structure.

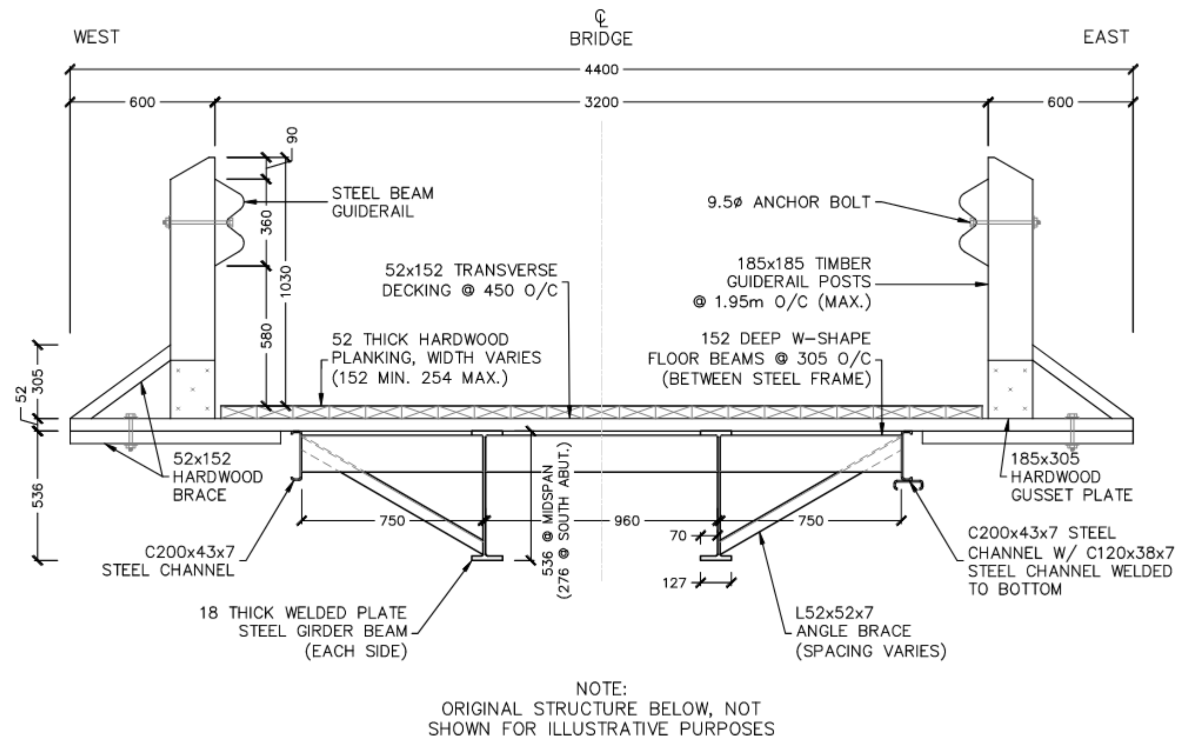


Figure 4.1: Typical Cross Section of Structure

5. Condition of Structure

5.1. Longitudinal Steel Girders

The steel girders appear to be in overall good to fair condition with corrosion noted throughout. The bottom flanges of the girders were noted to have ±5% section loss. The steel girders have web stiffeners installed in seven (7) locations. In general, the web stiffeners are in similar condition to the girders with corrosion noted throughout and ±5% section loss near the bottom of the stiffeners.

Figure 5.1: Eastern Steel Plate Girder

5.2. Transverse Floor Beams

The floor beams appear to be in overall fair condition with moderate section loss noted. Portions of the webs are beginning to flake off with an estimated 25% section loss. The floor beams are 3.2m in length and run continuous across the bridge width (penetrating through longitudinal girders). The W-Shape beams are 152mm in depth with 50mm flange widths and 3.2mm flange thickness. The ends of the floor beams are capped by C-Channels. This load evaluation assumes the floor beams cantilever past both girders and are free on the east and west ends.

Figure 5.2: Typical Floor Beams

5.3. Deck Boards

The wooden deck appears to be in overall fair to poor condition. Significant rot and perforations are noted in ten (10) different locations throughout the deck. In general, the wooden deck was soft and beginning to deteriorate (rot) throughout. A previous inspection was completed on this bridge structure in June 2024 by PEARSON for the Biennial OSIM Inspections. Within the last 5 months since the inspection, the condition of the deck has significantly worsened, indicating that the deck has reached the end of it’s useful service life. It should be noted that minor rehabilitations to the deck had been completed after the review to repair the rotten deck boards.

Figure 5.3: View of Deck Wearing Surface

6. Evaluation Criteria

6.1. General Requirements

The load capacity evaluations for each critical structural member noted above were completed in accordance with Section 14 - Evaluation of the CHBDC. The evaluation has been carried out to assess the vertical loading only, and has assumed that the wind loading on the structure is negligible. The vertical load carrying members have been analysed utilizing ultimate limit states design with serviceability and fatigue limit states not being considered.

6.2. Target Reliability Index

The Target Reliability Index, β, is an index which is utilized to obtain the load factors used during the load analysis. The Target Reliability Index is selected from Table 14.5 of the CHBDC based on three categories: the system behaviour (S1, S2 or S3), the element behaviour (E1, E2 or E3), and the inspection level (INSP1, INSP2 and INSP3). The system behaviour takes into consideration the effect on the global structure if that particular element fails. i.e. the girders are category S1, as the failure of the girders would result in the total collapse of the structure. The element behaviour takes into consideration the rate at which a structural element will fail. The quicker and more sudden of a failure, the higher the element behaviour category. The inspection level takes into consideration the level of inspection which the evaluator completed with INSP1 indicating no inspection and INSP3 indicating inspection directly completed by the evaluator.

The target reliability index, system behaviour, element behaviour, and inspection level specified for the critical components of Bridge Structure S-0005 are outlined in Table 6.1.

TABLE 6.1: TARGET RELIABILITY INDEX (*B*)

Member Type	Behaviour of Interest	System Behaviour Category	Element Behaviour Category	Inspection Level	Target Reliability Index, β
Steel Plate Girders	Shear at the South Support	S1	E3	INSP3	3.0
	Moment at the Midspan	S1	E3	INSP3	3.0
	Moment 1.4m from the South Support	S1	E3	INSP3	3.0
Floor Beams	Shear at Supports	S3	E3	INSP3	2.5
	Moment at Midspan	S3	E3	INSP3	2.5
Deck Boards	Shear at Supports	S3	E1	INSP3	3.25
	Moment at Midspan	S3	E3	INSP3	2.5

The system behaviour for the floor beams is noted to be Category S3 as the floor beams are spaced at 305mm on-center, and therefore a failure of a single floor beam would not result in the total collapse of the structure. The element behaviour for each of the members noted above is noted to be Category E3 (except for the deck boards under shear failure). The Category E3 is to be used for elements which are subject to gradual failure with warning of probable failure.

6.3. Loading Conditions

6.3.1. Dead Loads

The CHBDC classifies dead loads into three categories, D1, D2, and D3. Category D1 is for the dead loads of factory produced components and cast-in-place concrete (excluding decks). Category D2 is for the dead loads of cast-in-place concrete decks, bituminous surfacing (when field measured), and non-structural components. Category D3 is for bituminous surfacing where the nominal thickness is assumed to be 90mm (no field measurements).

The dead loads for Bridge Structure S-0005 include the self-weight of the wooden deck system, the steel girders and floor beams (along with other secondary steel components), and the steel beam barrier system. These loads where distributed to the floor beams and carried onto the girders based on their respective tributary widths. The loads assumed for the elements are as follows:

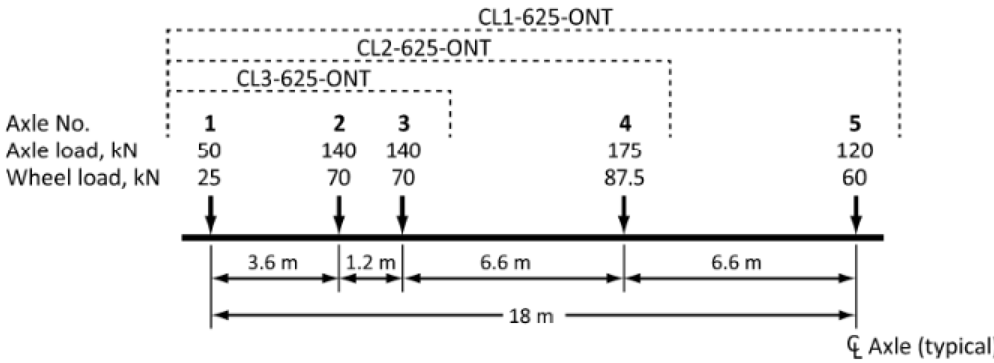
- Steel Beam Barrier System: 0.75 KN/m
- Wood Deck: 5.8 KN/m^3
- 45' Steel Flatbed Trailer: 1.64 KN/m^2

The steel beam barrier system and wood deck system are assumed to be in the D2 dead load category, while the flatbed steel superstructure is assumed to be in the D1 dead load category. The resulting dead load factors (α_d) were taken from Table 14.6 in the CHBDC, which depend on the target reliability index (β) outlined above.

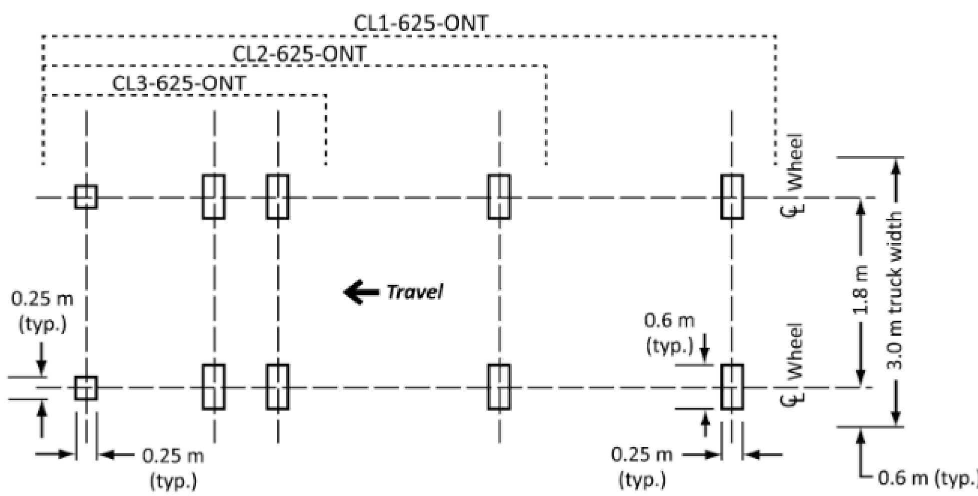
6.3.2. Normal Traffic Loads – Live Loads

The live loading used for the evaluation considered three categories of vehicles. Level one evaluations considered vehicle trains consisting of more than one trailer (CL1-625-ONT). Level two evaluations considered vehicle combinations with only one trailer (CL2-625-ONT). Level three evaluations considered single unit vehicles (CL3-625-ONT). The vehicle axel configuration and weight distribution can be seen in Figure 6.1 below.

Evaluation level	Live load model	Gross load (kN)
Level 1	CL1-625-ONT	625
Level 2	CL2-625-ONT	505
Level 3	CL3-625-ONT	330



CL1-625-ONT, CL2-625-ONT, and CL3-625-ONT truck load (elevation)



CL1-625-ONT, CL2-625-ONT, and CL3-625-ONT truck load (plan)

Figure 6.1: Ontario Truck Loading (Figure A14.2.1 of CHBDC)

The live load factors (α_L) were taken from Table 14.7 of the CHBDC and are equal to a factor of 1.35 when the target reliability index is 2.5, and a factor of 1.49 when the target reliability index is 3.0. In addition to the live load factors, a dynamic load allowance is applied in accordance with Clause 3.8.4.5 of the CHBDC. The dynamic load allowance for the girders is noted to be 0.3 for the moment and shear in all locations. The dynamic load allowance for the deck boards and the floor beams is noted to be 0.4, as it has been assumed that only a single axle of the vehicle load is being applied to each those elements at any point in time.

6.4. Material Properties

As the structure does not have original construction drawings, and the date of construction for the bridge is unknown, it was assumed that the steel superstructure was manufactured between 1976 and 1991 with an unknown grade of steel. Therefore, the minimum yield strength of steel (F_y) from this time period is assumed to be 300MPa as specified in Table 14.1 of the CHBDC.

The wood used for the deck is assumed to be spruce-pine-fir grade No. 1. From Table 14.9 of the CHBDC, the specified strength of bending at the extreme fibre (f_{bb}) is to be taken as 13.0MPa, and the specified strength in longitudinal shear is to be taken as 1.0MPa.

6.5. Resistance Adjustment Factor, U

Factored resistances of the structural components are to be multiplied by the appropriate resistance adjustment factors (U) as specified in Table 14.10 of the CHBDC. As the resistance of the members has been calculated based on the net section areas at locations exhibiting deterioration, the redistribution of load effects between members due to defects and deterioration shall be considered. The resistance adjustment factors for structural steel are as follows:

- Plastic Moment Adjustment Factor: 1.00
- Yield Moment Adjustment Factor: 1.06
- Shear Adjustment Factor (stocky web): 1.02

7. Structural load Evaluation Results

7.1. General

Our analysis has been completed in general conformance with the CHBDC, CSA S16 – Design of Steel Structures, and CSA O86 – Engineering Design in Wood. Our analysis has assumed the bridge is being utilized for a single lane of traffic, which is appropriate given the roadway width over the bridge of ±3.0m. To determine the worst-case loading scenario for each structural element considered in the evaluation, axel and wheel loads were evaluated as moving loads. Theoretical factored loads, which include the dynamic allowance and resistance adjustment factors, have been obtained at the critical locations for load bearing elements.

It should be noted that this structural load evaluation has not considered the original concrete T-beam structure. As the flatbed trailer clear spans over the original concrete structure, no vertical loads are being transferred to the deck top. In addition, any lateral pressure being applied to the abutment walls and wingwalls by the end bearing pressure of the steel flatbed structure is assumed to be adequately resisted by the foundations of the original structure and the lateral resistance provided by the concrete deck.

7.2. Member Section Properties and Resistances

The member resistances have been calculated based on field measurements completed by Pearson staff. The resistances account for the current condition of the structure including section loss due to deterioration. It should be noted that the load evaluation does not account for additional section loss caused by further deterioration in the future.

The member section properties and resistances are outlined below, with the location of the member (x) taken from the south support of the bridge.

TABLE 7.1: MEMBER RESISTANCES

Member	Depth (mm)	Flange Width (mm)	Flange Thickness (mm)	Web Thickness (mm)	Plastic Modulus (x10^3 mm^3)	Moment Resistance (KN•m)	Shear Resistance (KN)
Girders (x= 0m to 1.4m)	276	127	18	11	748.0	213.2	434
Girders (x= 3.4m to North Support)	536	127	18	11	1872.0	533.5	N/A
Floor Beams	152	50	3.2	3	39.7	11.3	74.9
Deck Boards	50.8	Width = 250mm	N/A	N/A	Section Mod. = 107.5	1.26	19.1

7.3. Evaluation Results

Evaluations of the load bearing members has been completed in accordance with Clause 14.15 of the CHBDC. The evaluations have been completed to output a Live Load Capacity Factor (LLCF) for the bending moment and shear stress for each critical load case. The formula used to calculate the LLCF is as follows:

$$LLCF = \frac{UR_r - \sum \propto_D D - \sum \propto_A A}{\propto_L L(1 + I_D)}$$

Where: LLCF: Live Load Capacity Factor

U: Resistance Adjustment Factor

R_r: Factored Resistance of Structural Component

$\propto_D$: Dead Load Factor

D: Nominal Dead Load Effect

$\propto_A$: Load Factors due to Additional Loads (including wind, creep, shrinkage, etc.)

A: Additional Load Force Effects

$\propto_L$: Live Load Factor

L: Nominal Live Load Effect

I_D: Dynamic Load Allowance

Full traffic loading conditions were established for each structural component utilizing the standard CL-625-ONT truck loading specified for the Province of Ontario.

During a structural load evaluation, the following rules are utilized to determine an appropriate load posting for the bridge structure:

1. If the LLCF is found to be greater than or equal to 1.0 utilizing a CL1-625-ONT truck load, a load restriction is not considered to be necessary for the bridge.
2. If the LLCF is less than 1.0 but greater than 0.3 utilizing a CL1-625-ONT truck load, then a triple load posting is recommended.
3. If the LLCF is less than 0.3 utilizing a CL1-625-ONT truck load, then a single load posting is recommended utilizing a CL3-625-ONT truck load.
4. If the LLCF is less than 0.3 utilizing a CL3-625-ONT truck load, consideration should be given for closing the bridge structure.

The load postings when the LLCF is between 1.0 and 0.3 is calculated by multiplying the load posting factor (P) by the gross vehicle weight (W) for which the evaluation considers. The load posting factor is taken from Figure 6.1 which is shown below.

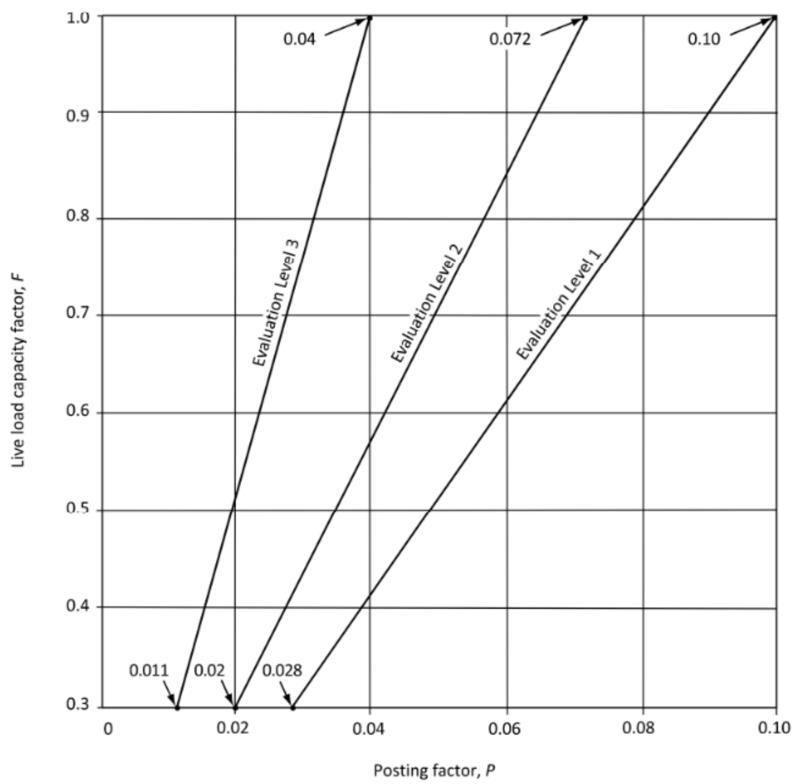


Figure 7.1: Posting Loads for Gross Vehicle Weight (Figure 14.6 of CHBDC)

Table 7.2 below outlines the results of the load evaluation for each critical member.

TABLE 7.2: LOAD EVALUATION SUMMARY

Structural Element	Applied Force	Location on Structure	LLCF (Level 1)	LLCF (Level 2)	LLCF (Level 3)	Recommendation
Girders	Moment	Midspan	0.438	N/A	N/A	Triple Load Posting
	Moment	X = 1.4m	0.230	0.230	0.416	Single Load Posting
	Shear	South Support	1.34	N/A	N/A	N/A
Floor Beams	Moment	Cantilever	0.15	0.18	0.18	Close Bridge
	Moment	Midspan	0.412	N/A	N/A	Triple Load Posting
	Shear	Above Girder	0.670	N/A	N/A	Triple Load Posting
Deck Boards	Moment	Midspan	0.19	0.24	0.24	Close Bridge or Rehab Deck
	Shear	Above Support	0.39	N/A	N/A	Triple Load Posting

The load evaluation resulted in two (2) critical load cases indicating the requirements of a temporary/permanent bridge closure. Firstly, the steel transverse floor beams failed in moment when the CL3-625-ONT truck load had its wheel load positioned along the cantilevered edge of the beam (adjacent to barrier system). The LLCF for the steel beams when considering moment resistance was determined to be 0.18, which is 40% below the minimum LLCF (0.3) required to establish a single load posting. Secondly, the top wood deck boards failed in moment when the CL3-625-ONT truck load had its wheel load positioned between the transverse boards below. The LLCF for the deck boards when considering moment resistance was determined to be 0.24, which is 20% below the minimum LLCF (0.3) required to establish a single load posting.

It should be noted that our review did not include an in-depth analysis of the steel beam barrier system. The current configuration for the barrier system is not considered code compliant, and the construction methodology does not appear to have the structural capacity to support the loading requirements (vehicle or pedestrian) outlined in the CHBDC. During the site inspections the barrier system could be laterally displaced with human force, and therefore was assumed to have minimal live loading capacity. Given the fact that the barrier system does not appear to meet the CHBDC requirements, if the bridge is rehabilitated and reopened to vehicular traffic, the barrier system will need to be replaced.

7.4. Structural Recommendations

The steel superstructure appears to be in overall fair condition with minor section loss and corrosion noted. Based on our load evaluation, the transverse floor beams and exposed deck boards are not structurally adequate to support vehicle loading. The wood deck is also exhibiting significant deterioration causing an uneven and dangerous riding surface. In addition, the construction methodology of the steel beam barrier system does not appear to be structurally adequate to support vehicle or pedestrian loading.

Beyond the bridge superstructure, the roadway approaches appear to be very steep and not constructed in conformance with the geometric standards for Ontario roadways. It is anticipated that the vertical alignment of the bridge approaches is impacting the line-of-sight for approaching vehicles, which could increase the chances of an accident in the future. This safety concern is amplified by the bridges narrow roadway width (±3.0m) and the lack of guiderail systems at each corner of the bridge. There is also no signage on the roadway indicating a narrow roadway, narrow bridge or one lane traffic.

Therefore, based on the current condition of the bridge structure, the non-code-compliant geometry of the roadway approaches, and the identified load carrying capacity issues, it is our recommendation to temporarily close the structure to vehicular and pedestrian traffic, including all maintenance and recreational vehicles. As the Township has completed minor repairs to the bridge deck to address the uneven riding surface, it is our opinion that the bridge structure can remain open until the end of the snowmobile season (March 31, 2025). This recommendation assumes that only small recreational vehicles will be permitted to cross the bridge prior to the structures permanent closure. No roadway vehicles and maintenance vehicles (snow groomer) should be permitted to cross the bridge.

It should be noted that we have reviewed the option of establishing a very low single load posting beyond the limit specified by the CHBDC. Based on our review of the CHBDC, the lowest load posting established utilizing an Evaluation Level 3 loading condition is 7 Tonnes. However, the MTO Structural Manual Rev.60 (January 2024) states that for low volume roads (AADT < 400) a lower load limit may be posted than outlined in the CHBDC. Based on our analysis, the floor beams have the capacity to support a 4 Tonne vehicular load. Considering that a majority of vehicles utilizing the bridge would weigh more than this load limit, and a deck and barrier rehabilitation would be required prior to reopening the existing bridge structure, we recommend that the Township consider either permanently closing the bridge or performing a major rehabilitation to increase the load carrying capacity of the structure (i.e. bridge superstructure replacement).

As the bridge structure will continue to deteriorate over time, we recommend that the Township begin budgeting for the permanent removal or rehabilitation of the bridge in the next 1-5 years. Further details regarding the available design alternatives with estimated construction costs have been provided to the Township under separate cover.

7.5. Signage Recommendations

We recommend that the Township install temporary signage until the closure of the structure indicating that the bridge is for trail use only. At the end of the snowmobile season (March 31, 2025), we recommend installing closure signage and barricade systems to restrict the usage of the bridge by the public. A notice should be issued to the public, local residents and all emergency services outlining the permanent closure of the bridge structure.

Based on our reviews, we recommend installing the following signage as a minimum while the bridge remains open:

- "Multi-Use Trail Only" sign to be installed at the trail entrance on both Church Sideroad West and Lindenwood Road
- "No Unauthorized Motorized Vehicles" sign to be installed at the trail entrance on both Church Sideroad West and Lindenwood Road
- "Maximum 4 Tonnes" sign to be installed near bridge approach on both sides.
- "Narrow Bridge Ahead" sign to be installed on both sides of the bridge.
- "Checkerboard Warning" sign to be installed (with barricades) at the intersection of Church Sideroad West and Sarawak-Keppel Townline, as well as the north approach of bridge, after closure of structure.
- "Bridge Closed" sign to be installed at the trail entrance on both Church Sideroad West and Lindenwood Road after closure of structure.


8. Conclusion

Given the current condition of the bridge and the results of the detailed load evaluation, we recommend the Township take the following steps:

- As the Town has completed minor rehabilitation to the bridge deck, the structure may stay open until March 31, 2025, or the end of the snowmobile season, whichever is sooner.
- At the end of the snowmobile season, the bridge shall be temporarily closed to all vehicular and pedestrian traffic. Roadway closure signage and barricades should be installed at each end of the bridge.
- The Township should begin budgeting for the permanent removal or rehabilitation of the structure in the next 1 – 5 years.

It is our understanding that after the temporary closure of the bridge structure, the Township will review the option to either rehabilitate the structure or prepare for the permanent removal of the bridge. As outlined above, additional information regarding available design alternatives with estimated construction costs has been provided to the Township under separate cover. Upon request, Pearson Engineering Ltd. is also prepared to assist the Township with the engineering services required to complete the preferred design solution.

9. Limitations

Our scope of work consisted of a visual, non-destructive review of the bridge superstructure. No physical / destructive testing was completed. Calculations were completed in accordance with Section 14 of the CHBDC utilizing Ultimate Limit States Design only. The original concrete structure below the steel superstructure was not considered in this review.

The information in this report is intended for the use of the Township of Georgian Bluffs for Structure S-0005 exclusively. The issuance of the results or information provided within this report to any potential contractors or future consultants is the responsibility of the parties noted above.

Pearson Engineering Ltd. accepts no liability for use of this information by third parties. Any decisions made by third parties based on information provided in this report are made at the sole risk of third parties. Pearson Engineering Ltd. accepts no responsibilities for damages incurred by any third parties as a result of any decisions or actions made as a result of this report.

Only the specific information identified has been reviewed. The consultant is not obligated to identify mistakes or insufficiencies in the information obtained from the various sources or to verify the accuracy of the information. The consultant may use such specific information obtained in performing its services and it's entitled to rely upon the accuracy and completeness thereof.

The evaluation does not wholly eliminate uncertainty regarding the potential for future costs, hazards or losses in connection with the structure. No site reviews, physical or destructive testing and no design calculations have been performed unless specifically recorded. Conditions existing but not recorded were not apparent given the level of study undertaken. We can perform further investigation on items of concern if so required.

I trust this report meets your needs at this time. Thank you for choosing Pearson Engineering Ltd. for your engineering needs and should you require further assistance or clarifications with this project, please do not hesitate to contact our office.

All of which is respectfully submitted,

Pearson Engineering Ltd.

Jesse Borges, P. Eng.
Structural Project Manager

LICENSED PROFESSIONAL ENGINEER
Feb 14/25
J. BORGES
100503702
PROVINCE OF ONTARIO

Appendix A

Photo Report

Photo 1 - View of Structure Facing East.

Photo 2 - View of Structure Facing West.

Photo 3 - View of Structure Facing South.

Photo 4 - View of Structure Facing North.

Date of Photos: November 7, 2024
Inspector: David DeBoer, E.I.T.

Photo 5 - View of Deteriorated Deck Boards.

Photo 6 - View of Barrier Post Connection Detail.

Photo 7 - View of Steel Beam Barrier System.

Photo 8 - View of Underside of Deck Cantilevered Edge and Steel Frame.

Date of Photos: November 7, 2024

Inspector: David DeBoer, E.I.T.

Photo 9 - View of Cantilevered Floor Beams.

Photo 10 - View of South Concrete Abutment.

Photo 11 - View of Steel Girder and Diagonal Strutting.

Photo 12 - View of Interior Floor Beams.

Date of Photos: November 7, 2024
Inspector: David DeBoer, E.I.T.

Photo 13 - View of Southwest Wingwall of Original Structure.

Photo 14 - View of Southeast Wingwall of Original Structure.

Photo 15 - View of Interior Soffit of Original Structure.

Photo 16 - View of Fascia and Curb of Original Structure.

Date of Photos: November 7, 2024

Inspector: David DeBoer, E.I.T.

Photo 5 - View of Concrete Structure Girder Deterioration.

Photo 6 - View of Concrete Structure Interior Soffit.

Photo 7 - View of Waterway Facing East.

Photo 8 - View of Waterway Facing West.

Date of Photos: November 7, 2024
Inspector: David DeBoer, E.I.T.