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Rehabilitation of the Broddring Empire Road 95 Bridge over the Toark River

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September 2022

Letter of Submission removed.

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Summary

The Broddring Empire Road 95 Bridge over the Toark River is a concrete slab on steel girder bridge located near Teirm, Alagaësia. The bridge was identified as in need of major rehabilitation to address issues with its concrete deck, including significant delamination, spalls, and corroding reinforcing steel. Two rehabilitation options were proposed to address these issues: a deck patch and sidewalk overlay, and a deck replacement. As well, the feasibility of widening the deck to accommodate bicycle lanes was explored.

The capacity of the structure was evaluated using CSA S6-19, the Canadian Highway Bridge Design Code, considering ultimate limit state during construction and service, and the serviceability limit state of permanent deflection. The structural evaluation determined that the existing steel girders are capable of supporting a new, thicker concrete deck in accordance with current standards. However, the deck cannot be widened to safely accommodate bicycle lanes without surpassing the serviceability limit state or increasing the capacity of the superstructure.

A deck replacement was recommended over the patch and overlay option due to concerns about the continued deterioration of the deck after work is complete, and the fixed costs of construction that do not scale with project size.

A two-stage construction and traffic staging plan was proposed that maintained a single direction of traffic over the bridge and routed the other direction through a nearby detour street. A potential lead contamination hazard was identified, as was a conflict with the Surdan National right-of-way underneath the structure.

Acknowledgements

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Key Plan

Broddring Empire Road 95 Bridge over the Toark River (Bridge No. 55795)

Teirm, Alagaësia

(Map data © OpenStreetMap contributors) [1]

1.0 Introduction

Roran Stronghammer & Associates Inc. (RSH) was retained by the Broddring Empire to undertake the preliminary and detailed design for the rehabilitation of 2 bridges under RFP-ANC-7985-782 No. 7985-890. The bridges in this assignment include the Broddring Empire Road 95 Bridge over the Toark River (Bridge No. 55795), hereafter referred to as the Toark River Bridge, and the Woadark Lake Bridge (Bridge No. 952471). This report provides a pre-design evaluation and assessment of the possible rehabilitation options for the Toark River Bridge on Broddring Empire Road 95.

1.1. Background

The Toark River Bridge was constructed in 1976, and is a four-span 143.26 m reinforced concrete deck on welded steel girder bridge [2]. As shown in the Key Plan on Page 1, the bridge is located on Broddring Empire Road 95. The Toark River Bridge is located approximately 0.4 km east north of Broddring Empire Road 53 in Teirm, Alagaësia [1]. The bridge carries one (1) lane of traffic in each direction.

1.2. Problem Statement

The most recent Ontario Structure Inspection Manual (OSIM) report completed in 2019 identified that the structure was in need of major rehabilitation within the next four years, and recommended that a Detailed Deck Condition Survey (DDCS) be conducted to determine the scale of the required rehabilitation [3]. The Broddring Empire retained RSH to assess the bridge's condition in greater depth and develop a rehabilitation strategy that will address the observed issues [4].

1.3. Scope

This report assesses two possible rehabilitation options for the Toark River Bridge that would restore its original intended functionality. As well, this report will explore the feasibility of a hypothetical improvement to the structure that would allow it to accommodate bicycle lanes on both sides of the road. Finally, the report will discuss other important considerations, such as the impact of construction on traffic and potential constraints set by local stakeholders.

2.0 Existing Structure

2.1. Structure Location

The structure is located in Teirm, Alagaësia, and carries two lanes of traffic over the Toark River, a pedestrian path, and a Surdan National right-of-way [4]. For the purposes of this report, the structure and Broddring Empire Road 95 are considered to have a north-south alignment.

The AADT of Broddring Empire Road 95 is 15 329 vehicles per day [5]. The posted speed limit is 50 km/h.

2.2. Description of Structure

The bridge superstructure consists of five welded steel girders with 1.42 m deep webs and varying flange thicknesses [2]. The girders are spaced at 2.74 m centre-to-centre and supported on neoprene bearing pads. From north to south, the girder spans are 27.43 m, 44.20 m, 44.20 m, and 27.43, for a total length of 143.26 m. The out-to-out deck width is 12.80 m, and the width of the curb-to-curb traveled surface is 9.14 m. Figure 1 shows the existing bridge deck cross-section, measured in Imperial units.

Figure removed.

Figure 1. Bridge deck cross-section [2]

Six field splices make each girder continuous over the entire span of the structure. [2]. Steel cross bracing is present at irregular, approximately 5 m intervals. The reinforced concrete deck is shown on the original drawings to be 191 mm thick. The DDCS determined that the actual deck thickness was between 185 mm and 205 mm thick, with an existing asphalt thickness between 30 mm and 70 mm [6].

The abutments are reinforced concrete supported by concrete-filled pipe pile foundations [2]. The piers are supported by interlocking steel sheet piles. Wingwalls are located on all four corners of the bridge. The existing barrier is a steel balustrade that does not meet current safety standards.

2.3. Rehabilitation History

The Broddring Empire retained Dûrgrimst Ingeitum Consulting Engineers Ltd. to design a rehabilitation of the structure in 1998 [7]. The following work was completed:

- Replaced webs and flanges of select existing girders;
- Abrasive blast cleaned and coated structural steel at repair locations;
- Patch repaired concrete deck and various locations;
- Replaced existing expansion joint assemblies; and
- Repaired and replaced existing deck drains.

Given the age of the bridge, it is unlikely that it has only undergone a single rehabilitation. For instance, it is unlikely that the coating on the structural steel is the original. However, no additional information was made available by the Empire.

3.0 Field Investigation

3.1. Detailed Deck Condition Survey

The DDCS was carried out by RSH subconsultant Du Weldenvarden (DWV) [6]. The following is a summary of the relevant findings.

Deck – Sixteen concrete cores were taken from the deck. Disintegration was noted in one core [6]. Observations of the exposed deck from fifteen sawn asphalt samples revealed spalling in four samples, delamination in one, and scaling in four.

Exposed corroded rebar on the surface of the concrete was noted in one sawn asphalt sample [6]. The cover on the upper rebar layer ranged from 0 mm to 53 mm, with an average of 41 mm. Light corrosion was observed in five cores.

The half-cell corrosion potential test detected probable active corrosion, defined as a corrosion potential value more negative than -0.350 V [8], in 0.3% of the deck area [6]. 92.2% of the deck area likely had no corrosion activity. 7.5% had uncertain corrosion activity.

Sidewalks – The concrete sidewalks were in generally good condition with medium cracks (38.1 m) and wide cracks (3.6 m), delamination (3.07 m^2), spalls (0.23 m^2) and scaling [6]. The spalls are a tripping hazard to pedestrians. The concrete cover on the upper reinforcing steel layer ranged from 40 mm to 112 mm, with an average thickness of 85 mm.

Probable active corrosion was detected for 35.1% of the sidewalk area. 10.3% of the sidewalk area likely had no corrosion activity [6]. 54.6% had uncertain corrosion activity.

Deck Soffit – The soffit and fascia were in fair condition, with extensive medium width cracking (128.5 m), delaminations (14.32 m^2), and spalls (14.19 m^2) [6]. A net is present below the soffit and girders, north of Pier 2, above the pedestrian path.

3.2. 2019 OSIM Report

The most recent OSIM report, provided by the Empire, also provides the following observations and recommendations that were not reiterated in the DDCS [3]:

Structural Steel – Light to heavy surface corrosion, pitting, and flaking was present throughout the girders and diaphragms [3]. The southeast exterior girder and the south abutment diaphragms had significant section loss.

Coating – The existing coating was flaking. Its observed condition was in worse over the SN right-of-way [3].

Asphalt – Transverse cracks and potholes were noted throughout the wearing surface [3]. Exposed waterproofing was visible in one pothole. A wide longitudinal crack was present across the entire span of the bridge. Additional bottom up defects were noted.

Barrier – The handrail had light corrosion throughout [3]. It does not meet current crash test safety standards.

4.0 Proposed Rehabilitation Options

4.1. Deck Rehabilitation

4.1.1. Sidewalk Overlay and Deck Patch

The top layer of the concrete is removed down to the first layer of reinforcing steel or deeper. Then, new concrete is poured to the original level of the curb. In combination with concrete patch repairs to the top surface, soffit, and fascia of the deck, and a new asphalt wearing surface, the integrity of the deck can be improved.

4.1.2. Deck Replacement

The existing 191 mm thick deck is removed and replaced with a new 225 mm thick deck. The new thickness matches the thickness typical of new concrete bridge decks, and provides additional concrete cover. The existing girders will be evaluated to ensure that they have sufficient capacity with the thicker deck and comply with newer design codes.

4.1.3. Optional Deck Widening

The current structure is configured to carry two lanes of traffic and two sidewalks. In one hypothetical scenario where the Broddring Empire wishes to install bicycle lanes on both sides the structure, the bridge deck would need to be widened, ideally during replacement. The suggested minimum width of a bicycle lane is 1.5 m [9]. This would widen the bridge deck by a total of 3.0 m if vehicle lane widths were not reduced.

4.2. Steel Repairs and Recoating

Structural steel is blasted with abrasive material to remove the existing coating. Repairs are made to the bare structural steel. Then, a two or three coat paint system is applied to the steel in accordance with MTO standards [10]. The degree of steel repairs depends on the results of a structural steel inspection.

4.3. Superstructure Replacement

If the existing girders have insufficient capacity to carry a new deck in accordance with current design codes, a superstructure replacement may be required. A superstructure replacement would also enable improvements to the structure, such as the aforementioned deck widening, and extend the service life of the structure further than a deck replacement would.

5.0 Evaluation

The structure was evaluated to determine the capacity of the existing girders to accommodate new 225 mm thick, and optionally wider, reinforced concrete deck. All structural evaluation was completed in accordance with CSA S6:19, the Canadian Highway Bridge Design Code [11]. A summary of the structural evaluation is below. Some details and equations are omitted for length. Calculation spreadsheets are available in Appendix A.

5.1. Properties

The five welded steel plate girders are identical, with varying top and bottom flange thicknesses and widths across their length [2]. Due to the symmetry of the structure, three locations on an exterior girder were chosen for analysis: at the abutment; at the midspan of the second span; and at the central pier.

Section properties were calculated with standard equations. At all locations, the girder section was classified as a Class 4 section due to the web height-width ratio [11, Cl. 10.9.2 and Table 10.3].

Table 1 summarizes the relevant geometric properties of the girders.

Table 1. Geometric Properties

Property	Abutment	Midspan	Pier
$b_{top\text{-}flange}$ (mm)	355.6	355.6	508
$t_{top\text{-}flange}$ (mm)	19.1	19.1	57.2
$b_{bot\text{-}flange}$ (mm)	406.4	457.2	508
$t_{bot\text{-}flange}$ (mm)	19.1	31.8	57.2
h (mm)		1431.9	
t_{web} (mm)		9.5	
C_w (mm^2)	8.7E+13	1.1E+14	6.4E+14
J (mm^4)	2168448	6109639	63627619
A (mm^2)	28155	34929	71073

N.A. (bottom) (mm)	710	584	773
Plastic N.A. (bottom) (mm)	684	341	773
I_x (mm^4)	9953696848	12738938701	34533471748
S_x (mm^3)	13097854	14178761	44668107
Z_x (mm^3)	15389205	18847388	48113660

Table 2 summarizes the relevant material properties.

Table 2. Material Properties

Property	Value	Comments
Φ_s (-)	0.95	Steel resistance factor (flexure and shear) [11, Cl. 10.5.7]
F_y (MPa)	280	Yield stress of existing steel [2] [12]
Φ_c (-)	0.75	Concrete resistance factor [11, Table 8.1]
f'c (MPa)	30	Minimum concrete compressive strength [11, Cl. 8.4.1.2.1]
Φ_r (-)	0.90	Concrete resistance factor [11, Table 8.1]
f_y (MPa)	400	Minimum rebar strength [11, Cl. 8.4.2.1.1]

The effective slab width was calculated with

$$b_s = b\left[1 - \left(1 - \frac{L_e}{15b}\right)^3\right] \leq b \tag{1}$$

and

$$b_e = b_{tf} + b_{s-cant} + b_{s-int} \tag{2}$$

where b is the distance from the top flange to the edge of the slab (for the cantilever portion) or half the distance between the top flanges of the girder (for the interior portion) [11, Fig 5.6], and

L_e is the effective length at that section calculated in accordance with [11, Cl. 5.6.4.6]. At all three locations, the effective slab width was calculated to be 2286 mm.

5.2. Loads

Dead loads were calculated using the dimensions of the existing girders and bridge deck (including curb), and the specific weights provided in [11, Table 3.4]. In addition to the thicker 225 mm bridge deck, the thickness of the asphalt wearing surface was increased to 90 mm. An additional 15 percent was added to the calculated girder weight to account for additional miscellaneous structural steel, including cross-bracing, stiffeners, and shear studs. The weight of the formwork, including workers and equipment, was taken to be 2.5 kPa as specified in [13, 04.01.03].

As the existing barrier does not meet current safety standards, the barrier load was instead calculated using barrier that met crash test requirements. The barrier exposure index was calculated as specified in [11, Cl. 12.4.3.2.3], using a design speed of 70 km/h, which is 20 km/h higher than the posted speed limit.

Table 3 summarizes the factors used.

Table 3. Barrier Exposure Factors

Factor		Comments
Type, K_h	1.375	Two-way undivided, two lanes. [11, Table 12.1]
Curvature, K_c	1.00	No curvature. [11, Table 12.2]
Grade, K_g	1.25	Grade is 3%. [11, Table 12.3]
Height, K_s	1.95	Superstructure is 16 m above deep water. [11, Table 12.4]

The barrier exposure index equation is

$$B_e = \frac{(AADT)K_h K_c K_g K_s}{1000} = 53.4 \quad (3)$$

The clearance beyond the barrier is less than 2.25 m [11, Table 12.5]. The truck percentage on the road is unknown, but by interpolation a TL-5 barrier would be required if the truck percentage exceeded 14.5% [11, Table 12.5]. It was assumed that the truck percentage would not exceed this value due to primarily residential areas north of the structure, and the concentration of industry south of the structure, near to major freeways. Hence, the barrier was taken to be TL-4. The linear load associated with the barrier type is 30 kN [11, Table 3.7] over 5.5 m [11, Fig. 12.2], or 5.5 kN/m.

The pedestrian load was taken as 1.6 kPa as specified in [11, Cl. 3.8.9]. The construction machine load was taken as 1 kPa [13, 04.01.03]. The vehicle load was calculated by software in section 5.4.

5.3. Truck Load Fractions

The simplified method of analysis [11, Cl. 5.6] was used to calculate the truck load fractions and determine the longitudinal load effects of traffic. A lane reduction factor of 0.9 was used to account for the two lanes of traffic present [11, Cl. 14.9.4.2]. The truck load fraction for ULS and SLS 1 [11, Cl. 5.6.4.3] was calculated with

$$F_T = \frac{S}{D_T \gamma_c (1 + \mu\lambda)} \geq 1.05 \frac{nR_L}{N} \quad (4)$$

where S is the spacing between girders, μ is the lane width modification factor [11, Cl 5.6.4.4], n is the number of design lanes, R_L is the lane reduction factor, N is the number of girders, and the remaining variables are determined with the equations in [11, Table 5.3]. Table 4 summarizes the results.

Table 4. Truck Load Fractions at ULS and SLS1

	Abutment		Midspan		Pier	
	Shear	Moment	Shear	Moment	Shear	Moment
D_T	3.40	3.44	3.40	3.44	3.40	3.47
λ	0.00	0.09	0.00	0.09	0.00	0.09
γ_c	1.00	1.00	1.00	1.00	0.90	1.00
R_L	0.9					
F_T	0.73	0.66	0.73	0.66	0.81	0.65

5.4. Demand

The demand was determined by modelling two load cases in SAP2000. Figure 2 and Figure 3 below depict the shear and moment diagrams from the first case, a 1 kN/m uniformly distributed live load across the entire length of the structure.

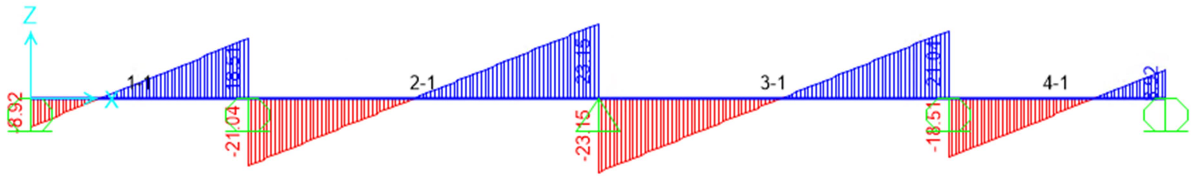


Figure 2. Shear force diagram caused by 1 kN/m live load

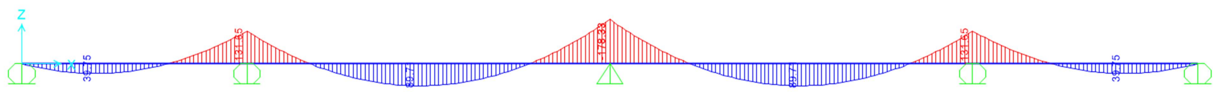


Figure 3. Bending moment diagram caused by 1 kN/m live load

The values were used to scale the shear and moment demand caused by uniformly distributed loads, such as the deck, girders, and asphalt.

The dead loads, except for the girders and sidewalk, were distributed based on the effective slab width. The sidewalk dead load, which would have only acted on a single girder per side, was divided equally across the width of the bridge. This is a permissible assumption due to the symmetry of the cross-section and identical girders [14, Cl. C5.6.3]. The barrier and pedestrian loads only acted on the exterior girder in question.

The second modelled load case was a CL-625-ONT moving live truck load, shown in Figure 3 and Figure 4.

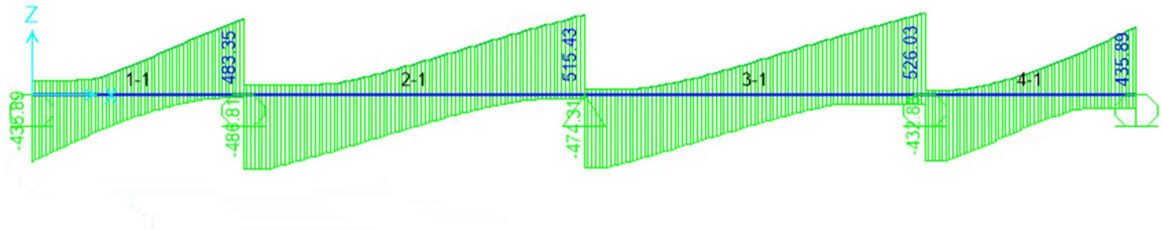


Figure 4. Shear force diagram caused by CL-625-ONT truck

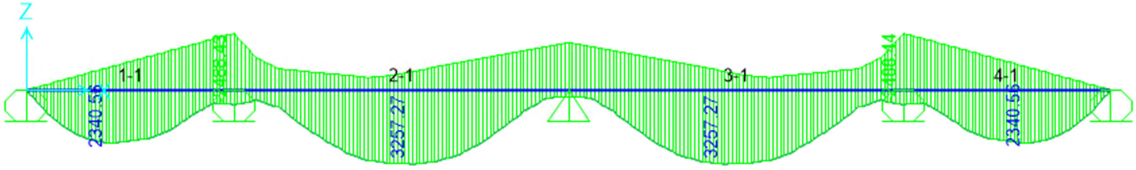


Figure 5. Bending moment diagram caused by CL-625-ONT truck

Two configurations of the truck load are possible [11, Cl. 3.8.3.1.1 and Annex 3.4]. The first increases the load by the dynamic load allowance (DLA), which is 0.25 for this structure [11, Cl. 3.8.4.5.3]. The second reduces the truck load to 80% of its original value and adds a 9 kN/m lane load. The greatest magnitude value from either case was taken to make calculations conservative. For the exterior girder, the shear and moment demand was determined for each location at ultimate limit state (ULS) under construction and service conditions. Construction condition only considers the construction loads (deck, formwork, machines, and girder self-weight). The service condition considers the composite deck and girder self-weight and the service loads (railing, asphalt, trucks, and pedestrians).

The load factors used are summarized in Table 5 [11, Table 3.1-3.3].

Table 5. Load Factors

Element	SLS	ULS
Deck	1	1.2
Girder	1	1.1
Formwork	1	1.2
Machine	1	1.45
Railing	1	1.1
Asphalt	1	1.5
CL-625-ONT truck	0.9	1.7
Pedestrian	0.9	1.7

The demands are summarized in Table 6.

Table 6. Demand at ULS

	Abutment		Midspan		Pier	
Element	Shear (kN)	Moment (kNm)	Shear (kN)	Moment (kNm)	Shear (kN)	Moment (kNm)
Deck	-200	892	-24	2014	520	-4005
Girder	-24	109	-3	246	63	-489
Formwork	-61	273	-7	615	159	-1223
Machine	-29	131	-4	296	76	-589
Railing	-54	240	-6	543	140	-1079
Asphalt	-13	58	-2	130	23	-259
CL-625-ONT truck	-677	401	370	4557	884	-3977
Pedestrian	-13	60	-2	134	35	-267
Construction (Non-Composite)	-315	1406	-37	3172	819	-6306
Service (Composite)	-982	1760	334	7625	1675	-10076

5.5. Capacity

The shear and moment resistances were determined for each location at ULS under construction and service conditions. Capacity-demand ratios were calculated for each location. Additionally, the combined effects of shear and moment were considered [11, Cl. 10.10.5.2].

The bare girder was treated as a Class 4 section. The ratio of web height to width is equal to 150, which allowed it to be treated as a laterally unbraced Class 3 member, subject to a reduction factor [11, Cl. 10.10.3.4 and Cl. 10.10.4.4].

The unfactored yield moment [11, Cl. 10.10.3.2] was calculated as

$$M_y = S_x F_y \tag{5}$$

The critical elastic moment [11, Cl. 10.10.3.3] was calculated as

$$M_u = \frac{\omega_2 \pi}{L}\left[\sqrt{E_S I_y G_s J}\left[B_1 + \sqrt{1 + B_2 + B_1^2}\right]\right] \tag{6}$$

where ω_2, B_1, and B_2 are coefficients calculated based on loading and geometry, and L is equal to the typical unbraced length between cross-bracing.

At all locations, the critical elastic moment was larger than 67% of the unfactored yield moment [11, Cl. 10.10.3.3]. Hence, the resistance was calculated with

$$M_r = 1.15\Phi_s M_y[1 - \frac{0.28M_y}{M_u}] \leq \Phi_s M_y \tag{7}$$

and the applied reduction factor was calculated with

$$1 - \frac{1}{300 + \frac{1200A_{cf}}{A_w}}\left[\frac{2d_c}{w} - \frac{1900}{\sqrt{\frac{M_f}{\Phi_s S}}}\right] \tag{8}$$

The factored shear resistance was taken as

$$V_r = \Phi_s A_w F_s \tag{9}$$

where A_w is the shear area of the web, calculated using the height and F_s is the ultimate shear stress given by

$$F_s = \frac{180000k_v}{(\frac{h}{w})^2} + \left[0.5F_y - 0.866 * \frac{180000k_v}{\left(\frac{h}{w}\right)^2}\right][\frac{1}{\sqrt{1+(\frac{a}{h})^2}}] \tag{10}$$

where a/h is considered infinite for unstiffened webs.

The limits of combined shear and moment were calculated with

$$0.727\frac{M_f}{M_r} + 0.455\frac{V_f}{V_r} < 1.0 \tag{11}$$

where M_f and V_f are the factored moment and shear demand, respectively.

Table 7 summarizes the results at ULS under construction conditions.

Table 7. Capacity at Construction ULS

	Abutment		Midspan		Pier	
	Shear (kN)	Moment (kNm)	Shear (kN)	Moment (kNm)	Shear (kN)	Moment (kNm)
Capacity	1949	4193	2417	4004	4962	12930
Demand	-315	1406	-37	3172	819	-6306
Capacity-Demand Ratio (-)	6.18	2.98	64.49	1.26	6.06	2.05
Combined Shear and Moment (-) *(Acceptable if < 1.0)*	0.32		0.58		0.43	

Analysis of the composite section found that the plastic neutral axis of the section was located in the concrete deck [11, Cl. 10.11.6.2.1]. The moment resistance [11, Cl. 10.11.6.2.2] was calculated with

$$M_r = C_c e_c + C_r e_r + +C_s e_s \tag{12}$$

where e_c, e_r, and e_s are lever arms associated with the concrete, rebar, and steel, measured from the neutral axis of the steel area in tension [11, Fig. 10.4], and C_c, C_r, and C_s are the factored resistances of the concrete, rebar and steel calculated with the following equations.

$$C_C = \propto_1 \Phi_c b_e t_c f'_c \tag{13}$$

where α_1 is the ratio of average stress in a rectangular compression stress block to the specified concrete strength [11, Cl. 8.8.3] and t_c is the thickness of the concrete.

$$C_r = \Phi_r A_r f_y \tag{14}$$

where A_r is the area of the rebar. In lieu of a detailed design, the rebar arrangement in the deck was assumed to be identical to that of the existing, though the contribution of the rebar was found to be negligible.

$$C_s = \Phi_s A'_{sc} F_y \tag{15}$$

where A'_{sc} is the area of steel in compression.

Table 8 summarizes the results at ULS under service conditions.

Table 8. Capacity at Service ULS

	Abutment		Midspan		Pier	
	Shear (kN)	Moment (kNm)	Shear (kN)	Moment (kNm)	Shear (kN)	Moment (kNm)
Capacity	1949	8640	2417	13652	4962	N/A
Demand	-982	1760	334	7625	1675	-10076
Capacity-Demand Ratio (-)	1.98	4.91	7.24	1.79	2.96	N/A

In the negative bending moment regions at the pier, the stresses on the steel and composite sections were checked against the lateral torsional buckling stress and the yield stresses of the steel and rebar [11, Cl. 10.11.6.3]. The inequalities used were

$$\frac{M_{fd}}{S} + \frac{(M_{fsd} + M_{fl})}{S'} \leq \Phi_s F_{cr} \tag{16}$$

$$\frac{M_{fd}}{S} + \frac{(M_{fsd} + M_{fl})}{S'} \leq \Phi_s F_y \tag{17}$$

$$\frac{(M_{fsd} + M_{fl})}{S'} \leq \Phi_r f_y \tag{18}$$

where S and S' are the elastic section moduli of the steel and composite sections, M_{fd}, M_{fsd}, and M_{fl} are the factored moment demands associated with the dead, superimposed dead, and live loads, and F_{cr} is the moment resistance of the steel section divided by the resistance factor and section modulus. As specified in [11, C.10.11.6.3.1], the contribution of the reinforcement and concrete to the resistance can be neglected for simplicity and conservatism. Regardless, as the girder section at the pier is doubly symmetric, the yielding of the tension flange and reinforcement is unlikely to govern. Table 9 summarizes the results for the negative moment region.

Table 9. Capacity at Service ULS in Negative Moment Region

	Capacity (MPa)		Demand (MPa)
$\Phi_s F_{cr}$	289	>=	226
$\Phi_s F_y$	266	>=	226
$\Phi_r f_y$	360	>=	196

5.6. Control of Permanent Deflection

The transformed composite section at the midspan location was verified to ensure that permanent deflections at serviceability limit state (SLS) were acceptable [11, Cl. 10.11.4]. The location was chosen because it experiences the greatest positive bending moment. The bending moment demand at SLS is summarized in Table 10.

Table 10. Midspan Bending Moment Demand at SLS

Element	Moment (kNm)
Deck	1679
Girder	224
Railing	493
Asphalt	87
CL-625-ONT truck	2413
Pedestrian	79
Service (Composite)	4974

The modulus of elasticity of concrete was calculated with [11, Cl. 8.4.1.5.3] to be 27 398 MPa. The modulus of elasticity of steel was taken as 200 000 MPa [11, Cl. 10.4.2]. The modular ratio, n, was calculated to be 7.30. The neutral axes of the transformed composite section were calculated with

$$y_a = \frac{A_s * y_s + (A_c * y_c)/a}{A_s + A_c/a} \tag{19}$$

where factor a is equal to n or 3n. The section moduli of the composite section was calculated with

$$S_a = (I_s + \frac{I_c}{a})/y_a \tag{20}$$

The maximum normal stress in the positive moment regions was calculated with

$$\frac{M_d}{S_x} + \frac{M_{sd}}{S_{3n}} + \frac{M_L}{S_n} \leq 0.90F_y \tag{21}$$

where M_d, M_{sd}, and M_L are the SLS factored moment demand caused by the dead, superimposed dead, and live loads, respectively. The left side was determined to be 239.5 MPa, less than 90% of the yield stress of the steel or 252 MPa [11, Cl. 10.11.4]

Control of permanent deflection is unlikely to govern for a Class 3 or Class 4 (stiffened plate girder) section in the negative moment region [14, Cl. C10.11.4].

5.7. Recommended Work

Removing and patching concrete at problematic locations on the deck would stop dangerous spalls, and reduce the risk to trains and pedestrians travelling underneath. However, the significant deterioration of the deck soffit, in spite of the generally low probability of active corrosion in the deck, suggests that the soffit is likely to continue to deteriorate in other locations. Similarly, overlaying the existing sidewalk would eliminate the tripping hazards currently present and remove some of the contaminated concrete causing corrosion. The findings of the DDCS suggest that the sidewalk is highly susceptible to corrosion, and it is unlikely that scarifying and overlaying the top of the sidewalk will remove all of the problematic concrete.

ASTM Standard C876-15 [8], the corrosion potential test method used in the DDCS, is suitable for slabs of any thickness or cover depth. However, concrete cover greater than 75 mm can cause adjacent corrosion potentials to average out and smooth out spikes in corrosion potential. This can obscure the severity of corrosion in a thick slab.

Evaluation of the structure found that the existing girders have sufficient capacity to support a new deck in accordance with current design standards. A detailed structural steel inspection should be conducted to confirm that the girders remain in good condition. A full replacement of the existing superstructure is not recommended provided that the results of the steel inspection are acceptable. Therefore, RSH recommends that the bridge undergo a deck replacement. Excel Goal Seek was used to determine the maximum allowable increase in bridge width. At ULS, the controlling factor was determined to be moment resistance of the bare girder at the midspan location. An increase in width of 662 mm on both sides reduced the capacity-demand ratio to 1.0. This means that the structure could accommodate two minimum-width bicycle lanes if the vehicle lane widths were reduced. However, this cannot be recommended for the following reasons:

- The vehicle lane widths would be reduced significantly to less than 3.5 m each;
- No buffer or shoulder would be present between the bicycle lanes and vehicle lanes;
- Increasing demand to the match capacity cannot be recommended, even with generally conservative calculations; and
- The increased demand induces unacceptably high stress in the steel, violating the serviceability limit state.

It is clear that a widening the bridge deck to accommodate bicycle lanes would not be possible without changes to the existing superstructure, for instance by the addition of an extra girder.

Each time work performed on a structure, there are fixed costs that do not directly scale with the size of the project, including the costs of engineering design, mobilization, access, roadway and environmental protection, traffic control, and railway flagging. These costs make it likely that the cost of additional overlays or patches for the rest of the expected service life of the bridge will be greater than the cost to replace the deck entirely. The 2019 OSIM report provided estimates of the capital costs for a full structure replacement and an extensive rehabilitation that includes concrete patching, substructure repairs, and bearing replacements [3]. For the rehabilitation, the cost of insurance, mobilization, and access was given as $300 000, railway flagging as $75 000, and roadside protection as $54 000. The difference in cost of engineering design was only 25% ($239 000 compared to $300 000) when comparing a $4M rehabilitation project and a $21M replacement project.

It is also recommended that the structural steel be abrasive blast cleaned, repaired, and recoated in accordance with MTO standards, given its overall poor condition noted in the OSIM. This work can be done after the existing deck is removed.

6.0 Construction and Traffic Staging

The AADT of Broddring Empire Road 95 is 15 329 vehicles per day [5].

For a typical two-stage construction procedure, the narrow 9.14 m curb-to-curb deck width of the existing structure would make it impossible to maintain simultaneous two-way traffic during construction. While past rehabilitation work has employed temporary traffic signalling to alternate the direction of traffic flow [7], the length of the structure, high traffic volume, and longer duration of construction make this impractical.

Figure 6 shows the nearest north-south detour, Oromis Street (Teirm Road 48), which can be accessed approximately 100 m from the north approach of the bridge and 400 m from the south [1]. The detour requires passage through a Surdan National level crossing and a local residential street, Brom Street. There is potential for traffic backups while the crossing is closed, and disruption to local residents due to increased traffic is assured.

Figure removed.

Figure 6. Nearest detour route

(Map data © OpenStreetMap contributors) [1]

Due to these factors, RSH proposes a modified two-stage construction sequence that would maintain a single lane of traffic over the bridge at all times. This would lessen the traffic burden for the detour route.

In Stage 1, the east half of the bridge would be closed to allow construction in the Stage 1 work area. After the completion of Stage 1, the east half would be reopened for vehicle traffic, and the west half of the bridge would be closed to allow construction in the Stage 2 work area. A longitudinal construction joint would connect the bridge deck near the centreline of the structure.

To reduce confusion for drivers, vehicle traffic would only be allowed in the same single direction for the entire duration of construction. In lieu of a traffic study, it is recommended that southbound traffic be detoured to Oromis Street to avoid forcing drivers to backtrack, and to prevent traffic backups on residential streets when a train is passing through the level crossing.

7.0 Stakeholder Constraints

7.1.1. Upper Toark River Conservation Authority

The bridge crosses over the Toark River and falls within the regulated area of the Upper Toark River Conservation Authority [15]. The UTRCA will need to be engaged to determine the permissible disruption to the local environment. The contractor will be directed to protect the waterway from debris generated by construction, and to restore any disrupted areas to their original state after work is completed.

One particular concern is the possibility that the structural steel coating contains hazardous lead. The 3 coat Alkyd paint coat system applied to most MTO bridges built prior to 1974 used a red lead primer [10]. It is extremely unlikely that the steel has not been recoated since its construction, as even coatings applied circa 2000 have estimated service lives of 20-25 years, but the possible presence of lead must still be considered.

7.1.2. Surdan National

The bridge crosses over a Surdan National right-of-way. The clearance from the top of the tracks to the bottom of the girders is 7.16 m [2]. Any work done over or in close proximity to the railway must be done under flagging protection. Access to the bridge soffit over the right-of-way must be portable.

8.0 Conclusion

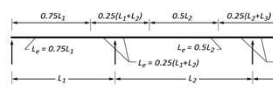
The Toark River Bridge is in need of major rehabilitation within the next two years. Based on the information from a Detailed Deck Condition Survey and the provided OSIM report, RSH recommends that the bridge undergo a deck replacement to eliminate concerns about continued deterioration in the sidewalk and soffit. The existing girders have sufficient capacity to accommodate a new, thicker deck in accordance with current codes, subject to a detailed structural steel inspection and steel repairs. It is also recommended that the structural steel be cleaned and coated. To facilitate the required construction, a modified two-stage construction sequence that maintains a single direction of traffic over the bridge is recommended to reduce the impact on nearby residential streets. Environmental concerns due to construction and the potential for lead contaminants must be considered, as should the impacts of construction on the Surdan National right-of-way below the structure. Future work on the rehabilitation of the bridge should include a cost estimation and detailed design of the new concrete deck.

References

References removed.

Appendix A: Structural Evaluation

Section									
Φ_s	-	0.95							
F_y	MPa	280						*http://dir.cisc-icca.ca/files/technical/techdocs/historical/historicalsteels.pdf*	
Φ_c	-	0.75							*G40.8 Grade A (1960)*
f'c	MPa	30							
Φ_r	-	0.9							
		Abutment		**Midspan**		**Pier**			
		Top	Bottom	Top	Bottom	Top	Bottom		
b	mm	355.6	406.4	355.6	457.2	508	508		*Flanges*
t	mm	19.05	19.05	19.05	31.75	57.15	57.15		
b/2t	-	9.3	10.7	9.3	7.2	4.4	4.4		
b/2t * SQRT(F_y)		156.2	178.5	156.2	120.5	74.4	74.4		
		Class 2	Class 2	Class 2	Class 1	Class 1	Class 1	*10.9.2.1*	
h	mm	1431.925							*Web*
w	mm	9.525							
h/w	-	150.3333333							
h/w * SQRT(F_y)		2515.557813							
		Class 4						*10.9.2.1*	*Section is Class 4*
k_v		5.34							
(h/w)/(SQRT(k_v/F_y)		1088.58833							
C_w	mm^2	8.76483E+13		1.14143E+14		6.40084E+14		*C10.10.2.3*	
J	mm^4	2168448		6109639.16		63627619.29		*C10.10.2.3*	
G_s	MPa	77000							
A_{tf}	mm^2	6774.18		6774.18		29032.2			
y_{ntf}	mm	1460.5		1473.2		1517.65			*From bottom*
A_w	mm^2	13639.08563		13639.08563		13639.08563			
y_{nw}	mm	735.0125		747.7125		773.1125			*From bottom*
A_{bf}	mm^2	7741.92		14516.1		29032.2			
y_{nbf}	mm	9.525		15.875		28.575			*From bottom*
NA	mm	710.0763024		584.2728463		773.1125			*From bottom*
PNA	mm	684.2125		341.3125		773.1125			
I_x	mm^4	9953696848		12738938701		34533471748			
I_y	mm^4	178042052.3		324347398.4		1248797395			
S_x	mm^3	13097854		14178761		44668107			
I_{yc}	mm^4	71383689.49		71383689.49		624347138.4		*C10.10.2.3*	
β_x	-	-255.249099		-721.005853		-0.10627631			
Z_x	mm^3	15389205		18847388		48113660			

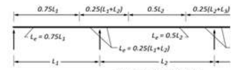


a) Continuous-span bridge

Structure								*5.6*	*Satisfies 5.6.2*
N	-	5							
L	mm	143260							*L_1 = 27432*
L_e	mm	20574		22098		35814		*5.6.4.6*	*L_2 = 44196*
W	mm	12801.6							
t_{slab}	mm	225							*225 mm for replacement, 191 (existing) for overlay*
S	mm	2743.2							
S_c	mm	914.4							
W_c	mm	9144							
n	-	2							
W_e	mm	4572							
		Interior	Overhang	Interior	Overhang	Interior	Overhang		
b	mm	1193.8	736.6	1193.8	736.6	1117.6	660.4		
b_s	mm	1193.8	736.6	1193.8	736.6	1117.6	660.4		
b_e	mm	2286.0		2286.0		2286.0			
inc	mm	0							
		2286.0							

Dead Load		2.286	1.8288	Existing		Rehabilitated			
	Element		γ_c [kN/m^3]	Area [m^2]	P [kN/m]	Area [m^2]	P [kN/m]	**Load Factor**	
		Non-Composite						**SLS1**	**ULS1**
	Reinforced Concrete Slab		24	0.436626	10.479024	0.51435	12.3444	1	1.2
	Reinforced Concrete Curb		24	0.74322432	6.37049417	0.743224	6.370494	1	1.2
	Girder		77		2.16794929	0	2.167949	1	1.1
	+15% Girder				0.32519239		0.325192	1	1.1
	Formwork		2.5	0	0	2.286	5.715	1	1.2
		Composite							
	Railing				11		11	1	1.1
	Asphalt		23.5	0.03483864	0.81870804	0.041148	0.966978	1	1.5
Live Load									
	Element								
		Non-Composite							
	Machine		1		2.286		2.286	0.9	1.445
		Composite							
	CL-625 ONT + DLA							0.9	1.7
	80% CL-625 ONT + Lane Load							0.9	1.7
	Pedestrian		1.6		2.44608		2.44608	0.9	1.7

Structure			Abutment		Midspan		Pier		5.6	*Satisfies 5.6.2*
N	-		5							
L	mm		143260							*L_1 = 27432*
L_e	mm		20574		22098		35814		*5.6.4.6*	*L_2 = 44196*
W	mm		12801.6							
t_{slab}	mm		225							*225 mm for replacement, 191 (existing) for overlay*
S	mm		2743.2							
S_c	mm		914.4							
W_c	mm		9144							
n	-		2							
W_e	mm		4572							
			Interior	Overhang	Interior	Overhang	Interior	Overhang		
b	mm		1193.8	736.6	1193.8	736.6	1117.6	660.4		
b_s	mm		1193.8	736.6	1193.8	736.6	1117.6	660.4		
b_e	mm		2286.0		2286.0		2286.0			
R_L			0.9						*3.8.4.2.1*	
μ			1						*5.6.4.4*	
DLA			0.25							
			Interior	Exterior	Interior	Exterior	Interior	Exterior		
D_T	ULS1/SLS1	Moment	3.55	3.44	3.58	3.44	3.77	3.47	*5.6.4.3*	
		Shear	3.40	3.40	3.40	3.40	3.40	3.40		
λ		Moment	0.09	0.09	0.09	0.09	0.09	0.09		
		Shear	0.00	0.00	0.00	0.00	0.00	0.00		
γ_c		Moment	1.00	1.00	1.00	1.00	1.00	1.00		
		Shear	1.00	1.00	1.00	1.00	0.90	0.90		
γ_e		Moment	N/A	N/A	N/A	N/A	N/A	N/A		
		Shear	N/A	N/A	N/A	N/A	N/A	N/A		
F_T		Moment	0.71	0.73	0.70	0.73	0.67	0.72		
		Shear	0.81	0.81	0.81	0.81	0.90	0.90		
		Moment	**0.64**	**0.66**	**0.63**	**0.66**	**0.60**	**0.65**		
		Shear	**0.73**	**0.73**	**0.73**	**0.73**	**0.81**	**0.81**		
D_T	SLS2	Moment	4.14	3.65	4.16	3.66	4.30	3.73		
		Shear	3.60	3.60	3.60	3.60	3.60	3.60		
λ		Moment	0.05	0.05	0.05	0.05	0.05	0.05		
		Shear	0.00	0.00	0.00	0.00	0.00	0.00		
γ_c		Moment	1.12	1.00	1.12	1.00	1.12	1.00		
		Shear	1.00	1.00	1.00	1.00	0.90	0.90		
γ_e		Moment	0.00	1.00	0.00	1.00	0.00	1.00		
		Shear	0.00	0.00	0.00	0.00	0.00	0.00		
F_T		Moment	0.56	0.37	0.56	0.37	0.54	0.36		
		Shear	0.76	0.76	0.76	0.76	0.85	0.85		
		Moment	**0.51**	**0.33**	**0.50**	**0.33**	**0.49**	**0.32**		
		Shear	**0.69**	**0.69**	**0.69**	**0.69**	**0.76**	**0.76**		

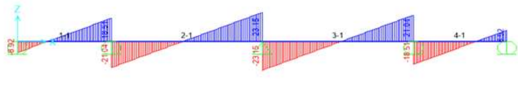


a) Continuous-span bridge

SLS	1kN/m		Deck		Girder		Formwork		Machine		Non-Composite	
			18.71489	1	2.493142	1	5.715	1	2.286	1		
	V	M	V	M	V	M	V	M	V	M	V	M
Abutment	-8.92	39.75	-167	744	-22	99	-51	227	-20	91	-261	1161
Midspan	-1.06	89.7	-20	1679	-3	224	-6	513	-2	205	-31	2620
Pier	23.15	-178.33	433	-3337	58	-445	132	-1019	53	-408	676	-5209
ULS			1.2		1.1		1.2		1.445			
			V	M	V	M	V	M	V	M	V	M
Abutment			-200	893	-24	109	-61	273	-29	131	-315	1406
Midspan			-24	2014	-3	246	-7	615	-4	296	-37	3172
Pier			520	-4005	63	-489	159	-1223	76	-589	819	-6306

	Railing		Asphalt		CL-625-ONT		Pedestrian		Composite	
	5.5	1	0.966978	1	Varies	Varies	0.978432	0.9		
	V	M	V	M	V	M	V	M	V	M
Abutment	-49	219	-9	38	-359	212	-8	35	-613	1347
Midspan	-6	493	-1	87	196	2413	-1	79	166	4974
Pier	127	-981	22	-172	468	-2106	20	-157	1129	-7198
	1.1		1.5		Varies	Varies	1.7			
	V	M	V	M	V	M	V	M	V	M
Abutment	-54	240	-13	58	-677	401	-13	60	-982	1760
Midspan	-6	543	-2	130	370	4557	-2	134	334	7625
Pier	140	-1079	34	-259	884	-3977	35	-267	1675	-10076

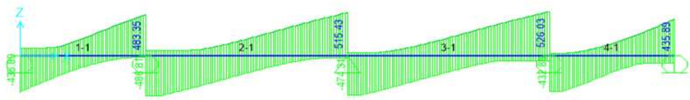
CL-625-ONT			80% CL-625-ONT + Lane Load		CL-625-ONT + DLA		
	V	M	V	M		V	M
Abutment	-438.89	0	-431	358	Abutment	-548.613	0
Midspan	240	3257.27	182	3413	Midspan	300	4071.588
Pier	515.43	-2488.43	621	-3596	Pier	644.2875	-3110.54

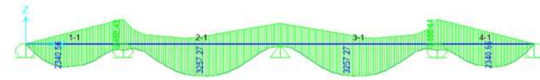


Governing Live Load

	V	M
Abutment	-549	358
Midspan	300	4072
Pier	644	-3596

	SLS1		ULS1	
	F_T	0.9	F_T	1.7
	V	M	V	M
Abutment	0.65	0.59	1.23	1.12
Midspan	0.65	0.59	1.23	1.12
Pier	0.73	0.59	1.37	1.11





		Abutment	Midspan	Pier		
Construction ULS						
Φ_s	-	0.95				
S_x	mm_3	13097853.68	14178761.4	44668106.84		
F_y	MPa	280	280	280		
M_y	Nmm	3.67E+09	3.97E+09	1.25E+10		*10.10.3.3*
β_x	-	-255.249099	-721.005853	-0.106276314		*10.10.2.3*
L	mm	5385.816	5690.616	4800.6		
La		1346.454	1422.654	1200.15		
Lb		2692.908	2845.308	2400.3		
Lc		4039.362	4267.962	3600.45		
E_s	MPa	200000				
I_y	mm^3	178042052.3	324347398	1248797395		
G_s	MPa	77000				
J	mm^4	2168448	6109639	63627619		
C_w	mm^2	8.76483E+13	1.1414E+14	6.40084E+14		
B_1		-1.08714888	-2.33704164	-0.000248287		
B_2		35.72155778	14.7894918	11.19022898		
M_{max}	Nmm	1760	7625	-10076		
M_a	Nmm	1760	7625	-10076		
M_b	Nmm	1760.466135	7625	-10076		
M_c	Nmm	1760	7625	-10076		
ω_2		1	1	-1	*conservatively (if Ma, Mb, Mc all equal to Mmax)*	
M_u	Nmm	7210303536 *Mu bigger*	6931670655 *Mu bigger*	-79920573018 *Mu bigger*		
Z_x		15389205	18847388	48113660		
M_y	Nmm	4308977519	5277268647	13471824838		
$0.67M_y$		2887014937	3535769993	9026122641		
Term 1		0.00111606	0.00111606	0.000350346		
Bracket Term		-21.764368	105.185251	-29.52715237		
Reduction Factor		1.024290349	0.88260691	1.01034471		
M_r	kNm	4192.961881	4003.85192	12930.62762		
Demand		1406	3172	-6306		
C/D		2.982990622	1.2622741	2.050516659		
h/w		150.3333333				*10.10.5*
k_v		5.34				
$(h/w)/(SQRT(k_v/F_y)$		1088.58833				
F_{cr}	N	42.53076435				
F_t	N	103.1683581				
F_s	N	145.6991224				
A_w		14077.59281	17464.6828	35851.74281		
V_r	N	1948538	2417359.51	4962389.092		
	kN	1948.538273	2417.35951	4962.389092		
Demand SLS	kN	-315	-37	819		
C/D	-	6.177492018	64.4916495	6.061888235		
Combined		0.317369634	0.58299982	0.429603911	*< 1.0*	*10.10.5.2*
Service ULS						
b_e	mm	2286				
Φ_c	-	0.75				
f'c	MPa	30				
Φ_r	-	0.9				
$F_{y\text{-}rebar}$	MPa	400			*400W*	
Moment region		Positive	Positive	Negative		
α_1	-	0.805				
C_{c1}	kN	9316	9316	9316	*same effective slab width and thickness*	*10.11.6.2.2*
A_r		4071	4071	1929	*assuming same rebar as existing*	
Cr	kN	262	262	4416		
C_1	kN	9578	9578	13733		
C_2	kN	7489	9291	19073		
PNA in...		Concrete	Concrete			
a	mm	145.4785565	188.998041			
C_{c2}		6023565.091	7825496.97			
C_s		1232325.508	1232325.51			
e_c	mm	1240.41875	1595.225			
e_r	mm	1302.91875	1657.725			
e_s	mm	948.0815186	948.081519	0.34101904		
M_r		8640.429138	13652.2073			
Demand		1760	7625			
C/D		4.908034848	1.7904553			
M_{fd}	Nmm			-4493974646	*Neglecting slab reinforcement is conservative*	*10.11.7.3.1*
M_{fsd}	Nmm			-1337558280	*C10.11.6.3.1*	
M_{fl}	Nmm			-4244119199		
S				44668107	*WRT top fibre*	
S'				44668107		
M_r	Nmm			12930627617		
F_{cr}	Mpa			304.7182415		
Φ_sF_{cr}		289.4823294	>=	225.5670284	*OK*	
Φ_sF_y		266	>=	225.5670284	*OK*	
Φ_rf_y		360	>=	195.6226593	*OK*	
Demand ULS	kN	-982	334	1675		
C/D	-	1.983702247	7.2358196	2.961941792		

F_y	MPa	280	*10.11.4*		
E_s	MPa	200000	*Ctrl of permanent deflections unlikely to govern for Class 3 and plate girder negative moment regions*		
E_c	MPa	27398.34869			
n	-	7.299710002			
3n	-	21.89913001			
A_s		34929.36563			
A_c		514350			
y_s	mm	584.2728463			
y_c		1544.425			
y_n		1226.205382			
y_{3n}		970.3158323			
I_c		2169914063			
I_x		12738938701	steel only		
$I_{x\text{-}n}$		34267754747			
$I_{x\text{-}3n}$		34873964060			
S_x		14178761			
$S_{x\text{-}n}$		27946179			
$S_{x\text{-}3n}$		35940838			
M_d	Nmm	1902360817	4974.205		
M_{sd}	Nmm	580087927			
M_L	Nmm	2491756261			
		239.4725068	<=	252	OK