BC Ministry of Transportation & Infrastructure

Old Spences Bridge No. 2411

Load Capacity Evaluation &

Rehabilitation Options

2010 June 10 Our Ref: 1884-RPT-SPE-002-2

Prepared by: Mark Steunenberg, P.Eng.

PROFESSIONAL ENGINEER PROVINCE OF BRITISH COLUMBIA M. STEUNENBERG #28418

Reviewed by: Darryl Matson, P.Eng.

PROFESSIONAL ENGINEER PROVINCE OF BRITISH COLUMBIA D. D. MATSON

Andrew Griezic, P.Eng.

PROFESSIONAL ENGINEER PROVINCE OF BRITISH COLUMBIA A.J. GRIEZIC

BUCKLAND & TAYLOR LTD.
101-788 Harbourside Drive
North Vancouver, BC, Canada V7P 3R7
Tel: (604) 986-1222 Fax: (604) 986-1302
E-mail: mainoffice@b-t.com
Website: www.b-t.com

Executive Summary

The Old Spences Bridge was constructed in 1931 and crosses the Thompson River providing a link between Highway 8 and Highway 1 in the Community of Spences Bridge, BC. In 1962, a new bridge was constructed approximately 900 m downstream that also connects Highway 8 and Highway 1.

The Old Spences Bridge is a single lane bridge composed of five truss spans and two girder spans. The truss spans vary in length with a single span of 21.0 m (69 ft.), two spans of 27.7 m (91 ft.) and two spans of 65.8 m (216 ft.). The girder spans are 11.3 m (37 ft.) and 12.2 m (40 ft.) making the total length of the bridge 231.6 m (760 ft.). Six concrete piers and two concrete abutments support the bridge.

Annual inspections of the Old Spences Bridge have been performed for many years and following the 2002 inspection the bridge was posted with a 25 tonne load limit. During the 2008 inspection, significant deterioration, corrosion and holes were identified in heavier structural components. Based on the 2008 visual inspection the bridge was closed to all vehicular traffic in 2009 in order to ensure public safety.

Subsequent to closing the crossing, the British Columbia Ministry of Transportation and Infrastructure (BC MoT) retained Buckland & Taylor Ltd. (B&T) to carry out a detailed inspection and load capacity evaluation of the structure. As part of their assignment, B&T was also tasked with developing conceptual rehabilitation options and cost estimates to restore the bridge to a range of acceptable levels of reliability.

Recommended maintenance, rehabilitation and evaluation items based on observations made during the detailed inspection are presented in B&T Report No. 1884-RPT-SPE-001-0, “Old Spences Bridge No. 2411 – Inspection Report.”

This report summarizes the findings of the load evaluation of the bridge, makes recommendations regarding conceptual rehabilitation options, and summarizes cost estimates to restore the bridge to a range of acceptable levels of reliability.

The results of the load evaluation for the various vehicular and pedestrian loadings applied to the bridge in its current state are summarized in Table 1.

It should be noted that in the evaluation, two pedestrian load cases have been established in order to satisfy the intent of the code, while at the same time being more representative of local conditions. Load case 1 is pedestrian loading applied to the sidewalk only, in accordance with CHBDC. Load case 2 is pedestrian loading applied anywhere on the bridge, but the loading, as

specified by BC MoT, is limited to a maximum of fifty (50) pedestrians. If the bridge is opened as a pedestrian-only bridge, BC MoT must post signage limiting the pedestrian load to a maximum of fifty (50) people on the bridge at any given time.

Table 1: Vertical Load Evaluation Conclusions – By Member Type

Item	Conclusions Regarding Live Load Models (without snow load)			
	CL1-625	**25 Tonne**	**5 Tonne**	**Pedestrians**
Concrete Deck [1]	Acceptable	Acceptable	Acceptable	N/A
Deck Stringers	**Not Acceptable**	Acceptable	Acceptable	Acceptable
Floorbeams	**Not Acceptable**	**Not Acceptable – some in bending**	Acceptable	Acceptable
Sidewalk	**Not Acceptable**	**Not Acceptable**	Acceptable	Acceptable
Truss System	**Not Acceptable**	**Not Acceptable – some diagonals**	Acceptable	Acceptable
Truss Bearings	**Not Acceptable**	Acceptable	Acceptable	Acceptable
Girders	**Not Acceptable**	**Not Acceptable – webs at bearings**	Acceptable	Acceptable
Concrete Piers	**Not Acceptable**	Acceptable	Acceptable	Acceptable
Overall Conclusion	**Not Acceptable**	**Not Acceptable**	Acceptable	Acceptable

Notes: 1. In addition to the conclusion that the strength of the deck is acceptable, there are potentially serviceability issues that may need to be addressed due to gaps that have developed between the stringers and the deck.

In its current condition, the bridge can be opened to 5 tonne vehicle traffic. However, it is recommended that repairs be carried out before the end of 2011 if the bridge is intended to remain in service beyond 2011.

In its current condition, the bridge can be opened as a pedestrian-only bridge, subject to a load limit of fifty (50) pedestrians. However, it is recommended that repairs to some of the concrete piers be carried out by the end of 2011 if the bridge is intended to remain in service beyond 2011.

Given the fact that the traffic barrier connection does not meet PL-1 requirements, if the bridge is opened to vehicular traffic, it is recommended that BC MoT assess the risks associated with the barrier and establish whether the barrier should be upgraded to a higher standard. The estimated cost associated with upgrading the barrier on both sides of the bridge is included in the summary of costs for various options with the bridge open to vehicles.

The results of the load evaluation demonstrate that it is important to perform snow removal if the bridge is reopened in order to ensure that maximum vehicular or pedestrian load is not coincident with maximum snow loads. If the bridge is open for vehicular loads, a maximum snow depth of 350 mm concurrent with vehicular load is established as the limit, beyond which snow removal is required. If the bridge is open as a pedestrian-only bridge, a maximum snow depth of 600 mm is established as the limit, beyond which snow removal by manual methods or lightweight equipment weighing less than 500 kg is required.

High-level cost estimates have been prepared for the different vehicle loadings considered in the evaluation and for the different rehabilitation design life options. The summary of the estimated costs is listed in Table 2.

Table 2: Summary of Costs for Various Rehabilitation Options

Option	Estimated Cost (2009 dollars)			Comment
	Project Costs: Rehabilitation, Construction & Management	Maintenance Inspections	Total Project Cost[1]	
1. Immediate Demolition	N/A	N/A	**$1.5 M**	
2. Repair				
(a) 2 years @ limited pedestrian	nil	$0.15 M	**$0.15 M**	Does not include costs associated with mitigating seismic and wind risk
(b) 2 years @ 5 tonne	$ 0.55 M (optional barrier repairs)	$ 0.15 M	**$0.15 -$0.70 M**	
(c) 10 years @ limited pedestrian	$ 0.18 M (pier repairs)	$ 0.60 M (bi-annual detailed)	**$ 0.78 M**	
3. Rehabilitation				
(a) 10 years @ 5 tonne	$1.90 M	$ 1.35 M	**$ 3.25 M**	
(b) 10 years @ 25 tonne	$ 3.29 M	$ 0.36 M	**$ 3.65 M**	
(c) 25 years @ 5 tonne	$ 24.84 M	$ 0.16 M	**$ 25.0 M**	
(d) 50 years @ 5 tonne	$ 26.64 M	$ 0.36 M	**$ 27.0 M**	
(e) 25 years @ 25 tonne	$ 25.34 M	$ 0.16 M	**$ 25.5 M**	
(f) 50 years @ 25 tonne	$ 27.14 M	$ 0.36 M	**$ 27.5 M**	
4. Replacement				
(a) New single lane bridge with sidewalk	$ 14.3 M	N/A	**$ 14.3 M[2]**	Seismic and wind risk mitigated
(b) New two lane bridge with sidewalk	$ 22.7 M	N/A	**$ 22.7 M[2]**	

Notes: 1 - For all options except immediate demolition, the life-cycle cost must be increased by $1.5 M to reflect demolition costs.

2 - An allowance of $0.5 M has been made for property acquisition, in the event that a revised location is chosen for the new structure.

Based on the estimated costs of rehabilitating Old Spences Bridge, it does not appear to be cost effective to upgrade the existing bridge beyond a 10 year life. If BC MoT intends to provide this extra crossing between Highway 1 and Highway 8, in addition to the bridge just downstream, replacement of the bridge should be considered within the next 10 years.

It is also noted that opening the bridge for a pedestrian-only crossing is more favourable than a vehicular crossing in terms of cost, public safety as well as confidence in achieving the estimated service life.

Table of Contents

1 Introduction

The Old Spences Bridge was constructed in 1931 and crosses the Thompson River providing a link between Highway 8 and Highway 1 in the Community of Spences Bridge, BC. In 1962, a new bridge was constructed approximately 900 m downstream that also connects Highway 8 and Highway 1.

The Old Spences Bridge is a single lane bridge composed of five truss spans and two girder spans. The truss spans vary in length with a single span of 21.0 m (69 ft.), two spans of 27.7 m (91 ft.) and two spans of 65.8 m (216 ft.). The girder spans are 11.3 m (37 ft.) and 12.2 m (40 ft.) making the total length of the bridge 231.6 m (760 ft.). Six concrete piers and two concrete abutments support the bridge. An elevation, plan and typical sections of the bridge are shown in Figure 1 and Figure 2. A general arrangement drawing is included in Appendix A.

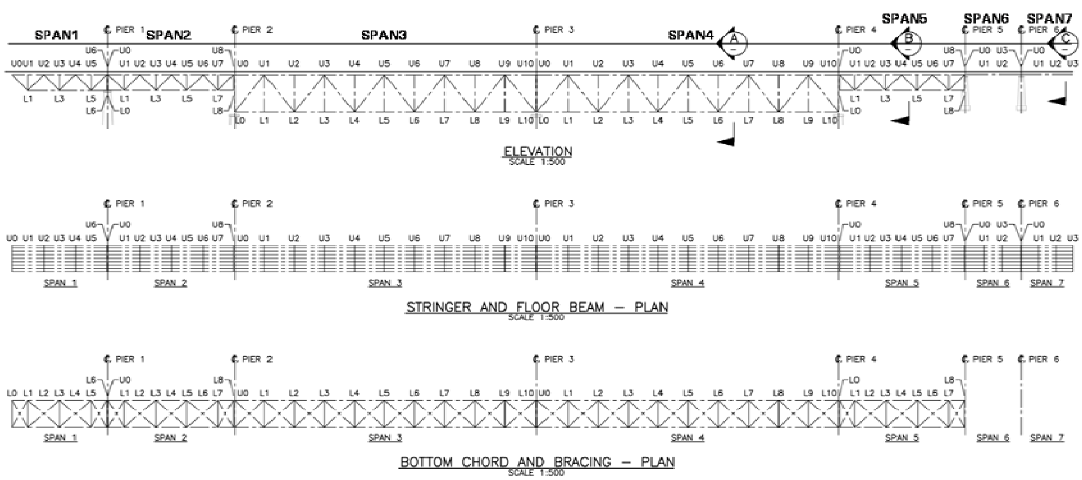


Figure 1: Old Spences Bridge – Elevation and Plan

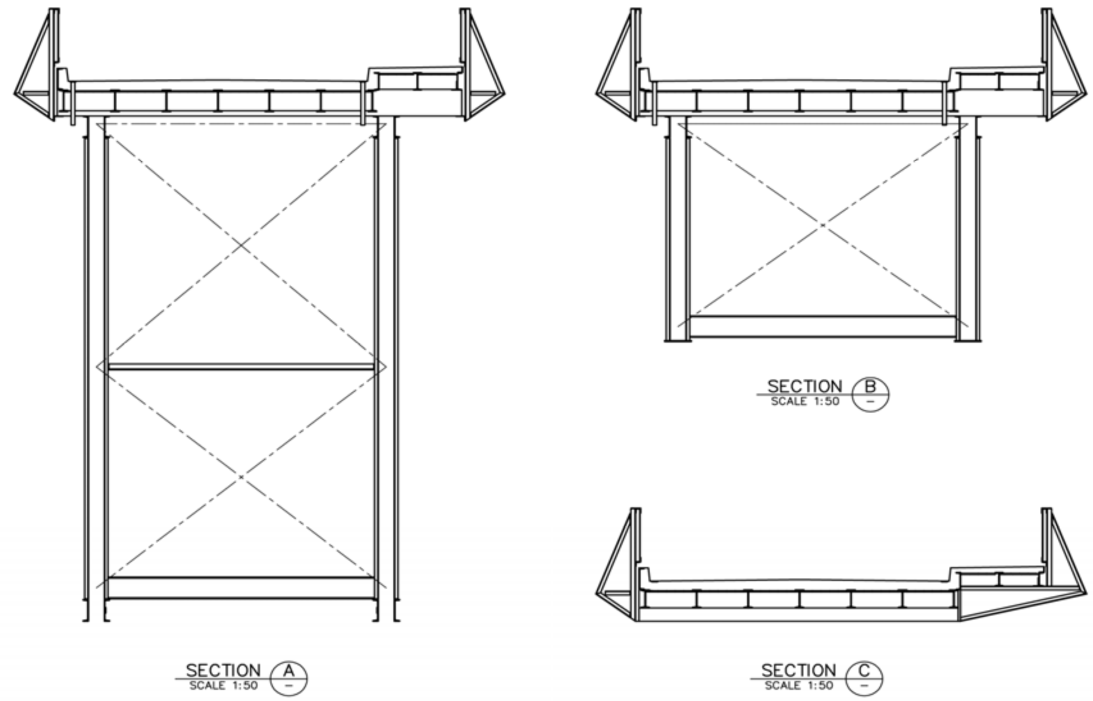


Figure 2: Old Spences Bridge – Typical Cross Sections

1.1 Past Studies

Annual inspections have been performed for many years and following the 2002 inspection the bridge was posted with a load limit. During the 2008 inspection, significant deterioration, corrosion and holes were identified in heavier structural components. Based on the 2008 visual inspection the bridge was closed to all vehicular traffic in 2009 in order to ensure public safety.

1.2 Current Assignment

Subsequent to closing the crossing, BC MoT retained B&T to carry out a detailed inspection and evaluation of the structure. As part of their assignment, B&T was also tasked with developing conceptual rehabilitation options and cost estimates to restore the bridge to a range of acceptable levels of reliability.

Recommended maintenance, rehabilitation and evaluation items based on observations made during the detailed inspection are presented in B&T Report No. 1884-RPT-SPE-001-0, “Old Spences Bridge No. 2411 – Inspection Report.”

This report summarizes the findings of the load evaluation of the bridge, makes recommendations regarding conceptual rehabilitation options, and summarizes high-level cost estimates for a variety of live load models and design life options.

2 Description of Bridge

The framing of the truss spans consists of top chords, top chord lateral bracing, verticals, diagonals, bottom chords, bottom chord lateral bracing and transverse sway bracing. The deck framing system consists of longitudinal stringers supported on transverse floorbeams, which bear on the top chord of the truss spans.

Each girder span consists of longitudinal stringers supported on two transverse floorbeams, which frame into two longitudinal edge girders. The edge girders are supported on concrete piers and abutments.

The bridge has been assembled using rivets although areas in which repairs have been made use high strength bolts.

The main bridge components are identified in Figure 3 to Figure 6, and are described in more detail in the subsections that follow.

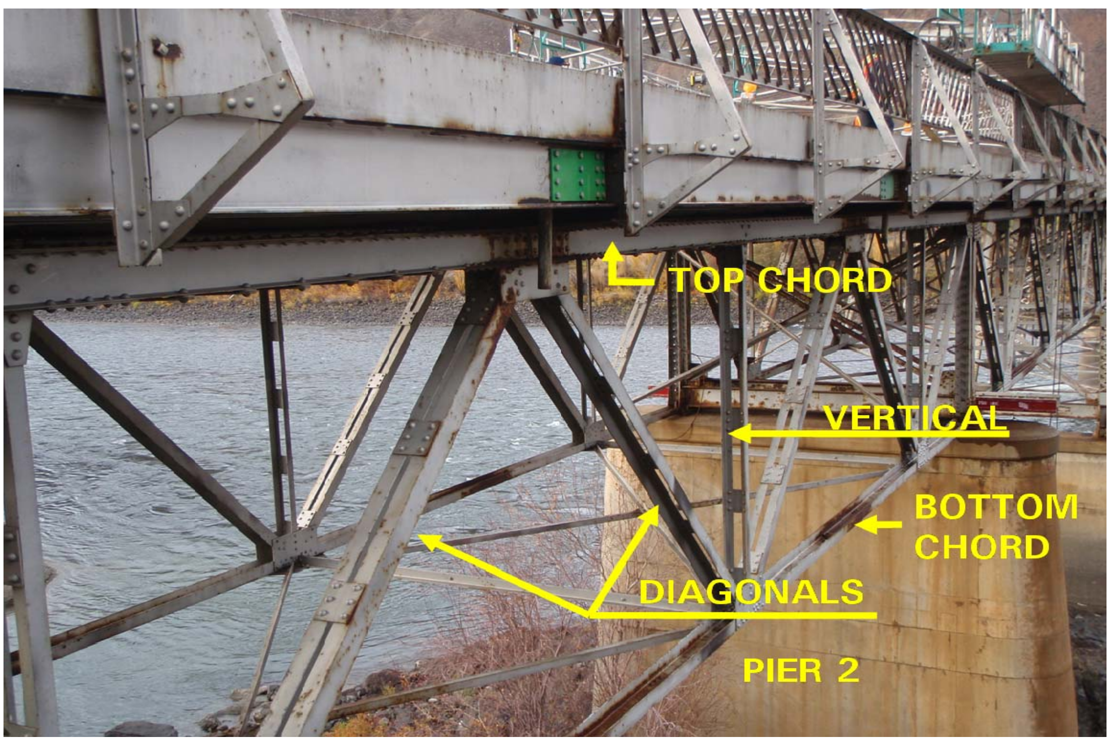


Figure 3: View of Typical Truss Span Showing Vertical Load Carrying Members

Figure 4: View of Typical Truss Span Showing Lateral Load Carrying Members

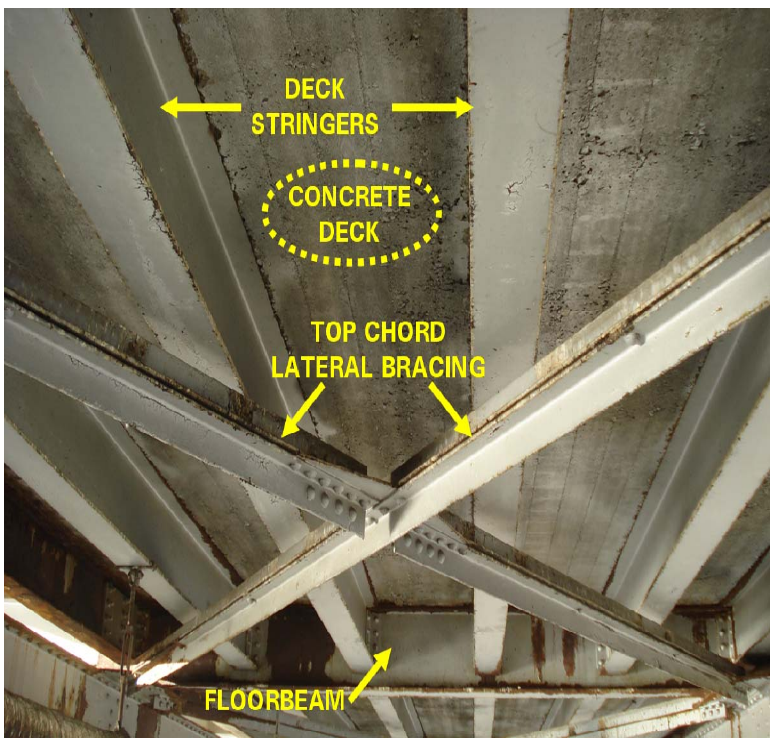


Figure 5: View of Typical Floor System in Truss Spans

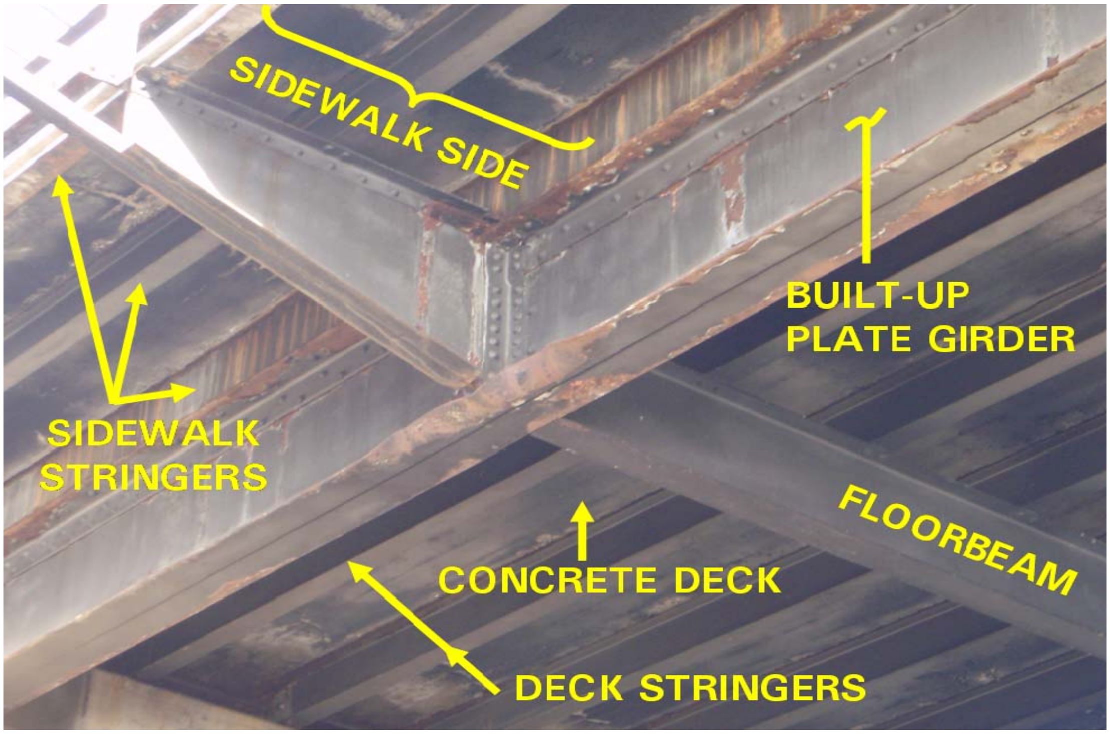


Figure 6: View of Typical Floor System in Girder Spans

2.1 Top Chord

The top chords of the truss spans are formed from back-to-back rolled channels that are connected along the top flange using a combination of batten plates and continuous cover plates. Along the bottom flange, the channels are connected using lacing bars. In spans 1, 2 and 5 the channels are 203 mm (8”) deep while in Spans 3 and 4 they are 380 mm (15”) deep.

2.2 Bottom Chord

Unlike the top chords, the type of members making up the bottom chords differ between the longer and shorter spans. In the longer spans, Spans 3 and 4, the bottom chord members are two back-to-back 380 mm (15”) deep channels connected by batten plates along the top and bottom flanges. However, in the shorter spans, Spans 1, 2 and 5, the bottom chords are formed by pairs of steel angles oriented toe-to-toe with the vertical leg extending upwards. The angles are connected with batten plates at approximately quarter points along their length.

2.3 Verticals

The vertical members throughout all of the truss spans are either formed from pairs of steel angles or pairs of steel channels. In the shorter spans, pairs of angles are used exclusively while steel channels are used in the longer spans where member demands are larger.

2.4 Diagonals

The diagonal members in the truss spans are similar to the vertical members with pairs of steel angles used in the shorter spans and pairs of steel channels used in the longer spans. However, the tension diagonals in Spans 3 and 4 are formed from four angles as opposed to the pair of angles used in the shorter spans. The four angles are arranged in a box pattern connected at intermediate points with batten plates. Batten plates are also used to provide intermediate connections between members.

2.5 Bottom Chord Lateral Bracing

The bottom chord lateral bracing in all of the truss spans comprises single steel angles as cross-bracing and pairs of angles as transverse struts. The pairs of angles are oriented back-to-back with vertical legs oriented upwards. At the bearing locations the transverse strut is a rolled I-shape girder in place of the pairs of angles. This girder serves as a jacking beam for bearing replacement and may provide a means of balancing loads between the bearings.

The cross-bracing members frame into gusset plates that are riveted to the underside of the bottom flange of the bottom chord in the case of the shorter spans, and to the top flange of the bottom chord in the case of the longer spans. A gusset plate is also located at the intersection of the two cross brace angles to provide a mid-length connection.

2.6 Top Chord Lateral Bracing

Similar to the bottom chord lateral bracing, the top chord lateral bracing is formed with single angles as cross-bracing members. Unlike the bottom lateral bracing however there are no transverse struts. These struts are replaced with the floorbeams that support the concrete deck.

The cross-bracing members are connected to gusset plates at each end of the member. These gusset plates are located between the top chord flange and the bottom flange of the floorbeams. A gusset plate is also located at the intersection of the two cross brace angles to provide a mid-length connection.

2.7 Sway Bracing

Sway bracing is provided between the east and west trusses at end points and intermediate points. The framing of the bracing is either single or double angles connected at their intersection point and at their endpoints to the east and west trusses. In Spans 3 and 4, the sway bracing is located at Panel Points 0, 2, 4, 6, 8 and 10. There is also a set of inclined sway bracing in the end bays of the truss where the top chord frames into the bearing point at the pier (eg. Panel Points L0 to U1). In the shorter spans, the sway bracing is oriented on a slope and is connected to the truss diagonals. In Span 1, sway bracing is located between Panel Points 0 and 1 and between Panel Points 5 and 6. In Spans 2 and 5, sway bracing is located between Panel Points 0 - 1, 2 - 3, 5 - 6 and 7 - 8.

2.8 Deck Components

A 150 mm (6”) concrete deck supported on longitudinal stringers, which are in turn supported on transverse floorbeams, makes up the deck system. The concrete deck is believed to be the original cast-in-place bridge deck. It appears that the deck was cast as individual panels between adjacent floorbeams resulting in joints in the concrete at each floorbeam location. The design drawings show a single mat with two layers of reinforcing located 37 mm (1½”) from the underside of the deck.

There is a 1220 mm (4 ft.) wide sidewalk on the west side of the bridge that extends beyond the west truss. This sidewalk is supported on three longitudinal stringers that are also connected to the transverse floorbeams.

2.9 Girder Spans

The two girder spans, Spans 6 and 7, are located at the north end of the bridge and measure 12.2 and 11.3 m (40 and 37 ft.), respectively. The south span, Span 6, crosses over an active CN Rail line containing two rail tracks. Both girder spans have the same framing arrangement with two 710 mm (28”) deep built-up plate girders supporting the spans. The plate girders are constructed with four angles riveted to a web plate. Each span has five longitudinal deck stringers that are continuous along the span. The stringers have bearing plates at each end where they rest on concrete pedestals. Intermediate support is provided at the third points where the stringers bear on transverse floorbeams. The floorbeams are connected to the edge girder with a web to web connection. Both the stringers and the floorbeams are rolled I-shaped sections.

3 Evaluation Criteria

3.1 General Requirements

The load capacity evaluation was conducted using CAN/CSA-S6-06 Canadian Highway Bridge Design Code (CHBDC). Relevant sections of the BC MoT Supplement to Section 14 of CHBDC, dated 2009 August, were incorporated as appropriate.

The load evaluation was based on the following information:

- Original terms of reference, contained in BC MoT's request for work plan, dated 2009 September 17;
- Bridge design drawings, shop drawings, BMIS inventory details and condition inspection reports provided by BC MoT;
- Information on changes from original construction provided by BC MoT, including an insulated water line and cable TV duct;
- 2003 detailed inspection and load evaluation reports prepared by Watson Engineering and provided by BC MoT;
- B&T workplan, dated 2009 October 6, prepared in response to the BC MoT terms of reference;
- Finding from B&T's detailed inspection, performed from 2009 October 19 to 27, and summarized in B&T Report No. 1884-RPT-SPE-001-0; and
- Continued correspondence with BC MoT to further refine the loading criteria.

The load capacity evaluation has been carried out to assess the vertical load carrying capacity of the bridge at the ultimate limit state only.

Effects from vertical loads such as dead, live and snow have been considered. In addition, the inspection has identified that sliding bearings are likely seized and piers have cracks. Therefore, thermally induced loads were considered for the substructure.

Vertical load carrying members, including their connections, that have been included in this evaluation include:

i. Concrete Deck;

ii. Deck stringers under the roadway;

iii. Floorbeams;

iv. Sidewalk components (sidewalk stringers and brackets);

v. Truss chords, diagonals, verticals and gusset plates;

vi. Truss Bearings;

vii. Girders; and

viii. Concrete piers.

Some members have not been included in the load evaluation. However, these members have been inspected and where significant section loss was observed, rehabilitation work may be recommended and the associated costs will be included in the total rehabilitation cost estimate. Members that have not been included in the load capacity evaluation are as follows:

- Lateral plan bracing and lateral cross section (sway) bracing. These members resist lateral loads and do not significantly influence the vertical load carrying capacity of the bridge in terms of promoting load sharing or providing bracing to compression members; and
- Concrete abutments.

3.2 Critical Members and Sections

3.2.1 Concrete Deck

The concrete deck is designed to span transversely between the deck stringers. The spacing of the stringers in the truss spans is slightly greater than in the girder spans (2’-9” versus 2’-6½”). Therefore, only the deck in the truss spans is evaluated.

The inspection findings highlight significant rust jacking at the floorbeam top flange, causing the deck to lift off the stringers and essentially span longitudinally between floorbeams. It is reasonable to assume that the load carrying capacity of the deck at the ultimate limit state can still be evaluated on the basis that the deck would eventually deflect down to a point where it touches the stringers and spans transversely. However, the cracking that may result in the deck as it deflects could reduce the service life of the deck. Therefore, to get an estimate of the initial bending behaviour of the concrete deck, it is also evaluated as a member spanning longitudinally between floorbeams. In this case, the deck is evaluated for the largest floorbeam spacing, which occurs in the truss spans.

It should be noted that the evaluation of the deck as intended by the original design (spanning transversely between stringers) is covered by CHBDC 14.14.1.3. However, the deck does not meet the requirements outlined in CHBDC 14.14.1.3.1 which references the empirical design method in CHBDC 8.18.4. Therefore the deck capacity will be evaluated for punching shear per CHBDC 14.14.1.3.2 and 14.14.1.3.3, and the live load capacity factor is computed for ultimate limit states per CHBDC 14.15.2.2.1.

3.2.2 Deck Stringers

Evaluation of deck stringers under the roadway is broken down as follows:

- Deck stringers in the truss spans are simply supported. All seven stringers in the cross-section have similar tributary widths for dead load demands and the code distribution factors for live load produces essentially the same live load demand in all the stringers. Therefore, only one stringer in the cross section is evaluated. Furthermore, the stringers spans vary slightly from 21’-6” to 22’-6”, and therefore only the longest span is evaluated; and
- Deck stringers in the girder spans are continuous over the floorbeams. All five stringers in the cross section have similar tributary widths for dead load demands and the code distribution factors for live load produces essentially the same live load demand in all the stringers. Therefore, only one stringer in the cross section is evaluated. Furthermore, the stringers spans vary slightly from 12’-3” to 13’-3”, and therefore only the longest span is evaluated.

Sidewalk stringers are evaluated as part of the sidewalk components, as described in Section 3.2.4.

3.2.3 Floorbeams

Floorbeams in the truss spans are all the same size. The floorbeam with the largest adjacent stringer span is evaluated.

Floorbeams in the girder spans are all the same size. The floorbeam with the largest adjacent stringer span is evaluated.

3.2.4 Sidewalk Components

Evaluation of the sidewalk components is broken down as follows:

- Sidewalk stringers along the truss spans are simply supported over the floorbeams. The middle beam is evaluated. The channel is not evaluated because it does not significantly influence the capacity of the sidewalk. Furthermore, the channel is located directly above the exterior deck stringer which is assumed to carry all the tributary loads. Therefore, even though the detailed inspection identified section loss in the web of the channel where it bears on the floorbeam, the structural consequence is minimal because the deck stringer is relied upon to carry the vertical load;
- Sidewalk stringers along the girder spans are continuous over the sidewalk brackets in the girder spans. The middle beam is evaluated based on the same rationale described in the previous bullet point;
- Sidewalk brackets in the girder spans have results reported for the critically loaded bracket that receives the largest loads delivered from adjacent stringer spans; and
- The sidewalk in the truss spans is supported by the floorbeams. The portion of the floorbeam under the sidewalk was initially not intended to be load rated, because the loads on this portion of the floorbeam are small compared to the portion of the floorbeam under the deck stringers. However, the detailed inspection identified significant section loss in the top flange of the floorbeam over the truss top chord where the floorbeam cantilevers out to support the sidewalk. Therefore, the negative bending capacity of the floorbeam will be evaluated. Section loss in the web was not observed to be nearly as severe and therefore shear and compression in the web were not evaluated.

3.2.5 Truss System

There are three different truss span lengths on the bridge:

- One 68’-9” span (span 1);
- Two identical 90’-9” spans (spans 2 and 5); and
- Two identical 216’-4 $^1/_2$” spans (spans 3 and 4).

The truss chords, diagonals and verticals have results reported for each member of the truss. Table 3 summarizes the distribution of loads between the upstream and downstream trusses, considering the cross-section geometry of the bridge and the eccentricity of the trusses with respect to various loads. The lateral load sharing between trusses and girders is expected to be minimal. Therefore, the truss and girder demands are based on simple lateral distribution assumptions for dead and live loads.

Table 3: Distribution of Loads to Upstream and Downstream Trusses

	Distribution Factor		
Load	**Downstream Truss**	**Upstream Truss**	**Used in Evaluation**
Dead Load	**51%**	49%	51%
Vehicular Live Load (can be shifting laterally, therefore the sum > 100%)	62%	**78%**	78**%**
Snow Load (can be on roadway and sidewalk)	**57%**	43%	57%

Since the upstream and downstream trusses have identical member sizes, the maximum demand is reported for the most heavily loaded truss, as shown in bold in Table 3. The conservatism in this approach is likely small in comparison to uncertainties associated with far more influential factors such as the extent and rate of corrosion.

3.2.6 Truss Bearings

The truss bearings for the 68'-9" spans and one end of the 90'-9" span consist of gusset plates riveted to angles that bear on the shoe plates. The gusset plates, rivets and angles will be evaluated for their ability to resist vertical loads.

At the other end of the 90'-9" span, the truss is connected into the vertical member of the 216'-4½" span. This connection and the additional compression in the vertical member are evaluated as part of the truss system.

The truss bearings for the 216'-4½" spans consist of a pin supported by vertical pin plates riveted to angles that bear on the shoe plates. The pin, pin plates, rivets and angles will be evaluated for their ability to resist vertical loads.

Due to the fact that that inspection identified that the truss bearings appear to be seized, the bearings may be susceptible to undesirable longitudinal shear demands due to temperature loading. Therefore, the anchor bolts in the truss bearings will be evaluated for these shears.

3.2.7 Girders

The two girder spans are 40’ and 37’, and the girder sizes are the same for the two spans. Therefore, results are reported for the longer span. If the live load capacity factors (LLCFs) are slightly less than 1.0, the shorter span may be revisited to assess whether the LLCFs are greater than 1.0.

The distribution of loads to the upstream and downstream girders is the same as that assumed for the trusses, refer back to Table 3.

3.2.8 Concrete Piers

Each concrete pier is evaluated for its ability to resist axial loads and moments resulting from vertical loads.

Longitudinal bending moments resulting from shear demands in the seized bearings are included in the evaluation.

3.3 Loads

3.3.1 Dead Loads

Dead load, D1, as defined in CHBDC 14.8.2.1(a), includes the weight of factory-produced components. In this evaluation, this includes all structural steel components such as trusses, bracing, stringers and floorbeams. Connections, battens and lacing are also included in this category.

Dead load, D2, as defined in CHBDC 14.8.2.1(b), includes the weight of cast-in-place concrete decks and non-structural components. In this evaluation, this includes the concrete deck, concrete sidewalks, railings and utilities such as an insulated waterline and cable TV duct.

BC MoT has confirmed that there is no known overlay or resurfacing that has increased the thickness of the concrete deck since original construction. Therefore, the original deck thickness is used in this evaluation.

Furthermore, this evaluation considers only the current dead load condition, therefore this evaluation has no allowance for future overlay or increased deck thickness.

3.3.1.1 Dead Load Effects

For evaluating the truss and girders, the weight takeoff of the stringers, floorbeams, lateral bracing, trusses and girders included main elements such as angles, channels and beams. The bare steel weight of the main elements was then increased by 20% to account for the weight of additional elements such as connections, gussets, batten plates and lacing. The 20% allowance appears reasonable, given that the resulting steel weight was then compared to the weight takeoff on the original design drawings, and the results were within 3%.

3.3.2 Live Loads

CHBDC 14.9.4.1 indicates that the number of design lanes shall be determined in accordance with the current or intended use of the bridge. BC MoT has confirmed that for this evaluation, the intended use is one lane.

The highway is designated as Class C, meaning that uniformly distributed loads included in lane loads are 7 kN/m.

Five live load models were considered for the load capacity evaluation and are described in the following subsections.

3.3.2.1 CL1-625 Loading

CL1-625 loading. This is considered Normal traffic, Evaluation Level 1, consisting of a CL1-625 Truck or Lane Load. The loading is shown in CHBDC Figure 14.1, and the effects reported are the largest from:

- CL1-625 truck plus dynamic load allowance; or
- 80% of the CL1-625 truck plus 7 kN/m, with no dynamic load allowance.

3.3.2.2 25 Tonne Loading

This is considered alternative loading, consisting of a 25 tonne vehicle Truck or Lane Load. The 25 tonne vehicle specified by BC MoT is shown in Figure 7, and the effects reported are the largest from:

- 25 tonne truck plus dynamic load allowance; or
- 80% of the 25 tonne truck plus 7 kN/m, with no dynamic load allowance.

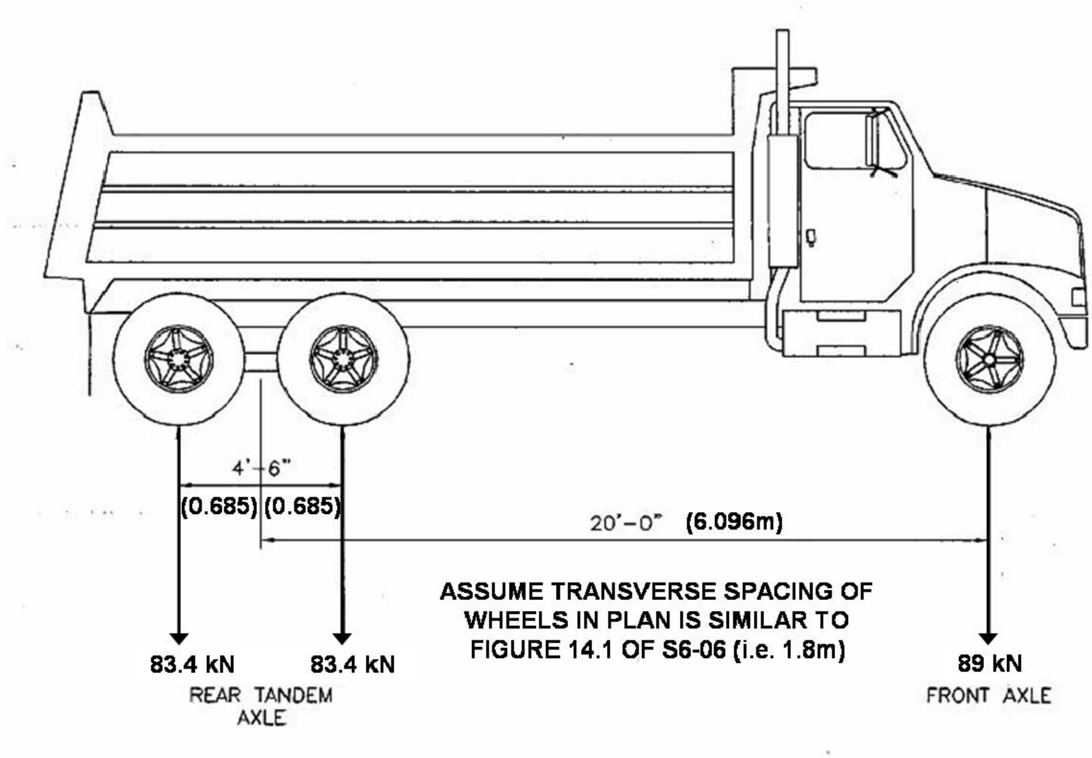


Figure 7: 25 Tonne Vehicle Axle Loads

3.3.2.3 5 Tonne Loading

This is considered alternative loading, consisting of a 5 tonne vehicle Truck or Lane Load. The 5 tonne vehicle specified by BC MoT is shown in Figure 8, and the effects reported are the largest from:

- 5 tonne truck plus dynamic load allowance; or
- 80% of the 5 tonne truck plus 7 kN/m, with no dynamic load allowance.

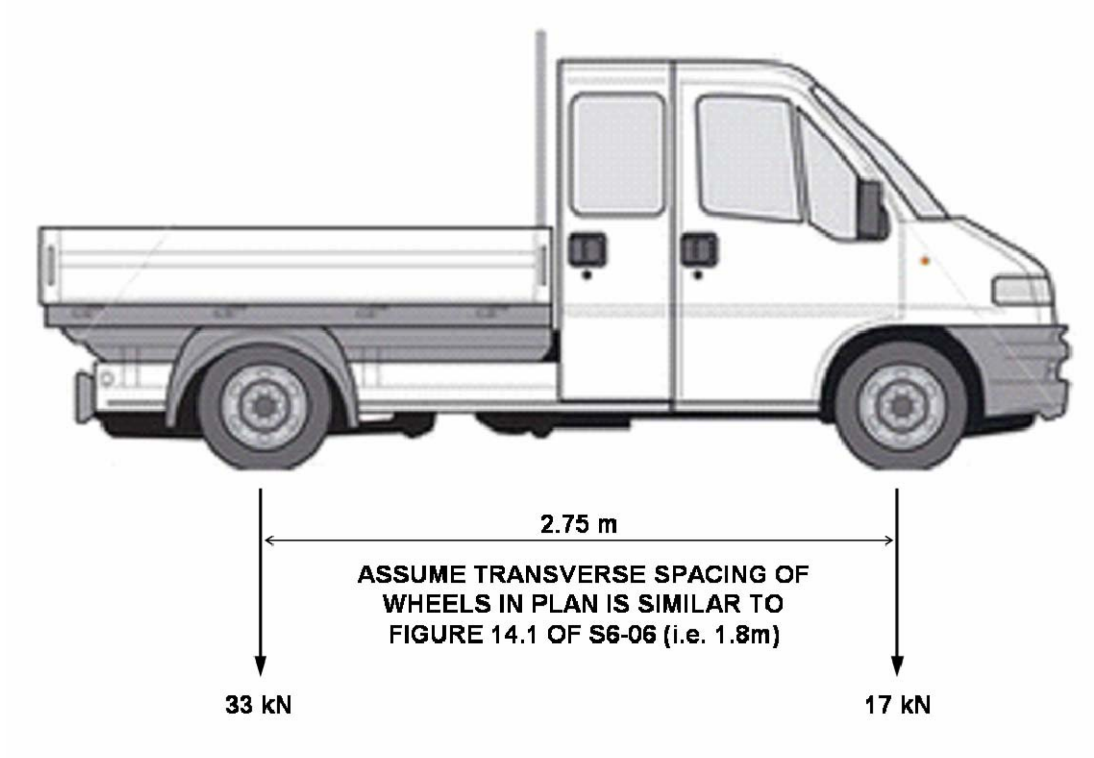


Figure 8: 5 Tonne Vehicle Axle Loads

3.3.2.4 Pedestrian Loading

The pedestrian load intensity specified in CHBDC was not applied to the full deck area (i.e., full width of the bridge). Applying the pedestrian load intensity specified in CHBDC to the full bridge width is deemed to be excessive and the resulting demands in the main members are significantly higher than the demands from the 5 and 25 tonne vehicle loadings. The pedestrian loading specified in CHBDC is associated with a crowd of spectators standing close together on a walkway area and is too severe for this bridge given its location.

Therefore, in this evaluation two pedestrian load cases have been established in order to satisfy the intent of the code, while at the same time being more representative of local conditions. The load cases are as follows:

Case 1: Pedestrian load on sidewalk only

This case is applicable to a scenario where the bridge is only open to pedestrian loads, or the bridge is open to both vehicles and pedestrians.

Pedestrian loading is applied to the sidewalk width only, with an intensity of up to 4 kPa as defined in CHBDC 3.8.9. The pedestrian loading is used only as a check of the sidewalk components, and is not applicable to the trusses, girders and piers. The presence of pedestrian loading applied to the sidewalk coincident with traffic loading is not considered. This is all consistent with CHBDC 14.9.5.1 and the commentary (C14.9.5.1).

The question may arise as to the capacity of the global elements to resist pedestrian loads on the sidewalk only. Section 3.3.3 describes the snow load for which the bridge is evaluated, and it should be noted that snow pattern 1, as shown in Figure 9, could be present when the bridge is closed. The snow load pressure associated with pattern 1 is 1.8 kPa over the entire bridge width, resulting in a factored reaction to one truss that is essentially equivalent to that of the factored pedestrian loading on the sidewalk only. Therefore, provided the bridge is shown to be adequate for the snow pattern 1 loading, then it follows that the truss and girders are also capable of resisting the maximum code specified pedestrian loading of up to 4 kPa applied to the sidewalk only.

<u>Case 2: Pedestrian load applied anywhere on the bridge, but load is limited to a maximum of fifty (50) pedestrians:</u>

This case is applicable to a scenario where the bridge is only open to pedestrian loads.

BC MoT has provided input as to a load intensity that it believes is representative of local conditions. The rationale is that a bus load of visitors might visit the area and walk on the bridge. A bus may carry approximately 50 people, each weighing 1 kN (225 pounds). Working backwards from the code specified maximum pedestrian load of 4 kPa, one can calculate that the intensity and loaded area is 4 kPa over an area of 12.5 m^2. This load case is applied anywhere on the bridge deck in order to assess all floor system components in a situation where it serves as a pedestrian-only bridge. If the bridge is opened as a pedestrian-only bridge, BC MoT must post signage limiting the pedestrian load to a maximum of fifty (50) people on the bridge at any given time.

The results of the evaluation present the most critical of the two pedestrian load cases described preceding.

3.3.2.5 Wheels on the Sidewalk

CHBDC 3.8.4.4 specifies that vehicular wheel loads on sidewalks should be considered using 70% of the wheel load. The code is not clear on whether this is applicable to bridge evaluations, but the possibility of accidental loads on the sidewalk exists. Therefore, the evaluation includes verification of sidewalk components for 70% of the wheel loads for CL1-625, 25 tonne and 5 tonne vehicles.

3.3.2.6 Dynamic Load Allowance

For the CL1-625 truck model, a dynamic load allowance is applied in accordance with CHBDC 14.9.1.7 and 3.8.4.5.

For alternative loading, CHBDC 14.9.1.6, 14.9.3 and 3.8.4.5 are not clear on dynamic load allowance for the 25 tonne and 5 tonne truck models when more than one axle are used. This evaluation assumes a dynamic load allowance of 0.4 when only one axle of the vehicle is used, and a dynamic load allowance of 0.3 when two or more axles are used.

For lane load models, no dynamic load allowance is applied to the reduced truck or uniformly distributed load.

3.3.2.7 Lateral Distribution of Live Loads to Stringers

For an axle load effect in the deck stringers, the simplified method in CHBDC Section 5 results in lateral distribution factors of 0.25 and 0.30 for bending and shear, respectively. This was confirmed by a simple grillage model representing the deck, stringers and floorbeams. Therefore, live load bending and shear demands in the deck stringers were computed based on a lateral distribution factor of 0.30. This represents a modestly conservative distribution for bending when compared to CHBDC.

Live loads distribution to stringers due to wheels on the sidewalk is based on assuming the full wheel is carried entirely by one stringer, with one wheel load directly above the stringer.

3.3.3 Snow Loads

CHBDC 3.1 indicates that snow load is generally not considered in the design of bridges because considerable snow load will cause a compensating reduction in traffic load. However, CHBDC 14.9.5.2 indicates that if significant loading on sidewalks is expected, it shall be considered in the evaluation. Furthermore, the possibility that the current status of a total bridge closure be continued raises the possibility that the bridge should be evaluated under its own weight with only snow present.

The National Building Code of Canada (2005), Section 4.1.6.2, was used as a reference to determine the magnitude of the snow load. Snow loads are estimated based on a 1-in-50-year ground snow load (S_s) and associated rain load (S_r) for surrounding areas such as Kamloops, Cache Creek, Ashcroft and Merritt. The resulting specified snow load, S, is 1.8 kPa. A noticeably higher snow load for the area of Lytton has not been included, as the snow load of 1.8 kPa is already considerably larger than snow loads derived from Western Bridge Co. original drawings E1 to E3.

Several snow load patterns, as shown in Figure 9, are assumed in the evaluation. In addition to assuming that the entire bridge width is uniformly loaded, concentrated loads are also considered resulting from snow ploughs providing enough width for traffic to pass through. The concentrated loads are computed assuming the overall weight from the uniform load is still present, but is piled high in the area adjacent to the clearing.

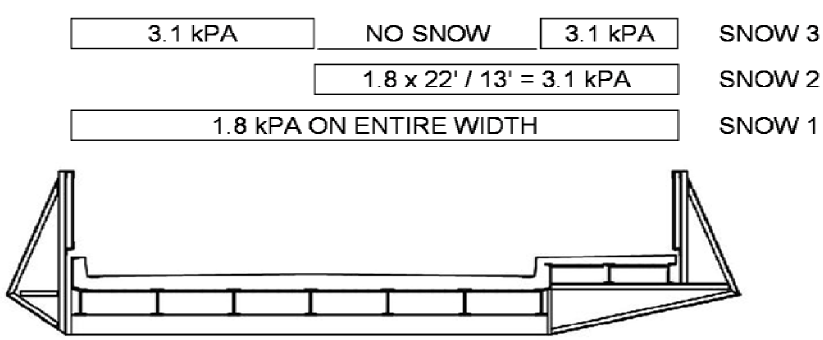


Figure 9: Snow Loading Considered in Evaluation

3.3.4 Temperature Loads

Thermally induced loads are included for the evaluation of the truss anchor bolts and concrete piers because previous inspections have identified that the bearings have seized and there are cracks in the concrete piers.

CHBDC 14.9.5.4 specifies that CHBDC 3.9.4 be used for computing temperature effects. Parameters assumed in the evaluation are as follows:

- Maximum mean daily temperature = 30ºC;
- Minimum mean daily temperature = -26ºC;
- Superstructure Type = B; and
- Effective construction temperature = 15ºC (assumed).

Table 3.7 and Figure 3.5 of CHBDC modify the above as follows to obtain the thermal ranges applied to the bridge:

- $T_{hot} = T_{max} – T_{effective\ construction}$ = (30+20-7) – 15 = +28ºC
- $T_{cold} = T_{min} – T_{effective\ construction}$ = (-26-5+10) – 15 = -36ºC

3.4 Target Reliability Index

The target reliability index, β, obtained from CHBDC Table 14.5 depends on three factors:

i. System Behaviour: whether or not failure of the member will lead to complete failure of the structure (refer to CHBDC 14.12.2);

ii. Element Behaviour: whether or not the failure of the member is sudden and whether or not the member has post-failure capacity (refer to CHBDC 14.12.3); and

iii. Inspection Level: how well the condition of the member is known (refer to CHBDC 14.12.4). The inspection level for all members is taken as INSP3 due to the detailed inspection that was performed as part of this project.

The system and element behaviours associated with members and behaviours of interest are summarized in Table 4. Note that not all of the behaviours listed in the table are applicable to the entire bridge. For instance, the stringer shear connection to floorbeam is only applicable to deck stringers in the truss spans. This behaviour is not applicable to sidewalk stringers and deck stringers in the girder spans because in those areas the stringers sit on top of the floorbeams and do not have a shear connection.

Table 4: Target Reliability Index for Inspection Level INSP3

Member Type	Behaviour of Interest, when applicable	System Behaviour Category	Element Behaviour Category	Target Reliability Index, β, (CHBDC Table 14.5)
Concrete Deck	Punching Shear & Positive Bending	S3	E3	2.50
Stringers	Positive bending near midspan	S3	E3	2.50
	Negative bending over floorbeam (including consideration of moment-shear interaction)		E3	2.50
	Shear in web		E3	2.50
	Shear connection to floorbeam		E1	3.25
	Compression in web over floorbeam		E1	3.25
Floorbeams	Compression in web below stringer	S2	E1	3.50
	Positive bending near midspan		E3	2.75
	Shear in web		E3	2.75
	Shear connection to girder		E1	3.50
	Compression in web over truss chord		E1	3.50
Sidewalk brackets in girder spans	Compression in web below stringer	S2	E1	3.50
	Negative bending (including consideration of moment-shear interaction)		E3	2.75
	Shear in web		E3	2.75
	Shear connection to girder		E1	3.50
	Connection of flanges to girder		E1	3.50
Truss system (chords, diagonals and verticals)	Axial compression in member	S1	E1	3.75
	Axial tension in member (gross section)		E3	3.00
	Axial tension in member (net section)		E1	3.75
	Connections of truss members (including gussets)		E1	3.75
Truss bearings	Shear and moment interaction in pin	S1	E3	3.00
	All other connecting parts within the bearing		E1	3.75
	Anchor Bolt Shear		E2	3.25
Girders	Positive bending near midspan	S1	E3	3.00
	Shear in web		E3	3.00
	Interface shear between flange & web		E3	3.00
	Compression in web over bearing		E1	3.75
	Angle and rivets over the bearing		E1	3.75
Concrete Piers	Axial and bending interaction in piers (plain concrete, unreinforced sections)	S1	E1	3.75

3.5 Load Factors and Combinations

3.5.1 Load Factors for Dead and Live Load Only

The load factors for dead load and live load are determined from the target reliability indices, and vary for the different members on the bridge. Each dead and live load code reference is summarized in Table 5.

Table 5: Dead and Live Load Factors

Load Effect	Load Factor	Reference
Dead Load, D1	α_{D1max}	CHBDC Table 14.7
	α_{D1min}	CHBDC Table 3.2
Dead Load, D2	α_{D2max}	CHBDC Table 14.7
	α_{D2min}	CHBDC Table 3.2
Live Load, CL1-625	$\alpha_{L,CL1\text{-}625}$	CHBDC Table 14.8
Live Load, 25 tonne	$\alpha_{L,25t}$	CHBDC Table 14.9
Live Load, 5 tonne	$\alpha_{L,5t}$	CHBDC Table 14.9
Live Load, Pedestrian	$\alpha_{L,ped}$	CHBDC Table 14.8*

*CHBDC 14.9.5.1 is not clear on the load factor that should be used when pedestrian loading is considered. However, since the magnitude of pedestrian loading used in the evaluation is quite conservative, the load factors for pedestrian loading are taken from CHBDC Table 14.8, rather than Table 14.9.

3.5.2 Ultimate Limit States Combinations

The load factors associated with the ultimate limit states (ULS) load combinations that include snow and temperature are summarized in Table 6. The load factor for snow is taken from NBCC 2005, Table 4.1.3.2 for Case 3. The load factor for temperature is taken from CHBDC Table 3.1 for CHBDC Combination ULS 2, and is only included for evaluation of the piers and the truss bearings.

Table 6: ULS Combinations and Load Factors

ULS combination	Dead Load D1	Dead Load D2	Live Load L	Snow Load S	Thermal Load K
ULS1a (bridge open)	α_{D1}max or α_{D1}min	α_{D2}max or α_{D2}min	$\alpha_{L(CL1\text{-}625)}$ or $\alpha_{L(25t,\ 5t)}$	1.5 (on snow pattern 2 or 3)	1.15
ULS1b (bridge open)	α_{D1}max or α_{D1}min	α_{D2}max or α_{D2}min	$\alpha_{L(CL1\text{-}625)}$ or $\alpha_{L(25t,\ 5t)}$	0	1.15
ULS1c (bridge open)	α_{D1}max or α_{D1}min	α_{D2}max or α_{D2}min	$\alpha_{L(ped)}$	0	1.15
ULS1d (bridge open or closed)	α_{D1}max or α_{D1}min	α_{D2}max or α_{D2}min	0	1.5 (on snow pattern 1)	1.15
ULS9 (bridge open or closed)	1.35	1.35	0	0	0

ULS1a and ULS1b are combinations for vehicular traffic. ULS1a is a conservative combination when the maximum load factors for snow and temperature are considered to occur at the same time as maximum live load. Therefore, ULS1b has been developed without snow loads in order to provide reasonable insight regarding the influence of snow loads and the potential need for snow clearing efforts.

ULS1c is a combination for pedestrian loading. Snow loads are not included in the combination because it is highly unlikely that maximum pedestrian loading would occur simultaneously with maximum snow load. Furthermore, snow removal guidelines are developed later in this report as a means of reducing the magnitude of simultaneous loads.

ULS1d is a combination for snow loading only. In the event that the evaluation concludes that the bridge cannot be open to vehicular traffic, then the ability of the bridge to carry snow loads could be of particular interest.

ULS9 is a combination for dead load only. In the event that the evaluation concludes that the bridge should not be open to vehicular traffic, then the ability of the bridge to carry its own self-weight could be of particular interest.

3.5.3 Reporting of Capacity Factors

For ULS1a, ULS1b and ULS1c, a live load capacity factor (LLCF) is computed as described in CHBDC 14.15.2.1 or 14.15.4, while holding all other factored loads constant (dead, snow and temperature). A LLCF greater than 1.0 indicates that the bridge has sufficient capacity to resist the ULS combination of interest.

For ULS1d and ULS9, an LLCF cannot be computed because there is no live load in the combination. Therefore, a capacity to demand ratio (C/D) is presented in the results. A C/D greater than 1.0 indicates that the bridge has sufficient capacity to resist the ULS of interest. The C/D is similar to an LLCF, except that the C/D is an indicator as to how much reserve capacity there is for all loads to be scaled up, whereas the LLCF is an indicator as to how much reserve capacity there is for only the live load to be scaled up.

3.6 Evaluation of Resistances

3.6.1 Material Strengths

Material strengths assumed in the resistance calculations for the load evaluation are taken from Section 14 of CHBDC based on year of construction, because no material grades were specified on the drawings.

The bridge was designed in 1929, and this is used as the date of construction for the purposes of determining material strengths. Table 7 and Table 8 summarize the material strengths assumed in the evaluation.

Table 7: Material Strengths for Steel Superstructure

Element	Code Reference	Assumed in Evaluation	
		Yield Strength (Fy)	Tensile Strength (Fu)
Structural Steel	CHBDC Table 14.1	210 MPa	420 MPa
Rivets	CHBDC 14.7.4.6 (a)	n/a	320 MPa

Table 8: Material Strengths for Concrete Substructure

Material Strength	Code Reference	Assumed in Evaluation
Concrete Substructure 28-day compressive strength, f_c’	CHBDC 14.7.4.3	15 MPa
Concrete Deck 28-day compressive strength, f_c’	CHBDC 14.7.4.3	20 MPa
Reinforcing steel, yield strength, f_y	CHBDC Table 14.2	230 MPa

3.6.2 Resistance Adjustment Factors

As per CHBDC 14.14.2, factored resistances are multiplied by the appropriate Resistance Adjustment Factor, U, specified in CHBDC Table 14.15. Some behaviours are not clearly identified in CHBDC Table 14.15. Therefore, Table 9 summarizes some failure modes and the associated Resistance Adjustment Factors that are assumed in the evaluation.

Table 9: Resistance Adjustment Factors, U, for Behaviour Not Clearly Defined in CHBDC

Behaviour	Specific Comments	Resistance Adjustment Factor, U
Connection Capacities	Factored bearing resistance, Br, of structural steel in rivet connection. Br calculated per CHBDC 14.14.1.4.2(a). Note that Fu is always the structural steel, even if the rivet material is weaker than the structural steel.	1.20 (assumed), per CHBDC Table 14.15*
	Factored shear resistance, Vr, of rivet material in rivet connection. Vr calculated per CHBDC 14.14.1.4.2(b).	1.81 per CHBDC Table 14.15
	Factored shear and tension resistance for block failure, Tr, for locations such as web copes and truss member ends. Tr calculated per CHBDC 10.8.2(b) and (c).	1.18 per CHBDC Table 14.15
Beam webs	Web crippling and yielding resistance, Br, for webs in compression at supports. Br calculated per CHBDC 10.10.8.1(a) or (b).	1.00 (assumed)

* For bearing on structural steel, using U of 1.81 may prove to be unconservative, while U of 1.00 may prove to be overly conservative. Therefore, the U of 1.20 is selected as one would not expect significant difference in the bearing capacity of riveted and bolted connections.

4 Evaluation Results

Evaluation results in this section are presented in two parts:

- Part 1 of the evaluation results is based on assuming uncorroded capacities and the original design intention for particular bearing to be sliding; and
- Part 2 of the evaluation is based on revisiting key areas of the structure where the detailed inspection identified substantial corrosion, damage, or change in conditions such as seized bearings that could affect the vertical load carrying capacity of the bridge.

4.1 LLCFs for Uncorroded Original Design Bearing Restraints

For the uncorroded original design articulation, evaluation results for the various live load models are summarized in Table 10. The results are shown for ULS1b and ULS1c (dead plus live load, with no snow load), as well as ULS1d (dead plus snow load).

The table does not summarize ULS9 (dead load only) because ULS1d is more severe, and all findings for ULS1d are acceptable.

Results for ULS1a (dead plus live plus snow load) are not summarized in the table or the body of this report due to the severity of the assumption that maximum snow load and live load occur at the same time. Recommendations regarding snow removal are summarized later in Section 4.4 in order to ensure that maximum live load and maximum snow load are not coincident.

The table makes reference to Appendix B, where comprehensive tabular output is summarized. The comprehensive output in Appendix B includes results for ULS1a.

Table 10: Summary of Governing LLCFs for Various Live Load Models - Uncorroded Capacities

Item and Appendix Table	Governing Behaviour	Minimum LLCF (ULS 1b and ULS 1c)				C/D
		CL1-625 Loading	25 Tonne Loading	5 Tonne Loading	Pedestrian Loading	ULS1d
Concrete Deck App. Table B1	Deck spanning transversely between stringers (as designed)	1.02	1.50	4.04	N/A	N/A
Deck Stringers App. Table B2	Positive bending (truss spans)	**0.86**	1.00	3.30	6.05	4.62
	Shear in web connection	1.79	2.19	8.22	14.79	11.35
Floorbeams App. Table B3	Positive bending (truss spans)	**0.77**	1.21	3.34	4.08	2.40
	Shear in web connection	1.25	1.92	4.84	7.89	4.00
Sidewalk App. Table B4	Positive bending in sidewalk stringer (truss spans)	See note 4	**0.49**	1.63	3.84	See note 3
Truss – span 1 App. Table B5	Compression in diagonal	**0.39**	**0.62**	1.51	See note 3	1.35
Truss – span 2 App. Table B5	Compression in diagonal	**0.37**	**0.57**	1.48	See note 3	1.50
Truss – span 3 App. Table B5	Compression in diagonal	**0.62**	1.07 See note 1	1.68 See note 1	See note 3	1.22 (note 1)
Truss Bearings App. Table B6	Generally at pier 2, shear in rivets	**0.75 See note 2**	1.12	1.60 See note 2	See note 3	1.18
Girders App. Table B7	Compression in web	**0.44**	**0.64**	2.04	See note 3	2.03
Concrete Piers App. Table B8	Tensile stress due to M-N interaction in pier 6	**0.84 See note 6**	1.18	3.65	See note 5	See note 5

Notes:
1. Compression in a gusset plate.
2. Span 2 above pier 1, bearing of base angle.
3. Values are not reported because other more severe loading within the summary indicated that the member was ok. Therefore a less severe loading would be ok.
4. CL1-625 wheel load on sidewalk not evaluated because of all the widespread severe overstress in the other elements, indicating CL1-625 upgrade not reasonable.
5. Not computed. Will have very large C/D and will not govern based on engineering judgement.
6. Bearing stress in concrete.

Although Table 10 gives key results for behaviours of interest for the floor system, the truss has numerous members that are difficult to visualize and summarize in a simple tabular format within the body of the report.

Therefore, a graphical summary of overstressed truss members in Spans 1 and 2 for the 25 tonne loading is shown in Figure 10. Span 3 is not shown for the 25 tonne loading because there are no overstresses.

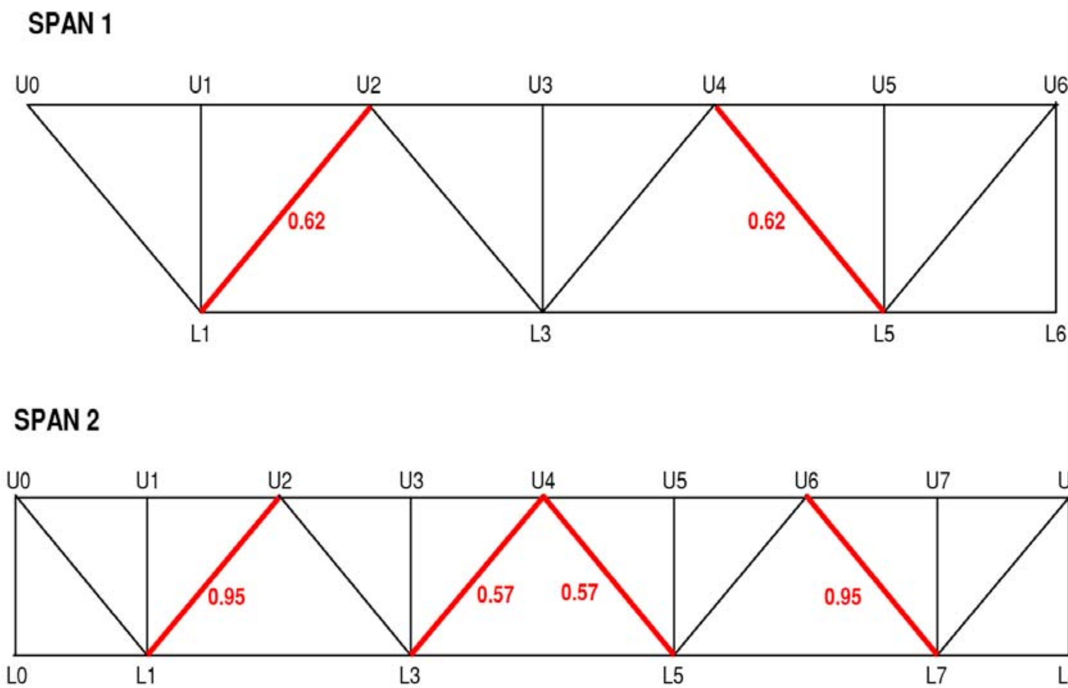


Figure 10: Overstressed LLCFs for 25 Tonne Loading - Uncorroded Capacities

A summary of overstressed truss members for the CL1-625 loading is not shown graphically due to the widespread overstress that extends into the chords and bearings.

A summary of overstressed truss members for the 5 tonne loading is not shown because there are no overstressed members.

4.2 LLCFs Including Effects of Section Loss Due to Corrosion and Seized Bearings

Based on the results of the detailed inspection, numerous components require additional consideration in terms of a reduced capacity due to section loss. The items in B&T's Inspection Report No. 1884-RPT-SPE-001-0 were identified as Evaluation Items, with reference numbers from E-1 to E-11.

The calculations for a reduced capacity due to section loss are handled in one of two ways:

- **Compute corrosion threshold to arrive at LLCF = 1.0**. The amount of section loss that can be tolerated is computed such that an LLCF of 1.0 is reached. This is best suited to observations that are applicable to several identical members in the structure (i.e. stringers); or
- **LLCF based on measured corrosion**. The reduced capacity is computed based on site measurements of the remaining section and the reduced LLCF is reported. This approach is best suited to an observation applying to a single point on the structure.

It is noted that due to the high-level nature of this load evaluation, the condition of each individual structural component will not be investigated in detail. Instead, the inspection findings and engineering judgement will be used to assign the extent of corrosion for a given component (e.g. stringer, floorbeam, etc.) and the remaining section to use in determining the capacities.

4.2.1 Concrete Deck (Evaluation item E-8)

The site observation regarding rust jacking on top of the floorbeam top flange has led to evaluating the concrete deck for potentially spanning longitudinally between floorbeams as the deck appears to have lifted off the stringers. The evaluation for this alternate arrangement of spanning longitudinally has resulted in the conclusion that the deck is in fact not able to carry its own weight spanning longitudinally between floorbeams, without even including the effects due to live load.

This indicates that the deck would likely experience substantial cracking as it deflects down to be supported on the stringers. This is more of a serviceability consideration than a structural strength issue, since once the deck sits back down on the stringers it behaves as originally designed.

It is also possible that while it appears that the deck is currently spanning between floorbeams, the deck may likely be supported on some intermediate points along the stringers. This could potentially explain why the deck appears to be able to span between floorbeams under its own weight.

It appears that a design life of 10 years for the concrete deck in its current condition is achievable for 5 tonne loading, with localized repairs of spalls to the soffit as identified in the inspection report. However, improved reliability for a 5 tonne loading and 10 year design life would be achieved if the gap between the deck and stringers is shimmed.

If the bridge is upgraded to the 25 tonne loading for a design life of up to 10 years, it is recommended that the gap between the deck and stringers be shimmed

It appears that a design life of 10 years for the concrete deck in its current condition is achievable for pedestrian-only loading. However, future detailed inspections may identify the need for localized repairs of spalls to the soffit.

If a design life beyond 10 years is desired for vehicular loads, it is recommended that the concrete deck be replaced in its entirety.

4.2.2 Deck Stringers (Evaluation Item E-9)

The deck stringers typically have moderate corrosion at midspan and severe corrosion in their webs at their ends where they connect to the floorbeams.

Corrosion thresholds have been computed on the basis of assuming a fixed amount of minor corrosion in less severely corroded portions of the stringer, and then computing how much corrosion can be tolerated in the remaining portion. For instance, as shown in Figure 11, because the condition at the end of a stringer is characterized by severe section loss in the web, a fixed amount of corrosion is assumed in the flanges (in this case, zero corrosion). Then the amount of corrosion that can be tolerated in the web is computed such that LLCF = 1.0. The results of the exercise are summarized in Table 11.

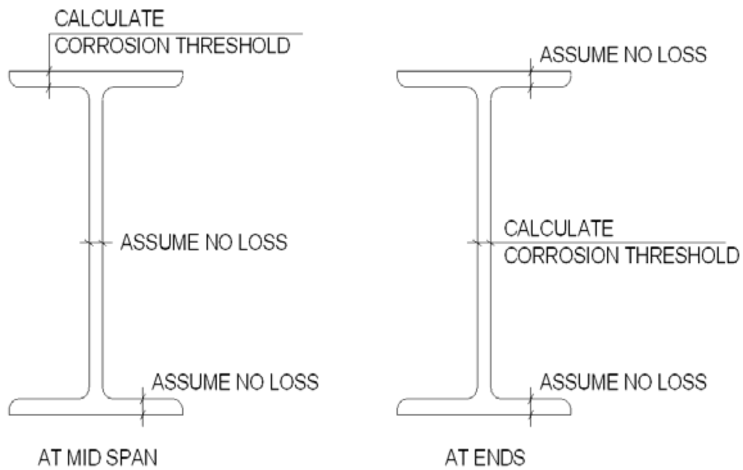


Figure 11: Stringer Section Loss and Corrosion Threshold Locations of Interest

The findings for the stringers suggest that they have barely enough positive bending strength in the truss spans for the 25 tonne loading, and cannot tolerate any section loss. However, this is based on a conservative live load lateral distribution factor of 0.30. Revisiting this location with the actual computed code simplified approach distribution factor of 0.254 produces the results summarized in Table 12. These results show that the stringers can accommodate some corrosion midspan for the 25 tonne loading, and that based on the extent of section loss/corrosion observed during the 2009 detailed inspection all of the stringers have sufficient capacity to withstand the 25 tonne loading.

Table 11: Tolerable Corrosion for Deck Stringers

Behaviour		25 Tonne Loading		5 Tonne Loading		Pedestrian Loading	
		Truss Spans	Girder Spans	Truss Spans	Girder Spans	Truss Spans	Girder Spans
Positive bending at midspan	Uncorroded LLCF	1.00	1.29	3.30	4.22	6.05	11.60
	Tolerable corrosion in top flange (mm)	0	5.49	6.1	8.2	6.1	8.2
	% tolerable section loss in top flange	0%	44%	49%	65%	49%	65%
	% of stringers that are acceptable (i.e., the observed corrosion is less than the tolerable corrosion)	90%	100%	100%	100%	100%	100%
Shear or web compression at ends	Uncorroded LLCF	3.21	1.62	10.88	5.89	16.36	18.35
	Tolerable corrosion in web (mm)	4.2	1.6	5.0	4.3	5.2	5.4
	% tolerable section loss in web	56%	20%	67%	54%	69%	68%
	% of stringers that are acceptable (i.e., the observed corrosion is less than the tolerable corrosion)	95%	100%	100%	100%	100%	100%

Table 12: Refined Tolerable Corrosion for Deck Stringers in the Truss Spans

Behaviour		25 Tonne Loading	
		Live Load Lateral Distribution = 0.30	**Live Load Lateral Distribution = 0.254**
Positive bending at midspan	Uncorroded LLCF	1.00	1.18
	Tolerable corrosion in top flange (mm)	0	3.7
	% tolerable section loss in top flange	0%	29%
	% of stringers that are acceptable (i.e., the observed corrosion is less than the tolerable corrosion)	90%	**100%**

A design life of up to 10 years for the stringers in their current condition appears to be achievable for the 5 tonne and 25 tonne loading, as well as for the pedestrian-only loading.

It is recommended that, if a design life beyond 10 years is desired for 5 tonne or 25 tonne loading, the stringers be replaced at the same time that the deck is replaced. This is due to comparing the cost of recoating the stringers to replacement, combined with the risk that recoating the existing stringers will not completely eliminate future deterioration.

4.2.3 Floorbeams (Evaluation Item E-10)

The floorbeams are severely corroded in their top flange. Near their ends, the web is also observed to have a varying degree of section loss.

Corrosion thresholds are computed the same way as described for the stringers. Figure 12 shows the assumptions regarding a fixed amount of corrosion and the corrosion threshold that will be computed, and the results of the exercise are summarized in Table 13.

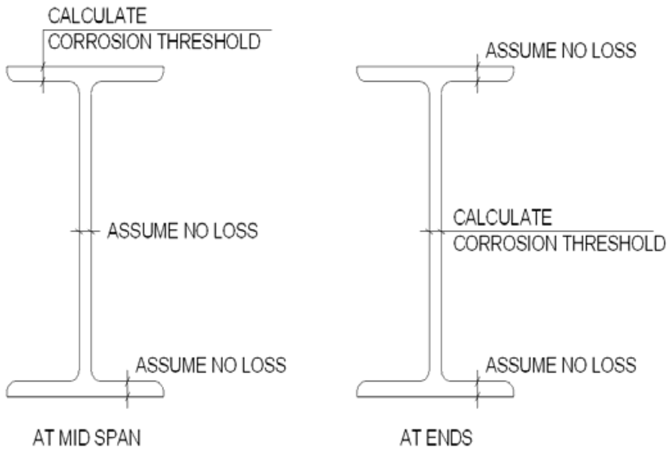


Figure 12: Floorbeam Section Loss and Corrosion Threshold Locations of Interest

A design life of up to 10 years for the floorbeams in their current condition appears to be achievable for the 5 tonne loading, as well as for the pedestrian-only loading.

If the bridge is upgraded to 25 tonnes for a design life of up to 10 years, some floorbeams must be strengthened, likely in the neighbourhood of 10% of the floorbeams.

It is recommended that, if a design life beyond 10 years is desired for 5 tonne or 25 tonne loading, the floorbeams be replaced at the same time that the deck is replaced. This is due to comparing the cost of recoating the floorbeams to the cost of replacement, combined with the risk that recoating of the existing floorbeams will not completely eliminate future deterioration.

Table 13: Tolerable Corrosion for Floorbeams

Behaviour		25 Tonne Loading		5 Tonne Loading		Pedestrian Loading	
		Truss Spans	**Girder Spans**	**Truss Spans**	**Girder Spans**	**Truss Spans**	**Girder Spans**
Positive bending at midspan	Uncorroded LLCF	1.21	0.92	3.34	2.83	4.08	4.67
	Tolerable corrosion in top flange (mm)	5.0	No Good	9.0	6.9	9.0	7.3
	% tolerable section loss in top flange	29%	No Good	52%	48%	52%	51%
	% of floorbeams that are acceptable (i.e., the observed corrosion is less than the tolerable corrosion)	**90%**	**0%**	100%	100%	100%	100%
Shear or web compression at ends	Uncorroded LLCF	2.48	1.92	6.11	5.91	9.94	9.76
	Tolerable corrosion in web (mm)	3.0	3.6	4.7	5.6	5.3	6.0
	% tolerable section loss in web	30%	38%	47%	59%	53%	63%
	% of floorbeams that are acceptable (i.e., the observed corrosion is less than the tolerable corrosion)	100%	100%	100%	100%	100%	100%

4.2.4 Sidewalk (Continuation of Evaluation Item E-10)

The floorbeam cantilevers that support the sidewalk in the truss spans are corroded in their top flange.

Corrosion thresholds are computed the same way as described for the stringers. Figure 13 shows the assumptions regarding a fixed amount of corrosion and the corrosion threshold that will be computed, and the results of the exercise are summarized in Table 14.

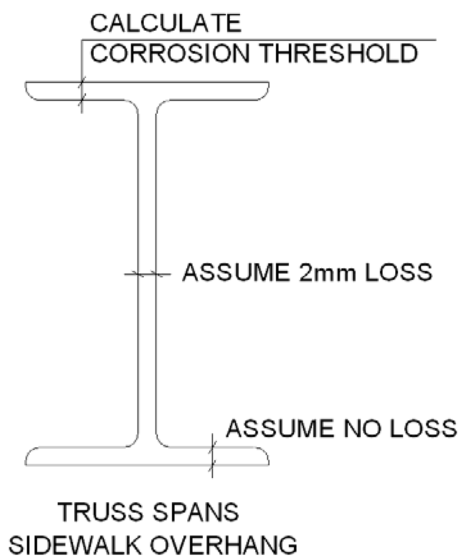


Figure 13: Sidewalk Floorbeam Section Loss and Corrosion Threshold

Table 14: Tolerable Corrosion for Sidewalk Floorbeams

Behaviour		25 Tonne Loading	5 Tonne Loading	Pedestrian Loading
Negative bending at truss top chord	Uncorroded LLCF	3.03	7.75	5.41
	Tolerable corrosion in top flange (mm)	10.2	10.2	10.2
	% tolerable section loss in top flange	59%	59%	59%
	% of floorbeams that are acceptable (i.e., the observed corrosion is less than the tolerable corrosion)	100%	100%	100%

The inspection also identified that some sidewalk stringers have distorted webs near the end. This is potentially the result of rust jacking such that the stringers are only loaded near midspan, with reduced capacity due to reduced lateral support. The capacity of the sidewalk stringer in the truss span was revisited assuming lateral support only at midspan. The results summarized in Table 15 indicate that the capacity of the sidewalk stringer is not overly sensitive to the assumption of continuous lateral support versus being supported at midspan only.

Table 15: Corroded and Uncorroded LLCF for Sidewalk Stringers

	25 Tonne Loading	5 Tonne Loading	Pedestrian Loading
Uncorroded LLCF (continuous lateral support)	0.49	1.63	3.84
Corroded LLCF (lateral support at midspan only)	0.44	1.36	3.19

A design life of up to 10 years for the floorbeams in their current condition appears to be achievable for the 5 tonne loading, as well as for the pedestrian-only loading.

If the bridge is upgraded to 25 tonnes for a design life of up to 10 years, the centre sidewalk stringer in the truss spans must be strengthened.

It is recommended that, if a design life beyond 10 years is desired for 5 tonne or 25 tonne loading, the sidewalk stringers be replaced along with the deck stringers and floorbeams.

4.2.5 Truss System (Evaluation Items E-2 to E-7)

The truss system has section loss due to corrosion in numerous areas that may influence the vertical load carrying capacity of the bridge.

4.2.5.1 Top Chord

Evaluation Item E-2. The truss has localized section loss in the web of the top chord in Span 4, Top chord panel point U3. The approach to computing the LLCF at this location was to conservatively assume that there was a 76 mm portion of the web completely missing, as shown in Figure 14. The LLCFs are summarized in Table 16, and the LLCFs indicate that the bridge would still possess adequate capacity for 25 tonne, 5 tonne and pedestrian loading (by virtue of the more severe snow loading being acceptable).

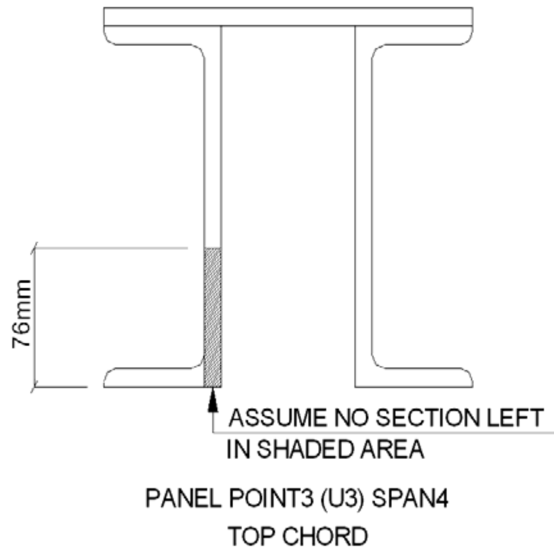


Figure 14: Truss Top Chord – Section Loss in Web

Table 16: Corroded and Uncorroded LLCF for Top Chord in Span 4 at Panel Point U3

	25 Tonne Loading	5 Tonne Loading	Snow Load (ULS 1d)
Uncorroded LLCF	2.16	3.38	C/D = 1.71
Corroded LLCF	1.81	2.84	C/D = 1.55

Evaluation Item E-3. The truss has corrosion in Span 2, at top chord panel point U6, where there is a perforation in the continuous cover plate on top of the top chord. The approach to computing the LLCF at this location was to simply assume that there is no cover plate present, clearly a conservative approach given that the perforation is approximately only 25% of the overall width.

The reduced strength of the top chord is based on the cross sectional strength of two channels, where slenderness effects are ignored due to the cover plate being competent enough to maintain overall stability. Figure 15 shows area of interest, and the LLCFs are summarized in Table 17. The corroded LLCFs indicate that the bridge would still possess adequate capacity for 25 tonne, 5 tonne and pedestrian loading (by virtue of the more severe snow loading being acceptable).

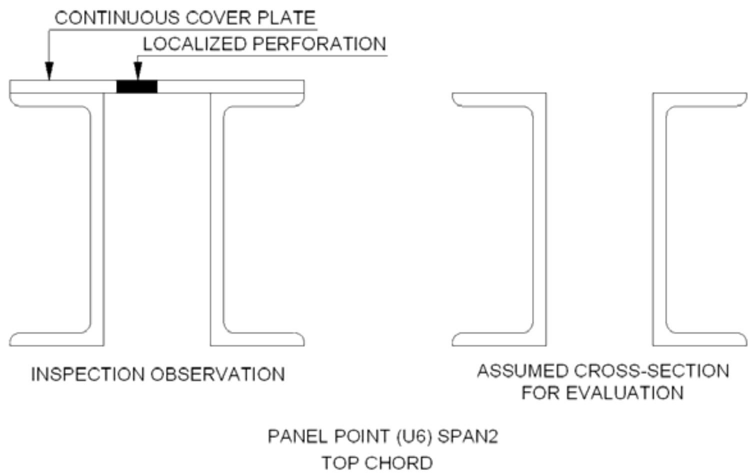


Figure 15: Truss Top Chord Cover Plate Section Loss

Table 17: Corroded and Uncorroded LLCF for Top Chord in Span 2 at Panel Point U6

	25 Tonne Loading	**5 Tonne Loading**	**Snow Load (ULS 1d)**
Uncorroded LLCF Based on member strength	4.73	10.12	C/D = 4.71
Corroded LLCF Based on cross sectional strength	2.82	6.03	C/D = 3.08

4.2.5.2 Bottom Chord

Corrosion thresholds for the following areas are computed the same way as described for the stringers. Figure 16 shows the assumptions regarding a fixed amount of corrosion and the corrosion threshold that will be computed, and the results of the exercise are summarized in Table 18.

- **Evaluation Item E-4**. Spans 1,2 and 5: Bottom chord, section loss in the vertical leg of the angle;
- **Evaluation Item E-5**. Spans 3 and 4: Bottom chord, section loss in the web and top flange of the channel; and
- **Evaluation Item E-7**. Spans 3 and 4: Bottom chord gusset plates, section loss along a horizontal line just above the top flange of the bottom chord.

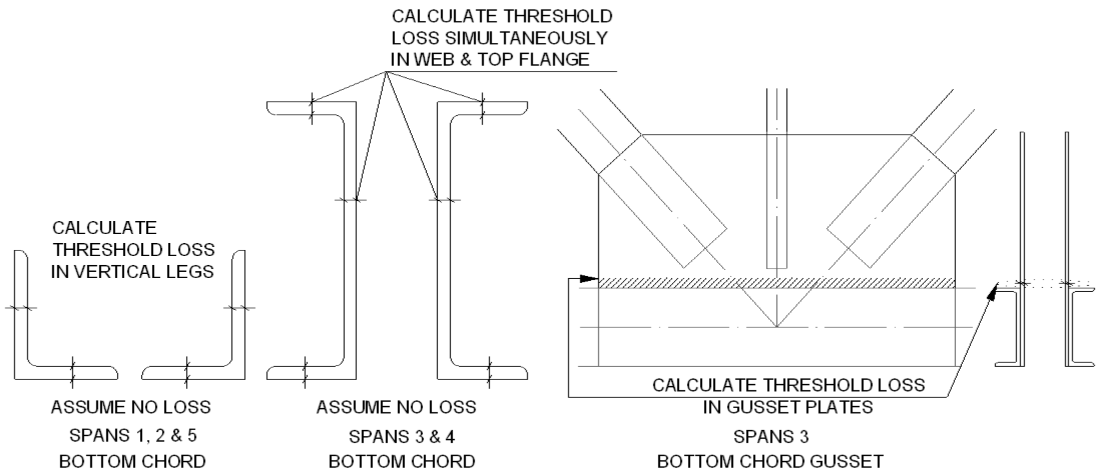


Figure 16: Truss System Section Loss and Corrosion Threshold

Additional Observation on Evaluation Item E-5. It should also be noted that the bottom chord in spans 3 and 4 has had reinforcing plates added to the web of the channels in order to compensate for section loss due to corrosion in the channel web and top flange, refer to Figure 17.The question arises as to the effectiveness of the new bolts that are bearing on the deteriorated web, especially because all of the bolts in the connection are required for the shear capacity of the bolts to equal the tensile capacity of the original channel web and top flange. However, the web can experience as much as 65% section loss before the bearing capacity of the web on a bolt governs over the shear capacity of the bolt. Therefore, the reinforcing detail is adequate for matching the tensile capacity of the bottom chord away from the repair provided that 35% of the web thickness remains.

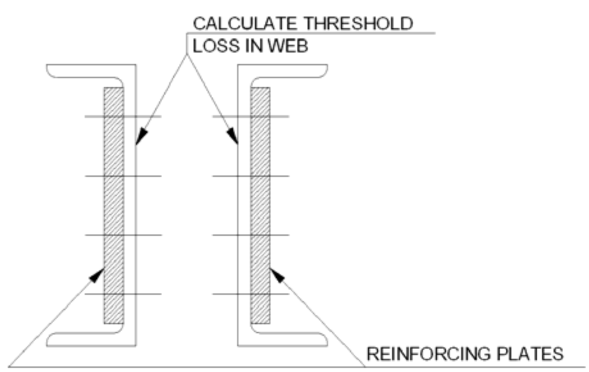


Figure 17: Reinforcing Plates Added to Bottom Chords in Spans 3 and 4

Table 18: Tolerable Corrosion for Bottom Chord

Behaviour		25 Tonne Loading	5 Tonne Loading	Snow Load (ULS 1d)
Spans 1,2 and 5: bottom chord in tension	Uncorroded LLCF	1.32	3.19	C/D = 1.94
	Tolerable corrosion in vertical leg (mm)	2.6	7.5	8.1
	% tolerable section loss in vertical leg	27%	78%	84%
	% of panel points in 3 spans that are acceptable (i.e., the observed corrosion is less than the tolerable corrosion)	90%	100%	100%
Spans 3 and 4: bottom chord in tension	Uncorroded LLCF.	2.47	3.88	C/D = 1.86
	Tolerable corrosion in top flange (mm)	7.1	8.7	8.6
	Tolerable corrosion in web (mm)	7.0	8.6	8.5
	% tolerable section loss in top flange & web	45%	55%	54%
	% of panel points in 2 spans that are acceptable (i.e., the observed corrosion is less than the tolerable corrosion)	100%	100%	100%
Spans 3 and 4: bottom chord gusset thickness just above top flange of bottom chord	Uncorroded LLCF	2.76	4.59	C/D = 2.16
	Tolerable corrosion in each gusset (mm)	4.0	4.9	5.0
	% tolerable section loss in gusset plates	42%	52%	53%
	% of panel points in 2 spans that are acceptable (i.e., the observed corrosion is less than the tolerable corrosion)	95%	100%	100%

Additional Observation on Evaluation Item E-6. The truss has localized section loss in the top flange of the bottom chord in Span 4 at panel point L4. The approach to computing the LLCF at this location was to assume that there was a portion of the flange completely missing, as shown in Figure 18, and the results of the exercise are summarized in Table 19.

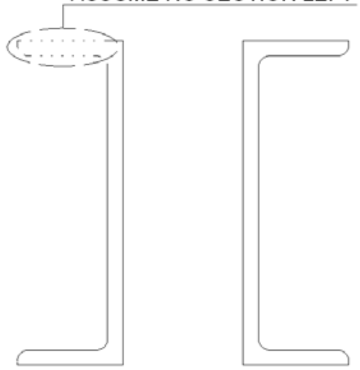


Figure 18: Truss Bottom Chord – Section Loss in Top Flange

Table 19: Corroded and Uncorroded LLCF for Bottom Chord in Span 4 at Panel Point L4

	25 Tonne Loading	5 Tonne Loading	Snow Load (ULS 1d)
Uncorroded LLCF (member strength)	2.47	3.88	C/D = 1.86
Corroded LLCF (cross sectional strength)	1.94	3.04	C/D = 1.52

4.2.5.3 Conclusions Regarding Truss System Capacity

The truss system in its current condition has sufficient strength to carry the 5 tonne and pedestrian loading (by virtue of the more severe snow loading being acceptable).

If the bridge is upgraded to 25 tonnes:

- some truss diagonals in spans 1, 2 and 5 must be strengthened at locations previously identified in Figure 10 of Section 4.1;
- some bottom chord members in spans 1, 2 and 5 must be strengthened (see Table 18); and

- some bottom chord gussets in spans 3 and 4 must be strengthened (see Table 18).

4.2.6 Truss Bearings

The governing capacity for vertical loads in the bearings is shear in the rivets of the bearing at Pier 2. The inspection did not identify any substantial section loss in the rivets. The bearing capacity of the pin plates, which is the next lowest capacity in the bearing, is approximately 20% larger than the rivet capacity. Therefore, a reduction in the vertical load carrying capacity of the pin plates is not likely to enter into the governing LLCF and as such a reduced LLCF need not be considered.

Shear in the anchor bolts that may be present due to temperature loading is discussed later as part of the concrete piers in Section 4.2.8.

A design life of up to 10 years for the truss bearings in their current condition appears to be achievable for the 5 tonne, 25 tonne and pedestrian loading.

It is recommended that, if a design life beyond 10 years is desired for 5 tonne or 25 tonne loading, the truss bearings be rehabilitated to reinstate the sliding of seized bearings.

4.2.7 Girders (Evaluation Item E-11)

A girder bottom flange has been damaged in Span 6 approximately one-third of the span length from Pier 6. The approach to computing the LLCF at this location was to assume that there is a portion of the bottom flange missing. Figure 19 shows the area of interest, and the LLCFs are summarized in Table 20. The damaged LLCFs indicate that the bridge would still possess adequate capacity for 25 tonne, 5 tonne and pedestrian loading (by virtue of the more severe snow loading being acceptable).

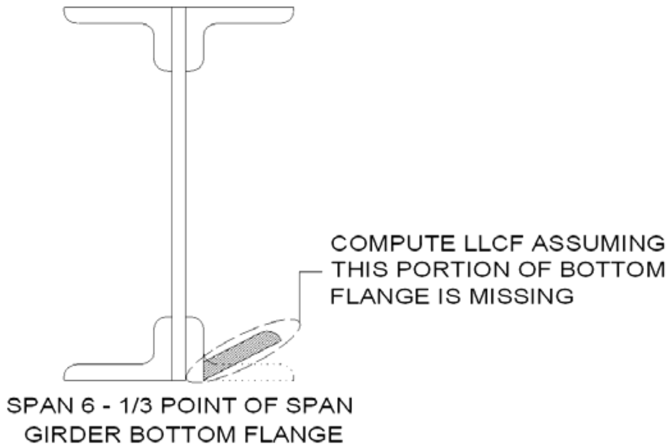


Figure 19: Damaged Girder Bottom Flange in Span 6

Table 20: LLCF for Girder Bottom Flange at Damaged Location in Span 6

	25 Tonne Loading	5 Tonne Loading	Snow Load (ULS 1d)
Undamaged LLCF	2.03	5.83	C/D = 4.20
Damaged LLCF	1.29	3.70	C/D = 2.84

A design life of up to 10 years for the girders in their current condition appears to be achievable for the 5 tonne and pedestrian loading.

If the bridge is upgraded to 25 tonnes for a design life of up to 10 years, bearing stiffeners must be added to address web in compression overstresses previously identified in Table 10 of Section 4.1.

It is recommended that, if a design life beyond 10 years is desired for 5 tonne or 25 tonne loading, the girders be recoated.

4.2.8 Concrete Piers (Evaluation Item E-1)

Based on the inspection findings, the truss bearings that are intended to allow expansion and contraction appear to be seized. This produces a situation where there are likely unintended longitudinal loads on the bearing anchor bolts and concrete piers due to temperature changes.

Load rating with consideration of thermal effects due to seized bearings is influenced by the bending stiffness and bending strength of the concrete piers. The original design drawings show that all of the piers are unreinforced concrete, with the exception of a portion of Pier 5. Evaluating the ultimate bending strength of an unreinforced concrete column per CHBDC results in essentially no bending strength due to the fact that the code does not allow consideration of the tensile strength of the concrete. However, in order to generate an upper bound shear demand in the anchor bolts, the concrete is assumed to have a tensile capacity equal to its cracking strength.

The effects of temperature loading on the bridge when the bearings are seized can be manifested in one of two ways:

- Build-up of large shear in the anchor bolts and moment in the concrete piers as the piers restrain movement. This is not likely due to the low cracking strength of the piers; or
- Relief of forces as the concrete piers crack or the anchor bolts bend or shear to accommodate the movements. This is likely and is further evidenced by the inspection findings.

The anchor bolts at the sliding bearings extend beyond the top of the concrete pier by several inches, resulting in cantilevering anchor bolts. Site observations appear to indicate that the bolts are bent and there are no signs of sliding between the anchor bolts and the slotted holes. Furthermore, field observations indicate that there are horizontal cracks and vertical splitting cracks in the piers, refer to Figure 20.

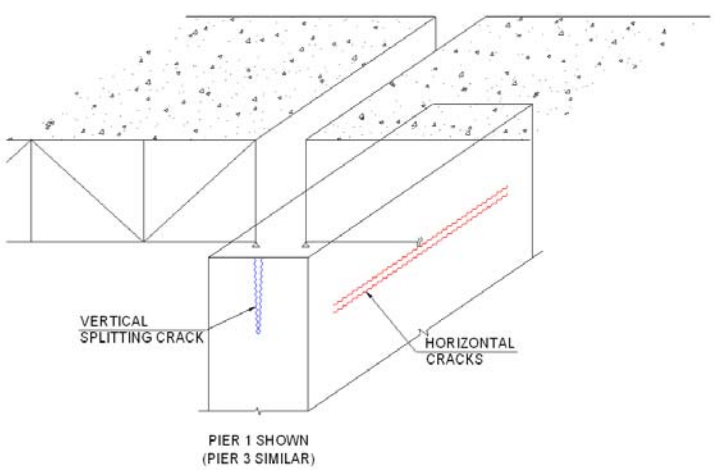


Figure 20: Typical Cracking Patterns Observed in Piers 1 and 3

The approach used to evaluate the piers and anchor bolts was to compute and compare the longitudinal shear that causes the pier to have horizontal cracks, the longitudinal shear that causes the anchor bolt to yield in bending, and the longitudinal shear that causes the anchor bolt to fail in shear. Table 21 shows results that are representative of typical results at piers.

Table 21: Summary of Capacity Limiting Shears at Piers

Longitudinal Shear	Pier 2 (Representative of Piers with Fixed Bearings)	Pier 3 (Representative of Piers with Seized Bearings)
Shear corresponding to the cracking moment in pier	105 kN	175 kN
Shear corresponding to the yield moment in anchor bolts	103 kN	58 kN
Shear corresponding to shear failure of anchor bolt	686 kN (concrete bearing)	1372 kN (concrete bearing)

The results indicate that the order of thermal relief for the piers from weakest to strongest is bending of the anchor bolts, followed by cracking of the piers and finally shearing of the bolts. Thermal relief of piers as the piers crack or the anchor bolts yield is not expected in itself to lead to instability of the piers, as the maximum thermal movement anticipated in any pier is expected to be less than 50 mm.

The other behaviour of interest that is harder to quantify is the consequence of vertical splitting cracks in piers that support two bearings, as observed at pier 1. This could be the result of two neighbouring spans contracting on a cold day. The contraction could be leading to each bearing on the pier of interest pulling away from the pier, thus leading to the vertical cracks and a more slender individual unreinforced column of concrete.

It is recommended that vertical splitting cracks of piers 1 and 3 be arrested by rehabilitating the tops of the piers in order to maintain reliable behaviour for a short term design life. This could be achieved by coring holes through the pier and installing loose steel tie rods. Care should be taken to ensure that the behaviour of the pier is not modified prior to replacement of the bearings. This is due to the fact that thermal stresses induced in the superstructure due to the seized bearings are currently relieved by the cracking in the pier. If the pier is fixed without releasing the bearings, the steel superstructure will be forced to resist the thermal loads.

It is also recommended that, if a life span beyond 10 years is desired, all piers be rehabilitated by encasing their tops in reinforced concrete and injecting all cracks with epoxy. Furthermore, the truss bearings that are seized should be rehabilitated to reinstate the intended sliding and a seismic evaluation for the unreinforced piers should be performed.

4.3 Traffic Barrier

In addition to assessing the vertical load carrying capacity of the bridge, it is noted that the traffic barriers are not a standard form. Therefore, only the anchorage of the barriers was briefly assessed for a collision load based on a barrier classification of PL-1.

Although this was not an exhaustive investigation, the barrier anchorage is overstressed at the connection to the deck (C/D = 0.2) and therefore does not satisfy the PL-1 requirements.

Given the fact that the traffic barrier connection does not meet PL-1 requirements, if the bridge is opened to vehicular traffic, it is recommended that BC MoT assess the risks associated with the barrier and establish whether the barrier should be upgraded to a higher standard.

4.4 Snow Removal Guidelines

As was previously noted, assuming that maximum snow load and maximum live load occurs at the same time is likely an overly-conservative assumption. Yet, considering each load on its own is likely unconservative.

The snow removal guidelines provided below could be refined further by computing the actual amount of snow permissible on the bridge, likely resulting in a larger amount of snow tolerable on the bridge. This could be accomplished by assessing the remaining capacity under a specified load and determining how much snow load could be tolerated. However, at this stage, given the numerous live load models and states of corrosion this is not deemed practicable.

For a situation where the bridge is open to vehicular traffic, the proposed approach is to assume that during plowing, the snow plow is limited to the posted load. Assuming there is little other traffic on the bridge, it could be assumed that the 7 kN/m uniformly distributed portion of the lane load is the equivalent snow load permitted on the bridge. Based on the commentary in NBCC, the approximate depth of compacted snow over the entire bridge width that produces a line load of 7 kN/m can be back-calculated as approximately 350 mm.

As a matter of interest, 7 kN/m over 7 m bridge width = 1 kPa, which is about half of the maximum assumed design snow load of 1.8 kPa.

Based on the results of the load evaluation, it is recommended that:

- if the bridge is open to vehicular traffic, the depth of snow anywhere on the bridge be limited to 350 mm and that snow in excess of this depth be removed from the bridge with a snow plow not exceeding the posted load limit; or
- if the bridge is open to pedestrians-only, the depth of snow anywhere on the bridge be limited to 600 mm and that snow in excess of this depth be removed from the bridge by manual methods or lightweight equipment weighing less than 500 kg.

4.5 Results - Pedestrians-only

The results in Table 10 show that the floor system components and main members in their uncorroded state are capable of carrying pedestrian loads.

Table 11 and Table 13 to Table 15 show that the floor system components that were revisited due to observed section loss still have adequate capacity to carry pedestrian loads.

In addition, Table 16 to Table 20 show that main members in the trusses and girders that were revisited due to observed section loss and damage still have adequate capacity to carry pedestrian loads. These tables do not show the pedestrian loading results; however, the demands from the snow loading in the members of interest are comparable to the pedestrian loads on the sidewalk only and therefore conclusions regarding the pedestrian loading can be deduced from the results shown in the tables.

The above conclusions regarding the ability of the bridge to carry pedestrian loads are subject to the following constraints being re-emphasized:

- The code specified pedestrian loads are not applied to the entire bridge width as this would be too severe for this bridge given its location;

- The code specified pedestrian load intensity is only applied to the sidewalk width as this is in keeping with the intent of the code for highway bridges with a sidewalk;
- An additional load case assuming fifty (50) pedestrians on the bridge, resulting in an intensity of 4 kPa over 12.5 m^2, was used to evaluate the floor system under the roadway; and
- The snow removal guidelines previously described in Section 4.4 limiting the depth of snow to 600 mm must be followed in order to reduce the magnitude of simultaneous loads from snow and pedestrians.

5 Repair Concepts and Cost Estimates

The observations made during the detailed inspection combined with the load evaluation and target design life options result in various types of repairs.

Based on the widespread and severe overstresses computed in the load evaluation for the CL1-625 loading, the structural feasibility and cost associated with upgrading the bridge to satisfy CL1-625 loading is expected to be prohibitive. Therefore, conceptual repairs and cost estimates to upgrade to this live load model have not been prepared. The cost of a new bridge is included in the concepts as a means of establishing a benchmark for the cost to achieve a CL1-625 loading.

Strengthening is related to upgrades required to increase the load carrying capacity of the bridge. Rehabilitation is related to upgrades required to increase the design life of the bridge. Appendix C contains Conceptual Rehabilitation drawings prepared by B&T as part of this evaluation report.

Maintenance items identified in the inspection report are deemed to be relevant to all concepts and are considered to be part of a routine maintenance program, and are therefore not addressed in this report. Some maintenance items may not be applicable to the 25 and 50 year design life options as they might be superseded by recommendations to replace the member.

High-level cost estimates were prepared based on the field inspection, the evaluation results and engineering judgment. It is considered that these estimates are appropriate for comparing the relative costs of the different rehabilitation options. However, detailed cost estimates of the rehabilitation work will need to be developed for the rehabilitation option chosen for final design. The costs are based on 2009 dollars.

5.1 Pedestrians-only “Do Nothing” Option – up to 2 Year Life

In its current condition, the bridge can safely carry pedestrian and snow loads if the bridge were opened as a pedestrian-only bridge with a posted limit of 50 pedestrians. Therefore, it is possible to open the bridge to pedestrians without any upgrades to the vertical load carrying capacity of the bridge. However, the following points must be acknowledged if BC MoT chooses this “Do Nothing Option”:

i. A detailed annual inspection is required in order to confirm that the condition of the bridge is in a condition reflective of the load evaluation assumptions; and

ii. This option is essentially a “do nothing” option. Therefore, the amount of remaining life associated with this option is minimal due to the continued degradation of the structure (i.e., it is anticipated that the structure will need upgrades to the piers within 2 years in order to extend the design life).

The costs associated with the Pedestrians-only “Do Nothing” Option are summarized in Table 22. It should be noted that while this is the least cost approach, there is little confidence in predicting a serviceable life of the bridge beyond a 2 year horizon unless the piers have minor upgrades.

Table 22: Pedestrian-only “Do Nothing” Option - up to 2 Year Life

Inspection Report Rehab Item (if Applicable)	Upgrades and Reference Drawing (if Applicable)	Approx. Quantity	Estimated Cost (2009 $)
	Annual detailed inspection with swing stage access @ $150 k/year	1 EA	150 k
TOTAL			**0.15 M**[1]

Notes: 1 - The life-cycle cost must be increased by $1.5 M to reflect demolition costs.

5.2 5 tonne Rehabilitation “Do Nothing” Option - up to 2 Year Life

In its current condition, the bridge can safely carry the 5 tonne vehicle loading. Therefore, it is possible to open the bridge to a posted load of 5 tonnes without any upgrades to the vertical load carrying capacity of the bridge. However, the following points must be acknowledged if BC MoT chooses the “Do Nothing Option”:

i. The traffic barrier on both sides of the bridge does not meet the requirements of a PL-1 barrier and the approximate cost of upgrading the barrier is included in the cost estimate for this option. However, BC MoT may choose to leave the barrier in its current state, thereby reducing the cost of this option, subject to a policy decision;

ii. A detailed annual inspection is required in order to confirm that the condition of the bridge is in a condition reflective of the load evaluation assumptions; and

iii. This option is essentially a “do nothing” option. Therefore, the amount of remaining life associated with this option is minimal due to the continued degradation of the structure (i.e., it is anticipated that the structure will need to be closed to all traffic and decommissioned in 2 years).

The costs associated with the 5 tonne Rehabilitation "Do Nothing" Option are summarized in Table 23. It should be noted that while this is the least cost approach for opening to vehicular traffic, there is little confidence in predicting a serviceable life of the bridge beyond a 2 year horizon. Therefore, it is assumed that if BC MoT moves forward with this option, the bridge will need to be closed to all traffic and decommissioned in 2 years.

Table 23: 5 tonne Loading Upgrades for "Do Nothing" Option - up to 2 Year Life

Inspection Report Rehab Item (if Applicable)	Upgrades and Reference Drawing (if Applicable)	Approx. Quantity	Estimated Cost (2009 $)
R-14	Upgrade traffic barrier on both sides of bridge	464 m	300 k
	Contingency (25%)		75 k
	Engineering (20%)		75 k
	Additional Project Costs (management, etc.)		100 k
	Annual detailed inspection with swing stage access @ $150 k/year	1 EA	150 k
TOTAL			**0.70 M**[1]

Notes: 1 - The life-cycle cost must be increased by $1.5 M to reflect demolition costs.

5.3 Pedestrians-only Rehabilitation – 10 Year Life

In its current condition, the bridge can safely carry the pedestrian and snow loading if the bridge were opened as a pedestrian-only bridge with a posted limit of 50 pedestrians. However, minor rehabilitation work is required to achieve a service life of 10 years and bi-annual detailed inspections will be required to monitor the condition of the bridge.

Table 24 summarizes the rehabilitation required to keep the bridge open as a pedestrian-only crossing for up to 10 years. In order to achieve a 10 year life, the upgrades to piers 1 and 3 should be undertaken and completed by the end of 2011.

Table 24: Pedestrian-only Upgrades - 10 Year Life

Inspection Report Rehab Item (if applicable)	Upgrades and Reference Drawing (if applicable)	Approx. Quantity	Estimated Cost (2009 $)
	Rehab tops of concrete piers 1 and 3, core through and install tie rods. B&T Dwg R07	20 EA	100 k
	Contingency (25%)		25 k
	Engineering (20%)		25 k
	Additional Project Costs (management, etc.)		30 k
	Bi-annual detailed inspection with swing stage access @ 150 k/year	4 EA	600 k
TOTAL			**0.78 M**[1]

Notes: 1 - The life-cycle cost must be increased by $1.5 M to reflect demolition costs.

The costs listed in the above table do not include shimming the gaps between the tops of the stringers and the underside of the concrete deck. At this time, it is deemed more advisable to monitor the behaviour of the gaps and proceed with rehabilitation of the gaps if undesirable behaviour is observed.

The unit costs associated with rehabilitating the pier tops for this option are higher than presented in other 10 Year Life options for vehicular traffic. This is because the pier rehabilitation is the only work item anticipated for this option, thus resulting in higher mobilization costs as a percentage of the total cost.

5.4 5 tonne Rehabilitation – 10 Year Life

In its current condition, the bridge can safely carry the 5 tonne vehicle loading. However, rehabilitation work is required to achieve a service life of 10 years, see B&T Dwg. R01 in Appendix C.

Table 25 summarizes the rehabilitation required to keep the bridge open to a 5 tonne loading for up to 10 years. In order to achieve a 10 year life, $1.6 million dollars in repairs must be undertaken and completed by the end of 2011.

Table 25: 5 tonne Loading Upgrades - 10 Year Life

Inspection Report Rehab Item (if applicable)	Upgrades and Reference Drawing (if applicable)	Approx. Quantity	Estimated Cost (2009 $)
R-3	Recoat under-deck elements	Touchup 25%	400 k
R-6	Replace gussets between truss verticals and bottom chord. B&T Dwg R05	20 EA	150 k
R-9	Replace sway bracings & connections	10 EA	100 k
R-11	Repair holes in floorbeams 4 & 8 in Span 4	2 EA	15 k
	Extend drain pipes. B&T Dwg R01	36 EA	75 k
	Rehab tops of concrete piers 1 and 3, core through and install tie rods. B&T Dwg R07	20 EA	50 k
R-14	Upgrade traffic barrier on both sides of bridge	464 m	300 k
	Contingency (25%)		270 k
	Engineering (20%)		270 k
	Additional Project Costs (management, etc.)		270 k
	Annual detailed inspection with swing stage access @ 150 k/year	9 EA	1.35 M
TOTAL			**3.25 M**[1]

Notes: 1 - The life-cycle cost must be increased by $1.5 M to reflect demolition costs.

The costs listed in the above table do not include shimming the gaps between the tops of the stringers and the underside of the concrete deck. At this time, it is deemed more advisable to monitor the behaviour of the gaps and proceed with rehabilitation of the gaps if undesirable behaviour is observed.

5.5 25 tonne Rehabilitation – 10 Year Life

The bridge requires structural strengthening and rehabilitation for the 25 tonne loading, as shown on B&T Dwg. R02 in Appendix C.

Table 26 summarizes the rehabilitation required to keep the bridge open for up to 10 years and the estimated costs of the repairs. Items in the table that are shown in **bold** are the items requiring immediate action to upgrade the bridge prior to opening it to 25 tonne loading, while it is recommended that the remaining items are performed before the end of 2011. It should be noted that these repairs are required to achieve a short term life of up to 10 years.

Table 26: 25 tonne Loading Upgrades - 10 Year Life

Inspection Report Rehab Item (if Applicable)	Upgrades and Reference Drawing (if Applicable)	Approx. Quantity	Estimated Cost (2009 $)
R-3	Recoat under-deck elements	Touchup 25%	400 k
R-6	Replace gussets between truss verticals and bottom chord. B&T Dwg R05	20 EA	150 k
R-9	Replace sway bracings & connections	10 EA	100 k
R-11	Repair holes in floorbeams 4 & 8 in Span 4	2 EA	15 k
R-12	Address gaps between stringers and deck	261 EA	500 k
	Extend drain pipes. B&T. Dwg R02	36 EA	75 k
	Rehab tops of concrete piers 1 and 3, core through and install tie rods. B&T Dwg R07	20 EA	50 k
	Strengthen truss diagonals in Spans 1, 2 and 5. Add an angle to each existing angle to decrease slenderness ratio. B&T Dwg. R05	20 EA	240 k
R-13	**Strengthen girder webs at bearings. Add a vertical stiffener to the web. B&T Dwg R05**	8 EA	30 k
	Strengthen 10% of floorbeams for positive bending. Add a cover plate to the bottom flange. B&T Dwg R06	4 EA	25 k
R-14	Upgrade traffic barrier on both sides of bridge	464 m	300 k
	Contingency (25%)		470 k
	Engineering (20%)		470 k
	Additional Project Costs (management, etc.)		460 k
	Annual detailed inspections with inspection vehicle @ 40 k/year	9 EA	360 k
TOTAL			**3.65 M**[1]

Notes: 1 - The life-cycle cost must be increased by $1.5 M to reflect demolition costs.

5.6 5 tonne Rehabilitation – 25 or 50 Year Life

If an extended lifespan of 25 to 50 years is desired for a 5 tonne posting, significant rehabilitation efforts will be required as shown on B&T Dwg. R03 in Appendix C.

The concrete deck and coating is expected to require replacement in the near term. When the deck is replaced, one could elect to recoat the floor system or replace it. It is recommended that the floor system be replaced. This is due to comparing the cost of recoating the floor system to replacement, combined with the risk that recoating of the floor system will not completely eliminate future deterioration. Furthermore, the concrete substructure and truss bearings will require rehabilitation. Table 27 summarizes the recommended rehabilitation work and the estimated costs required to keep the bridge open for a service life of 25 years and 50 years.

In order to increase the likelihood that the repair concepts are successful in extending the design life of the structure to 25 or 50 years, it is advisable that the repairs be implemented by the end of 2011. Delay of the implementation will likely decrease the effectiveness of the repairs and will increase the cost of such repairs as the corrosion continues.

Table 27: 5 tonne Loading Upgrades - 25 & 50 Year Life

Inspection Report Rehab Item (if applicable)	Upgrades and reference drawing (if Applicable)	Approx. Quantity	Estimated Cost (2009 $)
R-6	Replace gussets between truss verticals and bottom chord. B&T dwg R05	20 EA	150 k
R-9	Replace sway bracings & connections	10 EA	100 k
	Reinforce tops of all concrete piers and inject all cracks with epoxy. B&T R07	6 EA	840 k
R-1 and R-2	Rehabilitate truss sliding bearings B&T dwg R08	10 EA	3 M
R-8	Reinforce jacking beams. B&T dwg R08	5 EA	50 k
R-10	Replace entire concrete deck, stringers, floorbeams, barriers and railings	1750 m^2	3.5 M
R-4	Recoat the truss and girders		6.2 M
	Contingency (25%)		3.5 M
	Engineering (20%)		3.5 M
	Additional Project Costs (management, etc.)		4.0 M
	Detailed inspections every 5 years with inspection vehicle @ 40 k/inspection	4 EA	160 k
TOTAL - 25 Year Service Life			**25.0 M[1]**
	Added costs of extending coating life to 50 years		1.5 M
	Additional Project Costs (management, etc.)		300 k
	Additional inspections every 5 years	5 EA	200 k
TOTAL - 50 Year Service Life			**27.0 M[1]**

Notes: 1 - The life-cycle cost must be increased by $1.5 M to reflect demolition costs.

Note that the above costs do not include a seismic assessment and rehabilitation of the structure, which could range anywhere from $500 k to $5 M.

5.7 25 tonne Rehabilitation – 25 or 50 Year Life

If an extended lifespan of 25 to 50 years is desired for a 25 tonne posting, significant rehabilitation efforts will be required, as shown on B&T Dwg. R04 in Appendix C.

The concrete deck and coating is expected to require replacement in the near term. When the deck is replaced, one could elect to recoat the floor system or replace it. It is recommended that the floor system be replaced. This is due to comparing the cost of recoating the floor system to replacement, combined with the risk that recoating of the floor system will not completely eliminate future deterioration. Furthermore, the concrete substructure and truss bearings will require rehabilitation. Table 28 summarizes the recommended rehabilitation work and the estimated costs required to keep the bridge open for a service life of 25 years and 50 years.

Items in the table that are shown in **bold** are the items requiring immediate action to upgrade the bridge prior to opening it to 25 tonne loading, and it is recommended that the remaining items are performed before the end of 2011.

Table 28: 25 tonne Loading Upgrades - 25 & 50 Year Life

Inspection Report Rehab Item (if Applicable)	Upgrades and reference drawing (if Applicable)	Approx. Quantity	Estimated Cost (2009 $)
R-6	Replace gussets between truss verticals and bottom chord. B&T dwg R03	20 EA	150 k
R-9	Replace sway bracings & connections	10 EA	100 k
	Reinforce tops of all concrete piers and inject all cracks with epoxy. B&T R07	6 EA	840 k
R-1 and R-2	Rehabilitate truss sliding bearings. B&T dwg R08	10 EA	3 M
	Reinforce jacking beams. B&T dwg R08	5 EA	50 k
	Strengthen truss diagonals in Spans 1 and 2. Add an angle to each existing angle to decrease slenderness ratio. B&T dwg. R05	20 EA	240 k
R-13	**Strengthen girder webs at bearings. Add a vertical stiffener to the web. B&T dwg R05**	8 EA	30 k
R-10	**Replace entire concrete deck, stringers, floorbeams, barriers and railings**	1750 m^2	3.5 M
R-4	Recoat the truss and girders		6.2 M
	Contingency (25%)		3.5 M
	Engineering (20%)		3.5 M
	Additional Project Costs (management, etc.)		4.2 M
	Detailed inspections every 5 years with inspection vehicle @ 40 k/inspection	4 EA	160 k
TOTAL - 25 Year Service Life			**25.5 M[1]**
	Additional recoating of steel		1.5 M
	Additional Project Costs (management, etc.)		300 k
	Additional inspections every 5 years	5 EA	200 k
TOTAL - 50 Year Service Life			**27.5 M[1]**

Notes: 1 - The life-cycle cost must be increased by $1.5 M to reflect demolition costs.

Note that the above costs do not include a seismic assessment and rehabilitation of the structure, which could range anywhere from $500 k to $5 M.

5.8 New Bridge

The cost of a new bridge has been estimated based on the following assumptions:

- An allowance of $0.5 M has been made for property acquisition, in the event that a revised location is chosen for the new structure;

- Construction cost of $7500 per square metre, which includes an allowance of 25 % for contingency and 20% for engineering; and
- Additional project costs (management, etc.) estimated as $2.4 M for a new single-lane bridge and as $4.5 M for a new two lane bridge.

The two options and associated costs requested by BC MoT are as follows:

i. A single lane bridge with sidewalk, assuming a deck and sidewalk width that matches the existing overall width. The estimated cost associated with this option is $14.3 M; and

ii. A two lane bridge with sidewalk with an overall width of 10.5 m. The estimated cost associated with this option is $22.7 M.

The life-cycle costs of these new bridge options must be increased by $1.5 M to reflect the demolition costs.

5.9 Comparison of 5 tonne and Pedestrian-only Options

The option of opening the bridge for pedestrian use only (50 people maximum) is the least cost option. It should be noted that there are additional reasons why a pedestrian-only bridge is attractive from a risk perspective. These observations primarily revolve around public safety as well as confidence in achieving the estimated service life, and are listed following:

- Keeping the bridge open to pedestrians-only improves structural safety since it is less likely to experience an overload when compared to a bridge with a posted limit of 5 tonnes that could be exposed to truck overloads on a more regular basis.
- A pedestrian-only bridge should experience less degradation due to less dynamic loading than a bridge open to vehicular traffic. As they cross the bridge, vehicles would subject the structure to some pounding due to bumps in the roadway and loose connections such as gaps above the stringers. The pedestrian loading is expected to be more of a static loading, where amplification of load effects and pounding is not expected.
- The rate of corrosion should be slower for the bridge open to pedestrians-only than for a bridge open to vehicular traffic, since it will be exposed to less de-icing salts.

- Since the 5 tonne loading is generally more severe than the pedestrian and snow loading, the bridge open for pedestrian use only can generally tolerate a greater amount of corrosion.

6 Closing

The Old Spences Bridge has been evaluated incorporating the recent detailed inspection and considering a variety of live load models.

Repair concepts and cost estimates have been presented for the various live load models, considering immediate reopening and design lives of 10, 25 and 50 years.

6.1 Summary of Load Evaluation

The results of the load evaluation for the various vehicular and pedestrian loadings applied to the bridge in its current state are summarized in Table 29. It should be noted that in the evaluation, two pedestrian load cases have been established in order to satisfy the intent of the code, while at the same time being more representative of local conditions. Load case 1 is pedestrian loading applied to the sidewalk only, in accordance with CHBDC. Load case 2 is pedestrian loading applied anywhere on the bridge, but the loading, as specified by BC MoT, is limited to a maximum of fifty (50) pedestrians. If the bridge is opened as a pedestrian-only bridge, BC MoT must post signage limiting the pedestrian load to a maximum of fifty (50) people on the bridge at any given time.

Table 29: Vertical Load Evaluation Conclusions by Member Type

Item	Conclusions regarding live load models (without snow)				ULS 1d
	CL1-625	25 tonne	5 tonne	Pedestrians on Sidewalk Only	
Concrete Deck [1]	Acceptable	Acceptable	Acceptable	N/A	N/A
Deck Stringers	**Not Acceptable**	Acceptable	Acceptable	Acceptable	Acceptable
Floorbeams	**Not Acceptable**	**Not Acceptable – some in bending**	Acceptable	Acceptable	Acceptable
Sidewalk	**Not Acceptable**	**Not Acceptable**	Acceptable	Acceptable	Acceptable
Truss System	**Not Acceptable**	**Not Acceptable – some diagonals**	Acceptable	Acceptable	Acceptable
Truss Bearings	**Not Acceptable**	Acceptable	Acceptable	Acceptable	Acceptable
Girders	**Not Acceptable**	**Not Acceptable – webs at bearings**	Acceptable	Acceptable	Acceptable

Item	Conclusions regarding live load models (without snow)				ULS 1d
	CL1-625	25 tonne	5 tonne	Pedestrians on Sidewalk Only	
Concrete Piers	**Not Acceptable**	Acceptable	Acceptable	Acceptable	Acceptable
Overall Conclusion	**Not Acceptable**	**Not Acceptable**	Acceptable	Acceptable	Acceptable

Notes: 1. In addition to the conclusion that the strength of the deck is acceptable, there are potentially serviceability issues that may need to be addressed due to gaps that have developed between the stringers and the deck.

In its current condition, the bridge can be opened to 5 tonne vehicle traffic. However, it is recommended that repairs be carried out before the end of 2011 if the bridge is intended to remain in service beyond 2011.

In its current condition, the bridge can be opened as a pedestrian-only bridge, subject to a load limit of fifty (50) pedestrians. However, it is recommended that repairs to some of the concrete piers be carried out by the end of 2011 if the bridge is intended to remain in service beyond 2011.

The CL1-625 loading is not acceptable due to widespread substantial overstress in numerous parts of the floor system, truss system and girders.

In its current condition, the 25 tonne loading is not acceptable due to overstress in numerous locations, including the floorbeams, truss system and girders.

The concrete deck is not capable of spanning longitudinally between floorbeams. This is an indication that there could be extensive cracking in the deck affecting its service life if the rust jacking on top of the floorbeam top flange is not mitigated and the bearing on the stringers reinstated.

The anchor bolts and concrete piers are adequate for resisting thermally induced longitudinal shears, although this is due in large part to the absence of reinforcing steel in the piers. This means that thermally induced loads are easily relieved by cracking of the piers. The cracking of the piers in our estimation is more of design life and serviceability question than a question of strength for vertical load carrying capacity. However, vertical splitting cracks in piers 1 and 3 are of particular interest for strengthening if the bridge is intended to remain in service beyond a 2 year life.

Given that traffic barrier connection does not meet PL-1 requirements, if the bridge is opened to vehicular traffic, it is recommended that BC MoT assess the risks associated with the barrier and establish whether the barrier should be upgraded on both sides of the bridge to a higher standard.

6.2 Summary of Costs for Various Rehabilitation Options

High-level cost estimates have been prepared for the different vehicle loadings considered in the evaluation and for the different rehabilitation design life options. The summary of the estimated costs is listed in Table 30.

Table 30: Summary of Costs for Various Rehabilitation Options

Option	Estimated Cost (2009 dollars)			Comment
	Project Costs: Rehabilitation, Construction & Management	Maintenance Inspections	Total Project Cost[1]	
1. Immediate Demolition	N/A	N/A	**$1.5 M**	
2. Repair				
(a) 2 years @ limited pedestrian	nil	$0.15 M	**$0.15 M**	Does not include costs associated with mitigating seismic and wind risk
(b) 2 years @ 5 tonne	$ 0.55 M (optional barrier repairs)	$ 0.15 M	**$0.15-0.70 M**	
(c) 10 years @ limited pedestrian	$ 0.18 M (pier repairs)	$ 0.60 M (bi-annual detailed)	**$ 0.78 M**	
3. Rehabilitation				
(a) 10 years @ 5 tonne	$1.90 M	$ 1.35 M	**$ 3.25 M**	
(b) 10 years @ 25 tonne	$ 3.29 M	$ 0.36 M	**$ 3.65 M**	
(c) 25 years @ 5 tonne	$ 24.84 M	$ 0.16 M	**$ 25.0 M**	
(d) 50 years @ 5 tonne	$ 26.64 M	$ 0.36 M	**$ 27.0 M**	
(e) 25 years @ 25 tonne	$ 25.34 M	$ 0.16 M	**$ 25.5 M**	
(f) 50 years @ 25 tonne	$ 27.14 M	$ 0.36 M	**$ 27.5 M**	
4. Replacement				
(a) New single lane bridge with sidewalk	$ 14.3 M	N/A	**$ 14.3 M[2]**	Seismic and wind risk mitigated
(b) New two lane bridge with sidewalk	$ 22.7 M	N/A	**$ 22.7 M[2]**	

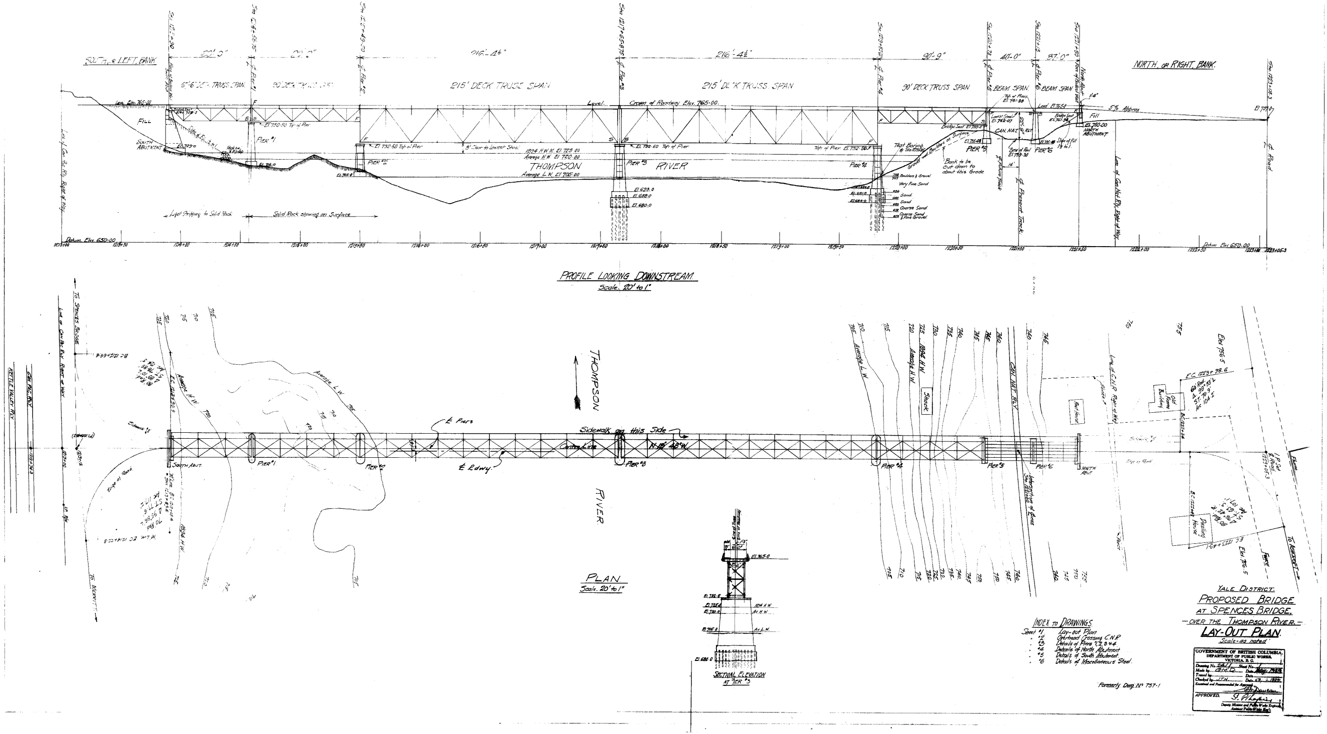
Notes: 1 - For all options except immediate demolition, the life-cycle cost must be increased by $1.5 M to reflect demolition costs.

2 - An allowance of $0.5 M has been made for property acquisition, in the event that a revised location is chosen for the new structure.

Based on the estimated costs of rehabilitating Old Spences Bridge, it does not appear to be cost effective to upgrade the existing bridge beyond a 10 year life. If BC MoT intends to provide this extra crossing between Highway 1 and Highway 8, in addition to the bridge just downstream, replacement of the bridge should be considered within the next 10 years.

It is also noted that opening the bridge for a pedestrian-only crossing is more favourable than a vehicular crossing in terms of cost, public safety as well as confidence in achieving the estimated service life.

Appendix A
General Arrangement Drawing


SOUTH or LEFT BANK
NORTH or RIGHT BANK
90' DECK TRUSS SPAN
215' DECK TRUSS SPAN
215' DECK TRUSS SPAN
90' DECK TRUSS SPAN
BEAM SPAN
BEAM SPAN
216'-4½"
216'-4½"
90'-9"
40'-0"
37'-0"
Crown of Roadway Elev. 765.00
Level
5% Approx.
FILL
SOUTH ABUTMENT
NORTH ABUTMENT
PIER #1
PIER #2
PIER #3
PIER #4
PIER #5
PIER #6
El. 732.50 Top of Pier
1894 H.W.M. El. 725.00
Average H.W. El. 720.00
Average L.W. El. 705.00
THOMPSON
RIVER
Test Boring
Bank to be cut down to about this Grade
CAN. NAT. RLY.
Solid Rock showing on Surface
Datum Elev. 650.00
PROFILE LOOKING DOWNSTREAM
Scale: 20' to 1"
THOMPSON
RIVER
Sidewalk on this Side
Centre Line
N 15° 48' W
℄ Piers
℄ Rdwy.
SOUTH ABUT.
NORTH ABUT.
Average H.W.
Average L.W.
1894 H.W.
Shack
CAN. NAT. RLY.
Old Frame Building
Dwelling House
Line of C.N.R. Right of Way
To SPENCES BRIDGE
To MERRITT
To ASHCROFT
CAN. PAC. RLY.
NICOLA VALLEY RLY.
Line of Can. Pac. Rly. Right of Way
PLAN
Scale: 20' to 1"
SECTIONAL ELEVATION AT PIER #3
El. 765.0
1894 H.W.
Av. H.W.
Av. L.W.
INDEX to DRAWINGS
Sheet #1 Lay-out Plan
" #2 Overhead Crossing C.N.R.
" #3 Details of Piers 1, 2, 3 & 4
" #4 Details of North Abutment
" #5 Details of South Abutment
" #6 Details of Miscellaneous Steel
Formerly Dwg. No. 757-1
YALE DISTRICT.
PROPOSED BRIDGE
AT SPENCES BRIDGE,
—OVER THE THOMPSON RIVER.—
LAY-OUT PLAN.
Scale-as noted
GOVERNMENT OF BRITISH COLUMBIA,
DEPARTMENT OF PUBLIC WORKS,
VICTORIA, B. C.
Drawing No. 2611 Sheet No. 1
Made by G.M.D. Date May 1929
Traced by Date
Checked by J.T.H. Date
Examined and Recommended for Approval
APPROVED.
Deputy Minister and Public Works Engineer
Assistant Public Works Eng'r.

Appendix B
LLCFs (Uncorroded, Original Design Articulation)

Appendix B Index

TABLE B1 - LOAD CAPACITY EVALUATION FOR CONCRETE DECK - ULS COMBINATIONS

BUCKLAND & TAYLOR LTD. Bridge Engineering

Notes:

1. Load rating method is referenced to CSA - S6 - 06, Section 14.
2. Evaluation procedure: ULS Method
3. Highway Class C (as per CSA-S6-06 Clause 1.4.2.2)
4. Evaluation was carried out for the following three live load models.
   CL1 - CL1-625 Truck or Lane Load traffic;
   25T - 25t review vehicle or Lane Load traffic;
   5T - 5t passenger vehicle;
5. Inspection Level considered: "INSP3" for all structural components
6. Target reliability index from Table 14.5.
7. Dead load factors from Table 14.7. and 3.2.
8. Live load factors are from: - Table 14.8, for normal traffic (CL1-625) and pedestrain load.
   - Table 14.9, for normal traffic (alternative loading)
9. Resistance adjustment factor from Table 14.15.
10. Live load capacity factor as per Clause 14.15.2.2.1.
11. Material strength: fy = 210 MPa, fu = 420 MPa for Structural steel
    fu = 320 MPa for Rivet
    fc' = 15 MPa for Reinforced concrete
    fy = 230 MPa for Reinforcing steel

Elt. #	Element - Force effect	Effect Units	Target reliability index				Live load							Resistance		LLCF
			Syst Behav	Elem Behav	Insp Level	Beta	LL Model	Lat. Distr.	Type span	Unfact. Wheel Loads	Load factor	DLA factor	Fact. Loads L_{wf}	Fact Resist	Adjust Fact	
1	Concrete deck at truss span	Vmax	S3	E3	INSP3	2.50	CL1	-	All	88	1.35	0.40	165	168	1.00	1.02
	Punching shear	[kN]					25T	-	Short	45	1.80	0.40	112			1.50
							5T	-	Short	17	1.80	0.40	42			4.04

Note: ALL in "Type Span" Column indicates that the live load factor is applicable to all span types (Section 14.13.3, CAN/CSA S6-06).
DLA = 0 indicates lane load governs
DLA > 0 indicates truck load governs

Note: An additional evaluation was performed for one way positive moment of a concrete deck of 3 m width.
This was addressed because the field inspection indicated a departure between the conrecte deck and the top flange of the stringers due to rust jacking.
The observations of the additional evaluation is as below:
(1) The concrete deck as a one way beam does not satisfy the minimum reinforcement ratio requirement.
(2) The concrete deck as a one way beam can not even hold its self weight, i.e. LLCF < 0 for any traffic load.

TABLE B2 - LOAD CAPACITY EVALUATION FOR DECK STRINGERS - ULS COMBINATIONS

BUCKLAND & TAYLOR LTD.
Bridge Engineering

Notes:

1. Load rating method is referenced to CSA - S6 - 06, Section 14.
2. Evaluation procedure: ULS Method
3. Highway Class C (as per CSA-S6-06 Clause 1.4.2.2)
4. Evaluation was carried out for the following three live load models.
   CL1 - CL1-625 Truck or Lane Load traffic;
   25T - 25t review vehicle or Lane Load traffic;
   5T - 5t passenger vehicle;
   PED - Pedestrian loading only.
5. Inspection Level considered: "INSP3" for all structural components
6. Target reliability index from Table 14.5.
7. Dead load factors from Table 14.7. and 3.2.
8. Live load factors are from: - Table 14.8, for normal traffic (CL1-625) and pedestrain load.
   - Table 14.9, for normal traffic (alternative loading)
9. Resistance adjustment factor from Table 14.15.
10. Live load capacity factor as per Clause 14.15.2.1.
11. Material strength: fy = 210 MPa, fu = 420 MPa for Structural steel
    fu = 320 MPa for Rivet
    fc' = 15 MPa for Reinforced concrete
    fy = 230 MPa for Reinforcing steel

UNCORRODED

Elt. #	Element - Force effect	Effect Units	Target reliability index: Syst Behav	Elem Behav	Insp Level	Beta	Dead load: Unfact. loads D1	D2	Load factors D1	D2	Fact. loads D1	D2	Snow load: Unfact. Loads	Load factor ULS1a	ULS1d	Fact.Loads ULS1a	ULS1d	Live load: LL Model	Lat. Distr.	Type span	Unfact. Loads	Load factor	DLA factor	Fact. Loads	Resistance: Fact Resist	Adjust Fact	Live Load Capacity Factor: ULS1a (w/ snow)	ULS1b (w/o snow)	C/D ULS1d & ULS9 (w/o live)
1	Deck stringer at truss span	Mmax	S3	E3	INSP3	2.50	3	22	1.05	1.10	3	24	0	1.50	0	0	0	CL1	Simplified	All	106	1.35	0.30	186	188	1.00	0.86	0.86	-
	(I CB14@36)	[kN.m]							1.05	1.10	3	24	0	1.50	0	0	0	25T	Simplified	Short	69	1.80	0.30	161			1.00	1.00	-
	Positive moment								1.05	1.10	3	24	0	1.50	0	0	0	5T	Simplified	Short	27	1.80	0.00	49			3.30	3.30	-
	near midspan								1.05	1.10	3	24	-	-	-	-	-	Ped.	-	All	20	1.35	0.00	27			-	6.05	-
									1.05	1.10	3	24	9	-	1.50	-	13	-	-	-	-	-	-	-			-	-	4.62
									1.35	1.35	4	30	-	-	-	-	-	-	-	-	-	-	-	-			-	-	5.55
2	Deck stringer at truss span	Vmax	S3	E3	INSP3	2.50	2	13	1.05	1.10	2	14	0	1.50	0	0	0	CL1	Simplified	All	73	1.35	0.30	128	265	1.02	1.98	1.98	-
	(I CB14@36)	[kN]							1.05	1.10	2	14	0	1.50	0	0	0	25T	Simplified	Other	45	1.35	0.30	79			3.21	3.21	-
	Web shear at floor beam support								1.05	1.10	2	14	0	1.50	0	0	0	5T	Simplified	Other	17	1.35	0.00	23			10.88	10.88	-
									1.05	1.10	2	14	-	-	-	-	-	Ped.	-	All	12	1.35	0.00	16			-	16.36	-
									1.05	1.10	2	14	5	-	1.50	-	8	-	-	-	-	-	-	-			-	-	11.35
									1.35	1.35	2	17	-	-	-	-	-	-	-	-	-	-	-	-			-	-	13.64
3	Deck stringer at truss span	Vmax	S3	E1	INSP3	3.25	2	13	1.08	1.16	2	15	0	1.50	0	0	0	CL1	Simplified	All	73	1.56	0.30	148	235	1.20	1.79	1.79	-
	(I CB14@36)	[kN]							1.08	1.16	2	15	0	1.50	0	0	0	25T	Simplified	Other	45	1.56	0.30	91			2.90	2.90	-
	Web connection to floor beam								1.08	1.16	2	15	0	1.50	0	0	0	5T	Simplified	Other	17	1.56	0.00	27			9.83	9.83	-
									1.08	1.16	2	15	-	-	-	-	-	Ped.	-	All	12	1.56	0.00	18			-	14.79	-
									1.08	1.16	2	15	5	-	1.50	-	8	-	-	-	-	-	-	-			-	-	11.46
									1.35	1.35	2	17	-	-	-	-	-	-	-	-	-	-	-	-			-	-	14.25
4	Deck stringer at girder span	Mmax	S3	E3	INSP3	2.50	0	5	1.05	1.10	1	5	0	1.50	0	0	0	CL1	Simplified	All	43	1.35	0.40	82	92	1.00	1.05	1.05	-
	(I 10@25.4)	[kN.m]							1.05	1.10	1	5	0	1.50	0	0	0	25T	Simplified	Short	29	1.80	0.30	67			1.29	1.29	-
	Positive moment								1.05	1.10	1	5	0	1.50	0	0	0	5T	Simplified	Short	8	1.80	0.40	20			4.22	4.22	-
	near midspan								1.05	1.10	1	5	-	-	-	-	-	Ped.	-	All	6	1.35	0.00	7			-	11.60	-
									1.05	1.10	1	5	2	-	1.50	-	3	-	-	-	-	-	-	-			-	-	10.63
									1.35	1.35	1	6	-	-	-	-	-	-	-	-	-	-	-	-			-	-	13.14
5	Deck stringer at girder span	Mmax	S3	E3	INSP3	2.50	1	6	1.05	1.10	1	6	0	1.50	0	0	0	CL1	Simplified	All	31	1.35	0.30	55	71	1.04	1.18	1.18	-
	(I 10@25.4)	[kN.m]							1.05	1.10	1	6	0	1.50	0	0	0	25T	Simplified	Short	18	1.80	0.30	41			1.51	1.51	-
	Negative moment								1.05	1.10	1	6	0	1.50	0	0	0	5T	Simplified	Short	8	1.80	0.00	15			4.53	4.53	-
	over floorbeam *								1.05	1.10	1	6	-	-	-	-	-	Ped.	-	All	6	1.35	0.00	8			-	8.00	-
									1.05	1.10	1	6	2	-	1.50	-	4	-	-	-	-	-	-	-			-	-	6.93
									1.35	1.35	1	8	-	-	-	-	-	-	-	-	-	-	-	-			-	-	8.56
6	Deck stringer at girder span	Vmax	S3	E3	INSP3	2.50	1	9	1.05	1.10	1	10	0	1.50	0	0	0	CL1	Simplified	All	68	1.35	0.30	119	231	1.02	1.90	1.90	-
	(I 10@25.4)	[kN]							1.05	1.10	1	10	0	1.50	0	0	0	25T	Simplified	Short	44	1.80	0.30	103			2.19	2.19	-
	Web shear at floor beam support								1.05	1.10	1	10	0	1.50	0	0	0	5T	Simplified	Short	12	1.80	0.30	27			8.22	8.22	-
									1.05	1.10	1	10	-	-	-	-	-	Ped.	-	All	8	1.35	0.00	11			-	20.09	-
									1.05	1.10	1	10	4	-	1.50	-	5	-	-	-	-	-	-	-			-	-	14.78
									1.35	1.35	1	12	-	-	-	-	-	-	-	-	-	-	-	-			-	-	18.25
7	Deck stringer at girder span	Bmax	S3	E1	INSP3	3.25	2	16	1.08	1.16	2	18	0	1.50	0	0	0	CL1	Simplified	All	73	1.56	0.30	148	292	1.00	1.84	1.84	-
	(I 10@25.4)	[kN]							1.08	1.16	2	18	0	1.50	0	0	0	25T	Simplified	Short	47	2.10	0.30	130			2.10	2.10	-
	Compression in web								1.08	1.16	2	18	0	1.50	0	0	0	5T	Simplified	Short	20	2.10	0.00	42			6.45	6.45	-
	over floorbeam								1.08	1.16	2	18	-	-	-	-	-	Ped.	-	All	16	1.56	0.00	25			-	10.84	-
									1.08	1.16	2	18	7	-	1.50	-	10	-	-	-	-	-	-	-			-	-	9.66
									1.35	1.35	2	21	-	-	-	-	-	-	-	-	-	-	-	-			-	-	12.35
8	Deck stringer at girder span	Bmax	S3	E1	INSP3	3.25	1	6	1.08	1.16	1	7	0	1.50	0	0	0	CL1	Simplified	All	61	1.56	0.30	124	182	1.00	1.41	1.41	-
	(I 10@25.4)	[kN]							1.08	1.16	1	7	0	1.50	0	0	0	25T	Simplified	Short	39	2.10	0.30	108			1.62	1.62	-
	Comp. in web over								1.08	1.16	1	7	0	1.50	0	0	0	5T	Simplified	Short	11	2.10	0.30	30			5.89	5.89	-
	concrete at piers								1.08	1.16	1	7	-	-	-	-	-	Ped.	-	All	6	1.56	0.00	10			-	18.35	-
									1.08	1.16	1	7	2	-	1.50	-	4	-	-	-	-	-	-	-			-	-	16.48
									1.35	1.35	1	8	-	-	-	-	-	-	-	-	-	-	-	-			-	-	21.06

Note: ALL in "Type Span" Column indicates that the live load factor is applicable to all span types (Section 14.13.3, CAN/CSA S6-06).
DLA = 0 indicates lane load governs
DLA > 0 indicates truck load governs
LLCFs that are circled in this Table are recomputed in Section 4 of this report to investigate the effects of capacity loss due to corrosion or damage
* Two possibilities were considered here: Maximum momont alone; Or conservative interaction of the max. moment and max. shear which may not be concurrent

■ GOVERNING LL CAPACITY FACTOR

CL1	0.86	0.86	-
25T	1.00	1.00	-
5T	3.30	3.30	-
Ped.	-	6.05	-
	-	-	4.62
	-	-	5.55

TABLE B3 - LOAD CAPACITY EVALUATION FOR FLOOR BEAMS - ULS COMBINATIONS

BUCKLAND & TAYLOR LTD. Bridge Engineering

Notes:
1. Load rating method is referenced to CSA - S6 - 06, Section 14.
2. Evaluation procedure: ULS Method
3. Highway Class C (as per CSA-S6-06 Clause 1.4.2.2)
4. Evaluation was carried out for the following three live load models.
   CL1 - CL1-625 Truck or Lane Load traffic;
   25T - 25t review vehicle or Lane Load traffic;
   5T - 5t passenger vehicle;
   PED - Pedestrian loading only.
5. Inspection Level considered: "INSP3" for all structural components
6. Target reliability index from Table 14.5.
7. Dead load factors from Table 14.7. and 3.2.
8. Live load factors are from: - Table 14.8, for normal traffic (CL1-625) and pedestrain load.
   - Table 14.9, for normal traffic (alternative loading)
9. Resistance adjustment factor from Table 14.15.
10. Live load capacity factor as per Clause 14.15.2.1.
11. Material strength: fy = 210 MPa, fu = 420 MPa for Structural steel
    fu = 320 MPa for Rivet
    fc' = 15 MPa for Reinforced concrete
    fy = 230 MPa for Reinforcing steel

UNCORRODED

Elt. #	Element - Force effect	Effect Units	Target reliability index: Syst Behav	Elem Behav	Insp Level	Beta	Dead load: Unfact. loads D1	D2	Load factors D1	D2	Fact. loads D1	D2	Snow load: Unfact. Loads	Load factor ULS1a	ULS1d	Fact.Loads ULS1a	ULS1d	Live load: LL Model	Lat. Distr.	Type span	Unfact. Loads	Load factor	DLA factor	Fact. Loads	Resistance: Fact Resist	Adjust Fact	Live Load Capacity Factor: ULS1a (w/ snow)	ULS1b (w/o snow)	C/D ULS1d & ULS9 (w/o live)
1	Floor beam at truss span	Mmax	S2	E3	INSP3	2.75	15	83	1.06	1.12	16	93	33	1.50	0	50	0	CL1	Static	All	195	1.42	0.30	360	386	1.00	0.63	0.77	-
	(I CB18@58)	[kN.m]							1.06	1.12	16	93	33	1.50	0	50	0	25T	Static	Other	124	1.42	0.30	229			0.99	1.21	-
	Positive moment at midspan								1.06	1.12	16	93	33	1.50	0	50	0	5T	Static	Other	58	1.42	0.00	83			2.73	3.34	-
									1.06	1.12	16	93	-	-	-	-	-	Ped.	-	All	48	1.42	0.00	68			-	4.08	-
									1.06	1.12	16	93	34	-	1.50	-	51	-	-	-	-	-	-	-			-	-	2.40
									1.35	1.35	20	112	-	-	-	-	-	-	-	-	-	-	-	-			-	-	2.91
2	Floor beam at truss span	Vmax	S2	E3	INSP3	2.75	13	70	1.06	1.12	14	79	38	1.50	0	57	0	CL1	Static	All	196	1.42	0.30	362	534	1.02	1.09	1.25	-
	(I CB18@58)	[kN]							1.06	1.12	14	79	38	1.50	0	57	0	25T	Static	Other	125	1.42	0.30	230			1.72	1.97	-
	Web shear at floor beam								1.06	1.12	14	79	38	1.50	0	57	0	5T	Static	Other	66	1.42	0.00	93			4.24	4.84	-
	support								1.06	1.12	14	79	-	-	-	-	-	Ped.	-	All	40	1.42	0.00	57			-	7.89	-
									1.06	1.12	14	79	29	-	1.50	-	44	-	-	-	-	-	-	-			-	-	4.00
									1.35	1.35	17	95	-	-	-	-	-	-	-	-	-	-	-	-			-	-	4.85
3	Floor beam at truss span	Bmax	S2	E1	INSP3	3.50	13	70	1.09	1.18	14	83	38	1.50	0	57	0	CL1	Static	All	196	1.63	0.30	416	752	1.00	1.44	1.58	-
	(I CB18@58)	[kN]							1.09	1.18	14	83	38	1.50	0	57	0	25T	Static	Other	125	1.63	0.30	264			2.27	2.48	-
	Compression in web over								1.09	1.18	14	83	38	1.50	0	57	0	5T	Static	Other	66	1.63	0.00	107			5.58	6.11	-
	truss chord								1.09	1.18	14	83	-	-	-	-	-	Ped.	-	All	40	1.63	0.00	66			-	9.94	-
									1.09	1.18	14	83	29	-	1.50	-	44	-	-	-	-	-	-	-			-	-	5.35
									1.35	1.35	17	95	-	-	-	-	-	-	-	-	-	-	-	-			-	-	6.70
4	Intermediate FB at girder span	Mmax	S2	E3	INSP3	2.75	7	49	1.06	1.12	7	55	20	1.50	0	30	0	CL1	Static	All	181	1.42	0.30	334	330	1.00	0.71	0.80	-
	(I CB18@51)	[kN.m]							1.06	1.12	7	55	20	1.50	0	30	0	25T	Static	Short	118	1.90	0.30	291			0.82	0.92	-
	Positive moment								1.06	1.12	7	55	20	1.50	0	30	0	5T	Static	Short	50	1.90	0.00	95			2.51	2.83	-
	near midspan								1.06	1.12	7	55	-	-	-	-	-	Ped.	-	All	40	1.42	0.00	57			-	4.67	-
									1.06	1.12	7	55	21	-	1.50	-	31	-	-	-	-	-	-	-			-	-	3.54
									1.35	1.35	9	66	-	-	-	-	-	-	-	-	-	-	-	-			-	-	4.38
5	Intermediate FB at girder span	Vmax	S2	E3	INSP3	2.75	6	34	1.06	1.12	6	38	17	1.50	0	26	0	CL1	Static	All	152	1.42	0.30	280	502	1.02	1.58	1.67	-
	(I CB18@51)	[kN]							1.06	1.12	6	38	17	1.50	0	26	0	25T	Static	Short	99	1.90	0.30	244			1.81	1.92	-
	Web shear at end support								1.06	1.12	6	38	17	1.50	0	26	0	5T	Static	Short	42	1.90	0.00	79			5.59	5.91	-
									1.06	1.12	6	38	-	-	-	-	-	Ped.	-	All	34	1.42	0.00	48			-	9.76	-
									1.06	1.12	6	38	14	-	1.50	-	21	-	-	-	-	-	-	-			-	-	7.84
									1.35	1.35	7	46	-	-	-	-	-	-	-	-	-	-	-	-			-	-	9.60
6	Intermediate FB at girder span	Vmax	S2	E1	INSP3	3.50	6	34	1.09	1.18	6	40	17	1.50	0	26	0	CL1	Static	All	152	1.63	0.30	321	421	1.81	2.15	2.23	-
	(I CB18@51)	[kN]							1.09	1.18	6	40	17	1.50	0	26	0	25T	Static	Short	99	2.20	0.30	283			2.44	2.53	-
	Web connection at end support								1.09	1.18	6	40	17	1.50	0	26	0	5T	Static	Short	42	2.20	0.00	92			7.51	7.80	-
									1.09	1.18	6	40	-	-	-	-	-	Ped.	-	All	34	1.63	0.00	55			-	12.98	-
									1.09	1.18	6	40	14	-	1.50	-	21	-	-	-	-	-	-	-			-	-	11.26
									1.35	1.35	7	46	-	-	-	-	-	-	-	-	-	-	-	-			-	-	14.27
7	Intermediate FB at girder span	Bmax	S2	E1	INSP3	3.50	2	16	1.09	1.18	2	19	12	1.50	0	17	0	CL1	Static	All	73	1.63	0.30	154	341	1.00	1.96	2.07	-
	(I CB18@51)	[kN]							1.09	1.18	2	19	12	1.50	0	17	0	25T	Static	Short	48	2.20	0.30	136			2.23	2.36	-
	Compression in web								1.09	1.18	2	19	12	1.50	0	17	0	5T	Static	Short	20	2.20	0.00	44			6.85	7.25	-
	below stringer								1.09	1.18	2	19	-	-	-	-	-	Ped.	-	All	16	1.63	0.00	26			-	12.21	-
									1.09	1.18	2	19	7	-	1.50	-	10	-	-	-	-	-	-	-			-	-	11.16
									1.35	1.35	2	21	-	-	-	-	-	-	-	-	-	-	-	-			-	-	14.43

Note: ALL in "Type Span" Column indicates that the live load factor is applicable to all span types (Section 14.13.3, CAN/CSA S6-06).
DLA = 0 indicates lane load governs
DLA > 0 indicates truck load governs
LLCFs that are circled in this Table are recomputed in Section 4 of this report to investigate the effects of capacity loss due to corrosion or damage

■ GOVERNING LL CAPACITY FACTOR

	ULS1a	ULS1b	ULS1d & ULS9
CL1	0.63	0.77	-
25T	0.82	0.92	-
5T	2.51	2.83	-
Ped.	-	4.08	-
	-	-	2.40
	-	-	2.91

TABLE B4 - LOAD CAPACITY EVALUATION FOR SIDEWALK (STRINGERS AND BRACKETS) - ULS COMBINATIONS

BUCKLAND & TAYLOR LTD. Bridge Engineering

Notes:

1. Load rating method is referenced to CSA - S6 - 06, Section 14.
2. Evaluation procedure: ULS Method
3. Highway Class C (as per CSA-S6-06 Clause 1.4.2.2)
4. Evaluation was carried out for the following three live load models.
   25T - 25t review vehicle or Lane Load traffic;
   5T - 5t passenger vehicle;
   PED - Pedestrian loading only.
   Note - ULS 1a and ULS 1d were not included in this table because ULS 1c governs them
   as the lowest factored pedestrain load = 1.35*4 kPa = 5.4 kPa
   > the largest factored snow = 1.5 * 3.1 kPa = 4.65 kPa
5. Inspection Level considered: "INSP3" for all structural components
6. Target reliability index from Table 14.5.
7. Dead load factors from Table 14.7. and 3.2.
8. Live load factors are from: - Table 14.8, for normal traffic (CL1-625) and pedestrain load.
   - Table 14.9, for normal traffic (alternative loading)
9. Resistance adjustment factor from Table 14.15.
10. Live load capacity factor as per Clause 14.15.2.1.
11. Material strength: fy = 210 MPa, fu = 420 MPa for Structural steel
    fu = 320 MPa for Rivet
    fc' = 15 MPa for Reinforced concrete
    fy = 230 MPa for Reinforcing steel

UNCORRODED

Elt. #	Element - Force effect	Effect Units	Target reliability index: Syst Behav	Elem Behav	Insp Level	Beta	Dead load: Unfact. loads D1	Unfact. loads D2	Load factors D1	Load factors D2	Fact. loads D1	Fact. loads D2	Snow load: Unfact. Loads	Load factor ULS1a	Load factor ULS1d	Fact.Loads ULS1a	Fact.Loads ULS1d	Live load: LL Model	Lat. Distr.	Type span	Unfact. Loads	Load factor	DLA factor	Fact. Loads	Resistance: Fact Resist	Adjust Fact	w/ snow: Live Load Capacity Factor ULS1a	w/o snow: ULS1b/c	w/o live: C/D ULS9
1	Sidewalk stringer at truss span	Mmax	S3	E3	INSP3	2.50	2	11	1.05	1.10	2	12	-	-	-	-	-	25T	Static	Short	80	1.80	0.30	187	107	1.00	-	0.49	-
	(I 12@25)	[kN.m]							1.05	1.10	2	12	-	-	-	-	-	5T	Static	Short	32	1.80	0.00	57			-	1.63	-
	Positive moment near midspan								1.05	1.10	2	12	-	-	-	-	-	PED	-	All	18	1.35	0.00	24			-	3.84	-
									1.05	1.10	2	12	-	-	-	-	-	-	-	-	-	-	-	-			-	-	-
									1.35	1.35	3	15	-	-	-	-	-	-	-	-	-	-	-	-			-	-	6.09
2	Sidewalk stringer at truss span	Vmax	S3	E3	INSP3	2.50	1	6	1.05	1.10	1	7	-	-	-	-	-	25T	Static	Other	53	1.35	0.30	92	241	1.02	-	2.57	-
	(I 12@25)	[kN]							1.05	1.10	1	7	-	-	-	-	-	5T	Static	Other	20	1.35	0.00	27			-	8.70	-
	Web shear at floor beam								1.05	1.10	1	7	-	-	-	-	-	PED	-	All	10	1.35	0.00	14			-	16.90	-
	support								1.05	1.10	1	7	-	-	-	-	-	-	-	-	-	-	-	-			-	-	-
									1.35	1.35	2	9	-	-	-	-	-	-	-	-	-	-	-	-			-	-	23.63
3	Sidewalk stringer at truss span	Vmax	S3	E1	INSP3	3.25	1	6	1.08	1.16	1	7	-	-	-	-	-	25T	Static	Other	53	1.56	0.30	107	106	1.00	-	0.90	-
	(I 12@25)	[kN]							1.08	1.16	1	7	-	-	-	-	-	5T	Static	Other	20	1.56	0.00	31			-	3.11	-
	Compression in web								1.08	1.16	1	7	-	-	-	-	-	PED	-	All	10	1.56	0.00	16			-	5.99	-
	over floorbeam								1.08	1.16	1	7	-	-	-	-	-	-	-	-	-	-	-	-			-	-	-
									1.35	1.35	2	9	-	-	-	-	-	-	-	-	-	-	-	-			-	-	10.20
4	Sidewalk floor beam at truss span	Mmax	S2	E3	INSP3	2.75	6	27	1.06	1.12	7	30	-	-	-	-	-	25T	Static	Other	61	1.42	0.30	112	386	1.00	-	3.03	-
	(I CB18@58)	[kN.m]							1.06	1.12	7	30	-	-	-	-	-	5T	Static	Other	31	1.42	0.00	44			-	7.75	-
	Negative moment								1.06	1.12	7	30	-	-	-	-	-	PED	-	All	44	1.42	0.00	63			-	5.41	-
	over truss chord *								1.06	1.12	7	30	-	-	-	-	-	-	-	-	-	-	-	-			-	-	-
									1.35	1.35	9	37	-	-	-	-	-	-	-	-	-	-	-	-			-	-	8.56
5	Sidewalk floor beam at truss span	Vmax	S2	E3	INSP3	2.75	6	26	1.06	1.12	7	29	-	-	-	-	-	25T	Static	Other	56	1.42	0.30	103	534	1.02	-	4.93	-
	(I CB18@58)	[kN]							1.06	1.12	7	29	-	-	-	-	-	5T	Static	Other	28	1.42	0.00	40			-	12.81	-
	Web shear at truss								1.06	1.12	7	29	-	-	-	-	-	PED	-	All	41	1.42	0.00	58			-	8.79	-
	chord support								1.06	1.12	7	29	-	-	-	-	-	-	-	-	-	-	-	-			-	-	-
									1.35	1.35	9	35	-	-	-	-	-	-	-	-	-	-	-	-			-	-	12.61
6	Sidewalk stringer at girder span	Mmax	S3	E3	INSP3	2.50	0	2	1.05	1.10	0	3	-	-	-	-	-	25T	Static	Short	33	1.80	0.30	78	56	1.04	-	0.71	-
	(I 9@21.8)	[kN.m]							1.05	1.10	0	3	-	-	-	-	-	5T	Static	Short	10	1.80	0.40	24			-	2.32	-
	Positive moment near midspan								1.05	1.10	0	3	-	-	-	-	-	PED	-	All	6	1.35	0.00	8			-	7.35	-
									1.05	1.10	0	3	-	-	-	-	-	-	-	-	-	-	-	-			-	-	-
									1.35	1.35	1	3	-	-	-	-	-	-	-	-	-	-	-	-			-	-	15.52
7	Sidewalk stringer at girder span	Mmax	S3	E3	INSP3	2.50	1	3	1.05	1.10	1	3	-	-	-	-	-	25T	Static	Short	21	1.80	0.30	48	56	1.04	-	1.08	-
	(I 9@21.8)	[kN.m]							1.05	1.10	1	3	-	-	-	-	-	5T	Static	Short	10	1.80	0.00	17			-	3.17	-
	Negative moment over bracket *								1.05	1.10	1	3	-	-	-	-	-	PED	-	All	6	1.35	0.00	9			-	6.45	-
									1.05	1.10	1	3	-	-	-	-	-	-	-	-	-	-	-	-			-	-	-
									1.35	1.35	1	4	-	-	-	-	-	-	-	-	-	-	-	-			-	-	12.41
8	Sidewalk stringer at girder span	Vmax	S3	E3	INSP3	2.50	1	5	1.05	1.10	1	5	-	-	-	-	-	25T	Static	Short	51	1.80	0.30	120	194	1.02	-	1.61	-
	(I 9@21.8)	[kN]							1.05	1.10	1	5	-	-	-	-	-	5T	Static	Short	14	1.80	0.30	32			-	6.00	-
	Web shear at bracket support								1.05	1.10	1	5	-	-	-	-	-	PED	-	All	9	1.35	0.00	11			-	16.77	-
									1.05	1.10	1	5	-	-	-	-	-	-	-	-	-	-	-	-			-	-	-
									1.35	1.35	1	6	-	-	-	-	-	-	-	-	-	-	-	-			-	-	27.70
9	Sidewalk stringer at girder span	Bmax	S3	E1	INSP3	3.25	1	8	1.08	1.16	2	10	-	-	-	-	-	25T	Static	Short	55	2.10	0.30	151	272	1.00	-	1.73	-
	(I 9@21.8)	[kN]							1.08	1.16	2	10	-	-	-	-	-	5T	Static	Short	23	2.10	0.00	49			-	5.31	-
	Comp. in web over								1.08	1.16	2	10	-	-	-	-	-	PED	-	All	16	1.56	0.00	26			-	10.20	-
	interior bracket								1.08	1.16	2	10	-	-	-	-	-	-	-	-	-	-	-	-			-	-	-
									1.35	1.35	2	11	-	-	-	-	-	-	-	-	-	-	-	-			-	-	20.99
10	Sidewalk stringer at girder span	Bmax	S3	E1	INSP3	3.25	1	3	1.08	1.16	1	3	-	-	-	-	-	25T	Static	Short	46	2.10	0.30	126	158	1.00	-	1.23	-
	(I 9@21.8)	[kN]							1.08	1.16	1	3	-	-	-	-	-	5T	Static	Short	13	2.10	0.30	35			-	4.45	-
	Comp. in web over end bracket								1.08	1.16	1	3	-	-	-	-	-	PED	-	All	6	1.56	0.00	10			-	15.96	-
									1.08	1.16	1	3	-	-	-	-	-	-	-	-	-	-	-	-			-	-	-
									1.35	1.35	1	4	-	-	-	-	-	-	-	-	-	-	-	-			-	-	33.52

TABLE B4 - LOAD CAPACITY EVALUATION FOR SIDEWALK (STRINGERS AND BRACKETS) - ULS COMBINATIONS

BUCKLAND & TAYLOR LTD. Bridge Engineering

Notes:

1. Load rating method is referenced to CSA - S6 - 06, Section 14.
2. Evaluation procedure: ULS Method
3. Highway Class C (as per CSA-S6-06 Clause 1.4.2.2)
4. Evaluation was carried out for the following three live load models.
   25T - 25t review vehicle or Lane Load traffic;
   5T - 5t passenger vehicle;
   PED - Pedestrian loading only.
   Note - ULS 1a and ULS 1d were not included in this table because ULS 1c governs them
   as the lowest factored pedestrain load = 1.35*4 kPa = 5.4 kPa
   > the largest factored snow = 1.5 * 3.1 kPa = 4.65 kPa
5. Inspection Level considered: "INSP3" for all structural components
6. Target reliability index from Table 14.5.
7. Dead load factors from Table 14.7. and 3.2.
8. Live load factors are from: - Table 14.8, for normal traffic (CL1-625) and pedestrain load.
   - Table 14.9, for normal traffic (alternative loading)
9. Resistance adjustment factor from Table 14.15.
10. Live load capacity factor as per Clause 14.15.2.1.
11. Material strength: fy = 210 MPa, fu = 420 MPa for Structural steel
    fu = 320 MPa for Rivet
    fc' = 15 MPa for Reinforced concrete
    fy = 230 MPa for Reinforcing steel

UNCORRODED

Elt. #	Element - Force effect	Effect Units	Target reliability index				Dead load						Snow load					Live load							Resistance		w/ snow	w/o snow	w/o live
			Syst Behav	Elem Behav	Insp Level	Beta	Unfact. loads		Load factors		Fact. loads		Unfact. Loads	Load factor		Fact.Loads		LL Model	Lat. Distr.	Type span	Unfact. Loads	Load factor	DLA factor	Fact. Loads	Fact Resist	Adjust Fact	Live Load Capacity Factor		C/D ULS9
							D1	D2	D1	D2	D1	D2		ULS1a	ULS1d	ULS1a	ULS1d										ULS1a	ULS1b/c	
11	Sidewalk bracket at girder span	Mmax	S2	E3	INSP3	2.75	2	7	1.06	1.12	2	7	-	-	-	-	-	25T	Static	Other	63	1.42	0.30	116	133	1.01	-	1.08	-
		[kN.m]							1.06	1.12	2	7	-	-	-	-	-	5T	Static	Other	27	1.42	0.00	38			-	3.32	-
	Negative moment at girder support *								1.06	1.12	2	7	-	-	-	-	-	PED	-	All	14	1.42	0.00	19			-	6.42	-
									1.06	1.12	2	7	-	-	-	-	-	-	-	-	-	-	-	-			-	-	-
									1.35	1.35	3	9	-	-	-	-	-	-	-	-	-	-	-	-			-	-	11.57
12	Sidewalk bracket at girder span	Vmax	S2	E3	INSP3	2.75	2	6	1.06	1.12	2	7	-	-	-	-	-	25T	Static	Other	55	1.42	0.30	102	569	1.02	-	5.58	-
		[kN]							1.06	1.12	2	7	-	-	-	-	-	5T	Static	Other	23	1.42	0.00	33			-	17.18	-
	Web shear at girder support								1.06	1.12	2	7	-	-	-	-	-	PED	-	All	12	1.42	0.00	18			-	32.42	-
									1.06	1.12	2	7	-	-	-	-	-	-	-	-	-	-	-	-			-	-	-
									1.35	1.35	3	8	-	-	-	-	-	-	-	-	-	-	-	-			-	-	51.76
13	Sidewalk bracket at girder span	Bmax	S2	E1	INSP3	3.50	1	3	1.09	1.18	1	4	-	-	-	-	-	25T	Static	Other	55	1.63	0.30	117	177	1.00	-	1.47	-
		[kN]							1.09	1.18	1	4	-	-	-	-	-	5T	Static	Other	23	1.63	0.00	38			-	4.52	-
	Comp. in web below stringer								1.09	1.18	1	4	-	-	-	-	-	PED	-	All	6	1.63	0.00	10			-	17.07	-
									1.09	1.18	1	4	-	-	-	-	-	-	-	-	-	-	-	-			-	-	-
									1.35	1.35	2	4	-	-	-	-	-	-	-	-	-	-	-	-			-	-	31.75

Note: ALL in "Type Span" Column indicates that the live load factor is applicable to all span types (Section 14.13.3, CAN/CSA S6-06).
DLA = 0 indicates lane load governs
DLA > 0 indicates truck load governs
The web connection and flange connection of the brackets to the girder were not included in the table because calculation indicated they are not governing.
LLCFs that are circled in this Table are recomputed in Section 4 of this report to investigate the effects of capacity loss due to corrosion or damage
* Two possibilities were considered here: Maximum momont alone; Or conservative interaction of the max. moment and max. shear which may not be concurrent

■ GOVERNING LL CAPACITY FACTOR

25T	-	0.49	-
5T	-	1.63	-
PED	-	3.84	-
	-	-	-
	-	-	6.09

TABLE B5 - LOAD CAPACITY EVALUATION FOR TRUSS SYSTEM SPAN 1- ULS COMBINATIONS

BUCKLAND & TAYLOR LTD.
Bridge Engineering

Notes:
1. Load rating method is referenced to CSA - S6 - 06, Section 14.
2. Evaluation procedure: ULS Method
3. Highway Class C (as per CSA-S6-06 Clause 1.4.2.2)
4. Evaluation was carried out for the following three live load models.
   CL1 - CL1-625 Truck or Lane Load traffic;
   25T - 25t review vehicle or Lane Load traffic;
   5T - 5t passenger vehicle or Lane Load traffic.
5. Inspection Level considered: "INSP3" for all structural components
6. Target reliability index from Table 14.5.
7. Dead load factors from Table 14.7. and 3.2.
8. Live load factors are from: - Table 14.8, for normal traffic (CL1-625) and pedestrain load.
   - Table 14.9, for normal traffic (alternative loading)
9. Resistance adjustment factor from Table 14.15.
10. Live load capacity factor as per Clause 14.15.2.1.
11. Material strength: fy = 210 MPa, fu = 420 MPa for Structural steel
    fu = 320 MPa for Rivet
    fc' = 15 MPa for Reinforced concrete
    fy = 230 MPa for Reinforcing steel

UNCORRODED

Elt. #	Element - Force effect	Effect Units	Target reliability index: Syst Behav	Elem Behav	Insp Level	Beta	Dead load: Unfact. loads D1	D2	Load factors D1	D2	Fact. loads D1	D2	Snow load: Unfact. Loads	Load factor ULS1a	ULS1d	Fact.Loads ULS1a	ULS1d	Live load: LL Model	Lat. Distr.	Type span	Unfact. Loads	Load factor	DLA factor	Fact. Loads	Resistance: Fact Resist	Adjust Fact	Live Load Capacity Factor: w/ snow ULS1a	w/o snow ULS1b	w/o live C/D ULS1d & ULS9
1	Comp. in top chord	Pmax	S1	E1	INSP3	3.75	36	88	1.10	1.20	40	106	41	1.50	-	62	-	CL1	Static	All	167	1.70	0.25	356	1401	1.01	3.39	3.57	-
	Member U0-U1, U5-U6	[kN]					36	88	1.10	1.20	40	106	41	1.50	-	62	-	25T	Static	Other	101	1.70	0.30	223	1401	1.01	5.42	5.70	-
	1001 & 1006						36	88	1.10	1.20	40	106	41	1.50	-	62	-	5T	Static	Other	54	1.70	0.00	92	1401	1.01	13.14	13.82	-
							36	88	1.10	1.20	40	106	44	-	1.50	-	66	-	-	-	-	-	-	-	1401	1.01	-	-	6.70
							36	88	1.35	1.35	49	119	-	-	-	-	-	-	-	-	-	-	-	-	1401	1.01	-	-	8.43
2	Comp. in top chord	Pmax	S1	E1	INSP3	3.75	36	88	1.10	1.20	40	106	41	1.50	-	62	-	CL1	Static	All	167	1.70	0.25	356	1650	1.20	4.98	5.16	-
	Connection U0-U1, U5-U6	[kN]					36	88	1.10	1.20	40	106	41	1.50	-	62	-	25T	Static	Other	101	1.70	0.30	223	1650	1.20	7.95	8.23	-
	1001 & 1006						36	88	1.10	1.20	40	106	41	1.50	-	62	-	5T	Static	Other	54	1.70	0.00	92	1650	1.20	19.29	19.97	-
							36	88	1.10	1.20	40	106	44	-	1.50	-	66	-	-	-	-	-	-	-	1650	1.20	-	-	9.37
							36	88	1.35	1.35	49	119	-	-	-	-	-	-	-	-	-	-	-	-	1650	1.20	-	-	11.79
3	Comp. in top chord	Pmax	S1	E1	INSP3	3.75	36	88	1.10	1.20	40	106	41	1.50	-	62	-	CL1	Static	All	167	1.70	0.25	356	1401	1.01	3.39	3.57	-
	Member U1-U2 , U4-U5	[kN]					36	88	1.10	1.20	40	106	41	1.50	-	62	-	25T	Static	Other	101	1.70	0.30	223	1401	1.01	5.42	5.70	-
	1002 & 1005						36	88	1.10	1.20	40	106	41	1.50	-	62	-	5T	Static	Other	54	1.70	0.00	92	1401	1.01	13.14	13.82	-
							36	88	1.10	1.20	40	106	44	-	1.50	-	66	-	-	-	-	-	-	-	1401	1.01	-	-	6.70
							36	88	1.35	1.35	49	119	-	-	-	-	-	-	-	-	-	-	-	-	1401	1.01	-	-	8.43
4	Comp. in top chord	Pmax	S1	E1	INSP3	3.75	36	88	1.10	1.20	40	106	41	1.50	-	62	-	CL1	Static	All	167	1.70	0.25	356	1100	1.20	3.13	3.30	-
	Connection U1-U2 , U4-U5	[kN]					36	88	1.10	1.20	40	106	41	1.50	-	62	-	25T	Static	Other	101	1.70	0.30	223	1100	1.20	4.99	5.27	-
	1002 & 1005						36	88	1.10	1.20	40	106	41	1.50	-	62	-	5T	Static	Other	54	1.70	0.00	92	1100	1.20	12.11	12.79	-
							36	88	1.10	1.20	40	106	44	-	1.50	-	66	-	-	-	-	-	-	-	1100	1.20	-	-	6.25
							36	88	1.35	1.35	49	119	-	-	-	-	-	-	-	-	-	-	-	-	1100	1.20	-	-	7.86
5	Comp. in top chord	Pmax	S1	E1	INSP3	3.75	73	176	1.10	1.20	80	211	83	1.50	-	124	-	CL1	Static	All	319	1.70	0.25	677	1401	1.01	1.48	1.66	-
	Member U2-U3, U3-U4	[kN]					73	176	1.10	1.20	80	211	83	1.50	-	124	-	25T	Static	Other	189	1.70	0.30	418	1401	1.01	2.39	2.69	-
	1003 & 1004						73	176	1.10	1.20	80	211	83	1.50	-	124	-	5T	Static	Other	101	1.70	0.00	171	1401	1.01	5.83	6.55	-
							73	176	1.10	1.20	80	211	88	-	1.50	-	131	-	-	-	-	-	-	-	1401	1.01	-	-	3.35
							73	176	1.35	1.35	98	238	-	-	-	-	-	-	-	-	-	-	-	-	1401	1.01	-	-	4.21
6	Comp. in top chord	Pmax	S1	E1	INSP3	3.75	73	176	1.10	1.20	80	211	83	1.50	-	124	-	CL1	Static	All	319	1.70	0.25	677	1100	1.20	1.33	1.52	-
	Connection U2-U3, U3-U4	[kN]					73	176	1.10	1.20	80	211	83	1.50	-	124	-	25T	Static	Other	189	1.70	0.30	418	1100	1.20	2.16	2.46	-
	1003 & 1004						73	176	1.10	1.20	80	211	83	1.50	-	124	-	5T	Static	Other	101	1.70	0.00	171	1100	1.20	5.27	6.00	-
							73	176	1.10	1.20	80	211	88	-	1.50	-	131	-	-	-	-	-	-	-	1100	1.20	-	-	3.12
							73	176	1.35	1.35	98	238	-	-	-	-	-	-	-	-	-	-	-	-	1100	1.20	-	-	3.93
7	Tens. in bot chord	Pmax	S1	E3	INSP3	3.00	73	176	1.07	1.14	78	201	83	1.50	-	124	-	CL1	Static	All	335	1.49	0.25	624	785	1.01	0.63	0.82	-
	Member L1-L3	[kN]					73	176	1.07	1.14	78	201	83	1.50	-	124	-	25T	Static	Other	202	1.49	0.30	391	785	1.01	1.00	1.32	-
	3001						73	176	1.07	1.14	78	201	83	1.50	-	124	-	5T	Static	Other	108	1.49	0.00	161	785	1.01	2.42	3.19	-
							73	176	1.07	1.14	78	201	88	-	1.50	-	131	-	-	-	-	-	-	-	785	1.01	-	-	1.94
							73	176	1.35	1.35	98	238	-	-	-	-	-	-	-	-	-	-	-	-	785	1.01	-	-	2.36
8	Tens. in bot chord	Pmax	S1	E1	INSP3	3.75	73	176	1.10	1.20	80	211	83	1.50	-	124	-	CL1	Static	All	335	1.70	0.25	712	747	1.18	0.66	0.83	-
	Connection L1-L3	[kN]					73	176	1.10	1.20	80	211	83	1.50	-	124	-	25T	Static	Other	202	1.70	0.30	446	747	1.18	1.05	1.33	-
	3001						73	176	1.10	1.20	80	211	83	1.50	-	124	-	5T	Static	Other	108	1.70	0.00	184	747	1.18	2.54	3.21	-
							73	176	1.10	1.20	80	211	88	-	1.50	-	131	-	-	-	-	-	-	-	747	1.18	-	-	2.09
							73	176	1.35	1.35	98	238	-	-	-	-	-	-	-	-	-	-	-	-	747	1.18	-	-	2.63
9	Tens. in bot chord	Pmax	S1	E3	INSP3	3.00	73	176	1.07	1.14	78	201	83	1.50	-	124	-	CL1	Static	All	335	1.49	0.25	624	785	1.01	0.62	0.82	-
	Member L3-L5	[kN]					73	176	1.07	1.14	78	201	83	1.50	-	124	-	25T	Static	Other	202	1.49	0.30	391	785	1.01	1.00	1.32	-
	3002						73	176	1.07	1.14	78	201	83	1.50	-	124	-	5T	Static	Other	108	1.49	0.00	161	785	1.01	2.43	3.20	-
							73	176	1.07	1.14	78	201	88	-	1.50	-	131	-	-	-	-	-	-	-	785	1.01	-	-	1.94
							73	176	1.35	1.35	98	238	-	-	-	-	-	-	-	-	-	-	-	-	785	1.01	-	-	2.36
10	Tens. in bot chord	Pmax	S1	E1	INSP3	3.75	73	176	1.10	1.20	80	211	83	1.50	-	124	-	CL1	Static	All	335	1.70	0.25	712	747	1.18	0.65	0.83	-
	Connection L3-L5	[kN]					73	176	1.10	1.20	80	211	83	1.50	-	124	-	25T	Static	Other	202	1.70	0.30	446	747	1.18	1.05	1.33	-
	3002						73	176	1.10	1.20	80	211	83	1.50	-	124	-	5T	Static	Other	108	1.70	0.00	183	747	1.18	2.54	3.22	-
							73	176	1.10	1.20	80	211	88	-	1.50	-	131	-	-	-	-	-	-	-	747	1.18	-	-	2.09
							73	176	1.35	1.35	98	238	-	-	-	-	-	-	-	-	-	-	-	-	747	1.18	-	-	2.63

Note: * indicates load reversal
DLA = 0 indicates lane load governs
DLA > 0 indicates truck load governs
LLCFs circled in this Table are recomputed in Section 4 of this report to investigate the effects of capacity loss due to corrosion or damage

TABLE B5 - LOAD CAPACITY EVALUATION FOR TRUSS SYSTEM SPAN 1- ULS COMBINATIONS

BUCKLAND & TAYLOR LTD. Bridge Engineering

Notes:
1. Load rating method is referenced to CSA - S6 - 06, Section 14.
2. Evaluation procedure: ULS Method
3. Highway Class C (as per CSA-S6-06 Clause 1.4.2.2)
4. Evaluation was carried out for the following three live load models.
   CL1 - CL1-625 Truck or Lane Load traffic;
   25T - 25t review vehicle or Lane Load traffic;
   5T - 5t passenger vehicle or Lane Load traffic.
5. Inspection Level considered: "INSP3" for all structural components
6. Target reliability index from Table 14.5.
7. Dead load factors from Table 14.7. and 3.2.
8. Live load factors are from: - Table 14.8, for normal traffic (CL1-625) and pedestrain load.
   - Table 14.9, for normal traffic (alternative loading)
9. Resistance adjustment factor from Table 14.15.
10. Live load capacity factor as per Clause 14.15.2.1.
11. Material strength: fy = 210 MPa, fu = 420 MPa for Structural steel
    fu = 320 MPa for Rivet
    fc' = 15 MPa for Reinforced concrete
    fy = 230 MPa for Reinforcing steel

UNCORRODED

Elt. #	Element - Force effect	Effect Units	Target reliability index: Syst Behav	Elem Behav	Insp Level	Beta	Dead load: Unfact. loads D1	Unfact. loads D2	Load factors D1	Load factors D2	Fact. loads D1	Fact. loads D2	Snow load: Unfact. Loads	Load factor ULS1a	Load factor ULS1d	Fact.Loads ULS1a	Fact.Loads ULS1d	Live load: LL Model	Lat. Distr.	Type span	Unfact. Loads	Load factor	DLA factor	Fact. Loads	Resistance: Fact Resist	Adjust Fact	Live Load Capacity Factor: w/ snow ULS1a	w/o snow ULS1b	w/o live C/D ULS1d & ULS9
11	Tens. in diagonal	Pmax	S1	E3	INSP3	3.00	53	128	1.07	1.14	57	146	60	1.50	-	90	-	CL1	Static	All	243	1.49	0.25	453	618	1.01	0.73	0.93	-
	Member U0-L1, L5-U6	[kN]					53	128	1.07	1.14	57	146	60	1.50	-	90	-	25T	Static	Other	146	1.49	0.30	283	618	1.01	1.17	1.49	-
	4001 & 4006						53	128	1.07	1.14	57	146	60	1.50	-	90	-	5T	Static	Other	78	1.49	0.00	117	618	1.01	2.84	3.61	-
							53	128	1.07	1.14	57	146	63	-	1.50	-	95	-	-	-	-	-	-	-	618	1.01	-	-	2.10
							53	128	1.35	1.35	71	172	-	-	-	-	-	-	-	-	-	-	-	-	618	1.01	-	-	2.56
12	Tens. in diagonal	Pmax	S1	E1	INSP3	3.75	53	128	1.10	1.20	58	153	60	1.50	-	90	-	CL1	Static	All	243	1.70	0.25	516	574	1.18	0.73	0.90	-
	Connection U0-L1, L5-U6	[kN]					53	128	1.10	1.20	58	153	60	1.50	-	90	-	25T	Static	Other	146	1.70	0.30	323	574	1.18	1.16	1.44	-
	4001 & 4006						53	128	1.10	1.20	58	153	60	1.50	-	90	-	5T	Static	Other	78	1.70	0.00	133	574	1.18	2.82	3.49	-
							53	128	1.10	1.20	58	153	63	-	1.50	-	95	-	-	-	-	-	-	-	574	1.18	-	-	2.21
							53	128	1.35	1.35	71	172	-	-	-	-	-	-	-	-	-	-	-	-	574	1.18	-	-	2.78
13	Comp. in diagonal	Pmax	S1	E1	INSP3	3.75	53	128	1.10	1.20	58	153	60	1.50	-	90	-	CL1	Static	All	243	1.70	0.25	516	409	1.01	0.22	0.39	-
	Member L1-U2, U4-L5	[kN]					53	128	1.10	1.20	58	153	60	1.50	-	90	-	25T	Static	Other	146	1.70	0.30	323	409	1.01	0.34	0.62	-
	4002 & 4005						53	128	1.10	1.20	58	153	60	1.50	-	90	-	5T	Static	Other	78	1.70	0.00	133	409	1.01	0.84	1.51	-
							53	128	1.10	1.20	58	153	63	-	1.50	-	95	-	-	-	-	-	-	-	409	1.01	-	-	1.35
							53	128	1.35	1.35	71	172	-	-	-	-	-	-	-	-	-	-	-	-	409	1.01	-	-	1.69
14	Comp. in diagonal	Pmax	S1	E1	INSP3	3.75	53	128	1.10	1.20	58	153	60	1.50	-	90	-	CL1	Static	All	243	1.70	0.25	516	641	1.81	1.66	1.84	-
	Connection L1-U2, U4-L5	[kN]					53	128	1.10	1.20	58	153	60	1.50	-	90	-	25T	Static	Other	146	1.70	0.30	323	641	1.81	2.66	2.94	-
	4002 & 4005						53	128	1.10	1.20	58	153	60	1.50	-	90	-	5T	Static	Other	78	1.70	0.00	133	641	1.81	6.45	7.12	-
							53	128	1.10	1.20	58	153	63	-	1.50	-	95	-	-	-	-	-	-	-	641	1.81	-	-	3.79
							53	128	1.35	1.35	71	172	-	-	-	-	-	-	-	-	-	-	-	-	641	1.81	-	-	4.76
15	Comp. in diagonal	Pmax	S1	E1	INSP3	3.75	0	0	1.10	1.20	0	0	0	1.50	-	0	-	CL1	Static	All	87	1.70	0.30	192	240	1.01	1.26	1.26	-
	Member U2-L3, L3-U4	[kN]					0	0	1.10	1.20	0	0	0	1.50	-	0	-	25T	Static	Other	55	1.70	0.30	121	240	1.01	2.01	2.01	-
	4003 & 4004						0	0	1.10	1.20	0	0	0	1.50	-	0	-	5T	Static	Other	25	1.70	0.00	43	240	1.01	5.66	5.66	-
							0	0	1.10	1.20	0	0	0	-	1.50	-	0	-	-	-	-	-	-	-	240	1.01	-	-	-
							0	0	1.35	1.35	0	0	-	-	-	-	-	-	-	-	-	-	-	-	240	1.01	-	-	-
16	Comp. in diagonal	Pmax	S1	E1	INSP3	3.75	0	0	1.10	1.20	0	0	0	1.50	-	0	-	CL1	Static	All	87	1.70	0.30	192	275	1.81	2.59	2.59	-
	Connection U2-L3, L3-U4	[kN]					0	0	1.10	1.20	0	0	0	1.50	-	0	-	25T	Static	Other	55	1.70	0.30	121	275	1.81	4.13	4.13	-
	4003 & 4004						0	0	1.10	1.20	0	0	0	1.50	-	0	-	5T	Static	Other	25	1.70	0.00	43	275	1.81	11.64	11.64	-
							0	0	1.10	1.20	0	0	0	-	1.50	-	0	-	-	-	-	-	-	-	275	1.81	-	-	-
							0	0	1.35	1.35	0	0	-	-	-	-	-	-	-	-	-	-	-	-	275	1.81	-	-	-
17	Comp. in vertical	Pmax	S1	E1	INSP3	3.75	57	139	1.10	1.20	63	167	65	1.50	-	98	-	CL1	Static	All	309	1.70	0.25	656	659	1.01	0.52	0.66	-
	Member U6-L6	[kN]					57	139	1.10	1.20	63	167	65	1.50	-	98	-	25T	Static	Other	171	1.70	0.30	379	659	1.01	0.89	1.15	-
	5004						57	139	1.10	1.20	63	167	65	1.50	-	98	-	5T	Static	Other	87	1.70	0.00	148	659	1.01	2.29	2.96	-
							57	139	1.10	1.20	63	167	69	-	1.50	-	104	-	-	-	-	-	-	-	659	1.01	-	-	2.00
							57	139	1.35	1.35	78	187	-	-	-	-	-	-	-	-	-	-	-	-	659	1.01	-	-	2.51
18	Comp. in vertical	Pmax	S1	E1	INSP3	3.75	57	139	1.10	1.20	63	167	65	1.50	-	98	-	CL1	Static	All	309	1.70	0.25	656	733	1.81	1.52	1.67	-
	Connection U6-L6	[kN]					57	139	1.10	1.20	63	167	65	1.50	-	98	-	25T	Static	Other	171	1.70	0.30	379	733	1.81	2.64	2.90	-
	5004						57	139	1.10	1.20	63	167	65	1.50	-	98	-	5T	Static	Other	87	1.70	0.00	148	733	1.81	6.77	7.44	-
							57	139	1.10	1.20	63	167	69	-	1.50	-	104	-	-	-	-	-	-	-	733	1.81	-	-	3.98
							57	139	1.35	1.35	78	187	-	-	-	-	-	-	-	-	-	-	-	-	733	1.81	-	-	5.01
19	Tens. in gusset PL	Pmax	S1	E1	INSP3	3.75	53	128	1.10	1.20	58	153	60	1.50	0	90	-	CL1	Static	All	243	1.70	0.25	516	869	1.01	1.12	1.29	-
	Panel Point U0	[kN]					53	128	1.10	1.20	58	153	60	1.50	0	90	-	25T	Static	Other	146	1.70	0.30	323	869	1.01	1.78	2.06	-
							53	128	1.10	1.20	58	153	60	1.50	0	90	-	5T	Static	Other	78	1.70	0.00	133	869	1.01	4.32	5.00	-
							53	128	1.10	1.20	58	153	63	-	1.50	-	95	-	-	-	-	-	-	-	869	1.01	-	-	2.86
							53	128	1.35	1.35	71	172	-	-	-	-	-	-	-	-	-	-	-	-	869	1.01	-	-	3.60
20	Shear in gusset PL	Vmax	S1	E1	INSP3	3.75	59	143	1.10	1.20	65	172	67	1.50	0	101	-	CL1	Static	All	272	1.70	0.25	578	724	1.02	0.69	0.87	-
	Panel Point L1 & L5	[kN]					59	143	1.10	1.20	65	172	67	1.50	0	101	-	25T	Static	Other	202	1.70	0.30	446	724	1.02	0.90	1.13	-
							59	143	1.10	1.20	65	172	67	1.50	0	101	-	5T	Static	Other	108	1.70	0.00	184	724	1.02	2.18	2.73	-
							59	143	1.10	1.20	65	172	71	-	1.50	-	107	-	-	-	-	-	-	-	724	1.02	-	-	2.15
							59	143	1.35	1.35	80	193	-	-	-	-	-	-	-	-	-	-	-	-	724	1.02	-	-	2.71

Note: * indicates load reversal
DLA = 0 indicates lane load governs
DLA > 0 indicates truck load governs
LLCFs circled in this Table are recomputed in Section 4 of this report to investigate the effects of capacity loss due to corrosion or damage

TABLE B5 - LOAD CAPACITY EVALUATION FOR TRUSS SYSTEM SPAN 1- ULS COMBINATIONS

BUCKLAND & TAYLOR LTD. Bridge Engineering

Notes:

1. Load rating method is referenced to CSA - S6 - 06, Section 14.
2. Evaluation procedure: ULS Method
3. Highway Class C (as per CSA-S6-06 Clause 1.4.2.2)
4. Evaluation was carried out for the following three live load models.
   CL1 - CL1-625 Truck or Lane Load traffic;
   25T - 25t review vehicle or Lane Load traffic;
   5T - 5t passenger vehicle or Lane Load traffic.
5. Inspection Level considered: "INSP3" for all structural components
6. Target reliability index from Table 14.5.
7. Dead load factors from Table 14.7. and 3.2.
8. Live load factors are from: - Table 14.8, for normal traffic (CL1-625) and pedestrain load.
   - Table 14.9, for normal traffic (alternative loading)
9. Resistance adjustment factor from Table 14.15.
10. Live load capacity factor as per Clause 14.15.2.1.
11. Material strength: fy = 210 MPa, fu = 420 MPa for Structural steel
    fu = 320 MPa for Rivet
    fc' = 15 MPa for Reinforced concrete
    fy = 230 MPa for Reinforcing steel

UNCORRODED

Elt. #	Element - Force effect	Effect Units	Target reliability index				Dead load						Snow load					Live load							Resistance		Live Load Capacity Factor		C/D
			Syst Behav	Elem Behav	Insp Level	Beta	Unfact. loads		Load factors		Fact. loads		Unfact.	Load factor		Fact.Loads		LL	Lat.	Type	Unfact.	Load	DLA	Fact.	Fact	Adjust	w/ snow	w/o snow	w/o live
							D1	D2	D1	D2	D1	D2	Loads	ULS1a	ULS1d	ULS1a	ULS1d	Model	Distr.	span	Loads	factor	factor	Loads	Resist	Fact	ULS1a	ULS1b	ULS1d & ULS9
21	Comp. in gusset PL	Pmax	S1	E1	INSP3	3.75	53	128	1.10	1.20	58	153	60	1.50	0	90	-	CL1	Static	All	243	1.70	0.25	516	851	1.01	1.08	1.26	-
	Panel Point U2 & U4	[kN]					53	128	1.10	1.20	58	153	60	1.50	0	90	-	25T	Static	Other	146	1.70	0.30	323	851	1.01	1.73	2.00	-
							53	128	1.10	1.20	58	153	60	1.50	0	90	-	5T	Static	Other	78	1.70	0.00	133	851	1.01	4.18	4.86	-
							53	128	1.10	1.20	58	153	63	-	1.50	-	95	-	-	-	-	-	-	-	851	1.01	-	-	2.80
							53	128	1.35	1.35	71	172	-	-	-	-	-	-	-	-	-	-	-	-	851	1.01	-	-	3.53
22	Tens. in gusset PL	Pmax	S1	E1	INSP3	3.75	43	104	1.10	1.20	47	124	49	1.50	0	73	-	CL1	Static	All	197	1.70	0.25	419	688	1.01	1.08	1.25	-
	Panel Point L3	[kN]					43	104	1.10	1.20	47	124	49	1.50	0	73	-	25T	Static	Other	119	1.70	0.30	262	688	1.01	1.72	2.00	-
							43	104	1.10	1.20	47	124	49	1.50	0	73	-	5T	Static	Other	64	1.70	0.00	108	688	1.01	4.17	4.84	-
							43	104	1.10	1.20	47	124	51	-	1.50	-	77	-	-	-	-	-	-	-	688	1.01	-	-	2.80
							43	104	1.35	1.35	58	140	-	-	-	-	-	-	-	-	-	-	-	-	688	1.01	-	-	3.52
23	Tens. in gusset PL	Pmax	S1	E1	INSP3	3.75	53	128	1.10	1.20	58	153	60	1.50	0	90	-	CL1	Static	All	243	1.70	0.25	517	772	1.01	0.92	1.10	-
	Panel Point U6	[kN]					53	128	1.10	1.20	58	153	60	1.50	0	90	-	25T	Static	Other	146	1.70	0.30	323	772	1.01	1.48	1.76	-
							53	128	1.10	1.20	58	153	60	1.50	0	90	-	5T	Static	Other	78	1.70	0.00	133	772	1.01	3.59	4.27	-
							53	128	1.10	1.20	58	153	63	-	1.50	-	95	-	-	-	-	-	-	-	772	1.01	-	-	2.54
							53	128	1.35	1.35	71	172	-	-	-	-	-	-	-	-	-	-	-	-	772	1.01	-	-	3.20
24	Comp. in gusset PL	Pmax	S1	E1	INSP3	3.75	57	139	1.10	1.20	63	167	65	1.50	0	98	-	CL1	Static	All	309	1.70	0.25	656	1078	1.01	1.16	1.31	-
	Panel Point L6	[kN]					57	139	1.10	1.20	63	167	65	1.50	0	98	-	25T	Static	Other	171	1.70	0.30	379	1078	1.01	2.01	2.27	-
							57	139	1.10	1.20	63	167	65	1.50	0	98	-	5T	Static	Other	87	1.70	0.00	148	1078	1.01	5.16	5.82	-
							57	139	1.10	1.20	63	167	69	-	1.50	-	104	-	-	-	-	-	-	-	1078	1.01	-	-	3.27
							57	139	1.35	1.35	78	187	-	-	-	-	-	-	-	-	-	-	-	-	1078	1.01	-	-	4.11

Note: * indicates load reversal
DLA = 0 indicates lane load governs
DLA > 0 indicates truck load governs
LLCFs circled in this Table are recomputed in Section 4 of this report to investigate the effects of capacity loss due to corrosion or damage

- GOVERNING LL CAPACITY FACTOR

- FOR MEMBERS

CL1	0.22	0.39	-
25T	0.34	0.62	-
5T	0.84	1.51	-
SNOW	-	-	1.35
DL	-	-	1.69

- FOR GUSSET PLATES

CL1	0.69	0.87	-
25T	0.90	1.13	-
5T	2.18	2.73	-
SNOW	-	-	2.15
DL	-	-	2.71

TABLE B5 - LOAD CAPACITY EVALUATION FOR TRUSS SYSTEM SPAN 2- ULS COMBINATIONS

BUCKLAND & TAYLOR LTD. Bridge Engineering

Notes:
1. Load rating method is referenced to CSA - S6 - 06, Section 14.
2. Evaluation procedure: ULS Method
3. Highway Class C (as per CSA-S6-06 Clause 1.4.2.2)
4. Evaluation was carried out for the following three live load models.
   CL1 - CL1-625 Truck or Lane Load traffic;
   25T - 25t review vehicle or Lane Load traffic;
   5T - 5t passenger vehicle or Lane Load traffic.
5. Inspection Level considered: "INSP3" for all structural components
6. Target reliability index from Table 14.5.
7. Dead load factors from Table 14.7. and 3.2.
8. Live load factors are from: - Table 14.8, for normal traffic (CL1-625) and pedestrain load.
   - Table 14.9, for normal traffic (alternative loading)
9. Resistance adjustment factor from Table 14.15.
10. Live load capacity factor as per Clause 14.15.2.1.
11. Material strength: fy = 210 MPa, fu = 420 MPa for Structural steel
    fu = 320 MPa for Rivet
    fc' = 15 MPa for Reinforced concrete
    fy = 230 MPa for Reinforcing steel

UNCORRODED

Elt. #	Element - Force effect	Effect	Target reliability index				Dead load						Snow load					Live load							Resistance		Live Load Capacity Factor w/ snow	Live Load Capacity Factor w/o snow	C/D w/o live
		Units	Syst Behav	Elem Behav	Insp Level	Beta	Unfact. loads D1	Unfact. loads D2	Load factors D1	Load factors D2	Fact. loads D1	Fact. loads D2	Unfact. Loads	Load factor ULS1a	Load factor ULS1d	Fact.Loads ULS1a	Fact.Loads ULS1d	LL Model	Lat. Distr.	Type span	Unfact. Loads	Load factor	DLA factor	Fact. Loads	Fact Resist	Adjust Fact	ULS1a	ULS1b	ULS1d & ULS9
1	Comp. in top chord	Pmax	S1	E1	INSP3	3.75	59	132	1.10	1.20	65	158	62	1.50	-	93	-	CL1	Static	All	234	1.70	0.25	498	1496	1.01	2.40	2.59	-
	Member U0-U1, U7-U8	[kN]					59	132	1.10	1.20	65	158	62	1.50	-	93	-	25T	Static	Other	123	1.70	0.30	272	1496	1.01	4.39	4.73	-
	2001 & 2008						59	132	1.10	1.20	65	158	62	1.50	-	93	-	5T	Static	Other	75	1.70	0.00	127	1496	1.01	9.39	10.12	-
							59	132	1.10	1.20	65	158	66	-	1.50	-	98	-	-	-	-	-	-	-	1496	1.01	-	-	4.71
							59	132	1.35	1.35	79	178	-	-	-	-	-	-	-	-	-	-	-	-	1496	1.01	-	-	5.87
2	Comp. in top chord	Pmax	S1	E1	INSP3	3.75	59	132	1.10	1.20	65	158	62	1.50	-	93	-	CL1	Static	All	234	1.70	0.25	498	1375	1.20	2.68	2.87	-
	Connection U0-U1, U7-U8	[kN]					59	132	1.10	1.20	65	158	62	1.50	-	93	-	25T	Static	Other	123	1.70	0.30	272	1375	1.20	4.90	5.24	-
	2001 & 2008						59	132	1.10	1.20	65	158	62	1.50	-	93	-	5T	Static	Other	75	1.70	0.00	127	1375	1.20	10.48	11.21	-
							59	132	1.10	1.20	65	158	66	-	1.50	-	98	-	-	-	-	-	-	-	1375	1.20	-	-	5.14
							59	132	1.35	1.35	79	178	-	-	-	-	-	-	-	-	-	-	-	-	1375	1.20	-	-	6.42
3	Comp. in top chord	Pmax	S1	E1	INSP3	3.75	59	132	1.10	1.20	65	158	62	1.50	-	93	-	CL1	Static	All	234	1.70	0.25	498	1496	1.01	2.40	2.59	-
	Member U1-U2, U6-U7	[kN]					59	132	1.10	1.20	65	158	62	1.50	-	93	-	25T	Static	Other	123	1.70	0.30	272	1496	1.01	4.39	4.73	-
	2002 & 2007						59	132	1.10	1.20	65	158	62	1.50	-	93	-	5T	Static	Other	75	1.70	0.00	127	1496	1.01	9.39	10.12	-
							59	132	1.10	1.20	65	158	66	-	1.50	-	98	-	-	-	-	-	-	-	1496	1.01	-	-	4.71
							59	132	1.35	1.35	79	178	-	-	-	-	-	-	-	-	-	-	-	-	1496	1.01	-	-	5.87
4	Comp. in top chord	Pmax	S1	E1	INSP3	3.75	59	132	1.10	1.20	65	158	62	1.50	-	93	-	CL1	Static	All	234	1.70	0.25	498	1375	1.20	2.68	2.87	-
	Connection U1-U2 , U6-U7	[kN]					59	132	1.10	1.20	65	158	62	1.50	-	93	-	25T	Static	Other	123	1.70	0.30	272	1375	1.20	4.90	5.24	-
	2002 & 2007						59	132	1.10	1.20	65	158	62	1.50	-	93	-	5T	Static	Other	75	1.70	0.00	127	1375	1.20	10.48	11.21	-
							59	132	1.10	1.20	65	158	66	-	1.50	-	98	-	-	-	-	-	-	-	1375	1.20	-	-	5.14
							59	132	1.35	1.35	79	178	-	-	-	-	-	-	-	-	-	-	-	-	1375	1.20	-	-	6.42
5	Comp. in top chord	Pmax	S1	E1	INSP3	3.75	136	307	1.10	1.20	150	368	145	1.50	-	217	-	CL1	Static	All	515	1.70	0.25	1095	1496	1.01	0.71	0.91	-
	Member U2-U3, U5-U6	[kN]					136	307	1.10	1.20	150	368	145	1.50	-	217	-	25T	Static	Other	266	1.70	0.30	588	1496	1.01	1.32	1.69	-
	2003 & 2006						136	307	1.10	1.20	150	368	145	1.50	-	217	-	5T	Static	Other	168	1.70	0.00	286	1496	1.01	2.71	3.47	-
							136	307	1.10	1.20	150	368	153	-	1.50	-	229	-	-	-	-	-	-	-	1496	1.01	-	-	2.02
							136	307	1.35	1.35	184	414	-	-	-	-	-	-	-	-	-	-	-	-	1496	1.01	-	-	2.52
6	Comp. in top chord	Pmax	S1	E1	INSP3	3.75	136	307	1.10	1.20	150	368	145	1.50	-	217	-	CL1	Static	All	515	1.70	0.25	1095	1238	1.20	0.68	0.88	-
	Connection U2-U3, U5-U6	[kN]					136	307	1.10	1.20	150	368	145	1.50	-	217	-	25T	Static	Other	266	1.70	0.30	588	1238	1.20	1.28	1.65	-
	2003 & 2006						136	307	1.10	1.20	150	368	145	1.50	-	217	-	5T	Static	Other	168	1.70	0.00	286	1238	1.20	2.62	3.38	-
							136	307	1.10	1.20	150	368	153	-	1.50	-	229	-	-	-	-	-	-	-	1238	1.20	-	-	1.99
							136	307	1.35	1.35	184	414	-	-	-	-	-	-	-	-	-	-	-	-	1238	1.20	-	-	2.48
7	Comp. in top chord	Pmax	S1	E1	INSP3	3.75	136	307	1.10	1.20	150	368	145	1.50	-	217	-	CL1	Static	All	515	1.70	0.25	1095	1496	1.01	0.71	0.91	-
	Member U3-U4, U4-U5	[kN]					136	307	1.10	1.20	150	368	145	1.50	-	217	-	25T	Static	Other	266	1.70	0.30	588	1496	1.01	1.32	1.69	-
	2004 & 2005						136	307	1.10	1.20	150	368	145	1.50	-	217	-	5T	Static	Other	168	1.70	0.00	286	1496	1.01	2.71	3.47	-
							136	307	1.10	1.20	150	368	153	-	1.50	-	229	-	-	-	-	-	-	-	1496	1.01	-	-	2.02
							136	307	1.35	1.35	184	414	-	-	-	-	-	-	-	-	-	-	-	-	1496	1.01	-	-	2.52
8	Comp. in top chord	Pmax	S1	E1	INSP3	3.75	136	307	1.10	1.20	150	368	145	1.50	-	217	-	CL1	Static	All	515	1.70	0.25	1095	1238	1.20	0.56	0.89	-
	Connection U3-U4, U4-U5	[kN]					136	307	1.10	1.20	150	368	145	1.50	-	217	-	25T	Static	Other	266	1.70	0.30	588	1238	1.20	1.05	1.66	-
	2004 & 2005						136	307	1.10	1.20	150	368	145	1.50	-	217	-	5T	Static	Other	168	1.70	0.00	286	1238	1.20	1.95	3.08	-
							136	307	1.10	1.20	150	368	153	-	1.50	-	229	-	-	-	-	-	-	-	1238	1.20	-	-	1.99
							136	307	1.35	1.35	184	414	-	-	-	-	-	-	-	-	-	-	-	-	1238	1.20	-	-	2.48
9	Tens. in bot chord	Pmax	S1	E3	INSP3	3.00	117	264	1.07	1.14	126	301	124	1.50	-	186	-	CL1	Static	All	469	1.49	0.25	873	1223	1.01	0.71	0.93	-
	Member L1-L3, L5-L7	[kN]					117	264	1.07	1.14	126	301	124	1.50	-	186	-	25T	Static	Other	246	1.49	0.30	477	1223	1.01	1.30	1.70	-
	3005 & 3007						117	264	1.07	1.14	126	301	124	1.50	-	186	-	5T	Static	Other	150	1.49	0.00	223	1223	1.01	2.79	3.62	-
							117	264	1.07	1.14	126	301	131	-	1.50	-	197	-	-	-	-	-	-	-	1223	1.01	-	-	1.98
							117	264	1.35	1.35	158	356	-	-	-	-	-	-	-	-	-	-	-	-	1223	1.01	-	-	2.40
10	Tens. in bot chord	Pmax	S1	E1	INSP3	3.75	117	264	1.10	1.20	129	316	124	1.50	-	186	-	CL1	Static	All	469	1.70	0.25	996	1236	1.20	0.56	0.89	-
	Connection L1-L3, L5-L7	[kN]					117	264	1.10	1.20	129	316	124	1.50	-	186	-	25T	Static	Other	246	1.70	0.30	544	1236	1.20	1.05	1.66	-
	3005 & 3007						117	264	1.10	1.20	129	316	124	1.50	-	186	-	5T	Static	Other	150	1.70	0.00	255	1236	1.20	1.95	3.08	-
							117	264	1.10	1.20	129	316	131	-	1.50	-	197	-	-	-	-	-	-	-	1236	1.20	-	-	1.99
							117	264	1.35	1.35	158	356	-	-	-	-	-	-	-	-	-	-	-	-	1236	1.20	-	-	2.48

Note: * indicates load reversal
DLA = 0 indicates lane load governs
DLA > 0 indicates truck load governs
LLCFs circled in this Table are recomputed in Section 4 of this report to investigate the effects of capacity loss due to corrosion or damage

TABLE B5 - LOAD CAPACITY EVALUATION FOR TRUSS SYSTEM SPAN 2- ULS COMBINATIONS

BUCKLAND & TAYLOR LTD. Bridge Engineering

Notes:
1. Load rating method is referenced to CSA - S6 - 06, Section 14.
2. Evaluation procedure: ULS Method
3. Highway Class C (as per CSA-S6-06 Clause 1.4.2.2)
4. Evaluation was carried out for the following three live load models.
   CL1 - CL1-625 Truck or Lane Load traffic;
   25T - 25t review vehicle or Lane Load traffic;
   5T - 5t passenger vehicle or Lane Load traffic.
5. Inspection Level considered: "INSP3" for all structural components
6. Target reliability index from Table 14.5.
7. Dead load factors from Table 14.7. and 3.2.
8. Live load factors are from: - Table 14.8, for normal traffic (CL1-625) and pedestrain load.
   - Table 14.9, for normal traffic (alternative loading)
9. Resistance adjustment factor from Table 14.15.
10. Live load capacity factor as per Clause 14.15.2.1.
11. Material strength: fy = 210 MPa, fu = 420 MPa for Structural steel
    fu = 320 MPa for Rivet
    fc' = 15 MPa for Reinforced concrete
    fy = 230 MPa for Reinforcing steel

UNCORRODED

Elt. #	Element - Force effect	Effect Units	Target reliability index: Syst Behav	Elem Behav	Insp Level	Beta	Dead load: Unfact. loads D1	Unfact. loads D2	Load factors D1	Load factors D2	Fact. loads D1	Fact. loads D2	Snow load: Unfact. Loads	Load factor ULS1a	Load factor ULS1d	Fact.Loads ULS1a	Fact.Loads ULS1d	Live load: LL Model	Lat. Distr.	Type span	Unfact. Loads	Load factor	DLA factor	Fact. Loads	Resistance: Fact Resist	Adjust Fact	Live Load Capacity Factor: w/ snow ULS1a	w/o snow ULS1b	w/o live C/D ULS1d & ULS9
11	Tens. in bot chord	Pmax	S1	E3	INSP3	3.00	156	350	1.07	1.14	166	399	165	1.50	-	248	-	CL1	Static	All	564	1.49	0.25	1051	1649	1.01	0.81	1.05	-
	Member L3-L5	[kN]					156	350	1.07	1.14	166	399	165	1.50	-	248	-	25T	Static	Other	315	1.49	0.30	610	1649	1.01	1.40	1.80	-
	3006						156	350	1.07	1.14	166	399	165	1.50	-	248	-	5T	Static	Other	198	1.49	0.00	295	1649	1.01	2.89	3.73	-
							156	350	1.07	1.14	166	399	174	-	1.50	-	261	-	-	-	-	-	-	-	1649	1.01	-	-	2.01
							156	350	1.35	1.35	210	473	-	-	-	-	-	-	-	-	-	-	-	-	1649	1.01	-	-	2.44
12	Tens. in bot chord	Pmax	S1	E1	INSP3	3.75	156	350	1.10	1.20	171	420	165	1.50	-	248	-	CL1	Static	All	564	1.70	0.25	1199	1656	1.18	0.93	1.14	-
	Connection L3-L5	[kN]					156	350	1.10	1.20	171	420	165	1.50	-	248	-	25T	Static	Other	315	1.70	0.30	696	1656	1.18	1.60	1.96	-
	3006						156	350	1.10	1.20	171	420	165	1.50	-	248	-	5T	Static	Other	198	1.70	0.00	337	1656	1.18	3.31	4.05	-
							156	350	1.10	1.20	171	420	174	-	1.50	-	261	-	-	-	-	-	-	-	1656	1.18	-	-	2.29
							156	350	1.35	1.35	210	473	-	-	-	-	-	-	-	-	-	-	-	-	1656	1.18	-	-	2.86
13	Tens. in diagonal	Pmax	S1	E3	INSP3	3.00	85	191	1.07	1.14	91	217	90	1.50	-	135	-	CL1	Static	All	339	1.49	0.25	631	880	1.01	0.71	0.92	-
	Member U0-L1, L7-U8	[kN]					85	191	1.07	1.14	91	217	90	1.50	-	135	-	25T	Static	Other	178	1.49	0.30	345	880	1.01	1.29	1.68	-
	4007 & 4014						85	191	1.07	1.14	91	217	90	1.50	-	135	-	5T	Static	Other	108	1.49	0.00	161	880	1.01	2.76	3.60	-
							85	191	1.07	1.14	91	217	95	-	1.50	-	142	-	-	-	-	-	-	-	880	1.01	-	-	1.97
							85	191	1.35	1.35	115	257	-	-	-	-	-	-	-	-	-	-	-	-	880	1.01	-	-	2.39
14	Tens. in diagonal	Pmax	S1	E1	INSP3	3.75	85	191	1.10	1.20	93	229	90	1.50	-	135	-	CL1	Static	All	339	1.70	0.25	720	877	1.18	0.80	0.99	-
	Connection U0-L1, L7-U8	[kN]					85	191	1.10	1.20	93	229	90	1.50	-	135	-	25T	Static	Other	178	1.70	0.30	394	877	1.18	1.47	1.81	-
	4007 & 4014						85	191	1.10	1.20	93	229	90	1.50	-	135	-	5T	Static	Other	108	1.70	0.00	184	877	1.18	3.14	3.87	-
							85	191	1.10	1.20	93	229	95	-	1.50	-	142	-	-	-	-	-	-	-	877	1.18	-	-	2.23
							85	191	1.35	1.35	115	257	-	-	-	-	-	-	-	-	-	-	-	-	877	1.18	-	-	2.78
15	Comp. in diagonal	Pmax	S1	E1	INSP3	3.75	85	191	1.10	1.20	93	229	90	1.50	-	135	-	CL1	Static	All	339	1.70	0.25	720	690	1.01	0.33	0.52	-
	Member L1-U2, U6-L7	[kN]					85	191	1.10	1.20	93	229	90	1.50	-	135	-	25T	Static	Other	178	1.70	0.30	394	690	1.01	0.61	0.95	-
	4008 & 4013						85	191	1.10	1.20	93	229	90	1.50	-	135	-	5T	Static	Other	108	1.70	0.00	184	690	1.01	1.30	2.04	-
							85	191	1.10	1.20	93	229	95	-	1.50	-	142	-	-	-	-	-	-	-	690	1.01	-	-	1.50
							85	191	1.35	1.35	115	257	-	-	-	-	-	-	-	-	-	-	-	-	690	1.01	-	-	1.87
16	Comp. in diagonal	Pmax	S1	E1	INSP3	3.75	85	191	1.10	1.20	93	229	90	1.50	-	135	-	CL1	Static	All	339	1.70	0.25	720	825	1.81	1.44	1.63	-
	Connection L1-U2, U6-L7	[kN]					85	191	1.10	1.20	93	229	90	1.50	-	135	-	25T	Static	Other	178	1.70	0.30	394	825	1.81	2.63	2.97	-
	4008 & 4013						85	191	1.10	1.20	93	229	90	1.50	-	135	-	5T	Static	Other	108	1.70	0.00	184	825	1.81	5.63	6.36	-
							85	191	1.10	1.20	93	229	95	-	1.50	-	142	-	-	-	-	-	-	-	825	1.81	-	-	3.22
							85	191	1.35	1.35	115	257	-	-	-	-	-	-	-	-	-	-	-	-	825	1.81	-	-	4.01
17	Tens. in diagonal	Pmax	S1	E3	INSP3	3.00	28	63	1.07	1.14	30	71	29	1.50	-	44	-	CL1	Static	All	174	1.49	0.25	325	417	1.01	0.85	0.99	-
	Member U2-L3, L5-U6	[kN]					28	63	1.07	1.14	30	71	29	1.50	-	44	-	25T	Static	Other	109	1.49	0.30	211	417	1.01	1.31	1.52	-
	4009 & 4012						28	63	1.07	1.14	30	71	29	1.50	-	44	-	5T	Static	Other	54	1.49	0.00	81	417	1.01	3.41	3.95	-
							28	63	1.07	1.14	30	71	31	-	1.50	-	47	-	-	-	-	-	-	-	417	1.01	-	-	2.85
							28	63	1.35	1.35	37	84	-	-	-	-	-	-	-	-	-	-	-	-	417	1.01	-	-	3.46
18	Tens. in diagonal	Pmax	S1	E1	INSP3	3.75	28	63	1.10	1.20	30	75	29	1.50	-	44	-	CL1	Static	All	174	1.70	0.25	371	389	1.18	0.83	0.95	-
	Connection U2-L3, L5-U6	[kN]					28	63	1.10	1.20	30	75	29	1.50	-	44	-	25T	Static	Other	109	1.70	0.30	241	389	1.18	1.28	1.47	-
	4009 & 4012						28	63	1.10	1.20	30	75	29	1.50	-	44	-	5T	Static	Other	54	1.70	0.00	92	389	1.18	3.35	3.83	-
							28	63	1.10	1.20	30	75	31	-	1.50	-	47	-	-	-	-	-	-	-	389	1.18	-	-	3.02
							28	63	1.35	1.35	37	84	-	-	-	-	-	-	-	-	-	-	-	-	389	1.18	-	-	3.77
19	Comp. in diagonal	Pmax	S1	E1	INSP3	3.75	28	63	1.10	1.20	30	75	29	1.50	-	44	-	CL1	Static	All	174	1.70	0.25	371	240	1.01	0.25	0.37	-
	Member L3-U4, U4-L5	[kN]					28	63	1.10	1.20	30	75	29	1.50	-	44	-	25T	Static	Other	109	1.70	0.30	241	240	1.01	0.38	0.57	-
	4010 & 4011						28	63	1.10	1.20	30	75	29	1.50	-	44	-	5T	Static	Other	54	1.70	0.00	92	240	1.01	1.00	1.48	-
							28	63	1.10	1.20	30	75	31	-	1.50	-	47	-	-	-	-	-	-	-	240	1.01	-	-	1.59
							28	63	1.35	1.35	37	84	-	-	-	-	-	-	-	-	-	-	-	-	240	1.01	-	-	1.99
20	Comp. in diagonal	Pmax	S1	E1	INSP3	3.75	28	63	1.10	1.20	30	75	29	1.50	-	44	-	CL1	Static	All	174	1.70	0.25	371	550	1.81	2.28	2.40	-
	Connection L3-U4, U4-L5	[kN]					28	63	1.10	1.20	30	75	29	1.50	-	44	-	25T	Static	Other	109	1.70	0.30	241	550	1.81	3.51	3.69	-
	4010 & 4011						28	63	1.10	1.20	30	75	29	1.50	-	44	-	5T	Static	Other	54	1.70	0.00	92	550	1.81	9.15	9.63	-
							28	63	1.10	1.20	30	75	31	-	1.50	-	47	-	-	-	-	-	-	-	550	1.81	-	-	6.54
							28	63	1.35	1.35	37	84	-	-	-	-	-	-	-	-	-	-	-	-	550	1.81	-	-	8.17

Note: * indicates load reversal
DLA = 0 indicates lane load governs
DLA > 0 indicates truck load governs
LLCFs circled in this Table are recomputed in Section 4 of this report to investigate the effects of capacity loss due to corrosion or damage

TABLE B5 - LOAD CAPACITY EVALUATION FOR TRUSS SYSTEM SPAN 2- ULS COMBINATIONS

BUCKLAND & TAYLOR LTD.
Bridge Engineering

Notes:

1. Load rating method is referenced to CSA - S6 - 06, Section 14.
2. Evaluation procedure: ULS Method
3. Highway Class C (as per CSA-S6-06 Clause 1.4.2.2)
4. Evaluation was carried out for the following three live load models.
   CL1 - CL1-625 Truck or Lane Load traffic;
   25T - 25t review vehicle or Lane Load traffic;
   5T - 5t passenger vehicle or Lane Load traffic.
5. Inspection Level considered: "INSP3" for all structural components
6. Target reliability index from Table 14.5.
7. Dead load factors from Table 14.7. and 3.2.
8. Live load factors are from: - Table 14.8, for normal traffic (CL1-625) and pedestrain load.
   - Table 14.9, for normal traffic (alternative loading)
9. Resistance adjustment factor from Table 14.15.
10. Live load capacity factor as per Clause 14.15.2.1.
11. Material strength: fy = 210 MPa, fu = 420 MPa for Structural steel
    fu = 320 MPa for Rivet
    fc' = 15 MPa for Reinforced concrete
    fy = 230 MPa for Reinforcing steel

UNCORRODED

Elt. #	Element - Force effect	Effect Units	Target reliability index				Dead load						Snow load					Live load							Resistance		Live Load Capacity Factor		C/D
			Syst Behav	Elem Behav	Insp Level	Beta	Unfact. loads		Load factors		Fact. loads		Unfact. Loads	Load factor		Fact.Loads		LL Model	Lat. Distr.	Type span	Unfact. Loads	Load factor	DLA factor	Fact. Loads	Fact Resist	Adjust Fact	w/ snow	w/o snow	w/o live
							D1	D2	D1	D2	D1	D2		ULS1a	ULS1d	ULS1a	ULS1d										ULS1a	ULS1b	ULS1d & ULS9
21	Comp. in vertical	Pmax	S1	E1	INSP3	3.75	82	184	1.10	1.20	90	221	87	1.50	-	130	-	CL1	Static	All	343	1.70	0.25	729	821	1.01	0.53	0.71	-
	Member U0-L0	[kN]					82	184	1.10	1.20	90	221	87	1.50	-	130	-	25T	Static	Other	178	1.70	0.30	394	821	1.01	0.98	1.31	-
	5005						82	184	1.10	1.20	90	221	87	1.50	-	130	-	5T	Static	Other	106	1.70	0.00	180	821	1.01	2.16	2.88	-
							82	184	1.10	1.20	90	221	92	-	1.50	-	137	-	-	-	-	-	-	-	821	1.01	-	-	1.85
							82	184	1.35	1.35	111	248	-	-	-	-	-	-	-	-	-	-	-	-	821	1.01	-	-	2.31
22	Comp. in vertical	Pmax	S1	E1	INSP3	3.75	82	184	1.10	1.20	90	221	87	1.50	-	130	-	CL1	Static	All	343	1.70	0.25	729	917	1.81	1.67	1.85	-
	Connection U0-L0	[kN]					82	184	1.10	1.20	90	221	87	1.50	-	130	-	25T	Static	Other	178	1.70	0.30	394	917	1.81	3.09	3.42	-
	5005						82	184	1.10	1.20	90	221	87	1.50	-	130	-	5T	Static	Other	106	1.70	0.00	180	917	1.81	6.77	7.49	-
							82	184	1.10	1.20	90	221	92	-	1.50	-	137	-	-	-	-	-	-	-	917	1.81	-	-	3.70
							82	184	1.35	1.35	111	248	-	-	-	-	-	-	-	-	-	-	-	-	917	1.81	-	-	4.62
23	Tens. in gusset PL	Pmax	S1	E1	INSP3	3.75	85	191	1.10	1.20	93	229	90	1.50	0	135	-	CL1	Static	All	339	1.70	0.25	720	965	1.01	0.72	0.91	-
	Panel Point U0	[kN]					85	191	1.10	1.20	93	229	90	1.50	0	135	-	25T	Static	Other	178	1.70	0.30	394	965	1.01	1.32	1.66	-
							85	191	1.10	1.20	93	229	90	1.50	0	135	-	5T	Static	Other	108	1.70	0.00	184	965	1.01	2.81	3.54	-
							85	191	1.10	1.20	93	229	95	-	1.50	-	142	-	-	-	-	-	-	-	965	1.01	-	-	2.10
							85	191	1.35	1.35	115	257	-	-	-	-	-	-	-	-	-	-	-	-	965	1.01	-	-	2.62
24	Comp. in gusset PL	Pmax	S1	E1	INSP3	3.75	82	184	1.10	1.20	90	221	87	1.50	0	130	-	CL1	Static	All	343	1.70	0.25	729	1078	1.01	0.89	1.07	-
	Panel Point L0	[kN]					82	184	1.10	1.20	90	221	87	1.50	0	130	-	25T	Static	Other	178	1.70	0.30	394	1078	1.01	1.64	1.97	-
							82	184	1.10	1.20	90	221	87	1.50	0	130	-	5T	Static	Other	106	1.70	0.00	180	1078	1.01	3.60	4.32	-
							82	184	1.10	1.20	90	221	92	-	1.50	-	137	-	-	-	-	-	-	-	1078	1.01	-	-	2.43
							82	184	1.35	1.35	111	248	-	-	-	-	-	-	-	-	-	-	-	-	1078	1.01	-	-	3.03
25	Comp. in gusset PL	Pmax	S1	E1	INSP3	3.75	85	191	1.10	1.20	93	229	90	1.50	0	135	-	CL1	Static	All	339	1.70	0.25	720	868	1.01	0.58	0.77	-
	Panel Point L1 & L8	[kN]					85	191	1.10	1.20	93	229	90	1.50	0	135	-	25T	Static	Other	178	1.70	0.30	394	868	1.01	1.07	1.41	-
							85	191	1.10	1.20	93	229	90	1.50	0	135	-	5T	Static	Other	108	1.70	0.00	184	868	1.01	2.28	3.01	-
							85	191	1.10	1.20	93	229	95	-	1.50	-	142	-	-	-	-	-	-	-	868	1.01	-	-	1.89
							85	191	1.35	1.35	115	257	-	-	-	-	-	-	-	-	-	-	-	-	868	1.01	-	-	2.36
26	Comp. in gusset PL	Pmax	S1	E1	INSP3	3.75	85	191	1.10	1.20	93	229	90	1.50	0	135	-	CL1	Static	All	339	1.70	0.25	720	794	1.01	0.48	0.67	-
	Panel Point U2 & U6	[kN]					85	191	1.10	1.20	93	229	90	1.50	0	135	-	25T	Static	Other	178	1.70	0.30	394	794	1.01	0.88	1.22	-
							85	191	1.10	1.20	93	229	90	1.50	0	135	-	5T	Static	Other	108	1.70	0.00	184	794	1.01	1.87	2.61	-
							85	191	1.10	1.20	93	229	95	-	1.50	-	142	-	-	-	-	-	-	-	794	1.01	-	-	1.73
							85	191	1.35	1.35	115	257	-	-	-	-	-	-	-	-	-	-	-	-	794	1.01	-	-	2.16
27	Tens. in gusset PL	Pmax	S1	E1	INSP3	3.75	70	158	1.10	1.20	77	190	75	1.50	0	112	-	CL1	Static	All	281	1.70	0.25	597	735	1.01	0.61	0.80	-
	Panel Point L3 & L5	[kN]					70	158	1.10	1.20	77	190	75	1.50	0	112	-	25T	Static	Other	148	1.70	0.30	327	735	1.01	1.11	1.45	-
							70	158	1.10	1.20	77	190	75	1.50	0	112	-	5T	Static	Other	90	1.70	0.00	153	735	1.01	2.38	3.11	-
							70	158	1.10	1.20	77	190	79	-	1.50	-	118	-	-	-	-	-	-	-	735	1.01	-	-	1.93
							70	158	1.35	1.35	95	214	-	-	-	-	-	-	-	-	-	-	-	-	735	1.01	-	-	2.41
28	Comp. in gusset PL	Pmax	S1	E1	INSP3	3.75	29	66	1.10	1.20	32	79	31	1.50	0	46	-	CL1	Static	All	176	1.70	0.25	374	719	1.01	1.52	1.65	-
	Panel Point U4	[kN]					29	66	1.10	1.20	32	79	31	1.50	0	46	-	25T	Static	Other	110	1.70	0.30	244	719	1.01	2.33	2.52	-
							29	66	1.10	1.20	32	79	31	1.50	0	46	-	5T	Static	Other	55	1.70	0.00	94	719	1.01	6.06	6.55	-
							29	66	1.10	1.20	32	79	33	-	1.50	-	49	-	-	-	-	-	-	-	719	1.01	-	-	4.54
							29	66	1.35	1.35	40	89	-	-	-	-	-	-	-	-	-	-	-	-	719	1.01	-	-	5.66

Note: * indicates load reversal
DLA = 0 indicates lane load governs
DLA > 0 indicates truck load governs
LLCFs circled in this Table are recomputed in Section 4 of this report to investigate the effects of capacity loss due to corrosion or damage

■ GOVERNING LL CAPACITY FACTOR

- FOR MEMBERS

CL1	0.25	0.37	-
25T	0.38	0.57	-
5T	1.00	1.48	-
SNOW	-	-	1.50
DL	-	-	1.87

- FOR GUSSET PLATES

CL1	0.48	0.67	-
25T	0.88	1.22	-
5T	1.87	2.61	-
SNOW	-	-	1.73
DL	-	-	2.16

TABLE B5 - LOAD CAPACITY EVALUATION FOR TRUSS SYSTEM SPAN 3- ULS COMBINATIONS

BUCKLAND & TAYLOR LTD. Bridge Engineering

Notes:

1. Load rating method is referenced to CSA - S6 - 06, Section 14.
2. Evaluation procedure: ULS Method
3. Highway Class C (as per CSA-S6-06 Clause 1.4.2.2)
4. Evaluation was carried out for the following three live load models.
   CL1 - CL1-625 Truck or Lane Load traffic;
   25T - 25t review vehicle or Lane Load traffic;
   5T - 5t passenger vehicle or Lane Load traffic.
5. Inspection Level considered: "INSP3" for all structural components
6. Target reliability index from Table 14.5.
7. Dead load factors from Table 14.7. and 3.2.
8. Live load factors are from: - Table 14.8, for normal traffic (CL1-625) and pedestrain load.
   - Table 14.9, for normal traffic (alternative loading)
9. Resistance adjustment factor from Table 14.15.
10. Live load capacity factor as per Clause 14.15.2.1.
11. Material strength: fy = 210 MPa, fu = 420 MPa for Structural steel
    fu = 320 MPa for Rivet
    fc' = 15 MPa for Reinforced concrete
    fy = 230 MPa for Reinforcing steel

UNCORRODED

Elt. #	Element - Force effect	Effect	Target reliability index				Dead load						Snow load					Live load							Resistance		Live Load		C/D
		Units	Syst Behav	Elem Behav	Insp Level	Beta	Unfact. loads D1	Unfact. loads D2	Load factors D1	Load factors D2	Fact. loads D1	Fact. loads D2	Unfact. Loads	Load factor ULS1a	Load factor ULS1d	Fact.Loads ULS1a	Fact.Loads ULS1d	LL Model	Lat. Distr.	Type span	Unfact. Loads	Load factor	DLA factor	Fact. Loads	Fact Resist	Adjust Fact	Capacity Factor ULS1a (w/ snow)	Capacity Factor ULS1b (w/o snow)	ULS1d & ULS9 (w/o live)
1	Comp. in top chord	Pmax	S1	E1	INSP3	3.75	400	545	1.10	1.20	439	654	253	1.50	-	380	-	CL1	Static	All	514	1.70	0.25	1092	2979	1.01	1.41	1.75	-
	Member U1-U2, U8-U9	[kN]					400	545	1.10	1.20	439	654	253	1.50	-	380	-	25T	Static	Other	402	1.70	0.00	683	2979	1.01	2.25	2.80	-
	2010 & 2017						400	545	1.10	1.20	439	654	253	1.50	-	380	-	5T	Static	Other	255	1.70	0.00	434	2979	1.01	3.54	4.42	-
							400	545	1.10	1.20	439	654	267	-	1.50	-	401	-	-	-	-	-	-	-	2979	1.01	-	-	2.01
							400	545	1.35	1.35	539	736	-	-	-	-	-	-	-	-	-	-	-	-	2979	1.01	-	-	2.36
2	Comp. in top chord	Pmax	S1	E1	INSP3	3.75	400	545	1.10	1.20	439	654	253	1.50	-	380	-	CL1	Static	All	514	1.70	0.25	1092	4902	1.20	4.04	4.39	-
	Connection U1-U2, U8-U9	[kN]					400	545	1.10	1.20	439	654	253	1.50	-	380	-	25T	Static	Other	402	1.70	0.00	683	4902	1.20	6.45	7.01	-
	2010 & 2017						400	545	1.10	1.20	439	654	253	1.50	-	380	-	5T	Static	Other	255	1.70	0.00	434	4902	1.20	10.16	11.04	-
							400	545	1.10	1.20	439	654	267	-	1.50	-	401	-	-	-	-	-	-	-	4902	1.20	-	-	3.94
							400	545	1.35	1.35	539	736	-	-	-	-	-	-	-	-	-	-	-	-	4902	1.20	-	-	4.61
3	Comp. in top chord	Pmax	S1	E1	INSP3	3.75	400	545	1.10	1.20	439	654	253	1.50	-	380	-	CL1	Static	All	514	1.70	0.25	1092	2979	1.01	1.41	1.75	-
	Member U2-U3, U7-U8	[kN]					400	545	1.10	1.20	439	654	253	1.50	-	380	-	25T	Static	Other	402	1.70	0.00	683	2979	1.01	2.25	2.80	-
	2011 & 2016						400	545	1.10	1.20	439	654	253	1.50	-	380	-	5T	Static	Other	255	1.70	0.00	434	2979	1.01	3.54	4.42	-
							400	545	1.10	1.20	439	654	267	-	1.50	-	401	-	-	-	-	-	-	-	2979	1.01	-	-	2.01
							400	545	1.35	1.35	539	736	-	-	-	-	-	-	-	-	-	-	-	-	2979	1.01	-	-	2.36
4	Comp. in top chord	Pmax	S1	E1	INSP3	3.75	400	545	1.10	1.20	439	654	253	1.50	-	380	-	CL1	Static	All	514	1.70	0.25	1092	3994	1.20	3.04	3.39	-
	Connection U2-U3, U7-U8	[kN]					400	545	1.10	1.20	439	654	253	1.50	-	380	-	25T	Static	Other	402	1.70	0.00	683	3994	1.20	4.86	5.41	-
	2011 & 2016						400	545	1.10	1.20	439	654	253	1.50	-	380	-	5T	Static	Other	255	1.70	0.00	434	3994	1.20	7.65	8.53	-
							400	545	1.10	1.20	439	654	267	-	1.50	-	401	-	-	-	-	-	-	-	3994	1.20	-	-	3.21
							400	545	1.35	1.35	539	736	-	-	-	-	-	-	-	-	-	-	-	-	3994	1.20	-	-	3.76
5	Comp. in top chord	Pmax	S1	E1	INSP3	3.75	601	820	1.10	1.20	661	984	381	1.50	-	571	-	CL1	Static	All	757	1.70	0.25	1608	3808	1.01	1.01	1.37	-
	Member U3-U4, U6-U7	[kN]					601	820	1.10	1.20	661	984	381	1.50	-	571	-	25T	Static	Other	600	1.70	0.00	1020	3808	1.01	1.60	2.16	-
	2012 & 2015						601	820	1.10	1.20	661	984	381	1.50	-	571	-	5T	Static	Other	383	1.70	0.00	651	3808	1.01	2.50	3.38	-
							601	820	1.10	1.20	661	984	402	-	1.50	-	603	-	-	-	-	-	-	-	3808	1.01	-	-	1.71
							601	820	1.35	1.35	811	1107	-	-	-	-	-	-	-	-	-	-	-	-	3808	1.01	-	-	2.01
6	Comp. in top chord	Pmax	S1	E1	INSP3	3.75	601	820	1.10	1.20	661	984	381	1.50	-	571	-	CL1	Static	All	757	1.70	0.25	1608	2933	1.81	1.92	2.28	-
	Connection U3-U4, U6-U7	[kN]					601	820	1.10	1.20	661	984	381	1.50	-	571	-	25T	Static	Other	600	1.70	0.00	1020	2933	1.81	3.03	3.59	-
	2012 & 2015						601	820	1.10	1.20	661	984	381	1.50	-	571	-	5T	Static	Other	383	1.70	0.00	651	2933	1.81	4.75	5.63	-
							601	820	1.10	1.20	661	984	402	-	1.50	-	603	-	-	-	-	-	-	-	2933	1.81	-	-	2.36
							601	820	1.35	1.35	811	1107	-	-	-	-	-	-	-	-	-	-	-	-	2933	1.81	-	-	2.77
7	Comp. in top chord	Pmax	S1	E1	INSP3	3.75	601	820	1.10	1.20	661	984	381	1.50	-	571	-	CL1	Static	All	757	1.70	0.25	1608	3808	1.01	1.01	1.37	-
	Member U4-U5, U5-U6	[kN]					601	820	1.10	1.20	661	984	381	1.50	-	571	-	25T	Static	Other	600	1.70	0.00	1020	3808	1.01	1.60	2.16	-
	2013 & 2014						601	820	1.10	1.20	661	984	381	1.50	-	571	-	5T	Static	Other	383	1.70	0.00	651	3808	1.01	2.50	3.38	-
							601	820	1.10	1.20	661	984	402	-	1.50	-	603	-	-	-	-	-	-	-	3808	1.01	-	-	1.71
							601	820	1.35	1.35	811	1107	-	-	-	-	-	-	-	-	-	-	-	-	3808	1.01	-	-	2.01
8	Comp. in top chord	Pmax	S1	E1	INSP3	3.75	601	820	1.10	1.20	661	984	381	1.50	-	571	-	CL1	Static	All	757	1.70	0.25	1608	2933	1.81	1.92	2.28	-
	Connection U4-U5, U5-U6	[kN]					601	820	1.10	1.20	661	984	381	1.50	-	571	-	25T	Static	Other	600	1.70	0.00	1020	2933	1.81	3.03	3.59	-
	2013 & 2014						601	820	1.10	1.20	661	984	381	1.50	-	571	-	5T	Static	Other	383	1.70	0.00	651	2933	1.81	4.75	5.63	-
							601	820	1.10	1.20	661	984	402	-	1.50	-	603	-	-	-	-	-	-	-	2933	1.81	-	-	2.36
							601	820	1.35	1.35	811	1107	-	-	-	-	-	-	-	-	-	-	-	-	2933	1.81	-	-	2.77
9	Tens. in bot chord	Pmax	S1	E3	INSP3	3.00	224	305	1.07	1.14	239	348	142	1.50	-	213	-	CL1	Static	All	286	1.49	0.25	532	1846	1.18	2.59	2.99	-
	Member L0-L1, L9-L10	[kN]					224	305	1.07	1.14	239	348	142	1.50	-	213	-	25T	Static	Other	226	1.49	0.00	337	1846	1.18	4.09	4.72	-
	3009 & 3018						224	305	1.07	1.14	239	348	142	1.50	-	213	-	5T	Static	Other	143	1.49	0.00	213	1846	1.18	6.46	7.45	-
							224	305	1.07	1.14	239	348	149	-	1.50	-	224	-	-	-	-	-	-	-	1846	1.18	-	-	2.68
							224	305	1.35	1.35	302	412	-	-	-	-	-	-	-	-	-	-	-	-	1846	1.18	-	-	3.05
10	Tens. in bot chord	Pmax	S1	E1	INSP3	3.75	224	305	1.10	1.20	246	366	142	1.50	-	213	-	CL1	Static	All	286	1.70	0.25	607	917	1.81	1.37	1.72	-
	Connection L0-L1, L9-L10	[kN]					224	305	1.10	1.20	246	366	142	1.50	-	213	-	25T	Static	Other	226	1.70	0.00	384	917	1.81	2.17	2.72	-
	3009 & 3018						224	305	1.10	1.20	246	366	142	1.50	-	213	-	5T	Static	Other	143	1.70	0.00	244	917	1.81	3.42	4.30	-
							224	305	1.10	1.20	246	366	149	-	1.50	-	224	-	-	-	-	-	-	-	917	1.81	-	-	1.98
							224	305	1.35	1.35	302	412	-	-	-	-	-	-	-	-	-	-	-	-	917	1.81	-	-	2.32

Note: * indicates load reversal
DLA = 0 indicates lane load governs
DLA > 0 indicates truck load governs
LLCFs circled in this Table are recomputed in Section 4 of this report to investigate the effects of capacity loss due to corrosion or damage

TABLE B5 - LOAD CAPACITY EVALUATION FOR TRUSS SYSTEM SPAN 3- ULS COMBINATIONS

BUCKLAND & TAYLOR LTD.
Bridge Engineering

Notes:

1. Load rating method is referenced to CSA - S6 - 06, Section 14.
2. Evaluation procedure: ULS Method
3. Highway Class C (as per CSA-S6-06 Clause 1.4.2.2)
4. Evaluation was carried out for the following three live load models.
   CL1 - CL1-625 Truck or Lane Load traffic;
   25T - 25t review vehicle or Lane Load traffic;
   5T - 5t passenger vehicle or Lane Load traffic.
5. Inspection Level considered: "INSP3" for all structural components
6. Target reliability index from Table 14.5.
7. Dead load factors from Table 14.7. and 3.2.
8. Live load factors are from: - Table 14.8, for normal traffic (CL1-625) and pedestrain load.
   - Table 14.9, for normal traffic (alternative loading)
9. Resistance adjustment factor from Table 14.15.
10. Live load capacity factor as per Clause 14.15.2.1.
11. Material strength: fy = 210 MPa, fu = 420 MPa for Structural steel
    fu = 320 MPa for Rivet
    fc' = 15 MPa for Reinforced concrete
    fy = 230 MPa for Reinforcing steel

UNCORRODED

Elt. #	Element - Force effect	Effect	Target reliability index				Dead load						Snow load					Live load							Resistance		w/ snow	w/o snow	w/o live
		Units	Syst	Elem	Insp	Beta	Unfact. loads		Load factors		Fact. loads		Unfact.	Load factor		Fact.Loads		LL	Lat.	Type	Unfact.	Load	DLA	Fact.	Fact	Adjust	Live Load Capacity Factor		C/D ULS1d
			Behav	Behav	Level		D1	D2	D1	D2	D1	D2	Loads	ULS1a	ULS1d	ULS1a	ULS1d	Model	Distr.	span	Loads	factor	factor	Loads	Resist	Fact	ULS1a	ULS1b	& ULS9
11	Tens. in bot chord	Pmax	S1	E3	INSP3	3.00	224	305	1.07	1.14	239	348	142	1.50	-	213	-	CL1	Static	All	286	1.49	0.25	532	2564	1.01	3.37	3.76	-
	Member L1-L2, L8-L9	[kN]					224	305	1.07	1.14	239	348	142	1.50	-	213	-	25T	Static	Other	226	1.49	0.00	337	2564	1.01	5.31	5.94	-
	3010 & 3017						224	305	1.07	1.14	239	348	142	1.50	-	213	-	5T	Static	Other	143	1.49	0.00	213	2564	1.01	8.39	9.38	-
							224	305	1.07	1.14	239	348	149	-	1.50	-	224	-	-	-	-	-	-	-	2564	1.01	-	-	3.19
							224	305	1.35	1.35	302	412	-	-	-	-	-	-	-	-	-	-	-	-	2564	1.01	-	-	3.63
12	Tens. in bot chord	Pmax	S1	E1	INSP3	3.75	224	305	1.10	1.20	246	366	142	1.50	-	213	-	CL1	Static	All	286	1.70	0.25	607	917	1.81	1.37	1.72	-
	Connection L1-L2, L8-L9	[kN]					224	305	1.10	1.20	246	366	142	1.50	-	213	-	25T	Static	Other	226	1.70	0.00	384	917	1.81	2.17	2.72	-
	3010 & 3017						224	305	1.10	1.20	246	366	142	1.50	-	213	-	5T	Static	Other	143	1.70	0.00	244	917	1.81	3.42	4.30	-
							224	305	1.10	1.20	246	366	149	-	1.50	-	224	-	-	-	-	-	-	-	917	1.81	-	-	1.98
							224	305	1.35	1.35	302	412	-	-	-	-	-	-	-	-	-	-	-	-	917	1.81	-	-	2.32
13	Tens. in bot chord	Pmax	S1	E3	INSP3	3.00	525	717	1.07	1.14	562	817	333	1.50	-	499	-	CL1	Static	All	670	1.49	0.25	1248	3390	1.01	1.24	1.64	-
	Member L2-L3, L7-L8	[kN]					525	717	1.07	1.14	562	817	333	1.50	-	499	-	25T	Static	Other	526	1.49	0.00	784	3390	1.01	1.97	2.61	-
	3011 & 3016						525	717	1.07	1.14	562	817	333	1.50	-	499	-	5T	Static	Other	335	1.49	0.00	499	3390	1.01	3.10	4.10	-
							525	717	1.07	1.14	562	817	351	-	1.50	-	527	-	-	-	-	-	-	-	3390	1.01	-	-	1.80
							525	717	1.35	1.35	709	967	-	-	-	-	-	-	-	-	-	-	-	-	3390	1.01	-	-	2.04
14	Tens. in bot chord	Pmax	S1	E1	INSP3	3.75	525	717	1.10	1.20	577	860	333	1.50	-	499	-	CL1	Static	All	670	1.70	0.25	1424	2017	1.81	1.20	1.55	-
	Connection L2-L3, L7-L8	[kN]					525	717	1.10	1.20	577	860	333	1.50	-	499	-	25T	Static	Other	526	1.70	0.00	894	2017	1.81	1.92	2.47	-
	3011 & 3016						525	717	1.10	1.20	577	860	333	1.50	-	499	-	5T	Static	Other	335	1.70	0.00	570	2017	1.81	3.01	3.88	-
							525	717	1.10	1.20	577	860	351	-	1.50	-	527	-	-	-	-	-	-	-	2017	1.81	-	-	1.86
							525	717	1.35	1.35	709	967	-	-	-	-	-	-	-	-	-	-	-	-	2017	1.81	-	-	2.18
15	Tens. in bot chord	Pmax	S1	E3	INSP3	3.00	525	717	1.07	1.14	562	817	333	1.50	-	499	-	CL1	Static	All	670	1.49	0.25	1248	3390	1.01	1.24	1.64	-
	Member L3-L4, L6-L7	[kN]					525	717	1.07	1.14	562	817	333	1.50	-	499	-	25T	Static	Other	526	1.49	0.00	784	3390	1.01	1.97	2.61	-
	3012 & 3015						525	717	1.07	1.14	562	817	333	1.50	-	499	-	5T	Static	Other	335	1.49	0.00	499	3390	1.01	3.10	4.10	-
							525	717	1.07	1.14	562	817	351	-	1.50	-	527	-	-	-	-	-	-	-	3390	1.01	-	-	1.80
							525	717	1.35	1.35	709	967	-	-	-	-	-	-	-	-	-	-	-	-	3390	1.01	-	-	2.04
16	Tens. in bot chord	Pmax	S1	E1	INSP3	3.75	525	717	1.10	1.20	577	860	333	1.50	-	499	-	CL1	Static	All	670	1.70	0.25	1424	2017	1.81	1.20	1.55	-
	Connection L3-L4, L6-L7	[kN]					525	717	1.10	1.20	577	860	333	1.50	-	499	-	25T	Static	Other	526	1.70	0.00	894	2017	1.81	1.92	2.47	-
	3012 & 3015						525	717	1.10	1.20	577	860	333	1.50	-	499	-	5T	Static	Other	335	1.70	0.00	570	2017	1.81	3.01	3.88	-
							525	717	1.10	1.20	577	860	351	-	1.50	-	527	-	-	-	-	-	-	-	2017	1.81	-	-	1.86
							525	717	1.35	1.35	709	967	-	-	-	-	-	-	-	-	-	-	-	-	2017	1.81	-	-	2.18
17	Tens. in bot chord	Pmax	S1	E3	INSP3	3.00	626	854	1.07	1.14	670	974	397	1.50	-	595	-	CL1	Static	All	777	1.49	0.25	1447	4147	1.01	1.35	1.76	-
	Member L4-L5, L5-L6	[kN]					626	854	1.07	1.14	670	974	397	1.50	-	595	-	25T	Static	Other	624	1.49	0.00	929	4147	1.01	2.10	2.74	-
	3013 & 3014						626	854	1.07	1.14	670	974	397	1.50	-	595	-	5T	Static	Other	399	1.49	0.00	594	4147	1.01	3.28	4.28	-
							626	854	1.07	1.14	670	974	419	-	1.50	-	628	-	-	-	-	-	-	-	4147	1.01	-	-	1.84
							626	854	1.35	1.35	845	1153	-	-	-	-	-	-	-	-	-	-	-	-	4147	1.01	-	-	2.10
18	Tens. in bot chord	Pmax	S1	E1	INSP3	3.75	626	854	1.10	1.20	688	1025	397	1.50	-	595	-	CL1	Static	All	777	1.70	0.25	1651	2567	1.81	1.42	1.78	-
	Connection L4-L5, L5-L6	[kN]					626	854	1.10	1.20	688	1025	397	1.50	-	595	-	25T	Static	Other	624	1.70	0.00	1060	2567	1.81	2.20	2.77	-
	3013 & 3014						626	854	1.10	1.20	688	1025	397	1.50	-	595	-	5T	Static	Other	399	1.70	0.00	678	2567	1.81	3.45	4.33	-
							626	854	1.10	1.20	688	1025	419	-	1.50	-	628	-	-	-	-	-	-	-	2567	1.81	-	-	1.98
							626	854	1.35	1.35	845	1153	-	-	-	-	-	-	-	-	-	-	-	-	2567	1.81	-	-	2.33
19	Comp. in diagonal	Pmax	S1	E1	INSP3	3.75	372	508	1.10	1.20	409	610	236	1.50	-	354	-	CL1	Static	All	473	1.70	0.25	1004	2434	1.01	1.08	1.43	-
	Member L0-U1, U9-L10	[kN]					372	508	1.10	1.20	409	610	236	1.50	-	354	-	25T	Static	Other	375	1.70	0.00	637	2434	1.01	1.70	2.26	-
	4015 & 4024						372	508	1.10	1.20	409	610	236	1.50	-	354	-	5T	Static	Other	238	1.70	0.00	404	2434	1.01	2.68	3.56	-
							372	508	1.10	1.20	409	610	249	-	1.50	-	374	-	-	-	-	-	-	-	2434	1.01	-	-	1.76
							372	508	1.35	1.35	503	686	-	-	-	-	-	-	-	-	-	-	-	-	2434	1.01	-	-	2.07
20	Comp. in diagonal	Pmax	S1	E1	INSP3	3.75	372	508	1.10	1.20	409	610	236	1.50	-	354	-	CL1	Static	All	473	1.70	0.25	1004	3300	1.81	4.58	4.93	-
	Connection L0-U1, U9-L10	[kN]					372	508	1.10	1.20	409	610	236	1.50	-	354	-	25T	Static	Other	375	1.70	0.00	637	3300	1.81	7.22	7.78	-
	4015 & 4024						372	508	1.10	1.20	409	610	236	1.50	-	354	-	5T	Static	Other	238	1.70	0.00	404	3300	1.81	11.39	12.26	-
							372	508	1.10	1.20	409	610	249	-	1.50	-	374	-	-	-	-	-	-	-	3300	1.81	-	-	4.29
							372	508	1.35	1.35	503	686	-	-	-	-	-	-	-	-	-	-	-	-	3300	1.81	-	-	5.02

Note: * indicates load reversal
DLA = 0 indicates lane load governs
DLA > 0 indicates truck load governs
LLCFs circled in this Table are recomputed in Section 4 of this report to investigate the effects of capacity loss due to corrosion or damage

TABLE B5 - LOAD CAPACITY EVALUATION FOR TRUSS SYSTEM SPAN 3- ULS COMBINATIONS

BUCKLAND & TAYLOR LTD. Bridge Engineering

Notes:
1. Load rating method is referenced to CSA - S6 - 06, Section 14.
2. Evaluation procedure: ULS Method
3. Highway Class C (as per CSA-S6-06 Clause 1.4.2.2)
4. Evaluation was carried out for the following three live load models.
   CL1 - CL1-625 Truck or Lane Load traffic;
   25T - 25t review vehicle or Lane Load traffic;
   5T - 5t passenger vehicle or Lane Load traffic.
5. Inspection Level considered: "INSP3" for all structural components
6. Target reliability index from Table 14.5.
7. Dead load factors from Table 14.7. and 3.2.
8. Live load factors are from: - Table 14.8, for normal traffic (CL1-625) and pedestrain load.
   - Table 14.9, for normal traffic (alternative loading)
9. Resistance adjustment factor from Table 14.15.
10. Live load capacity factor as per Clause 14.15.2.1.
11. Material strength: fy = 210 MPa, fu = 420 MPa for Structural steel
    fu = 320 MPa for Rivet
    fc' = 15 MPa for Reinforced concrete
    fy = 230 MPa for Reinforcing steel

UNCORRODED

Elt. #	Element - Force effect	Effect	Target reliability index				Dead load						Snow load					Live load							Resistance		w/ snow	w/o snow	w/o live
		Units	Syst	Elem	Insp	Beta	Unfact. loads		Load factors		Fact. loads		Unfact.	Load factor		Fact.Loads		LL	Lat.	Type	Unfact.	Load	DLA	Fact.	Fact	Adjust	Live Load Capacity Factor		C/D ULS1d
			Behav	Behav	Level		D1	D2	D1	D2	D1	D2	Loads	ULS1a	ULS1d	ULS1a	ULS1d	Model	Distr.	span	Loads	factor	factor	Loads	Resist	Fact	ULS1a	ULS1b	& ULS9
21	Tens. in diagonal	Pmax	S1	E3	INSP3	3.00	290	395	1.07	1.14	310	451	184	1.50	-	276	-	CL1	Static	All	411	1.49	0.25	766	1858	1.01	1.10	1.46	-
	Member U1-L2, L8-U9	[kN]					290	395	1.07	1.14	310	451	184	1.50	-	276	-	25T	Static	Other	312	1.49	0.00	465	1858	1.01	1.81	2.40	-
	4016 & 4023						290	395	1.07	1.14	310	451	184	1.50	-	276	-	5T	Static	Other	191	1.49	0.00	285	1858	1.01	2.95	3.92	-
							290	395	1.07	1.14	310	451	194	-	1.50	-	291	-	-	-	-	-	-	-	1858	1.01	-	-	1.79
							290	395	1.35	1.35	391	534	-	-	-	-	-	-	-	-	-	-	-	-	1858	1.01	-	-	2.03
22	Tens. in diagonal	Pmax	S1	E1	INSP3	3.75	290	395	1.10	1.20	319	474	184	1.50	-	276	-	CL1	Static	All	411	1.70	0.25	874	1887	1.18	1.32	1.64	-
	Connection U1-L2, L8-U9	[kN]					290	395	1.10	1.20	319	474	184	1.50	-	276	-	25T	Static	Other	312	1.70	0.00	531	1887	1.18	2.18	2.70	-
	4016 & 4023						290	395	1.10	1.20	319	474	184	1.50	-	276	-	5T	Static	Other	191	1.70	0.00	325	1887	1.18	3.57	4.41	-
							290	395	1.10	1.20	319	474	194	-	1.50	-	291	-	-	-	-	-	-	-	1887	1.18	-	-	2.05
							290	395	1.35	1.35	391	534	-	-	-	-	-	-	-	-	-	-	-	-	1887	1.18	-	-	2.41
23	Comp. in diagonal	Pmax	S1	E1	INSP3	3.75	207	283	1.10	1.20	228	339	131	1.50	-	197	-	CL1	Static	All	350	1.70	0.25	744	1252	1.01	0.67	0.94	-
	Member L2-U3, U7-L8	[kN]					207	283	1.10	1.20	228	339	131	1.50	-	197	-	25T	Static	Other	254	1.70	0.00	433	1252	1.01	1.16	1.61	-
	4017 & 4022						207	283	1.10	1.20	228	339	131	1.50	-	197	-	5T	Static	Other	150	1.70	0.00	254	1252	1.01	1.97	2.74	-
							207	283	1.10	1.20	228	339	139	-	1.50	-	208	-	-	-	-	-	-	-	1252	1.01	-	-	1.63
							207	283	1.35	1.35	279	382	-	-	-	-	-	-	-	-	-	-	-	-	1252	1.01	-	-	1.91
24	Comp. in diagonal	Pmax	S1	E1	INSP3	3.75	207	283	1.10	1.20	228	339	131	1.50	-	197	-	CL1	Static	All	350	1.70	0.25	744	1925	1.81	3.65	3.92	-
	Connection L2-U3, U7-L8	[kN]					207	283	1.10	1.20	228	339	131	1.50	-	197	-	25T	Static	Other	254	1.70	0.00	433	1925	1.81	6.29	6.74	-
	4017 & 4022						207	283	1.10	1.20	228	339	131	1.50	-	197	-	5T	Static	Other	150	1.70	0.00	254	1925	1.81	10.69	11.47	-
							207	283	1.10	1.20	228	339	139	-	1.50	-	208	-	-	-	-	-	-	-	1925	1.81	-	-	4.50
							207	283	1.35	1.35	279	382	-	-	-	-	-	-	-	-	-	-	-	-	1925	1.81	-	-	5.27
25	Tens. in diagonal	Pmax	S1	E3	INSP3	3.00	124	170	1.07	1.14	133	194	79	1.50	-	118	-	CL1	Static	All	289	1.49	0.25	537	994	1.01	1.04	1.26	-
	Member U3-L4, L6-U7	[kN]					124	170	1.07	1.14	133	194	79	1.50	-	118	-	25T	Static	Other	202	1.49	0.00	301	994	1.01	1.86	2.25	-
	4018 & 4021						124	170	1.07	1.14	133	194	79	1.50	-	118	-	5T	Static	Other	113	1.49	0.00	169	994	1.01	3.31	4.02	-
							124	170	1.07	1.14	133	194	83	-	1.50	-	125	-	-	-	-	-	-	-	994	1.01	-	-	2.22
							124	170	1.35	1.35	168	229	-	-	-	-	-	-	-	-	-	-	-	-	994	1.01	-	-	2.53
26	Tens. in diagonal	Pmax	S1	E1	INSP3	3.75	124	170	1.10	1.20	137	204	79	1.50	-	118	-	CL1	Static	All	289	1.70	0.25	613	1174	1.18	1.51	1.70	-
	Connection U3-L4, L6-U7	[kN]					124	170	1.10	1.20	137	204	79	1.50	-	118	-	25T	Static	Other	202	1.70	0.00	343	1174	1.18	2.70	3.04	-
	4018 & 4021						124	170	1.10	1.20	137	204	79	1.50	-	118	-	5T	Static	Other	113	1.70	0.00	192	1174	1.18	4.81	5.43	-
							124	170	1.10	1.20	137	204	83	-	1.50	-	125	-	-	-	-	-	-	-	1174	1.18	-	-	2.98
							124	170	1.35	1.35	168	229	-	-	-	-	-	-	-	-	-	-	-	-	1174	1.18	-	-	3.49
27	Comp. in diagonal	Pmax	S1	E1	INSP3	3.75	42	57	1.10	1.20	46	68	26	1.50	-	40	-	CL1	Static	All	227	1.70	0.25	483	408	1.01	0.53	0.62	-
	Member L4-U5, U5-L6	[kN]					42	57	1.10	1.20	46	68	26	1.50	-	40	-	25T	Static	Other	154	1.70	0.00	262	408	1.01	0.98	1.14	-
	4019 & 4020						42	57	1.10	1.20	46	68	26	1.50	-	40	-	5T	Static	Other	82	1.70	0.00	139	408	1.01	1.86	2.14	-
							42	57	1.10	1.20	46	68	28	-	1.50	-	42	-	-	-	-	-	-	-	408	1.01	-	-	2.64
							42	57	1.35	1.35	56	77	-	-	-	-	-	-	-	-	-	-	-	-	408	1.01	-	-	3.09
28	Comp. in diagonal	Pmax	S1	E1	INSP3	3.75	42	57	1.10	1.20	46	68	26	1.50	-	40	-	CL1	Static	All	227	1.70	0.25	483	917	1.81	3.12	3.20	-
	Connection L4-U5, U5-L6	[kN]					42	57	1.10	1.20	46	68	26	1.50	-	40	-	25T	Static	Other	154	1.70	0.00	262	917	1.81	5.74	5.89	-
	4019 & 4020						42	57	1.10	1.20	46	68	26	1.50	-	40	-	5T	Static	Other	82	1.70	0.00	139	917	1.81	10.83	11.12	-
							42	57	1.10	1.20	46	68	28	-	1.50	-	42	-	-	-	-	-	-	-	917	1.81	-	-	10.62
							42	57	1.35	1.35	56	77	-	-	-	-	-	-	-	-	-	-	-	-	917	1.81	-	-	12.45
29	Comp. in vertical	Pmax	S1	E1	INSP3	3.75	116	231	1.10	1.20	128	277	109	1.50	-	163	-	CL1	Static	All	362	1.70	0.25	769	897	1.01	0.44	0.65	-
	Member U0-L0	[kN]					116	231	1.10	1.20	128	277	109	1.50	-	163	-	25T	Static	Other	236	1.70	0.00	401	897	1.01	0.84	1.25	-
	5012						116	231	1.10	1.20	128	277	109	1.50	-	163	-	5T	Static	Other	123	1.70	0.00	210	897	1.01	1.61	2.39	-
							116	231	1.10	1.20	128	277	115	-	1.50	-	172	-	-	-	-	-	-	-	897	1.01	-	-	1.57
							116	231	1.35	1.35	157	312	-	-	-	-	-	-	-	-	-	-	-	-	897	1.01	-	-	1.93
30	Comp. in vertical	Pmax	S1	E1	INSP3	3.75	116	231	1.10	1.20	128	277	109	1.50	-	163	-	CL1	Static	All	362	1.70	0.25	769	733	1.81	0.99	1.20	-
	Connection U0-L0	[kN]					116	231	1.10	1.20	128	277	109	1.50	-	163	-	25T	Static	Other	236	1.70	0.00	401	733	1.81	1.89	2.30	-
	5012						116	231	1.10	1.20	128	277	109	1.50	-	163	-	5T	Static	Other	123	1.70	0.00	210	733	1.81	3.62	4.40	-
							116	231	1.10	1.20	128	277	115	-	1.50	-	172	-	-	-	-	-	-	-	733	1.81	-	-	2.30
							116	231	1.35	1.35	157	312	-	-	-	-	-	-	-	-	-	-	-	-	733	1.81	-	-	2.83

Note: * indicates load reversal
DLA = 0 indicates lane load governs
DLA > 0 indicates truck load governs
LLCFs circled in this Table are recomputed in Section 4 of this report to investigate the effects of capacity loss due to corrosion or damage

TABLE B5 - LOAD CAPACITY EVALUATION FOR TRUSS SYSTEM SPAN 3- ULS COMBINATIONS

BUCKLAND & TAYLOR LTD.
Bridge Engineering

Notes:

1. Load rating method is referenced to CSA - S6 - 06, Section 14.
2. Evaluation procedure: ULS Method
3. Highway Class C (as per CSA-S6-06 Clause 1.4.2.2)
4. Evaluation was carried out for the following three live load models.
   CL1 - CL1-625 Truck or Lane Load traffic;
   25T - 25t review vehicle or Lane Load traffic;
   5T - 5t passenger vehicle or Lane Load traffic.
5. Inspection Level considered: "INSP3" for all structural components
6. Target reliability index from Table 14.5.
7. Dead load factors from Table 14.7. and 3.2.
8. Live load factors are from: - Table 14.8, for normal traffic (CL1-625) and pedestrain load.
   - Table 14.9, for normal traffic (alternative loading)
9. Resistance adjustment factor from Table 14.15.
10. Live load capacity factor as per Clause 14.15.2.1.
11. Material strength: fy = 210 MPa, fu = 420 MPa for Structural steel
    fu = 320 MPa for Rivet
    fc' = 15 MPa for Reinforced concrete
    fy = 230 MPa for Reinforcing steel

UNCORRODED

Elt. #	Element - Force effect	Effect	Target reliability index				Dead load						Snow load					Live load							Resistance		w/ snow	w/o snow	w/o live
		Units	Syst Behav	Elem Behav	Insp Level	Beta	Unfact. loads		Load factors		Fact. loads		Unfact. Loads	Load factor		Fact.Loads		LL Model	Lat. Distr.	Type span	Unfact. Loads	Load factor	DLA factor	Fact. Loads	Fact Resist	Adjust Fact	Live Load Capacity Factor		C/D ULS1d & ULS9
							D1	D2	D1	D2	D1	D2		ULS1a	ULS1d	ULS1a	ULS1d										ULS1a	ULS1b	
31	Comp. in vertical	Pmax	S1	E1	INSP3	3.75	66	90	1.10	1.20	72	108	42	1.50	-	63	-	CL1	Static	All	194	1.70	0.30	428	496	1.01	0.60	0.75	-
	Member U2-L2, U8-L8	[kN]					66	90	1.10	1.20	72	108	42	1.50	-	63	-	25T	Static	Other	128	1.70	0.30	283	496	1.01	0.91	1.13	-
	5014 & 5020						66	90	1.10	1.20	72	108	42	1.50	-	63	-	5T	Static	Other	63	1.70	0.00	106	496	1.01	2.43	3.02	-
							66	90	1.10	1.20	72	108	44	-	1.50	-	66	-	-	-	-	-	-	-	496	1.01	-	-	2.04
							66	90	1.35	1.35	89	121	-	-	-	-	-	-	-	-	-	-	-	-	496	1.01	-	-	2.39
32	Comp. in vertical	Pmax	S1	E1	INSP3	3.75	66	90	1.10	1.20	72	108	42	1.50	-	63	-	CL1	Static	All	194	1.70	0.30	428	1100	1.20	2.52	2.66	-
	Connection U2-L2, U8-L8	[kN]					66	90	1.10	1.20	72	108	42	1.50	-	63	-	25T	Static	Other	128	1.70	0.30	283	1100	1.20	3.81	4.03	-
	5014 & 5020						66	90	1.10	1.20	72	108	42	1.50	-	63	-	5T	Static	Other	63	1.70	0.00	106	1100	1.20	10.12	10.71	-
							66	90	1.10	1.20	72	108	44	-	1.50	-	66	-	-	-	-	-	-	-	1100	1.20	-	-	5.37
							66	90	1.35	1.35	89	121	-	-	-	-	-	-	-	-	-	-	-	-	1100	1.20	-	-	6.29
33	Comp. in vertical	Pmax	S1	E1	INSP3	3.75	66	90	1.10	1.20	72	108	42	1.50	-	63	-	CL1	Static	All	195	1.70	0.30	430	496	1.01	0.60	0.75	-
	Member U4-L4, U6-L6	[kN]					66	90	1.10	1.20	72	108	42	1.50	-	63	-	25T	Static	Other	128	1.70	0.30	283	496	1.01	0.91	1.14	-
	5016 & 5018						66	90	1.10	1.20	72	108	42	1.50	-	63	-	5T	Static	Other	63	1.70	0.00	106	496	1.01	2.43	3.02	-
							66	90	1.10	1.20	72	108	44	-	1.50	-	66	-	-	-	-	-	-	-	496	1.01	-	-	2.04
							66	90	1.35	1.35	89	121	-	-	-	-	-	-	-	-	-	-	-	-	496	1.01	-	-	2.39
34	Comp. in vertical	Pmax	S1	E1	INSP3	3.75	66	90	1.10	1.20	72	108	42	1.50	-	63	-	CL1	Static	All	195	1.70	0.30	430	1100	1.20	2.51	2.65	-
	Connection U4-L4, U6-L6	[kN]					66	90	1.10	1.20	72	108	42	1.50	-	63	-	25T	Static	Other	128	1.70	0.30	283	1100	1.20	3.81	4.03	-
	5016 & 5018						66	90	1.10	1.20	72	108	42	1.50	-	63	-	5T	Static	Other	63	1.70	0.00	106	1100	1.20	10.12	10.71	-
							66	90	1.10	1.20	72	108	44	-	1.50	-	66	-	-	-	-	-	-	-	1100	1.20	-	-	5.37
							66	90	1.35	1.35	89	121	-	-	-	-	-	-	-	-	-	-	-	-	1100	1.20	-	-	6.29
35	Tens. in gusset PL	Pmax	S1	E1	INSP3	3.75	87	194	1.10	1.20	95	232	91	1.50	0	137	-	CL1	Static	All	341	1.70	0.25	724	965	1.01	0.70	0.89	-
	Panel Point U0	[kN]					87	194	1.10	1.20	95	232	91	1.50	0	137	-	25T	Static	Other	179	1.70	0.30	396	965	1.01	1.29	1.63	-
							87	194	1.10	1.20	95	232	91	1.50	0	137	-	5T	Static	Other	109	1.70	0.00	186	965	1.01	2.75	3.49	-
							87	194	1.10	1.20	95	232	96	-	1.50	-	144	-	-	-	-	-	-	-	965	1.01	-	-	2.06
							87	194	1.35	1.35	117	261	-	-	-	-	-	-	-	-	-	-	-	-	965	1.01	-	-	2.58
36	Comp. in gusset PL	Pmax	S1	E1	INSP3	3.75	372	508	1.10	1.20	409	610	236	1.50	0	354	-	CL1	Static	All	473	1.70	0.25	1004	1682	1.01	0.32	0.68	-
	Panel Point L0 & L10	[kN]					372	508	1.10	1.20	409	610	236	1.50	0	354	-	25T	Static	Other	375	1.70	0.00	637	1682	1.01	0.51	1.07	-
							372	508	1.10	1.20	409	610	236	1.50	0	354	-	5T	Static	Other	238	1.70	0.00	404	1682	1.01	0.80	1.68	-
							372	508	1.10	1.20	409	610	249	-	1.50	-	374	-	-	-	-	-	-	-	1682	1.01	-	-	1.22
							372	508	1.35	1.35	503	686	-	-	-	-	-	-	-	-	-	-	-	-	1682	1.01	-	-	1.43
37	Shear in gusset PL	Vmax	S1	E1	INSP3	3.75	402	548	1.10	1.20	442	658	255	1.50	0	382	-	CL1	Static	All	536	1.70	0.25	1140	1931	1.01	0.41	0.75	-
	Panel Point U1 & U9	[kN]					402	548	1.10	1.20	442	658	255	1.50	0	382	-	25T	Static	Other	226	1.70	0.00	385	1931	1.01	1.22	2.21	-
							402	548	1.10	1.20	442	658	255	1.50	0	382	-	5T	Static	Other	99	1.70	0.00	168	1931	1.01	2.79	5.07	-
							402	548	1.10	1.20	442	658	269	-	1.50	-	403	-	-	-	-	-	-	-	1931	1.01	-	-	1.30
							402	548	1.35	1.35	542	740	-	-	-	-	-	-	-	-	-	-	-	-	1931	1.01	-	-	1.52
38	Tens. in gusset PL	Pmax	S1	E1	INSP3	3.75	290	395	1.10	1.20	319	474	184	1.50	0	276	-	CL1	Static	All	411	1.70	0.25	874	1738	1.01	0.79	1.10	-
	Panel Point L2 & L8	[kN]					290	395	1.10	1.20	319	474	184	1.50	0	276	-	25T	Static	Other	312	1.70	0.00	531	1738	1.01	1.29	1.81	-
							290	395	1.10	1.20	319	474	184	1.50	0	276	-	5T	Static	Other	191	1.70	0.00	325	1738	1.01	2.11	2.96	-
							290	395	1.10	1.20	319	474	194	-	1.50	-	291	-	-	-	-	-	-	-	1738	1.01	-	-	1.62
							290	395	1.35	1.35	391	534	-	-	-	-	-	-	-	-	-	-	-	-	1738	1.01	-	-	1.90
39	Comp. in gusset PL	Pmax	S1	E1	INSP3	3.75	207	283	1.10	1.20	228	339	131	1.50	0	197	-	CL1	Static	All	350	1.70	0.25	744	1318	1.01	0.76	1.03	-
	Panel Point U3 & U7	[kN]					207	283	1.10	1.20	228	339	131	1.50	0	197	-	25T	Static	Other	254	1.70	0.00	433	1318	1.01	1.31	1.77	-
							207	283	1.10	1.20	228	339	131	1.50	0	197	-	5T	Static	Other	150	1.70	0.00	254	1318	1.01	2.23	3.00	-
							207	283	1.10	1.20	228	339	139	-	1.50	-	208	-	-	-	-	-	-	-	1318	1.01	-	-	1.72
							207	283	1.35	1.35	279	382	-	-	-	-	-	-	-	-	-	-	-	-	1318	1.01	-	-	2.01
40	Tens. in gusset PL	Pmax	S1	E1	INSP3	3.75	525	717	1.10	1.20	577	860	333	1.50	0	499	-	CL1	Static	All	670	1.70	0.25	1424	2655	1.01	0.52	0.87	-
	Panel Point L4 & L6	[kN]					525	717	1.10	1.20	577	860	333	1.50	0	499	-	25T	Static	Other	526	1.70	0.00	894	2655	1.01	0.83	1.39	-
							525	717	1.10	1.20	577	860	333	1.50	0	499	-	5T	Static	Other	335	1.70	0.00	570	2655	1.01	1.31	2.18	-
							525	717	1.10	1.20	577	860	351	-	1.50	-	527	-	-	-	-	-	-	-	2655	1.01	-	-	1.37
							525	717	1.35	1.35	709	967	-	-	-	-	-	-	-	-	-	-	-	-	2655	1.01	-	-	1.60

Note: * indicates load reversal
DLA = 0 indicates lane load governs
DLA > 0 indicates truck load governs
LLCFs circled in this Table are recomputed in Section 4 of this report to investigate the effects of capacity loss due to corrosion or damage

TABLE B5 - LOAD CAPACITY EVALUATION FOR TRUSS SYSTEM SPAN 3- ULS COMBINATIONS

BUCKLAND & TAYLOR LTD.
Bridge Engineering

Notes:

1. Load rating method is referenced to CSA - S6 - 06, Section 14.
2. Evaluation procedure: ULS Method
3. Highway Class C (as per CSA-S6-06 Clause 1.4.2.2)
4. Evaluation was carried out for the following three live load models.
   CL1 - CL1-625 Truck or Lane Load traffic;
   25T - 25t review vehicle or Lane Load traffic;
   5T - 5t passenger vehicle or Lane Load traffic.
5. Inspection Level considered: "INSP3" for all structural components
6. Target reliability index from Table 14.5.
7. Dead load factors from Table 14.7. and 3.2.
8. Live load factors are from: - Table 14.8, for normal traffic (CL1-625) and pedestrain load.
   - Table 14.9, for normal traffic (alternative loading)
9. Resistance adjustment factor from Table 14.15.
10. Live load capacity factor as per Clause 14.15.2.1.
11. Material strength: fy = 210 MPa, fu = 420 MPa for Structural steel
    fu = 320 MPa for Rivet
    fc' = 15 MPa for Reinforced concrete
    fy = 230 MPa for Reinforcing steel

UNCORRODED

Elt. #	Element - Force effect	Effect Units	Target reliability index: Syst Behav	Target reliability index: Elem Behav	Target reliability index: Insp Level	Beta	Dead load: Unfact. loads D1	Dead load: Unfact. loads D2	Dead load: Load factors D1	Dead load: Load factors D2	Dead load: Fact. loads D1	Dead load: Fact. loads D2	Snow load: Unfact. Loads	Snow load: Load factor ULS1a	Snow load: Load factor ULS1d	Snow load: Fact.Loads ULS1a	Snow load: Fact.Loads ULS1d	Live load: LL Model	Live load: Lat. Distr.	Live load: Type span	Live load: Unfact. Loads	Live load: Load factor	Live load: DLA factor	Live load: Fact. Loads	Resistance: Fact Resist	Resistance: Adjust Fact	w/ snow Live Load Capacity Factor ULS1a	w/o snow Live Load Capacity Factor ULS1b	w/o live C/D ULS1d & ULS9
41	Comp. in gusset PL	Pmax	S1	E1	INSP3	3.75	42	57	1.10	1.20	46	68	26	1.50	0	40	-	CL1	Static	All	227	1.70	0.25	483	989	1.01	1.75	1.83	-
	Panel Point U5	[kN]					42	57	1.10	1.20	46	68	26	1.50	0	40	-	25T	Static	Other	154	1.70	0.00	262	989	1.01	3.22	3.37	-
							42	57	1.10	1.20	46	68	26	1.50	0	40	-	5T	Static	Other	82	1.70	0.00	139	989	1.01	6.08	6.37	-
							42	57	1.10	1.20	46	68	28	-	1.50	-	42	-	-	-	-	-	-	-	989	1.01	-	-	6.40
							42	57	1.35	1.35	56	77	-	-	-	-	-	-	-	-	-	-	-	-	989	1.01	-	-	7.50

Note: * indicates load reversal
DLA = 0 indicates lane load governs
DLA > 0 indicates truck load governs
LLCFs circled in this Table are recomputed in Section 4 of this report to investigate the effects of capacity loss due to corrosion or damage

■ GOVERNING LL CAPACITY FACTOR

- FOR MEMBERS

CL1	0.44	0.62	-
25T	0.84	1.13	-
5T	1.61	2.14	-
SNOW	-	-	1.57
DL	-	-	1.91

- FOR GUSSET PLATES

CL1	0.32	0.68	-
25T	0.51	1.07	-
5T	0.80	1.68	-
SNOW	-	-	1.22
DL	-	-	1.43

TABLE B6 - LOAD CAPACITY EVALUATION FOR TRUSS BEARINGS - ULS COMBINATIONS

BUCKLAND & TAYLOR LTD.
Bridge Engineering

Notes:
1. Load rating method is referenced to CSA - S6 - 06, Section 14.
2. Evaluation procedure: ULS Method
3. Highway Class C (as per CSA-S6-06 Clause 1.4.2.2)
4. Evaluation was carried out for the following three live load models.
   CL1 - CL1-625 Truck or Lane Load traffic;
   25T - 25t review vehicle or Lane Load traffic;
   5T - 5t passenger vehicle;
5. Inspection Level considered: "INSP3" for all structural components
6. Target reliability index from Table 14.5.
7. Dead load factors from Table 14.7. and 3.2.
8. Live load factors are from: - Table 14.8, for normal traffic (CL1-625) and pedestrain load.
   - Table 14.9, for normal traffic (alternative loading)
9. Resistance adjustment factor from Table 14.15.
10. Live load capacity factor as per Clause 14.15.2.1.
11. Material strength: fy = 210 MPa, fu = 420 MPa for Structural steel
    fu = 320 MPa for Rivet
    fc' = 15 MPa for Reinforced concrete
    fy = 230 MPa for Reinforcing steel

UNCORRODED

Elt. #	Element - Force effect	Effect Units	Target reliability index: Syst Behav	Elem Behav	Insp Level	Beta	Dead load: Unfact. loads D1	D2	Load factors D1	D2	Fact. loads D1	D2	Snow load: Unfact. Loads	Load factor ULS1a	ULS1d	Fact.Loads ULS1a	ULS1d	Live load: LL Model	Lat. Distr.	Type span	Unfact. Loads	Load factor	DLA factor	Fact. Loads	Resistance: Fact Resist	Adjust Fact	w/ snow: Live Load Capacity Factor ULS1a	w/o snow: ULS1b	w/o live: C/D ULS1d & ULS9
1	Bearing above S.Abutment	Bmax	S1	E1	INSP3	3.75	58	139	1.10	1.20	64	167	65	1.50	0	98	0	CL1	Static	All	308	1.70	0.25	655	1280	1.01	1.47	1.62	-
	Bearing of the angle brackets	[kN]							1.10	1.20	64	167	65	1.50	0	98	0	25T	Static	Other	171	1.70	0.30	377			2.56	2.81	-
									1.10	1.20	64	167	65	1.50	0	98	0	5T	Static	Other	86	1.70	0.00	146			6.60	7.27	-
									1.10	1.20	64	167	69	-	1.50	-	104	-	-	-	-	-	-	-			-	-	3.87
									1.35	1.35	78	188	-	-	-	-	-	-	-	-	-	-	-	-			-	-	4.86
2	Bearing of span 1 above pier 1	Bmax	S1	E1	INSP3	3.75	58	139	1.10	1.20	64	167	65	1.50	0	98	0	CL1	Static	All	309	1.70	0.25	656	914	1.01	0.91	1.06	-
	Bearing of the angle brackets	[kN]							1.10	1.20	64	167	65	1.50	0	98	0	25T	Static	Other	172	1.70	0.30	379			1.57	1.83	-
									1.10	1.20	64	167	65	1.50	0	98	0	5T	Static	Other	87	1.70	0.00	148			4.03	4.69	-
									1.10	1.20	64	167	69	-	1.50	-	104	-	-	-	-	-	-	-			-	-	2.76
									1.35	1.35	78	188	-	-	-	-	-	-	-	-	-	-	-	-			-	-	3.47
3	Bearing of span 2 above pier 1	Bmax	S1	E1	INSP3	3.75	82	184	1.10	1.20	90	221	87	1.50	0	131	0	CL1	Static	All	343	1.70	0.25	729	914	1.01	0.66	0.84	-
	Bearing of the angle brackets	[kN]							1.10	1.20	90	221	87	1.50	0	131	0	25T	Static	Other	178	1.70	0.30	394			1.22	1.55	-
									1.10	1.20	90	221	87	1.50	0	131	0	5T	Static	Other	106	1.70	0.00	180			2.68	3.40	-
									1.10	1.20	90	221	92	-	1.50	-	138	-	-	-	-	-	-	-			-	-	2.06
									1.35	1.35	111	248	-	-	-	-	-	-	-	-	-	-	-	-			-	-	2.57
4	Bearing above pier 2	Vmax	S1	E1	INSP3	3.75	412	635	1.10	1.20	453	762	296	1.50	0	444	0	CL1	Static	All	606	1.70	0.00	1,030	1100	1.81	0.32	0.75	-
	Shear in rivets	[kN]							1.10	1.20	453	762	296	1.50	0	444	0	25T	Static	Other	407	1.70	0.00	692			0.48	1.12	-
									1.10	1.20	453	762	296	1.50	0	444	0	5T	Static	Other	285	1.70	0.00	485			0.68	1.60	-
									1.10	1.20	453	762	313	-	1.50	-	470	-	-	-	-	-	-	-			-	-	1.18
									1.35	1.35	556	857	-	-	-	-	-	-	-	-	-	-	-	-			-	-	1.41
5	Bearing above pier 3	Vmax	S1	E1	INSP3	3.75	328	448	1.10	1.20	361	538	208	1.50	0	312	0	CL1	Static	All	422	1.70	0.25	896	1100	1.81	0.87	1.22	-
	Shear in rivets	[kN]							1.10	1.20	361	538	208	1.50	0	312	0	25T	Static	Other	333	1.70	0.00	566			1.38	1.93	-
									1.10	1.20	361	538	208	1.50	0	312	0	5T	Static	Other	212	1.70	0.00	360			2.17	3.03	-
									1.10	1.20	361	538	220	-	1.50	-	330	-	-	-	-	-	-	-			-	-	1.62
									1.35	1.35	443	605	-	-	-	-	-	-	-	-	-	-	-	-			-	-	1.90

Note: ALL in "Type Span" Column indicates that the live load factor is applicable to all span types (Section 14.13.3, CAN/CSA S6-06).
DLA = 0 indicates lane load governs
DLA > 0 indicates truck load governs

■ GOVERNING LL CAPACITY FACTOR

CL1	0.32	0.75	-
25T	0.48	1.12	-
5T	0.68	1.60	-
	-	-	1.18
	-	-	1.41

TABLE B7 - LOAD CAPACITY EVALUATION FOR GIRDERS - ULS COMBINATIONS

BUCKLAND & TAYLOR LTD. Bridge Engineering

Notes:

1. Load rating method is referenced to CSA - S6 - 06, Section 14.
2. Evaluation procedure: ULS Method
3. Highway Class C (as per CSA-S6-06 Clause 1.4.2.2)
4. Evaluation was carried out for the following three live load models.
   CL1 - CL1-625 Truck or Lane Load traffic;
   25T - 25t review vehicle or Lane Load traffic;
   5T - 5t passenger vehicle or Lane Load traffic.
5. Inspection Level considered: "INSP3" for all structural components
6. Target reliability index from Table 14.5.
7. Dead load factors from Table 14.7. and 3.2.
8. Live load factors are from: - Table 14.8, for normal traffic (CL1-625) and pedestrain load.
   - Table 14.9, for normal traffic (alternative loading)
9. Resistance adjustment factor from Table 14.15.
10. Live load capacity factor as per Clause 14.15.2.1.
11. Material strength: fy = 210 MPa, fu = 420 MPa for Structural steel
    fu = 320 MPa for Rivet
    fc' = 15 MPa for Reinforced concrete
    fy = 230 MPa for Reinforcing steel

UNCORRODED

Elt. #	Element - Force effect	Effect Units	Target reliability index: Syst Behav	Elem Behav	Insp Level	Beta	Dead load: Unfact. loads D1	Unfact. loads D2	Load factors D1	Load factors D2	Fact. loads D1	Fact. loads D2	Snow load: Unfact. Loads	Load factor ULS1a	Load factor ULS1d	Fact.Loads ULS1a	Fact.Loads ULS1d	Live load: LL Model	Lat. Distr.	Type span	Unfact. Loads	Load factor	DLA factor	Fact. Loads	Resistance: Fact Resist	Adjust Fact	Live Load Capacity Factor ULS1a (w/ snow)	Live Load Capacity Factor ULS1b (w/o snow)	C/D ULS1d & ULS9 (w/o live)
1	Edge girder at span 6	Mmax	S1	E3	INSP3	3.00	41	120	1.07	1.14	44	137	123	1.50	-	184	-	CL1	Static	All	512	1.49	0.30	993	1393	1.00	1.04	1.22	-
	(4-L6x6x5/8 + PL 28x3/8)	[kN]					41	120	1.07	1.14	44	137	123	1.50	-	184	-	25T	Static	Other	429	1.49	0.30	831	1393	1.00	1.24	1.46	-
	Sagging moment near midspan						41	120	1.07	1.14	44	137	123	1.50	-	184	-	5T	Static	Other	158	1.49	0.00	236	1393	1.00	4.36	5.14	-
							41	120	1.07	1.14	44	137	128	-	1.50	-	192	-	-	-	-	-	-	-	1393	1.00	-	-	3.74
							41	120	1.35	1.35	55	162	-	-	-	-	-	-	-	-	-	-	-	-	1393	1.00	-	-	6.40
2	Edge girder at span 6	Mmax	S1	E3	INSP3	3.00	36	106	1.07	1.14	39	121	108	1.50	-	162	-	CL1	Static	All	460	1.49	0.30	890	1393	1.00	1.20	1.39	-
	(4-L6x6x5/8 + PL 28x3/8)	[kN]					36	106	1.07	1.14	39	121	108	1.50	-	162	-	25T	Static	Other	314	1.49	0.30	608	1393	1.00	1.76	2.03	-
	Sagging moment at 1/3 span						36	106	1.07	1.14	39	121	108	1.50	-	162	-	5T	Static	Other	142	1.49	0.00	211	1393	1.00	5.07	5.83	-
							36	106	1.07	1.14	39	121	115	-	1.50	-	173	-	-	-	-	-	-	-	1393	1.00	-	-	4.20
							36	106	1.35	1.35	49	143	-	-	-	-	-	-	-	-	-	-	-	-	1393	1.00	-	-	7.27
3	Edge girder at span 6	Vmax	S1	E3	INSP3	3.00	14	41	1.07	1.14	15	47	42	1.50	-	63	-	CL1	Static	All	204	1.49	0.30	394	557	1.02	1.13	1.28	-
	(4-L6x6x5/8 + PL 28x3/8)	[kN]					14	41	1.07	1.14	15	47	42	1.50	-	63	-	25T	Static	Other	140	1.49	0.30	271	557	1.02	1.64	1.87	-
	Web shear at the support						14	41	1.07	1.14	15	47	42	1.50	-	63	-	5T	Static	Other	57	1.49	0.00	85	557	1.02	5.21	5.94	-
							14	41	1.07	1.14	15	47	44	-	1.50	-	65	-	-	-	-	-	-	-	557	1.02	-	-	4.48
							14	41	1.35	1.35	19	55	-	-	-	-	-	-	-	-	-	-	-	-	557	1.02	-	-	7.68
4	Edge girder at span 6	Bmax	S1	E1	INSP3	3.75	14	41	1.10	1.20	15	49	42	1.50	-	63	-	CL1	Static	All	204	1.70	0.30	450	263	1.00	0.30	0.44	-
	(4-L6x6x5/8 + PL 28x3/8)	[kN]					14	41	1.10	1.20	15	49	42	1.50	-	63	-	25T	Static	Other	140	1.70	0.30	309	263	1.00	0.44	0.64	-
	Comp.in web at the support						14	41	1.10	1.20	15	49	42	1.50	-	63	-	5T	Static	Other	57	1.70	0.00	97	263	1.00	1.40	2.04	-
							14	41	1.10	1.20	15	49	44	-	1.50	-	65	-	-	-	-	-	-	-	263	1.00	-	-	2.03
							14	41	1.35	1.35	19	55	-	-	-	-	-	-	-	-	-	-	-	-	263	1.00	-	-	3.55

Note: * indicates load reversal
DLA = 0 indicates lane load governs
DLA > 0 indicates truck load governs
LLCFs circled in this Table are recomputed in Section 4 of this report to investigate the effects of capacity loss due to corrosion or damage

GOVERNING LL CAPACITY FACTOR

CL1	0.30	0.44	-
25T	0.44	0.64	-
5T	1.40	2.04	-
SNOW	-	-	2.03
DL	-	-	3.55

TABLE B8 - LOAD CAPACITY EVALUATION FOR CONCRETE PIERS - ULS COMBINATIONS

BUCKLAND & TAYLOR LTD. Bridge Engineering

Notes:

1. Load rating method is referenced to CSA - S6 - 06, Section 14.
2. Evaluation procedure: ULS Method
3. Highway Class C (as per CSA-S6-06 Clause 1.4.2.2)
4. Evaluation was carried out for the following three live load models.
   CL1 - CL1-625 Truck or Lane Load traffic;
   25T - 25t review vehicle or Lane Load traffic;
   5T - 5t passenger vehicle;
5. Inspection Level considered: "INSP3" for all structural components
6. Target reliability index from Table 14.5.
7. Dead load factors from Table 14.7. and 3.2.
8. Live load factors are from: - Table 14.8, for normal traffic (CL1-625) and pedestrain load.
   - Table 14.9, for normal traffic (alternative loading)
9. Resistance adjustment factor from Table 14.15.
10. Live load capacity factor as per Clause 14.15.2.1.
11. Material strength: fy = 210 MPa, fu = 420 MPa for Structural steel
    fu = 320 MPa for Rivet
    fc' = 15 MPa for Reinforced concrete
    fy = 230 MPa for Reinforcing steel

UNCORRODED

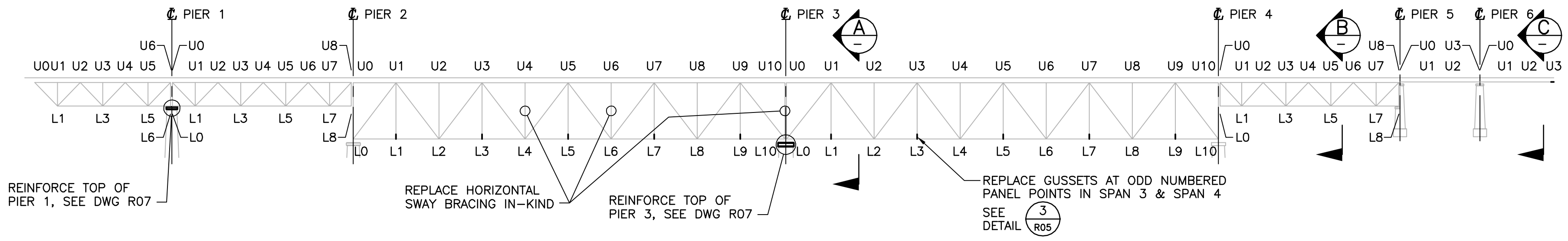
Elt. #	Element - Force effect	Effect Units	Target reliability index				Dead load						Snow load					Live load							Resistance		w/ snow	w/o snow	w/o live
							Unfact. loads		Load factors		Fact. loads		Unfact.	Load factor		Fact.Loads		LL	Lat.	Type	Unfact.	Load	DLA	Fact.	Fact	Adjust	Live Load Capacity Factor		C/D ULS1d
			Syst Behav	Elem Behav	Insp Level	Beta	D1	D2	D1	D2	D1	D2	Loads	ULS1a	ULS1d	ULS1a	ULS1d	Model	Distr.	span	Loads	factor	factor	Loads	Resist	Fact	ULS1a	ULS1b	& ULS9
1	Concrete pier 1	Br	S1	E1	INSP3	3.75	58	139	1.10	1.20	64	166	-	1.50	0	-	-	CL1	Static	All	309	1.70	0.25	656	779	1.00	-	0.84	-
	Bearing resistance	[kN]							1.10	1.20	64	166	-	1.50	0	-	-	25T	Static	Other	172	1.70	0.30	379			-	1.45	-
									1.10	1.20	64	166	-	1.50	0	-	-	5T	Static	Other	87	1.70	0.00	148			-	3.71	-
									1.10	1.20	64	166	-	-	1.50	-	-	-	-	-	-	-	-	-			-	-	3.39
									1.35	1.35	78	187	-	-	-	-	-	-	-	-	-	-	-	-			-	-	2.94
2	Concrete pier 2 & 4	Br	S1	E1	INSP3	3.75	412	635	1.10	1.20	453	762	-	1.50	0	-	-	CL1	Static	All	485	1.70	0.25	1,030	9077	1.00	-	7.63	-
	Bearing resistance	[kN]							1.10	1.20	453	762	-	1.50	0	-	-	25T	Static	Other	313	1.70	0.30	692			-	11.36	-
									1.10	1.20	453	762	-	1.50	0	-	-	5T	Static	Other	285	1.70	0.00	485			-	16.23	-
									1.10	1.20	453	762	-	-	1.50	-	-	-	-	-	-	-	-	-			-	-	7.47
									1.35	1.35	556	857	-	-	-	-	-	-	-	-	-	-	-	-			-	-	6.42
3	Concrete pier 3	Br	S1	E1	INSP3	3.75	328	448	1.10	1.20	361	538	-	1.50	0	-	-	CL1	Static	All	527	1.70	0.00	896	6006	1.00	-	5.70	-
	Bearing resistance	[kN]							1.10	1.20	361	538	-	1.50	0	-	-	25T	Static	Other	333	1.70	0.00	566			-	9.02	-
									1.10	1.20	361	538	-	1.50	0	-	-	5T	Static	Other	212	1.70	0.00	360			-	14.17	-
									1.10	1.20	361	538	-	-	1.50	-	-	-	-	-	-	-	-	-			-	-	6.68
									1.35	1.35	443	605	-	-	-	-	-	-	-	-	-	-	-	-			-	-	5.73
4	Concrete pier 5	Br	S1	E1	INSP3	3.75	82	184	1.10	1.20	90	221	-	1.50	0	-	-	CL1	Static	All	343	1.70	0.25	729	974	1.00	-	0.91	-
	Bearing resistance	[kN]							1.10	1.20	90	221	-	1.50	0	-	-	25T	Static	Other	178	1.70	0.30	394			-	1.68	-
									1.10	1.20	90	221	-	1.50	0	-	-	5T	Static	Other	106	1.70	0.00	180			-	3.68	-
									1.10	1.20	90	221	-	-	1.50	-	-	-	-	-	-	-	-	-			-	-	3.13
									1.35	1.35	111	248	-	-	-	-	-	-	-	-	-	-	-	-			-	-	2.71
5	Concrete pier 6 * +	Mmax	S1	E1	INSP3	3.75	0	-1	0.90	0.90	0	-1	-	1.50	0	-	-	CL1	Static	All	76	1.70	0.30	168	P-M	1.00	-	0.90	-
	Maximum bending moment at pier	[kN.m]							0.90	0.90	0	-1	-	1.50	0	-	-	25T	Static	Other	55	1.70	0.30	122			-	1.18	-
									0.90	0.90	0	-1	-	1.50	0	-	-	5T	Static	Other	20	1.70	0.00	34			-	3.65	-
									0.90	0.90	0	-1	-	-	1.50	-	-	-	-	-	-	-	-	-			-	-	-
									1.35	1.35	0	-1	-	-	-	-	-	-	-	-	-	-	-	-			-	-	-
	Concurrent axial force with Mmax	P	S1	E1	INSP3	3.75	54	156	0.90	0.90	49	140	-	1.50	0	-	-	CL1	Static	All	331	1.70	0.30	732			-	-	-
		[kN]							0.90	0.90	49	140	-	1.50	0	-	-	25T	Static	Other	243	1.70	0.30	537			-	-	-
									0.90	0.90	49	140	-	1.50	0	-	-	5T	Static	Other	89	1.70	0.00	151			-	-	-
									0.90	0.90	49	140	-	-	1.50	-	-	-	-	-	-	-	-	-			-	-	-
									1.35	1.35	73	211	-	-	-	-	-	-	-	-	-	-	-	-			-	-	-

Note: ALL in "Type Span" Column indicates that the live load factor is applicable to all span types (Section 14.13.3, CAN/CSA S6-06).
DLA = 0 indicates lane load governs
DLA > 0 indicates truck load governs
* The load factors for dead load has been reduced to 0.90 as the dead load is benefical (Table 3.2, CAN/CSA S6-06)
+ The force effect shown in the table is the critical one of three load combinations (Pmax and M, Mmax and M, Pmax and M=Pe as per 8.8.5.3, CAN/CSA S6-06).

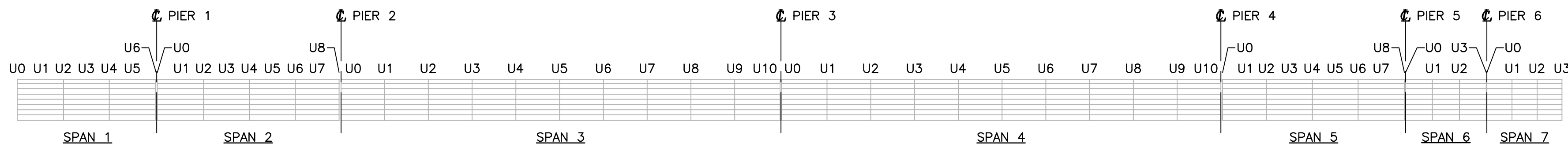
■ GOVERNING LL CAPACITY FACTOR

CL1	-	0.84	-
25T	-	1.18	-
5T	-	3.65	-
	-	-	3.13
	-	-	2.71

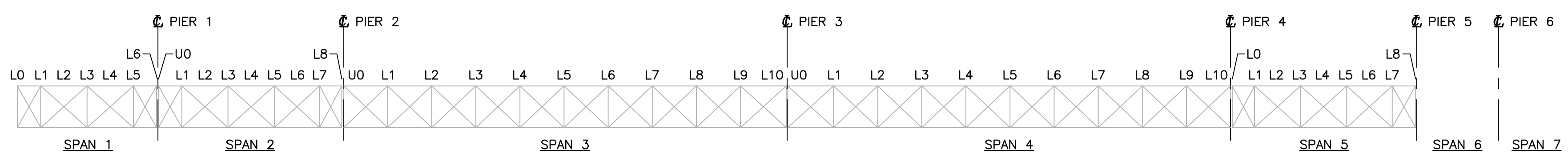
Appendix C
Buckland & Taylor Ltd. - Concept Rehabilitation Drawings



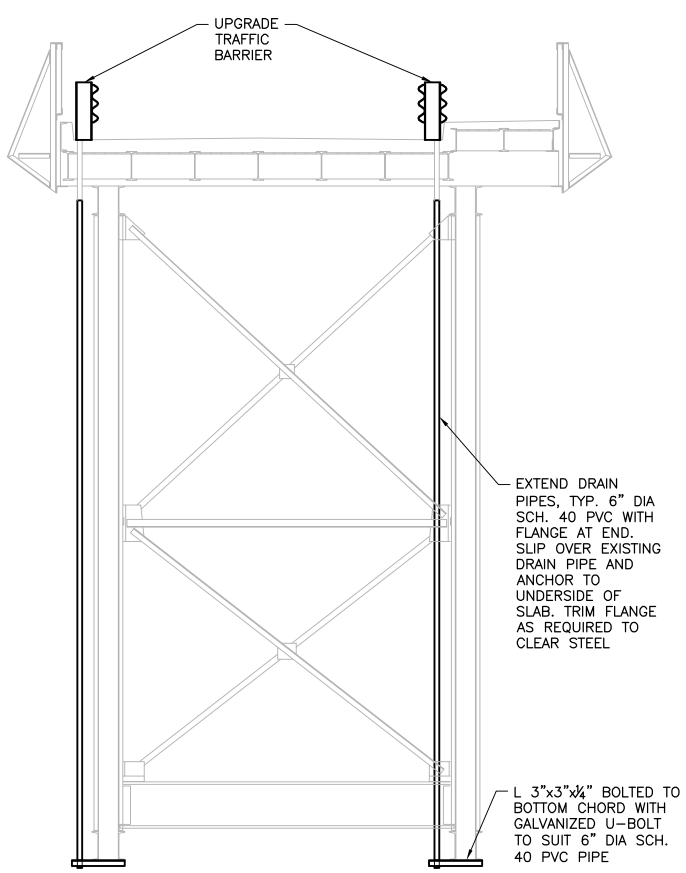
ELEVATION
SCALE 1:500



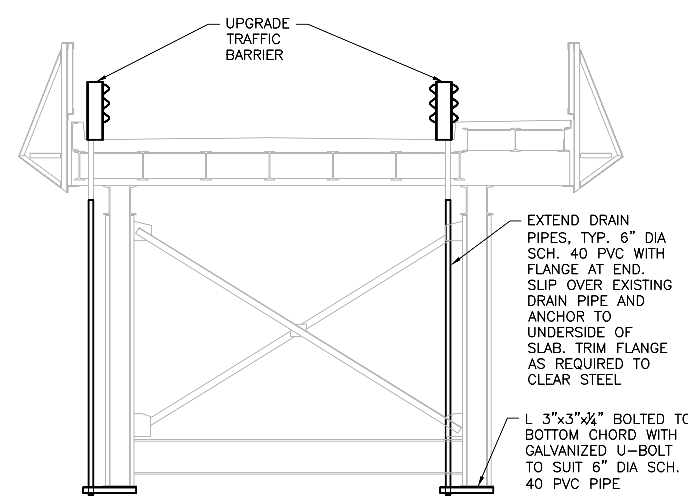
STRINGER AND FLOOR BEAM – PLAN
SCALE 1:500



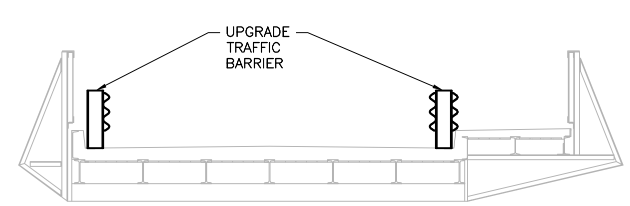
BOTTOM CHORD AND BRACING – PLAN
SCALE 1:500



SECTION A
SCALE 1:50



SECTION B
SCALE 1:50



SECTION C
SCALE 1:50

NOTES:

1. FOR 10 YEAR LIFE, RECOATING 25% OF ALL UNDER-DECK COMPONENTS IS REQUIRED.

PRELIMINARY 2010 FEB 11

Consultant Logo

BUCKLAND & TAYLOR LTD.
Bridge Engineering

Rev	Date	Description	Init
PA	09/11/20	REVISION IN PROGRESS...	

REVISIONS

BRITISH COLUMBIA
Ministry of Transportation & Infrastructure
Southern Interior Region

THOMPSON NICOLA DISTRICT
OLD SPENCES BRIDGE No. 2411
OLD SPENCES BRIDGE
5 TONNE REPAIRS – 10 YEAR LIFE

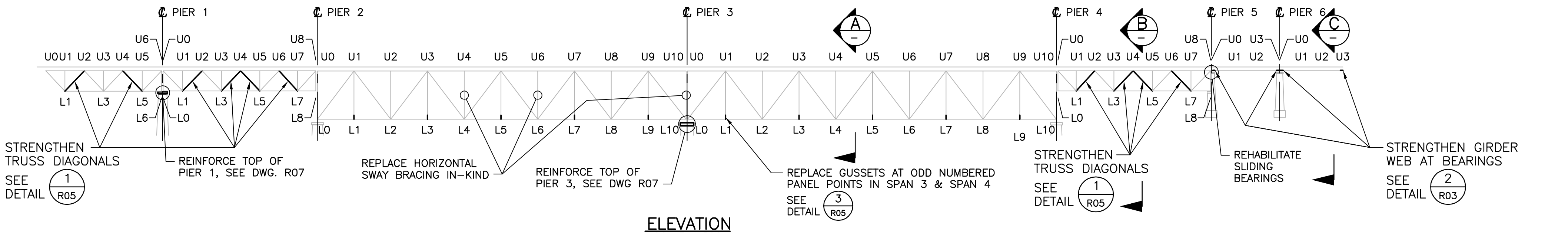
PREPARED UNDER THE DIRECTION OF

ENGINEER OF RECORD
DATE

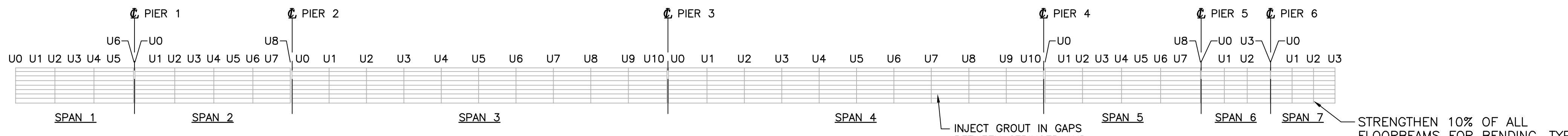
DESIGNED MS DATE 2009-11-20
CHECKED AG DATE 2009-11-20
DRAWN KAM DATE 2009-11-20
SCALE AS SHOWN
NEGATIVE No.

FILE No.	PROJECT No.	REG.	DRAWING No.
1884	2411	2	1884-2411-R01 PA

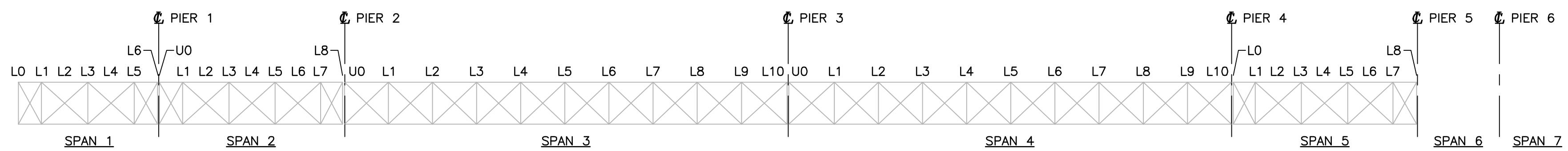
File: G:\1884\Old Spences Bridge\Inspection Report\Drawings\1884-2411-R01.dwg Save Date: Feb 11, 2010 8:39 AM Saved By: PREES



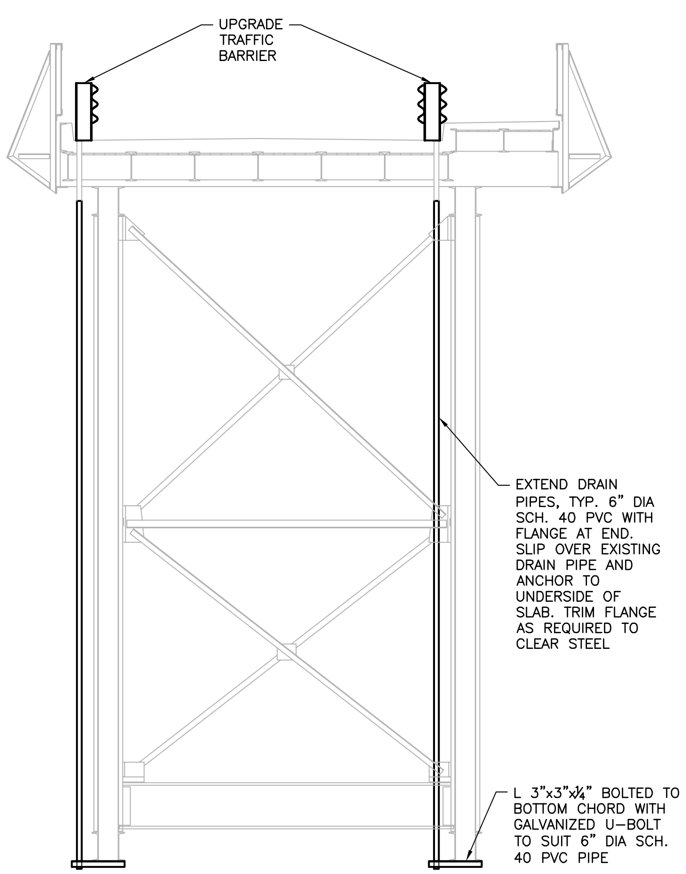
ELEVATION
SCALE 1:500



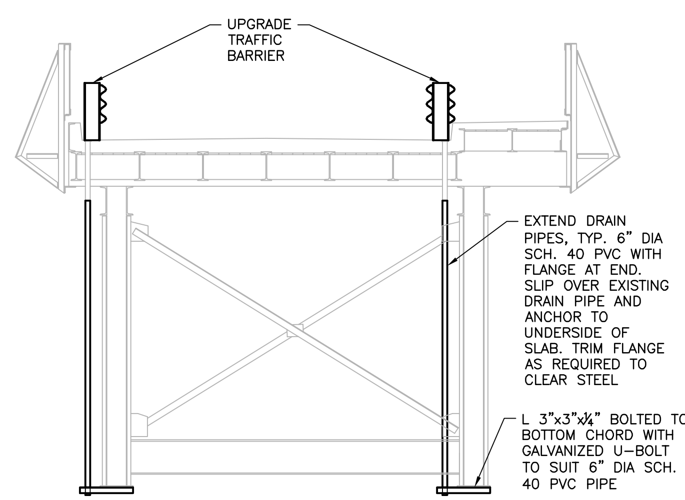
STRINGER AND FLOOR BEAM – PLAN
SCALE 1:500



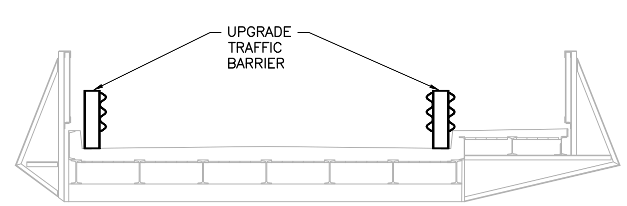
BOTTOM CHORD AND BRACING – PLAN
SCALE 1:500



SECTION A –
SCALE 1:50



SECTION B –
SCALE 1:50



SECTION C –
SCALE 1:50

NOTES:

1. FOR 10 YEAR LIFE, RECOATING 25% OF ALL UNDER-DECK COMPONENTS IS REQUIRED.

PRELIMINARY 2010 FEB 11

Consultant Logo

BUCKLAND & TAYLOR LTD. Bridge Engineering

Rev	Date	Description	Init
PA	09/11/20	REVISION IN PROGRESS...	

R E V I S I O N S

BRITISH COLUMBIA — Ministry of Transportation & Infrastructure, Southern Interior Region

THOMPSON NICOLA DISTRICT
OLD SPENCES BRIDGE No. 2411

OLD SPENCES BRIDGE
25 TONNE REPAIRS – 10 YEAR LIFE

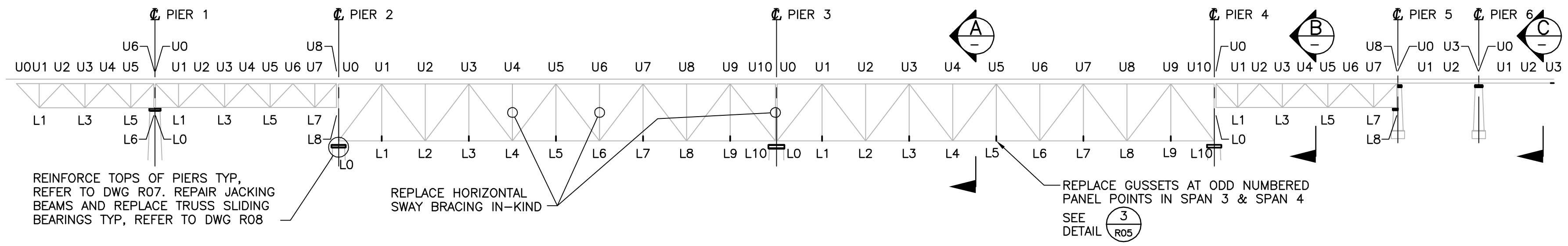
PREPARED UNDER THE DIRECTION OF

ENGINEER OF RECORD

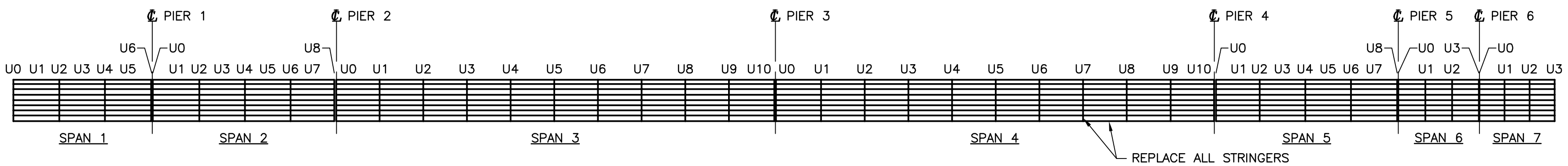
DATE

DESIGNED MS DATE 2009-11-20
CHECKED AG DATE 2009-11-20
DRAWN KAM DATE 2009-11-20
SCALE AS NOTED
NEGATIVE No.

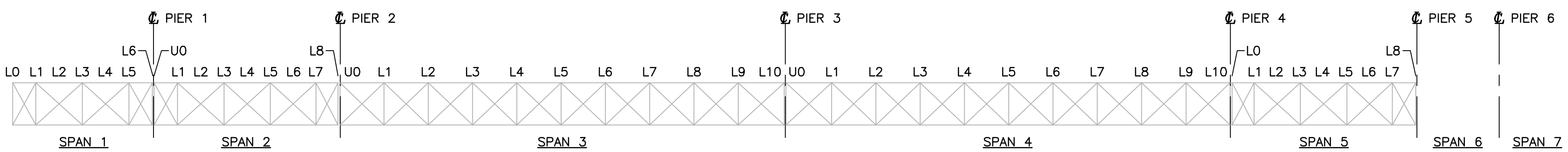
FILE No.	PROJECT No.	REG.	DRAWING No.
1884	2411	2	1884-2411-R02 PA



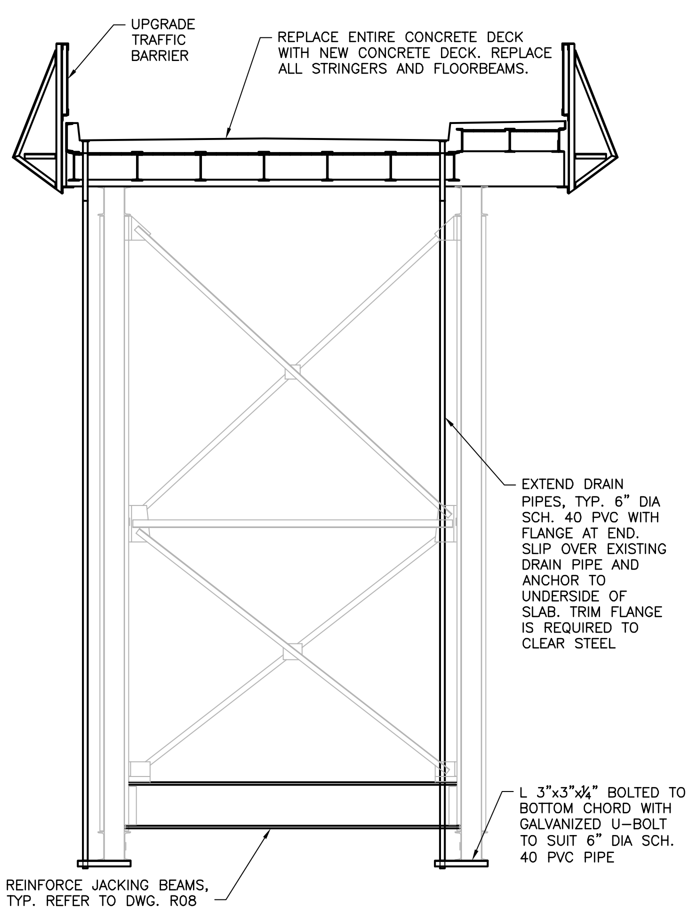
ELEVATION
SCALE 1:500



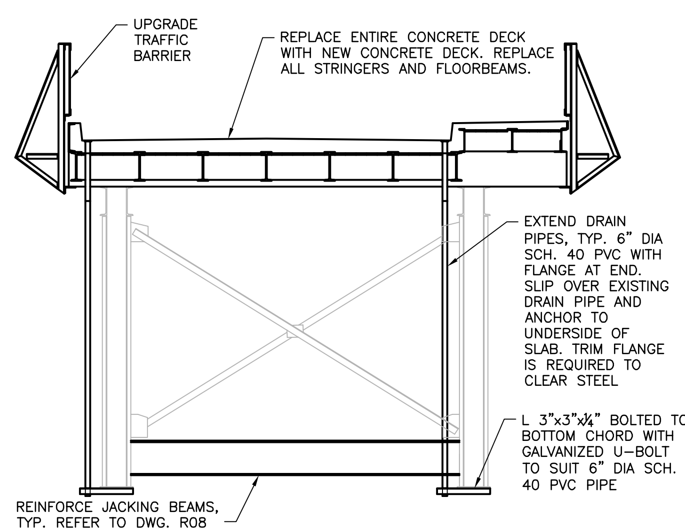
STRINGER AND FLOOR BEAM – PLAN
SCALE 1:500



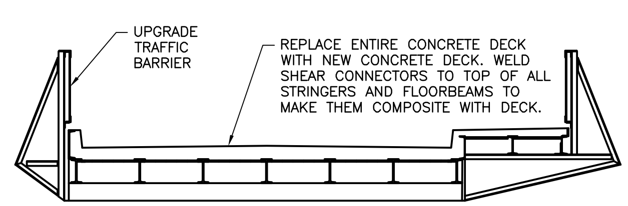
BOTTOM CHORD AND BRACING – PLAN
SCALE 1:500



SECTION A
SCALE 1:50



SECTION B
SCALE 1:50



SECTION C
SCALE 1:50

NOTES:

1. FOR 25 YEAR LIFE, RECOATING OF THE TRUSS AND GIRDERS IS REQUIRED.
2. FOR THE 50 YEAR LIFE, INITIAL TOUCHUPS WILL BE FOLLOWED BY AN ENTIRE RECOATING AFTER 20 YEARS.

PRELIMINARY 2009 DEC 10

Consultant Logo

BUCKLAND & TAYLOR LTD. Bridge Engineering

Rev	Date	Description	Init
PA	09/11/20	REVISION IN PROGRESS...	

R E V I S I O N S

BRITISH COLUMBIA
Ministry of Transportation & Infrastructure
Southern Interior Region

THOMPSON NICOLA DISTRICT
OLD SPENCES BRIDGE No. 2411

OLD SPENCES BRIDGE
5 TONNE REPAIRS – 25 & 50 YEAR LIFE

PREPARED UNDER THE DIRECTION OF

ENGINEER OF RECORD

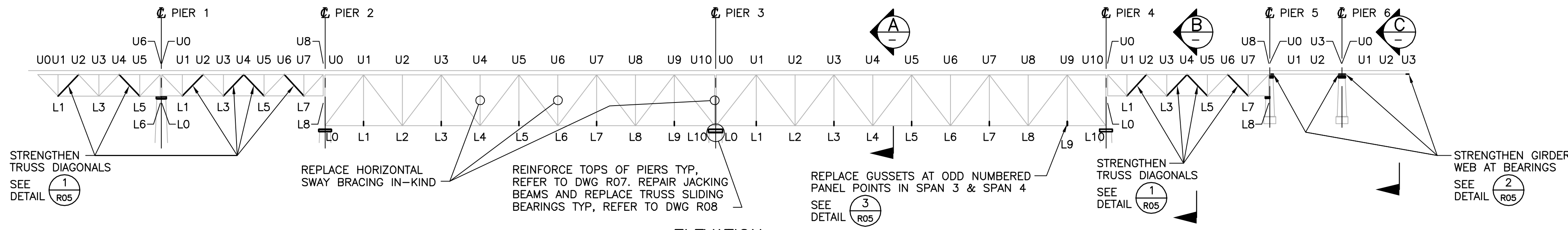
DATE

SEAL

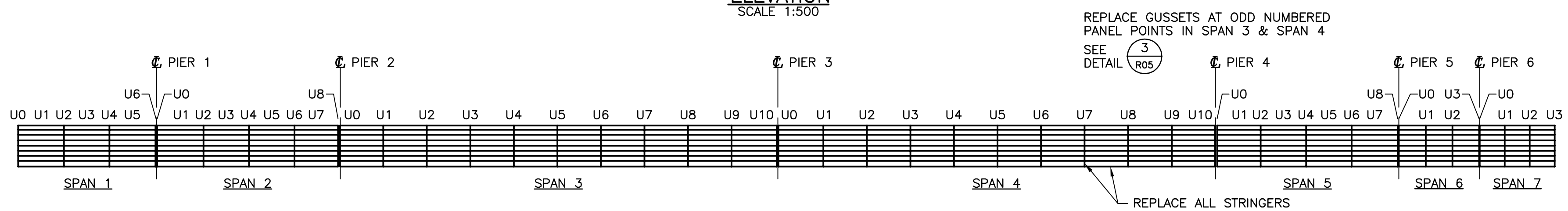
DESIGNED MS DATE 2009-11-20
CHECKED AG DATE 2009-11-20
DRAWN KAM DATE 2009-11-20
SCALE AS NOTED
NEGATIVE No.

FILE No.	PROJECT No.	REG.	DRAWING No.
1884	2411	2	1884-2411-R03 PA

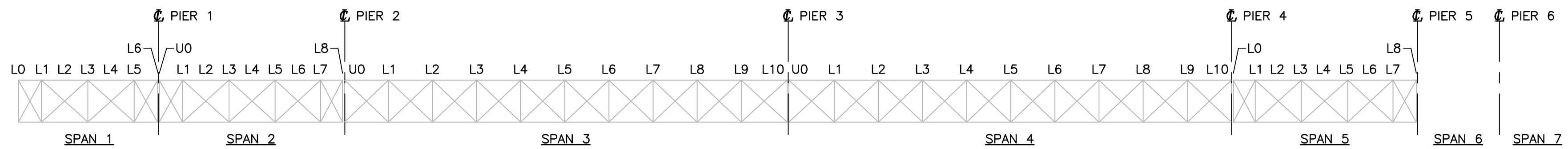
File: G:\1884\Old Spences Bridge\Inspection Report\Drawings\1884-2411-R03.dwg Save Date: Dec 10, 2009 2:26 PM Saved By: PREES



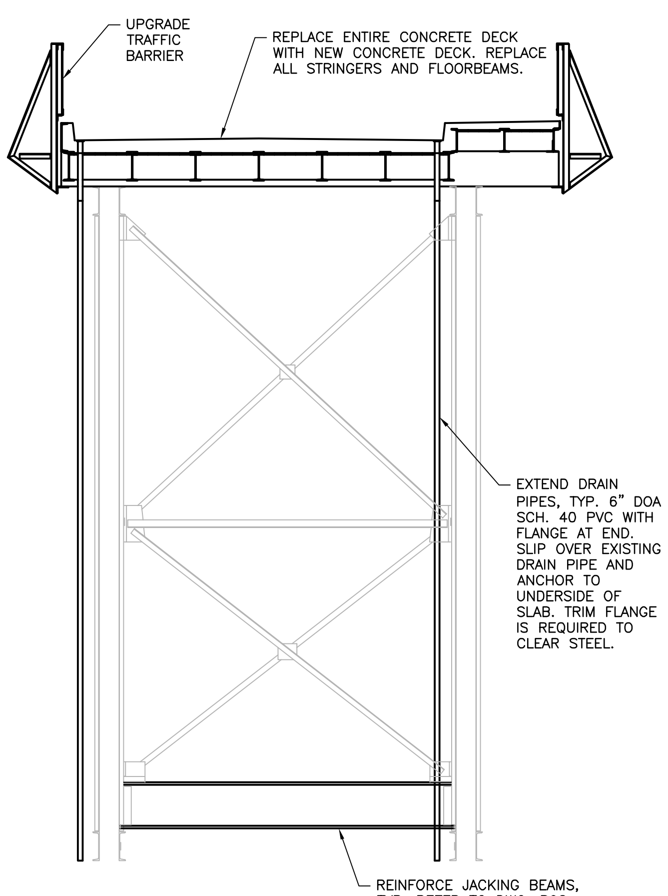
ELEVATION
SCALE 1:500



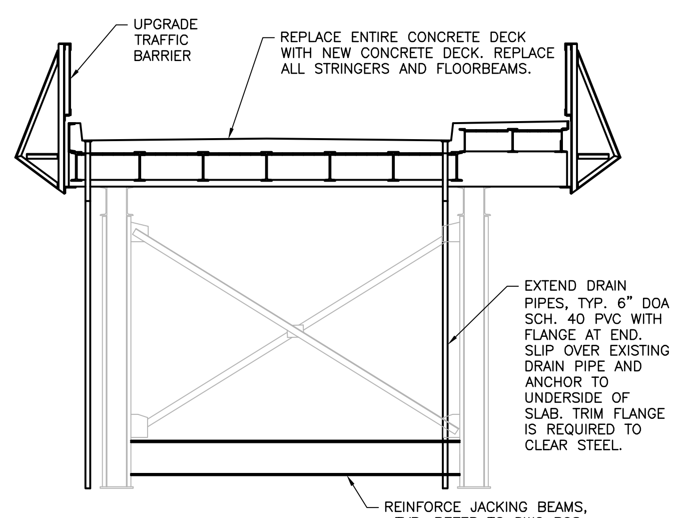
STRINGER AND FLOOR BEAM – PLAN
SCALE 1:500



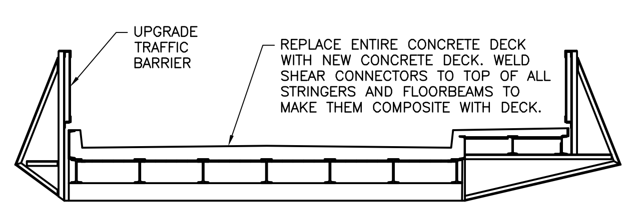
BOTTOM CHORD AND BRACING – PLAN
SCALE 1:500



SECTION A –
SCALE 1:50



SECTION B –
SCALE 1:50



SECTION C –
SCALE 1:50

NOTES:

1. FOR 25 YEAR LIFE, RECOATING OF THE TRUSS AND GIRDERS IS REQUIRED.
2. FOR THE 50 YEAR LIFE, INITIAL TOUCHUPS WILL BE FOLLOWED BY AN ENTIRE RECOATING AFTER 20 YEARS.

PRELIMINARY 2009 DEC 10

Consultant Logo
BUCKLAND & TAYLOR LTD. Bridge Engineering

Rev	Date	Description	Init
PA	09/11/20	REVISION IN PROGRESS...	

R E V I S I O N S

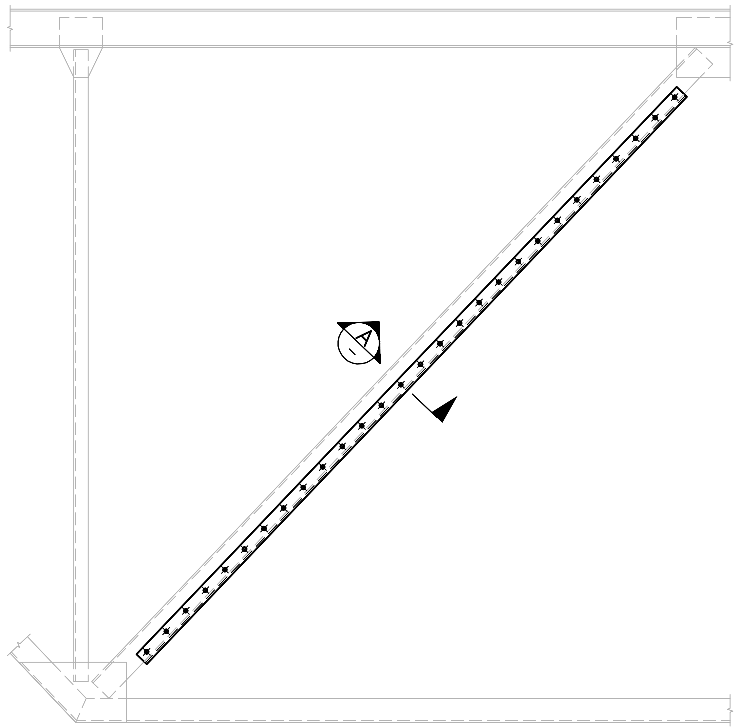
BRITISH COLUMBIA
Ministry of Transportation & Infrastructure
Southern Interior Region

THOMPSON NICOLA DISTRICT
OLD SPENCES BRIDGE No. 2411
OLD SPENCES BRIDGE
25 TONNE REPAIRS – 25 & 50 YEAR LIFE

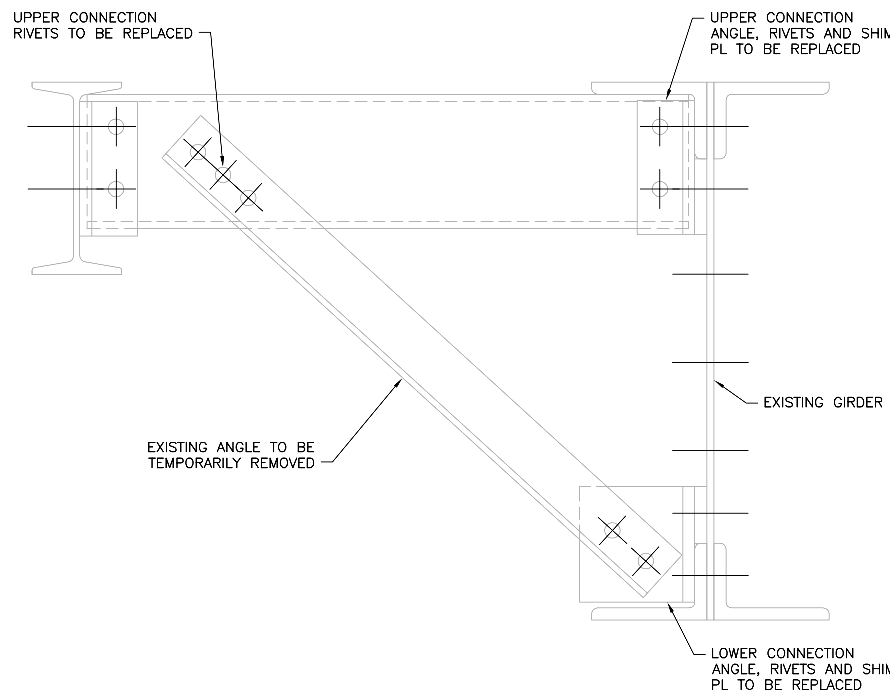
PREPARED UNDER THE DIRECTION OF
ENGINEER OF RECORD
DATE
SEAL

DESIGNED MS DATE 2009-11-20
CHECKED AG DATE 2009-11-20
DRAWN KAM DATE 2009-11-20
SCALE AS NOTED
NEGATIVE No.

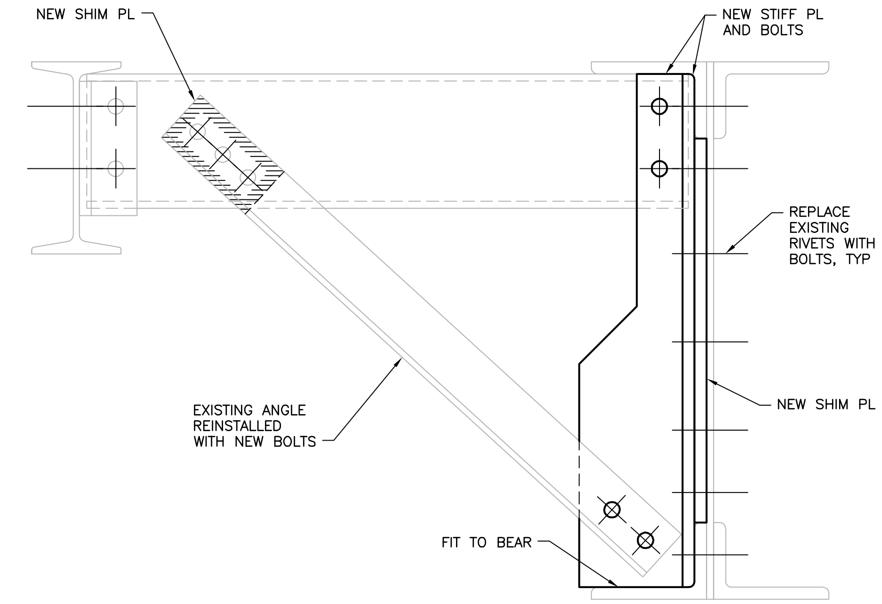
FILE No.	PROJECT No.	REG.	DRAWING No.
1884	2411	2	1884-2411-R04 PA



DETAIL 1
SCALE 1:20 R04/02

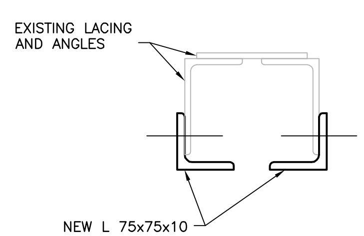


EXISTING GIRDER WEB AT BEARING

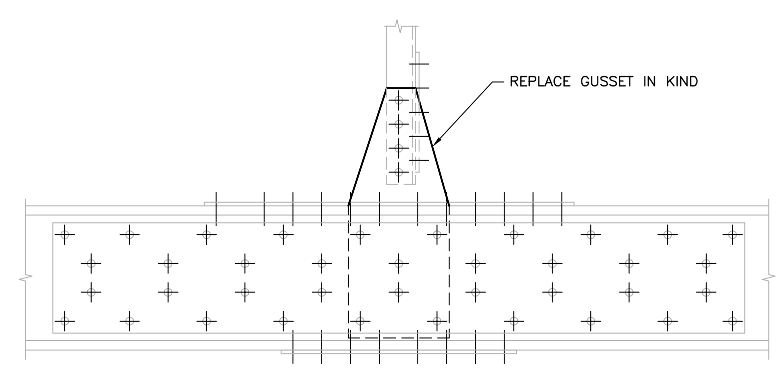


REPAIRED GIRDER WEB AT BEARING

DETAIL 2
SCALE 1:5 R02/04



SECTION A
SCALE 1:5



DETAIL 3
SCALE 1:5 R01/02/03/04

PRELIMINARY 2009 DEC 10

Consultant Logo

BUCKLAND & TAYLOR LTD.
Bridge Engineering

Rev	Date	Description	Init
PA	09/11/20	REVISION IN PROGRESS...	

R E V I S I O N S

BRITISH COLUMBIA
Ministry of Transportation & Infrastructure
Southern Interior Region

THOMPSON NICOLA DISTRICT
OLD SPENCES BRIDGE No. 2411

OLD SPENCES BRIDGE
25 TONNE STRENGTHENING TRUSS & GIRDERS

PREPARED UNDER THE DIRECTION OF

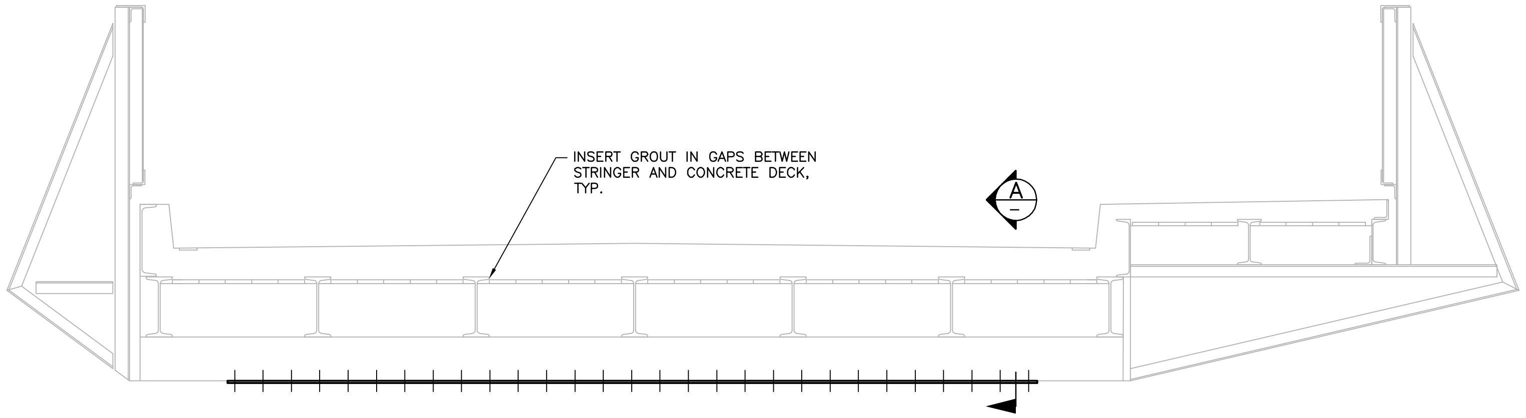
ENGINEER OF RECORD

DATE

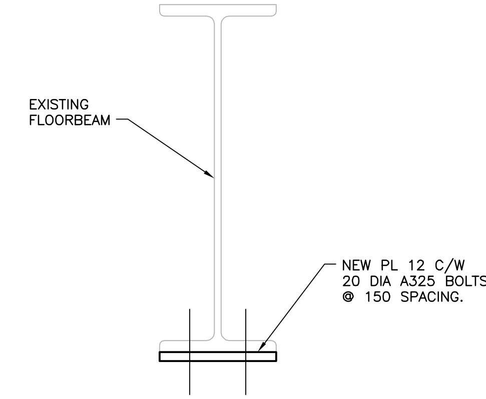
SEAL

DESIGNED MS DATE 2009-11-20
CHECKED AG DATE 2009-11-20
DRAWN KAM DATE 2009-11-20
SCALE AS NOTED
NEGATIVE No.

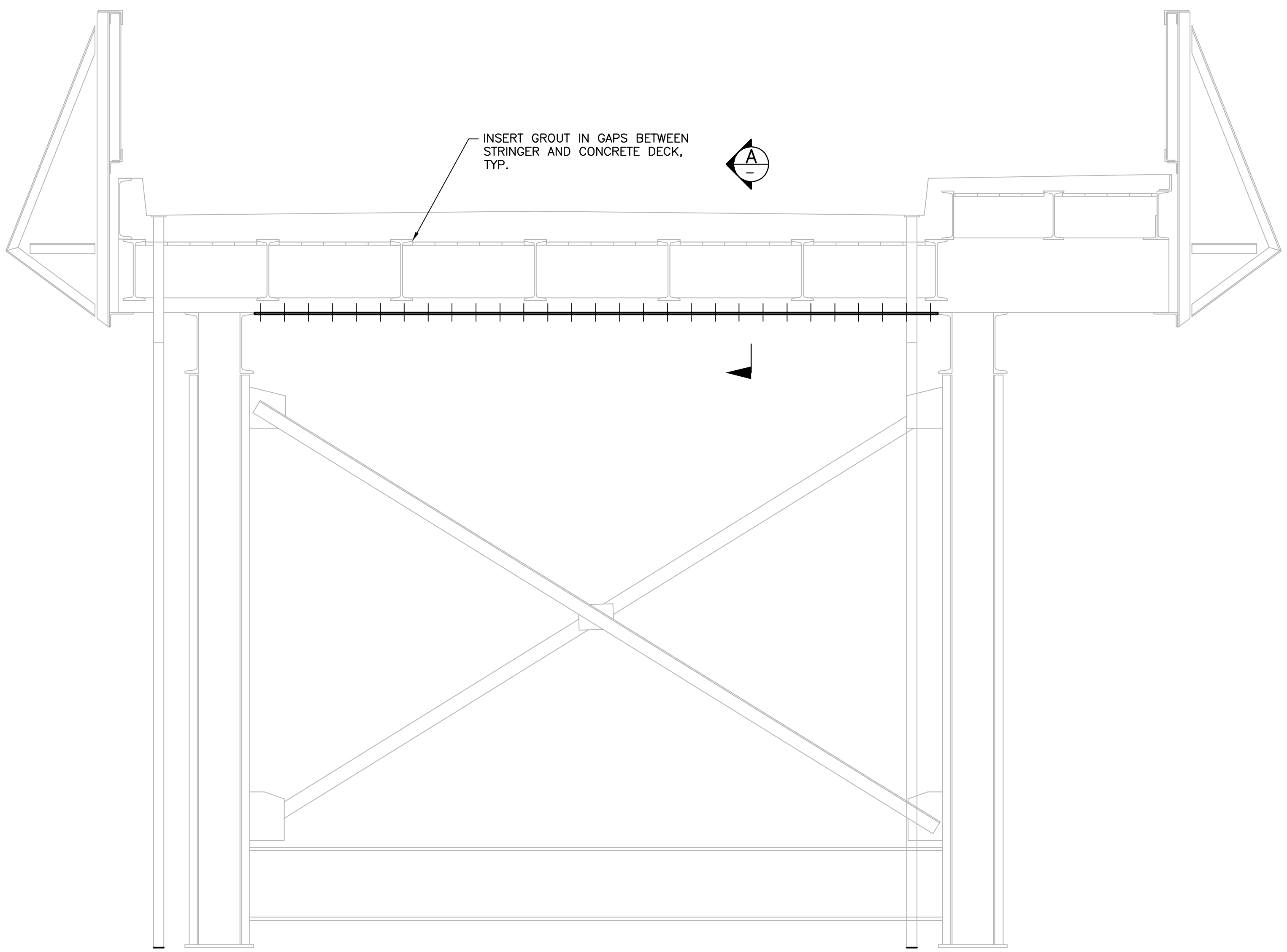
FILE No.	PROJECT No.	REG.	DRAWING No.
1884	2411	2	1884-2411-R05 PA



DETAIL 1
SCALE 1:20 R02



SECTION A
SCALE 1:5 –



DETAIL 2
SCALE 1:20 R02

PRELIMINARY 2009 DEC 10

Consultant Logo

BUCKLAND & TAYLOR LTD.
Bridge Engineering

Rev	Date	Description	Init
PA	09/11/20	REVISION IN PROGRESS...	

R E V I S I O N S

BRITISH COLUMBIA
Ministry of Transportation & Infrastructure
Southern Interior Region

THOMPSON NICOLA DISTRICT
OLD SPENCES BRIDGE No. 2411

OLD SPENCES BRIDGE
25 TONNE STRENGTHENING – FLOOR SYSTEM

PREPARED UNDER THE DIRECTION OF

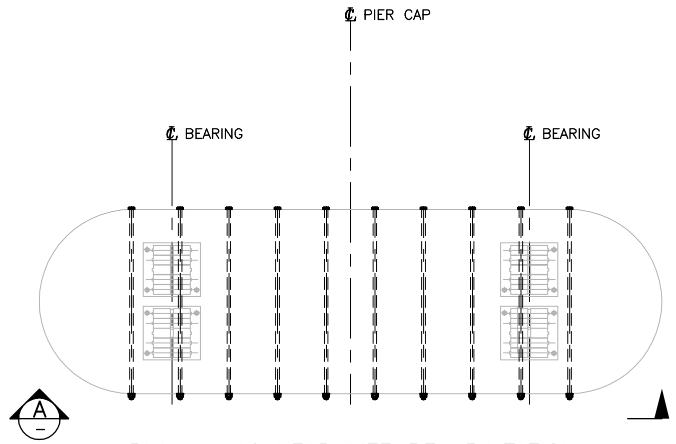
ENGINEER OF RECORD

DATE

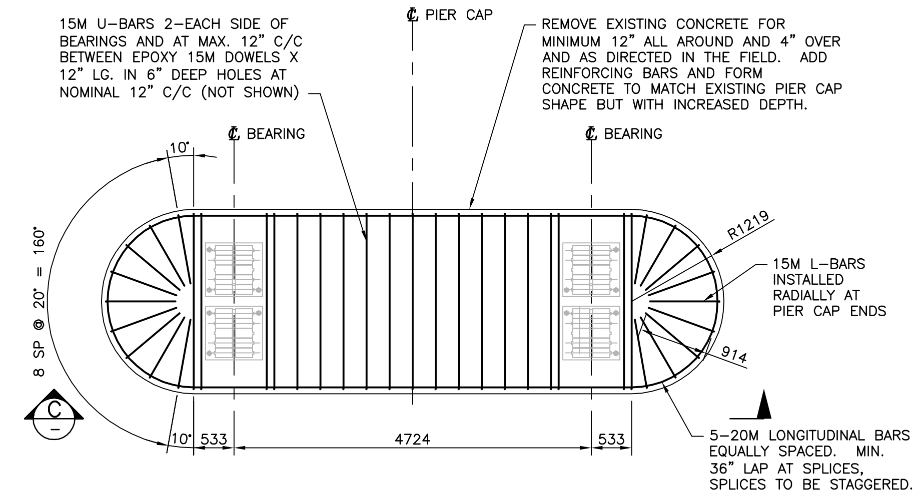
SEAL

DESIGNED MS DATE 2009-11-20
CHECKED AG DATE 2009-11-20
DRAWN KAM DATE 2009-11-20
SCALE AS NOTED
NEGATIVE No.

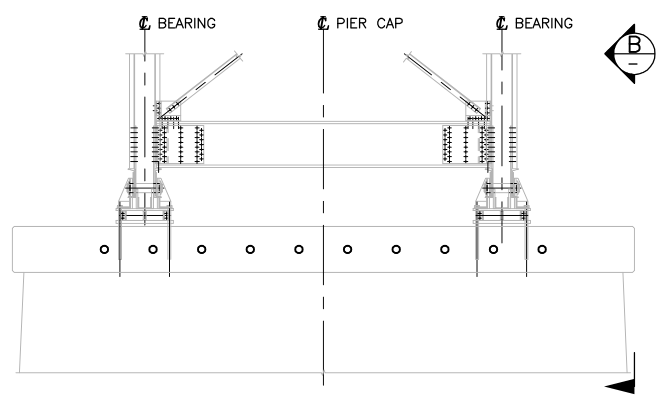
FILE No.	PROJECT No.	REG.	DRAWING No.
1884	2411	2	1884-2411-R06 PA



PLAN – 10 YEAR LIFE REHABILITATION
SCALE 1:50

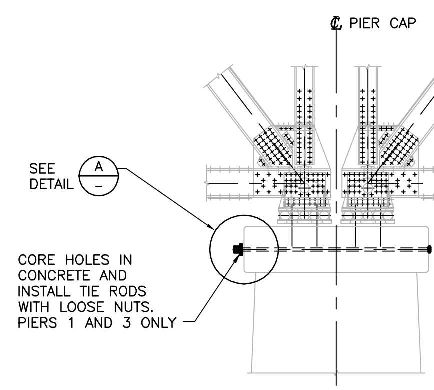


PLAN – 25 TO 50 YEAR LIFE REHABILITATION
SCALE 1:50



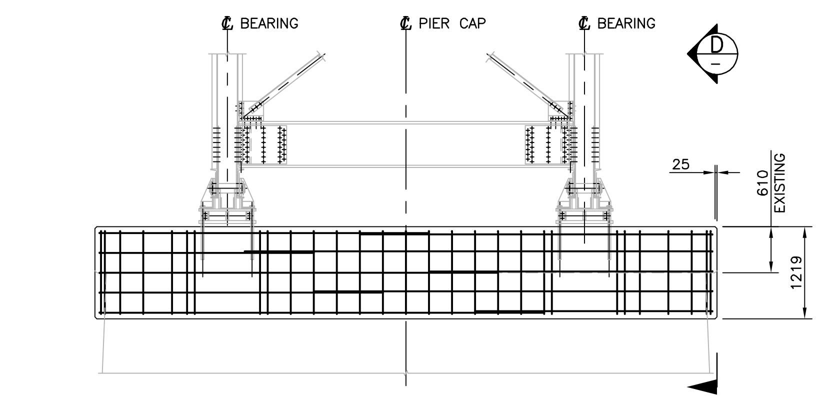
VIEW A
SCALE 1:50

10 YEAR LIFE REHABILITATION



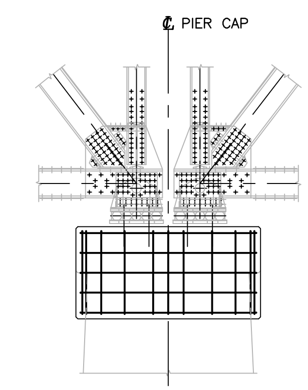
VIEW B
SCALE 1:50

10 YEAR LIFE REHABILITATION



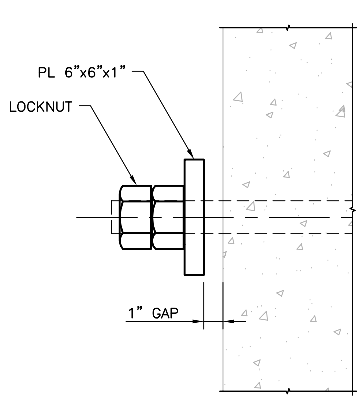
VIEW C
SCALE 1:50

25 TO 50 YEAR LIFE REHABILITATION



VIEW D
SCALE 1:50

25 TO 50 YEAR LIFE REHABILITATION



DETAIL 1
SCALE 1:5

PRELIMINARY 2009 DEC 10

Consultant Logo

BUCKLAND & TAYLOR LTD.
Bridge Engineering

Rev	Date	Description	Init
–	09/11/20	REVISION IN PROGRESS...	

REVISIONS

BRITISH COLUMBIA
Ministry of Transportation & Infrastructure
Southern Interior Region

THOMPSON NICOLA DISTRICT
OLD SPENCES BRIDGE No. 2411

OLD SPENCES BRIDGE
PIER CAP REHABILITATION CONCEPTS

PREPARED UNDER THE DIRECTION OF

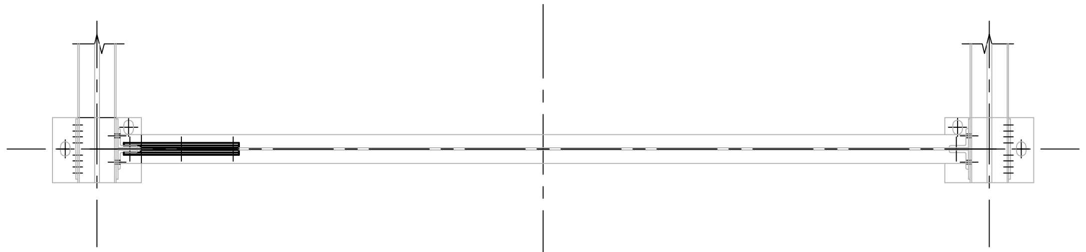
ENGINEER OF RECORD
DATE

SEAL

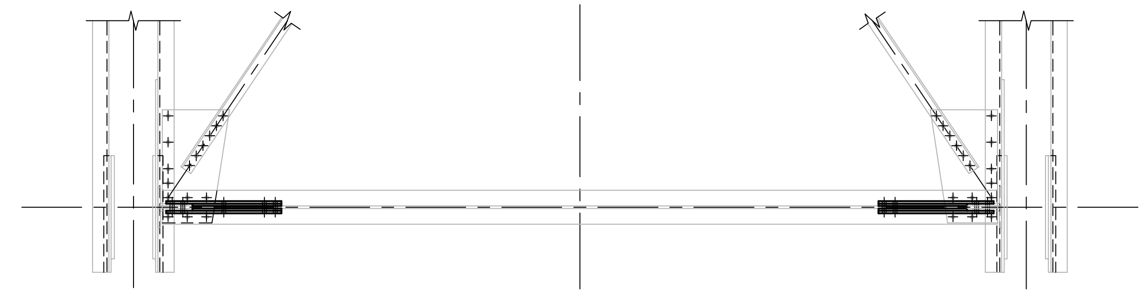
DESIGNED MS DATE 2009-12-10
CHECKED AG DATE 2009-12-10
DRAWN KAM DATE 2009-12-10
SCALE AS NOTED
NEGATIVE No.

FILE No.	PROJECT No.	REG.	DRAWING No.
1884	2411	2	1884-2411-R07 PA

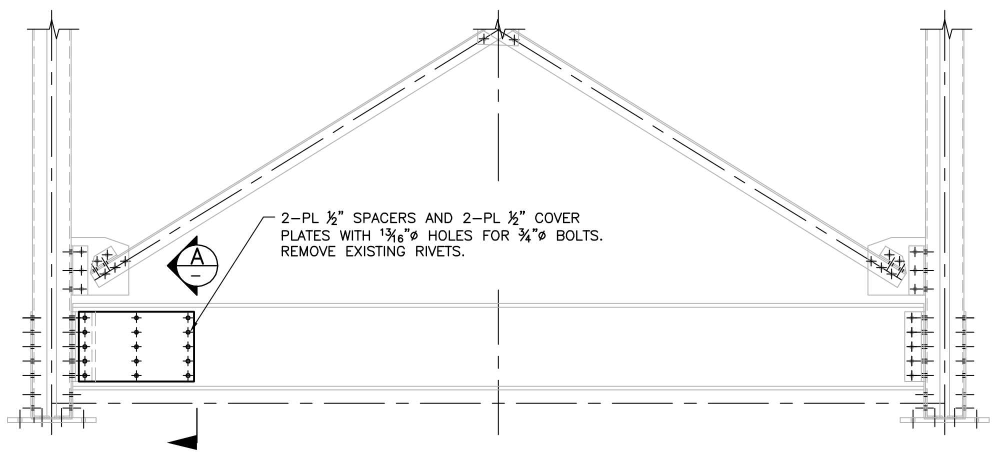
File: G:\1884\Old Spences Bridge\Inspection Report\Drawings\1884-2411-R07.dwg Save Date: Dec 10, 2009 2:54 PM Saved By: PREES



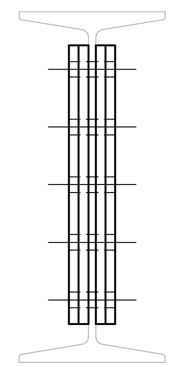
REPRESENTATIVE JACKING BEAM PLAN
SCALE 1:20



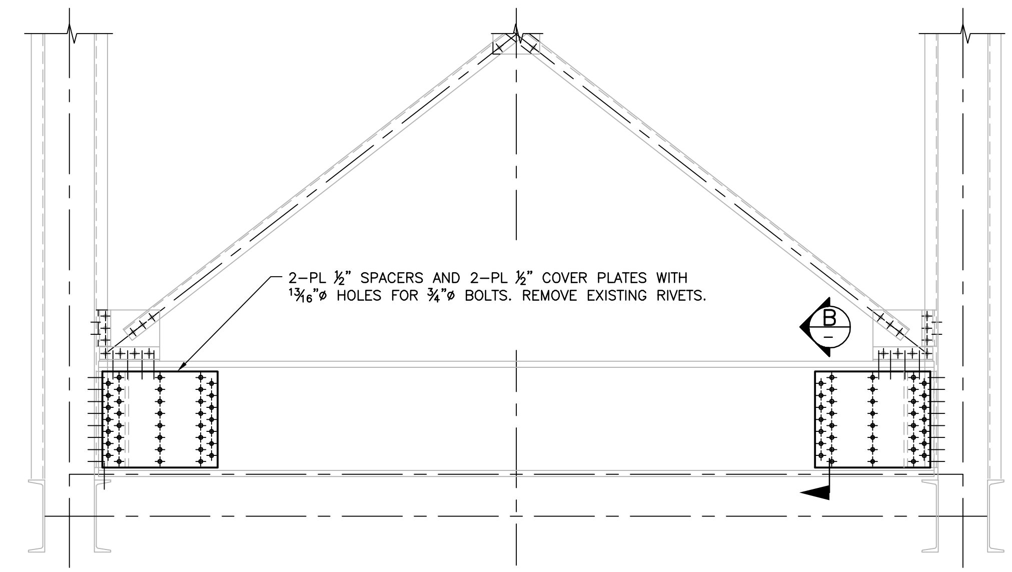
REPRESENTATIVE JACKING BEAM PLAN
SCALE 1:10



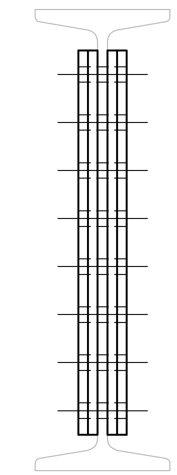
REPRESENTATIVE JACKING BEAM ELEVATION SPAN 1, 2 & 5
SCALE 1:20



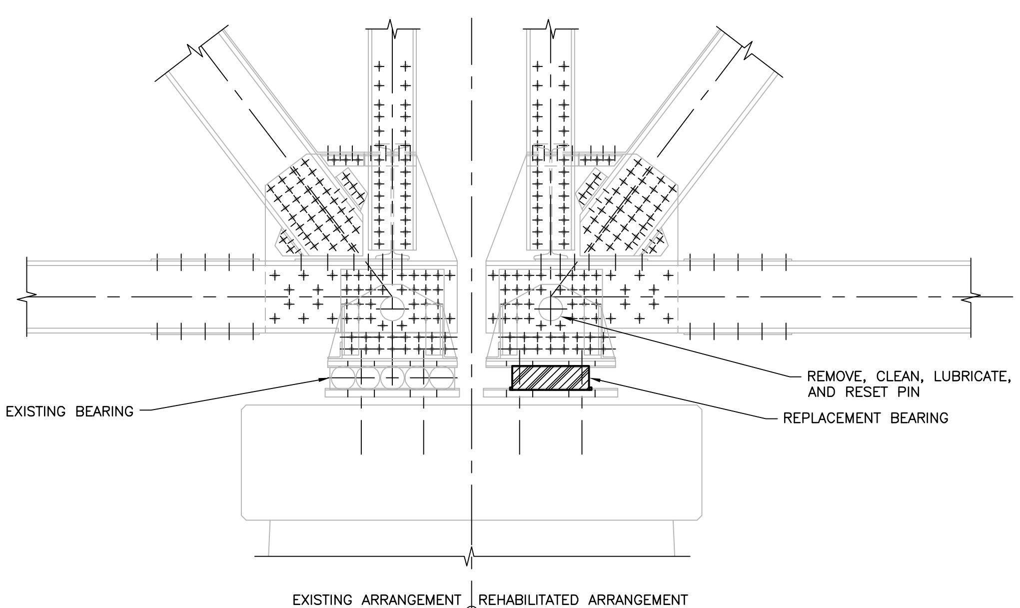
SECTION A –
SCALE 1:5



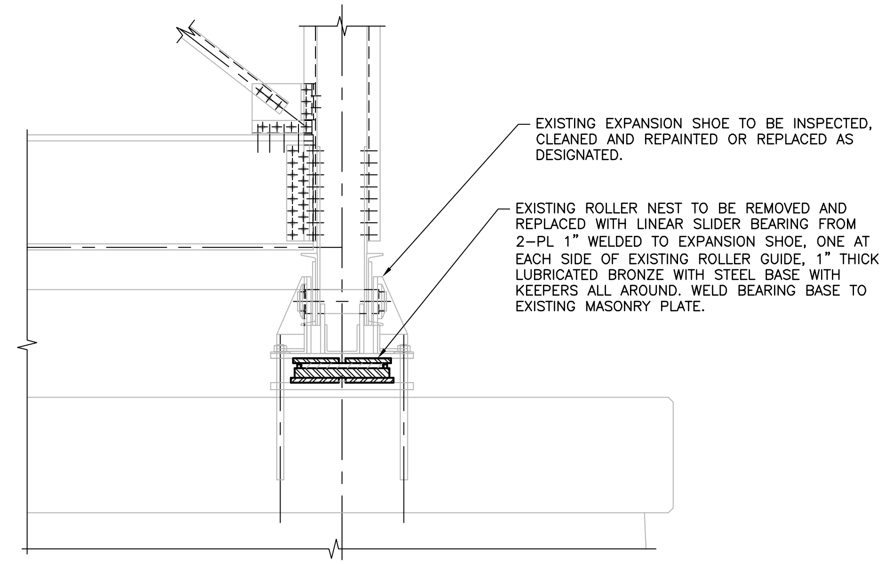
REPRESENTATIVE JACKING BEAM ELEVATION SPAN 3 & 4
SCALE 1:20



SECTION B –
SCALE 1:5



ELEVATION – BEARING REHABILITATION DETAILS AT PIER No. 3
SCALE 1:20
(ALL OTHER SLIDING TRUSS BEARINGS – SIMILAR CONCEPT)



SECTION – REPLACEMENT SLIDING BEARING
SCALE 1:20

PRELIMINARY 2009 DEC 10

Consultant Logo

BUCKLAND & TAYLOR LTD.
Bridge Engineering

Rev	Date	Description	Init
–	09/11/20	REVISION IN PROGRESS...	

REVISIONS

BRITISH COLUMBIA
Ministry of Transportation & Infrastructure
Southern Interior Region

THOMPSON NICOLA DISTRICT
OLD SPENCES BRIDGE No. 2411

OLD SPENCES BRIDGE
BEARING REHABILITATION CONCEPTS

PREPARED UNDER THE DIRECTION OF

ENGINEER OF RECORD

DATE

SEAL

DESIGNED MS DATE 2009-12-10
CHECKED AG DATE 2009-12-10
DRAWN PBR DATE 2009-12-10
SCALE AS NOTED
NEGATIVE No.

FILE No.	PROJECT No.	REG.	DRAWING No.
1884	2411	2	1884-2411-R08 PA

File: G:\1884\Old Spences Bridge\Inspection Report\Drawings\1884-2411-R08.dwg Save Date: Dec 10, 2009 3:01 PM Saved By: PREES