Bridge Inventory Re-Evaluation 2017 – 9th Avenue Viaduct Load Rating

Structure ID: 001130

Prepared for:
The Municipal Corporation of the City of Moose Jaw
228 Main Street North, Moose Jaw, Saskatchewan, S6H 3J8

Prepared by:
Stantec Consulting Ltd.
400-1820 Hamilton Street
Regina, SK. S4P 2B8

Project No. 111000232

July 9, 2018

Sign-off Sheet

This document entitled Bridge Inventory Re-Evaluation 2017 – 9th Avenue Viaduct Load Rating was prepared by Stantec Consulting Ltd. ("Stantec") for the account of Saskatchewan Ministry of Highways & Infrastructure (the "Client"). Any reliance on this document by any third party is strictly prohibited. The material in it reflects Stantec's professional judgment in light of the scope, schedule and other limitations stated in the document and in the contract between Stantec and the Client. The opinions in the document are based on conditions and information existing at the time the document was published and do not take into account any subsequent changes. In preparing the document, Stantec did not verify information supplied to it by others. Any use which a third party makes of this document is the responsibility of such third party. Such third party agrees that Stantec shall not be responsible for costs or damages of any kind, if any, suffered by it or any other third party as a result of decisions made or actions taken based on this document.

Prepared by ______________________________

(signature)

Ronald Luo, P.Eng., P.E.

Reviewed by ______________________________

(signature)

Kevin Paul, P.Eng.
Sr. Associate, Sector Leader, Transportation – Project Manager

Approved by ______________________________

(signature)

Kelvin DeGrow, P.Eng., PMP
Sr. Bridge Engineer, Transportation – 2nd Level Supervisor

Table of Contents

Sign-off Sheet

LIST OF TABLES

LIST OF FIGURES

LIST OF APPENDICES

1.0 INTRODUCTION

2.0 EXISTING CONDITIONS

3.0 LOAD RATING

3.1 STRUCTURAL SYSTEM AND LOADING

The bridge was built in 1980 and has a total length of 322.0m. It consists of nine continuous spans (24.0m + 33.5m + 39.2m + 43.3 m+ 48.5m + 36.0m + 36.0 m+ 32.0m + 29.5m). The bridge has two, welded steel girders. The girders are supported on concrete piers and abutments. The abutments and the piers are supported on reinforced concrete piles.

Currently the cast-in-place concrete deck has an overall width of 12.13m with a clear traffic width of 8.92 m. A concrete curb is on the east and a sidewalk is on the west sides of the bridge. A steel bridge rail with three tubes is anchored to the concrete curb and sidewalk on both sides of the bridge.

Traffic volumes at this bridge site show an Average Annual Daily Traffic (AADT) of 13,628 from 2011 traffic data. The original design vehicle was the three axle HS20-44 vehicle plus impact with a GVW of 32.2 tonnes. The load rating trucks are Typical Normal Traffic Vehicles for use on Primary Highways as the large AADT and shown in Saskatchewan's Ministry of Highways and Infrastructure's Bridge Evaluation Guidelines BE-100. The PS trucks and PC trucks for the load limit curve (chart) are also shown in BE-100.

The following elements were evaluated:

- Girders; and
- Floor beams.

Analysis of the steel girder spans begins with the existing configuration of the bridge. Two main girders, spaced 9.0 metres apart, support the cast in place concrete deck. Floor beams, spaced a maximum of 2.74 metres, span between the two steel girders. A review of the drawings provided by the City of Moose Jaw identified that there are shear studs placed on the top flanges of the steel girders within the continuous spans. This means the deck is composite with the girders and the girders alone support the dead and live loads. However, All through, there are shear studs placed on the top flanges of the floor beams, the studs are mainly within haunch area, not yet intrude into the concrete deck height of 200 mm. Therefore, the deck was not considered as composite with the floor beam to support the dead and live loads.

Loads from the girders are supported by steel pot bearings attached to the tops of the substructure units.

A load rating on the concrete deck was not carried out for this structure in accordance with BE-100.

3.2 CONDITION INSPECTION

2015 Bridge Deck Testing Assessment Report indicates there are few major concerns with the steel girders and the identified deterioration areas are most at expansion joints, bearings, deck drain, approach slabs, and wingwall. The 2013 Bearing Rope Access Inspection identifies that the most critical defects were found at Bearing 2 (on the north abutment west side) and 20 (on the south abutment west side). The bearings at the south abutment have since been replaced.

3.3 CRITICAL ELEMENTS

With only two-steel girders supporting the structure, this bridge is considered fracture critical. The girders have no redundancy, therefore, are the critical element in the structural system. The concrete deck is supported on the two-steel girders which are spaced 9.0m apart. The requirements in the Canadian Highway Bridge Design Code, S6-14 (CHBDC), for a bridge to be analyzed using the simplified method is not met for this bridge. There are only 2 girders and the girders are spaced more than 4.0m apart do not meet the requirements of Clause 5.6 of the CHBDC. The live load distribution was determined using statically determinate methods and the loads on the girders and beams were determined using beam theory and the methods identified in CHBDC.

Very little material information was found on the drawings provided by City of Moose Jaw. The material properties are shown in Table 3.1 based on the Clause 14.7 of CHBDC.

Table 3.1 Material Properties

Elements	Modulus of Elasticity [MPa]	Design Strength (MPa)	Structural Steel Grade	Density (kN/m^3)	CHBDC or BE100
Existing deck slab concrete	22,340	f'c = 20.0		24.0	14.7.4.3
Reinforcing bars in deck	200,000	Fy = 350		77.0	Table BE-2
Steel girder		Fy = 250			Table 14.1
Floor beam		Fy = 250			Table 14.1
Asphalt				23.5	

The unit weights of all materials were determined in accordance with Table 3.4 of CHBDC.

3.4 DEAD LOADS

3.4.1 Dead Loads on the Steel Girders

The dead loads include; steel girders, concrete deck, concrete curbs, sidewalk, high density topping wearing surface and the railing system. The configurations, size and dimensions of the elements were taken from the drawings and condition inspection report provided by the City of Moose Jaw. The weights of the girders, floor beams and other attachments were calculated per span as the uniform load applied to each span. The dead loads used are presented in Table 3.2.

Table 3.2 Dead Loads on Steel Girders

Elements	Dead Loads Per Girder of Continuous Spans [kN/m]
24.0 m span (first span at the north)	6.76
33.5 m span	6.71
39.2 m span	7.84
43.3 m span	8.94
48.5 m span	9.52
36.0 m span	8.97
36.0 m span	7.32
32.0 m span	7.41
29.5 m span	7.75
Concrete deck	31.26
Curb & Sidewalk	10.49
High density topping (wearing surface)	5.8
Utility Duct	1
Railing	0.75

3.4.2 Dead Loads on the Steel Floor Beam

The dead loads include; steel floor beam, concrete deck & curbs, and high density topping. The size and dimensions of the elements are taken from the drawings provided by the City of Moose Jaw. The dead loads of the deck are based on the contributing width of 2.74 m (maximum floor beam spacing). The deck has been analyzed as a one-way slab based on the spacing of the two girders and the spacing of the floor beams. The dead loads used are presented in Table 3.3.

Table 3.3 Dead Loads on Floor Beams

Elements	Dead Loads Per Floor Beam [kN/m]
Steel floor beam (W610x113)	1.11
Concrete deck & curb	13.15
High density topping (wearing surface)	3.56

3.5 LIVE LOADS

3.5.1 Live Loads

The bridge carries two-lane traffic on the deck. Since the AADT of 9th Avenue Viaduct is 13,628 (2011 numbers) and it is major routine for City of Moose Jaw, therefore, 9th Avenue Viaduct is classified as Class A as per Table 1.1 of CHBDC based on the Average Daily Traffic (ADT) per lane.

The dead loads are equally supported by the two girders. For lateral distribution of live loads, the system is treated as a beam that is simply supported at the steel girders. For the two-girder system, the lateral distributions of loads are calculated based on a statically determinate method. The steel girder closest to the curb side is subjected more live load.

SMHI's 8 Axle B-Train (NP8) for primary highways was considered as the analysis vehicle as shown in Figure 3.1. The NP8 is the heaviest truck.

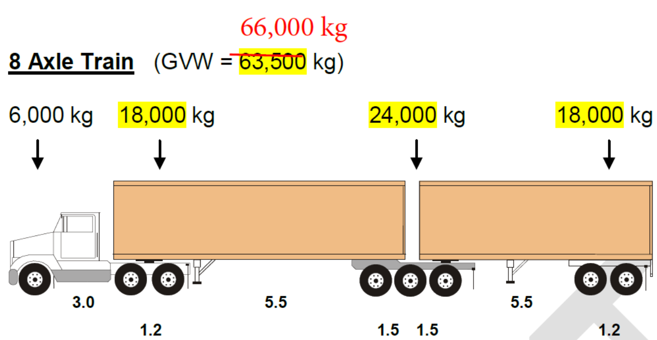


FIGURE BE-1: Typical Normal Traffic Vehicle Configurations for use on Primary Highways

Figure 3.1 8 Axle B-Train for Primary Highways

In accordance with Table 14.3 of CHBDC, the modification factor for multiple-lane loading is equal to 0.9 for two-lanes of traffic.

Truck lane loads consist of 80% of the considered truck, without a dynamic load allowance, but with an additional lane load of 9kN/m over the entire span to generate the maximum load responses.

3.5.2 Corresponding Live Load Distribution Factors for Steel Girder

There are two lanes on the bridge. For both the ULS and SLS load cases, the outside wheel of the analysis truck was set 600mm from the face of the curb. The other truck used in the analysis was set so that the truck would be free to move within the marked lanes of the bridge.

For FLS load case, the truck is placed at the center of the marked lane.

Table 3.4 Live Load Distribution Factors for Steel Girders

Elements	Two-Lanes of Normal Traffic	One-Lanes of Normal Traffic (FLS)
Steel girder moment and shear	1.336	0.782

Live Load Distribution Factors in the Table 3.4 do not include the modification factors for multiple-lane loading.

3.5.3 Target Reliability Index

The target reliability index, β, of the bridge was determined for each element of the structure. This index based on the system behavior, the element behavior, and the inspection level. β is used to determine the dead and live load factors used in the analysis.

The assumptions in determining the target reliability index for the steel girders in the continuous spans as an example are explained below:

1. The system behavior is a Category S2 in accordance with Section 4.7.1.1 of the draft version of SMHI BE-100.
2. The behavior of steel elements subjected to bending moment and shear can be characterized as a Category E3 (Clause 14.12.3 (c) of CHBDC), where failure occurs gradually with some warning such as steel beams in bending and shear.
3. The inspection level of the bridge is Level INSP2, since the inspection is to the satisfaction of the evaluator, with the results of each inspection recorded and available to the evaluator.

It is noted that one floor beam failure would not lead to a collapse of the entire bridge. The abutment roof T-beam and end beams could not be inspected as there was no access to these elements.

These parameters were applied using the same logic for each element of the bridge to produce the target reliability index which are shown in Table 3.5.

Table 3.5 Target Reliability Index

Component and Loading	System Behaviour	Element Behaviour	Inspection Level	Target Reliability Index, β
Steel Girder Moment (continuous. spans)	S2	E3	INSP2	3.00
Steel Girder Shear (continuous. spans)	S2	E3	INSP2	3.00
Floor beam moment	S2	E3	INSP2	3.00
Floor beam shear	S2	E3	INSP2	3.00

3.5.4 Load Factors

The dead load factors were determined in accordance with Clause 14.8.2.1 of CHBDC. Type D1 includes all factory produced components and cast-in-place concrete excluding decks. Type D2 is for cast-in-place concrete decks and high density topping that has been measured in the field. Using the above dead load rating system, the dead load factors used in the analysis are presented in Table 3.6

"Other spans" live load factors were used for the steel girders in shear and moment as the steel girder calculation spans are greater than 10 meter in accordance with Clause 14.13.3.1 of CHBDC. The live load factor for the steel floor beam in moment and shear were based on "Short Span" element since the maximum floor beam spacing (tributary span) is only 2.74 meter, which is less than 6 meter .

NP trucks are considered as alternative loading of normal traffic.

The girder and the floor beams are analyzed using simplified and beam theory methods.

A dynamic load allowance (DLA) of 0.25 was applied as per the Clause of 3.8.4.5.3 (d), for more than three axles of the design truck are on the bridge to produce the maximum moment or shear forces in the steel girder or floor beam.

Table 3.6 Load Factors

Component and Loading	Steel Girder/ Floor Beam	Concrete Deck & Curb	High Density Topping	Barrier (Railing)	Normal Traffic
Load category	D1	D2	D2	D1	Live
Steel Girder Moment (continuous spans)	1.07	1.14	1.14	1.07	1.49
Steel Girder Shear (continuous. spans)	1.07	1.14	1.14	1.07	1.49
Floor beam moment	1.07	1.14	1.14	1.07	2.00
Floor beam shear	1.07	1.14	1.14	1.07	2.00

3.6 LOAD RESPONSES

3.6.1 Load Response on Girders

The structure is modeled as nine-span continuous beam supported by the piers and the abutments. A computer model was created using a computer program (S-Frame) to calculate the loads. Load effects from a live load moving along the modelled beam, dead loads from Table 3.2 and the maximum and minimum shears and moments were inputted into the program. Exporting the results from S-Frame into an Excel spreadsheet allows for the application of the applicable load factors to determine the shears and moments at many point along the bridge.

3.6.2 Load Response on Steel Floor Beams

CSiBridge finite element models were created to analyze the span of 48.5 m with the maximum floor beam spacing, which includes the steel girders, floor beams. Pinned links were added between the floor beams and the concrete deck that allow the loads from the concrete deck adding to the floor beams. The moment and shear modification factors of the concrete deck were set zero to model the non-composite section of the steel girders, which let the steel floor beams take all loads . The load effects from live load and dead loads were entered into the computer model and exported the results of factored moments and shears from CSiBridge into Excel spreadsheets.

3.7 ELEMENT RESISTANCE

Moment and shear capacities for the steel girders and floor beams were computed as per Section 10 of CHBDC. Resistance adjustment factors were determined in accordance with CHBDC Table 14.15.

3.7.1 Girders

The steel girder is a built-up member with a top flange, web and bottom flange. The top flanges, the webs and the bottom flanges are varied in size (widths, thickness and heights). The material strengths of the webs and flanges are assumed to be the same as no information on the material grades are available. Since the shear connectors were installed on the steel girders in the continuous spans, the steel girders were a composite section. Clause of 10.11 – Composite beams and girders of CHBDC was used to calculate the moment resistances including positive moment regions and negative moment regions. Girders were laterally unbraced members with a laterally unbraced length of the floor beam spacing.

The girder shear resistance calculations are based on Clause 10.10.5 of CHBDC.

3.7.2 Steel Floor Beam

The floor beam is a rolled member. The steel section is a W610x113. There are studs welded on the top of the floor beams. The section strengths were calculated in accordance with the appropriate clauses in Section 10.10 of CHBDC.

3.8 SERVICEABILITY LIMIT STATE

The stresses and deflections at the SLS were not checked. The intent of this analysis was to determine the Ultimate Limit State loads and resistances to determine the vehicle configurations that can safely cross the bridge

3.9 FATIGUE LIMIT STATE

Fatigue is the tendency of a member to fail at a stress level below its yield stress when subject to cyclical loading. Fatigue is the primary cause of failure in fracture critical members (FCM), which is a steel member in tension or with a tension element, whose failure would likely result in failure of a span or the entire bridge.

The bottom flanges and the transverse stiffeners are considered fracture critical members as they are subjected tension stresses under the traffic loads. A two-girder system does not provide load path redundancy. Should one girder fail, there is a high probability that the entire structure would collapse.

3.9.1 Method of Analysis

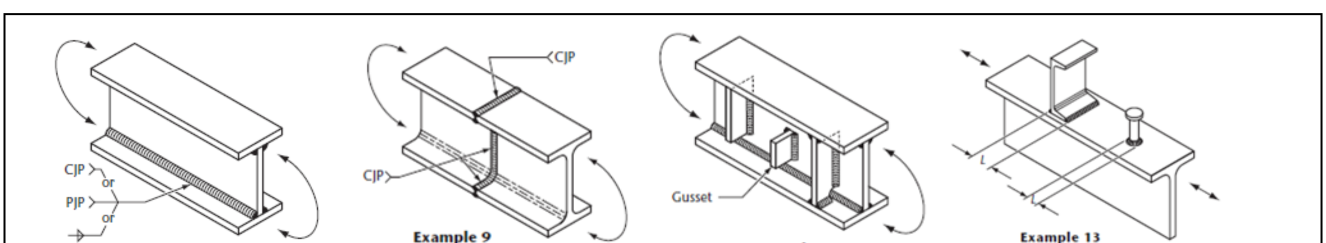
Steel girder bottom flanges, transverse stiffeners and shear connectors were evaluated to the fatigue limit states for stress range due to moment in accordance with Clause 14.18 and Clause 10.17 of CHBDC.

The element stress range resistances were determined by using formulas in Section 10 of CHBDC. The live load induced fatigue stresses were calculated by following Section 10 of CHDBC.

3.9.2 Fatigue categories

The bridge is a multi-span continuous beam structure. The steel girder is a built-up member, with the top flange, web and bottom flange connected with a continuous fillet weld parallel to the direction of applied stress.

The bottom of the bottom flange is categorized as fatigue detail category B. The top of the bottom flange, the bottom of the top flange and the transverse stiffeners' fatigue category is C1 since the fillet-welded connections have welds normal to the direction of load. The top of the top flange is categorized as fatigue detail category C since the flange at stud-type shear connections. The shear connectors on the floor beams has a fatigue category of D, which is also indicated by $F_{sr}{}^{D}$ of Clause 10.17.2.7 of CHBDC. The following Figure 5 are shown the detail categories for load induced fatigue.



Tension flange
Category B

Flange with welds
ground flush
Category B

Web Stiffener
Category C1

Shear Connector
Category D

Figure 3.2 Detail Categories for Load-Induced Fatigue

3.9.3 Traffic Data

The traffic data AADT of 13,628 was utilized for the fatigue analysis. An assumed 25 % truck volume, the single-lane average daily truck traffic is, $ADTT_f = 0.85 \times 13628 \times 0.25 \approx 2900$. It was assumed that the design truck operating in one lane has been running over the bridge since the bridge was built in 1980.

3.9.4 Stress Ranges

The stress range resistances and fatigue stress ranges computed for the evaluation trucks at FLS are listed in the Table 3.7.

Table 3.7 Stress Range Resistance of Fatigue

Elements	Steel Girder Bottom of Bottom Flanges (positive moment region)	Steel Girder Top of Bottom Flanges (positive moment region)	Steel Girder Top of Top Flanges (negative moment region)	Steel Girder Bottom of Top Flanges (negative moment region)
Fatigue Categories	B	C1	C	C1
Stress Range Resistance, F_{sr} (MPa)	55.0	41.5	36.49	41.5
Fatigue Stress Range, $0.52C_Lf_{sr}$ (MPa)	34.0	32.8	33.2	32.2

The calculation was based on having the design trucks crossing the bridge on a continual basis from the time the bridge was opened. Since the design trucks have not consistently crossed the since it was put into service the actual stress range will be lower.

3.9.5 Remaining Fatigue Life of Bridge

There appears to be enough residual fatigue life in the steel elements to reach an age of 100 years. Clause 1.4.2.3 of CHBDC states that the design life would be 75 years, but in our LCC analysis we are assuming that the bridge will be maintained to extend this design life to 100 years.

3.10 LIVE LOAD CAPACITY FACTORS

The capacity at ULS of each element is assessed by calculating the live load capacity factor (F-factors) at critical locations of section change or regions of high stress in accordance with Clause 14.15.2 of CHBDC. The value of F is the ratio of factored resistance minus all applied dead load effects to factored live load effects, including dynamic allowance. A value of F that is less than 1 indicates there is zero reserve capacity in the structural member and likewise, the maximum loading is reached.

The capacity factor for ultimate limit state was calculated using the following formula:

$$F = \frac{U\phi R - \sum \alpha_D D - \sum \alpha_A A}{\alpha_L L(1+I)}$$

The results for primary highway non-permit vehicles are summarized in Table 3.8.

Table 3.8 Live Load Capacity Factors

Elements	F Factor	Notes
Steel Girder – Positive Moment	1.16	
Steel Girder – Negative Moment	1.35	
Steel Girder - Shear	0.76	At pier 7
Floor beam - Moment	1.57	
Floor Beam - Shear	2.75	

The results from Table 3.8 show that the capacities of the steel girder are not adequate in the shear for the loads produced by primary highway non-permit vehicles.

In general, the steel girder vertical stiffener spacing over the piers should be smaller if the web or steel girder heights consist. The vertical stiffener spacing over pier 7 is1.37 m as other locations, which lead to smaller live load capacity factors.

The structure steel girders have few structural distress over pier 7. The real structural steel specified minimum yield stress Fy could be higher than the calculation value of 250 MPa. If Fy is set to 300 MPa or higher as it is common in nowadays, the structure could be structural sound in shear.

3.11 LOAD LIMIT CURVE

3.11.1 Analysis of Permit Single (PS) Vehicle and Permit Controlled (PC) Vehicle

The permit-single (PS) vehicle configurations considered in the permit vehicle analysis are shown in Figure BE-2 of Ministry's BE100 – Bridge Evaluation Guidelines. The permit controlled (PC) vehicle configurations considered in the permit vehicle analysis are shown in Figure BE-4 of Ministry's BE100 – Bridge Evaluation Guidelines. PS or PC truck mixed with NP8 truck was considered, that PC trucks occupy the traffic lane closed to the curb and NP8 is in the other marked traffic lane. The multi-lane factors are 1.0 and 0.7 for PC truck and NP8 truck respectively. Lane load was still considered for PC truck since the NP truck was included on the route. Load effects were calculated from the more severe of the mix of PC truck and NP8 truck with a dynamic load allowance of 0.25 or lane load that consists of 85% of the PC truck or NP8 truck without a dynamic load allowance but with an additional lane load of 9 kN/m over the entire length. The second load case could be that only PC trucks run on the bridge at the center as controlled.

The live load factor for PS truck is set 1.29 per Table 14.13 of CHBDC as the target reliability index, β, of the steel girder shear is 3.0 and type of analysis is statically determinate.

The live load factor for PC truck is set 1.18 per Table 14.12 of CHBDC as the target reliability index, β, of the steel girder shear is 3.0 and type of analysis is statically determinate.

3.11.2 Load Limit Curve

The PS or PC trucks were moved over the bridge in both directions and their total allowable weight was adjusted until an F-factor of unity was achieved. Figure 3.3 shows the maximum GVW allowed for every PS or PC axle configuration, incorporating deck resistance limitations for single, tandem and tridem axle groups.

3.12 BRIDGE STRENGTHENING AND PERMIT VEHICLES

Considering the F-Factors of shear are not greater than 1, strengthening should be required to resist the loads of SMHI's non-permit vehicle fleet.

The load rating indicates that the piers are structural deficient in shear over the pier 7 and pier , which requires to limit the weight of the legal NP trucks.

To strength the girder shear capacity can be done relatively easily by add two extra stiffeners at each side of the pier bearing center.

Appendix A

Appendix A LOAD RATING CALCULATIONS