Bridge Inventory Re-Evaluation 2017 – Main Street Southbound Bridge Load Rating

Structure ID: 001020

Prepared for:
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Sign-off Sheet

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3.1 STRUCTURAL SYSTEM AND LOADING

The Main Street Southbound Bridge over the Moose Jaw River was built in 1967. The bridge has a total span of 27.2 metre (m) and consists of three (3) spans (about 9 m each span). All the spans have nine (9) simply supported prestressed reinforced precast box girders. The box girders are seated side by side on plain neoprene pad continuous along the abutment caps and pier caps. Asphalt riding surface is placed directlyon top of the girders. The bridge provides two-lane traffic with a curb to curb width of 8.5 m. The concrete curbs were built on the both external box girders and the metal bridge railings were installed on the curbs on both sides.

The substructure consists of cast-in-place concrete piers and abutments. The piers consist of a continuous cap supported by six (6) pentagon shape (406 mm diameter) precast prestressed concrete piles.

The load rating (live load capacity analysis) has en performed on the girders, and pier caps following the procedures outlined in CAN/CSA-S6-14, Section 14. In lieu of the Normal Traffic vehicles identified in Section 14, the Saskatchewan Ministry of Highways and Infrastructure (SMHI) non-permit (NP) trucks were used. The SMHI NP vehicles include 5, 6, 7 and 8 axle trucks with axle loads and spacing outlined in Figure BE-1, as shown in SMHI Bridge Design Manual with subsequent updates.

The pier columns/ piles were not evaluated since no geotechnical information for the construction of this bridge is not available. The ground conditions and pile driving information is not available. Without this information, the elements cannot be evaluated.

Neither the top slab nor walls and bottom slab of the box girder were not carried out the load rating evaluation since they are locally effects and little problematic than the girder or the caps.

PS trucks were determined, and Load rating chart was created.

It was assumed that the box girders were made per SMHI standard precast prestressed concrete stringer 30 FT. shown on SMHI standard Plan 1804A, dated 28-2-63.The pier caps are similar as Manitoba Expressway Bridge. No as-built drawings were available at the time of the load rating calculations and the drawings for the Manitoba Street Expressway Bridges were used as the basis for the evaluation.

3.2 MATERIAL PROPERTIES

Material properties of the box girders were taken from the notes on SMHI Standard Precast Prestressed Concrete Stringer Plan 1804A. Pre-cast girder concrete strength was assumed to be

34.5 MPa (5,000 psi) at 28 days. All strands were assumed to be low relaxation strands with a yield strength of 1861 MPa (270 k) with the the initial tension or jacking force is 84.1 kN (18.9 kips). 15 strands with diameter of 11.1 mm (7/16 inch) are placed along the bottom of the girder and 2 strands with diameter of 7.9 mm (5/16 inch) are placed at the top corners of the girder.

The minimum yield strength of the reinforcing steel was assumed to be 345 MPa for both girders and the pier caps. A minimum yield strength was not found on the original drawings. The reinforcing bars were assumed to be intermediate grade. From Table BE-2 of SMHI's BE100 - Bridge Evaluation Guidelines, the intermediate grade reinforcing bar minimum yield strength is taken to be 345 MPa for the structures between 1956-1978. The strength of the concrete in the pier cap was assumed to be15.0 MPa in accordance with Clause 14.7.4.3 of CHBDC since plans and specifications are not available and cores have not been obtained.

The material properties are summarized in table 1.

Table 1 Material Properties

Material	Modulus of Elasticity [MPa]	Design Strength [MPa]
Prestressing Strands	200,000	1861
Reinforcing Bars	200,000	345
Precast Girders at service	27,790	34.5
Precast Girders at transfer	25,680	27.6
Deck and Pier Cap	20,360	15.0

3.3 LOADS

3.3.1 Dead Load

Dead loads identified for the structure included the component self-weight, asphalt, curbs and barriers. All dead loads are assumed to act on one-span simply supported girder.

Three types of dead loads were identified as outlined by CAN/CSA-S6-14 Clause 14.8.2.1:

- Type D1 includes all factory produced components and cast-in-place concrete excluding decks;
- Type D2 for cast-in-place concrete decks and non-structural components; and
- Type D3 for asphalt, which was not measured in the field.

Using the simplified method, the girders support the curb, barrier and asphalt equally. The unit-weights of each element were taken from table 3.3 in CHBDC and are as follows:

- Structural steel (railing) = 77. 0kN/m^3;
- Concrete girders = 24.5 kN/m3 for prestressed concrete ;

- Concrete deck, barrier, and pier caps = 24.0 kN/m^3 for reinforced concrete; and
- Asphalt = 23.5 kN/m^3.

The elements' dimensions were taken from the original drawings for the Manitoba Street Expressway Bridge. The average asphalt thickness was assumed to be 90mm.

Along one girder, the uniformly dead loads are: concrete girder = 8.04 kN/m ; asphalt = 2.58 kN/m ; curb and sidewalk = 2.23 kN/m ; and railing = 0.25 kN/m .

The self-weight of one pier cap is 8.93 kN/m.

3.3.2 Live Load

The live loads considered are according to the load rating system currently used in Saskatchewan. The four typical non-permit vehicles (NP trucks), as normal truck traffic are in accordance with SMHI Bridge Design Criteria (BE-100), version 2016. These trucks are shown in Figure BE-1. However, NP8, the results of the heaviest truck, were presented.

Lane loading was not considered in the evaluation as the girders are short spans of 9.0 m.

PS trucks in Figure BE-2 of BE100 were evaluated and determined in order to have the live load limits to this bridge.

3.4 SIMPLIFIED METHOD FOR GIRDER AND PIER CAP

3.4.1 Girder

The concrete girder is as a simply supported beam with the maximum span of 8.9 m between the supports.

The dead loads are uniformly applied along the girder.

The girders were considered individually resist to dead loads and live loads since there is no concrete deck above the girders. Only three connectors at the girder ends and middle wouldn't be able to transverse vertical shears or longitudinal bending moments. For 1.2 m wide girder, only one wheel can be within the girder width, therefore, the transverse live load distribution was calculated is 0.5.

3.4.2 Pier cap

The pier cap and its piles were modeled as a rigid frame. The dead loads are uniformly placed along the pier cap. The live load is treated as the wheel loads moving transversely along the pier cap. One-wheel load is equal to the total reaction at the pier divided by the number of the wheels. The total reaction at the pier is due to two-lane or one-lane traffic running along the girder.

3.5 TARGET RELIABILITY INDEX

The target reliability index, β, of the bridge is determined for each element of the structure (refer to Table 1.1). This index is a result of the system behavior, element behavior, and inspection level, which is used to determine the dead and live load factors used in the analysis.

Each component of the system has a unique combination of system and element behavior, which affects the resulting target reliability index. Inspection level was consistent for all elements so it was set to INSP2, which where inspection is to the satisfaction of the evaluator, with the results of each inspection recorded and available to the evaluator.

System behavior for each component was established as follows:

- Girders - Category S2 as failure of one girder line would not lead to total collapse, as multiple girders are present;
- Pier Cap – Category S2 as failure of one bay would not lead to total collapse, as multiple columns are present;

Element behavior for each component was determined as follows:

- Bending moment of concrete girder and concrete pier – E3, where the element being considered is subject to gradual failure with warning of probable failure. This can include under-reinforced concrete in bending;
- Shear of concrete girder and concrete pier – E2, where the element being considered is subject to sudden failure with little or no warning but will retain post-failure capacity. This can include concrete in shear and/or torsion with at least the minimum reinforcement required by Clause 14.14.1.6.2(a);

NP trucks were considered an alternative loading to normal traffic. The live load factors for alternative loadings as specified in Clause 14.9.1.6 and Table 14.9. The load factors for the girder moment where considered "short span" and for the girder shear per "other span" as the girder span is 9.0 m. The load factors for the pier moment and the pier shear were considered as "other span" as the reactions are associated the girder shear, or in other words, the reactions are dependent on the shear in the girder.

Using the above, the target reliability indices along with dead and live load factors were developed for each element and change in cross section identified on the structure and summarized in Table 2.

Table 2 Summary of Load Factors

Element	Girder Moment	Girder Shear	Pier Cap Moment	Pier Cap Shear
System Behavior	S2	S2	S2	S2
Element Behavior	E3	E2	E3	E2
Inspection Level	INSP2	INSP2	INSP2	INSP2
β =	3.00	3.25	3.00	3.25
Factors of Dead Load and Live Load				
D1 (α_D)	1.07	1.08	1.07	1.08
D2 (α_D)	1.14	1.16	1.14	1.16
D3 (α_D)	1.35	1.40	1.35	1.40
NP analysis (α_L)	1.49	1.56	1.49	1.56
PS analysis (α_L)	1.44	1.34	1.29	1.34

3.6 DYNAMIC LOAD ALLOWANCE

A dynamic load allowance (DLA) is applied to all trucks. In accordance with Clause 3.8.4.5.2 of CHBDC as shown below:

- 0.25 for the concrete girders and piers;

3.7 LIVE LOAD CAPACITY

Live load capacity factors were established at changes in cross section or at high stress regions for each element listed previously. The live load capacity factor was calculated using the following formula:

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

Where U is the element resistance adjustment factor, ϕ is a material reduction factor, R is the element resistance, αD, αA, and αL are the dead load, additional loads, and live load factors respectively, D, A, and L are the element applied dead load, additional load, and live load respectively, and I is the impact value, which is equal to the Dynamic Load Allowance (DLA).

Results for NP vehicles are summarized in Table 3.

Table 3 **Live Load Capacity Factor**

Location	Load Cases	8 Axle NP
Girder	Max. Moment	2.01
	Max. Shear	1.76
Pier Cap	Max. Moment	1.81
	Max. Shear	1.31

F-factors represent the load-carrying capacity remaining once dead loads have been accounted for. The resulting F-factors can be interpreted as follows:

- Values equal to or greater than 1.0 implies that once dead loads are accounted for, the element still has the capacity required to resist live loads introduced by the worst case loading condition of the evaluation vehicles;
- Values less than 1 indicate that the structure can safely support the dead loads of the structure, but cannot safely support the additional truck loading assumed in the evaluation; and
- Values less than 0 indicate that the structure cannot safely support its own weight (dead loads).

In review of Table 3, the critical element is the pier cap in shear with an F value of 1.31 for the NP trucks.

3.8 LOAD LIMIT CURVE

3.8.1 Permit Single (PS) Vehicle Analysis

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. Only one load case was considered, that PS trucks occupy both two lanes. Load effects were calculated from the more severe of PS truck alone with a dynamic load allowance of 0.25 or 85% of the PS truck without a dynamic load allowance but with an additional lane load of 8 kN/m over the entire length. The multi-lane factor of 0.9 was considered.

The case that PS truck is mixed with NP trucks was not considered.

The live load factor for PS truck is set 1.34 per Table 14.13 of CHBDC as the target reliability index, β, of the steel girder shear is 3.25 and type of analysis is statically determinate.

3.8.2 Load Limit Curve

The PS 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 axle configuration, incorporating deck resistance limitations for single, tandem and tridem axle groups.

3.9 BRIDGE STRENGTHENING AND PERMIT VEHICLES

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

LOAD RATING CALCULATIONS

Appendix A

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