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HIGHWAY BRIDGE EVALUATION FOR PARKS CANADA IN ATLANTIC PROVINCES

St. Peters Canal National Historic Site
Volume 2 of 2
Bridge Evaluation Report

MRC
Consultants in Transportation

February 2007

PARKS CANADA

ST. PETERS CANAL NATIONAL HISTORIC SITE

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2007
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VOLUME 2 of 2

BRIDGE EVALUATION REPORT

MRC

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BRIDGE EVALUATION PROCEDURE

The bridge evaluation procedure for Atlantic Provinces Parks Highway Bridges is summarized as follows:

1. COLLECTION OF DATA

The availability of contract drawings and shop drawings for the bridge was checked. Based on the drawings, the field inspection staff was informed about the necessary information to be collected from site. Various pieces of information about the bridge were obtained, i.e. determination of bridge type, number of spans, time of construction, materials of construction etc. The information was also obtained with regard to rehabilitation of the bridge. The inspection data, report, and photographs of the bridge were also collected.

The bridge dimensions and the component sizes were taken from the drawings subject to confirmation by the inspection. When the drawings were not available, measurements of the components recorded during inspection were used depending upon the type of material.

Once the above information was assembled, the following procedure was followed.

2. MATERIAL STRENGTHS

The contract and shop drawings provided by the department were first checked for information about material strengths. If the drawings did not provide adequate information about the material strengths "Canadian Highway Bridge Design Code" (CHBDC), CAN/CSA-S6-00 clause 14.6 provisions were used in the evaluation.

Based on provisions of this clause, the strengths of different types of steel were determined as follows:

Structural Steel

Date of Bridge Construction	**Specified Fy, MPa**	**Specified Fu, MPa**
Before 1905	180	360
1905 - 1932	210	420
1933 - 1975	230	420
After 1975	250	420

where Fy and Fu are yield and ultimate strengths respectively.

Reinforcing Steel

Based on the information available in the drawings about the grade of steel (structural grade, medium or hard), the yield strength fy, (MPa), of the reinforcing steel was determined as below:

Date of Bridge Construction	Grade Struct.	Medium	Hard	Unknown
Before 1914	-	-	-	210
1914 - 1955	230	275	345	230
1956 - 1978	275	345	415	275
After 1978				
- stirrups & ties	300	350	400	300
- remainder	300	350	400	350

Prestressing Steel

The tensile strength fpu, was taken as follows:

Date of Bridge Construction	fpu (MPa)
Before 1963	1600
Otherwise	1725

Concrete

CHBDC clause 14.6 recommends the use of following concrete strengths in the absence of any information in the drawings.

	fc´ (MPa)
Reinforced Concrete	
in substructure	15
in superstructure	20
Prestressed Concrete	25

When the above values in MRC's opinion, did not seem realistic, sample tests might be recommended to determine actual strengths in accordance with CHBDC clause A14.1.

3. MATERIAL DENSITIES

The following densities for various materials were used in evaluation in accordance with CHBDC clause 3.6:

Material	Unit Weight KN/m^3
Bituminous Wearing Surface	23.5
Concrete:	
Reinforced	24.0
Prestressed	24.5
Granular Soil	22.0
Wood:	
Hardwood	9.5
Softwood	6.0
Steel:	77.0

4. DEAD LOADS

The dead loads on various components of the bridge were calculated using the material densities in the previous section. Three types of dead loads were calculated:

D1: Dead loads of factory produced components, and cast-in-place concrete excluding decks

D2: All other dead loads (such as deck slab, curbs, railings, sidewalks etc.) except asphalt. Asphalt loads may be included if exact thicknesses are available

D3: Asphalt loads, where 90 mm of thickness is assumed in the absence of exact information

5. LIVE LOADS

The live load truck and lane loads were applied as per CHBDC 14.8.1 wherein, CL-1-W truck and corresponding lane loading is specified for level-1 evaluation. However, for bridges in the province of “New Brunswick”, CL-1-625-ONT truck and corresponding lane load were used for analysis. (See Figures)

Appropriate addition of “Dynamic Load Allowance” (DLA) was applied to the live load truck in accordance with CHBDC 3.8.4.5 as follows:

<u>DLA</u>

0.50 For deck joints

0.40 Where only one axle of the CL-W truck was used, except for deck joints

0.30 Where any two axles of CL-W truck, or axles 1, 2 and 3 were used

0.25 Where 3 axles of the CL-W truck, except for axles 1, 2 and 3, or more than 3 axles were used

When the results of analyses and resistance calculations showed that posting might be required, additional analyses using CL-2-W and CL-3-W trucks and corresponding lane loadings were performed.

Live load distribution factors for distribution of shear and moment were calculated based on provisions of CHBDC 5.7.1. The simplified method was used if the conditions set by CHBDC 5.7.1.1 were satisfied.

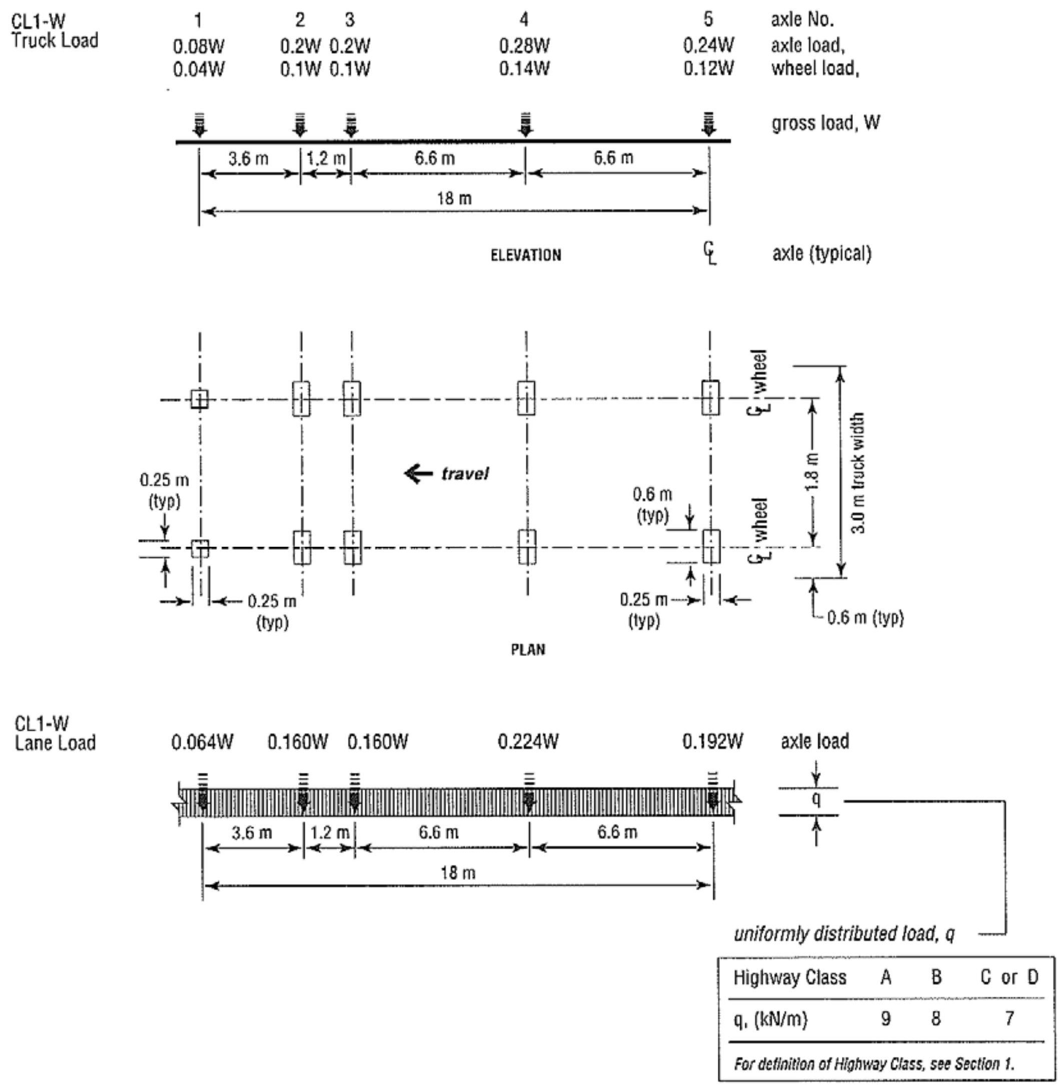
Live load for pedestrian bridge was taken in accordance with CHBDC 3.8.9 or as per manufacturer's drawing whichever was greater. Alternately, 80 KN maintenance vehicle load was also used in separate analysis and the controlling forces from the two analyses were used for evaluation.

6. LIMIT STATES

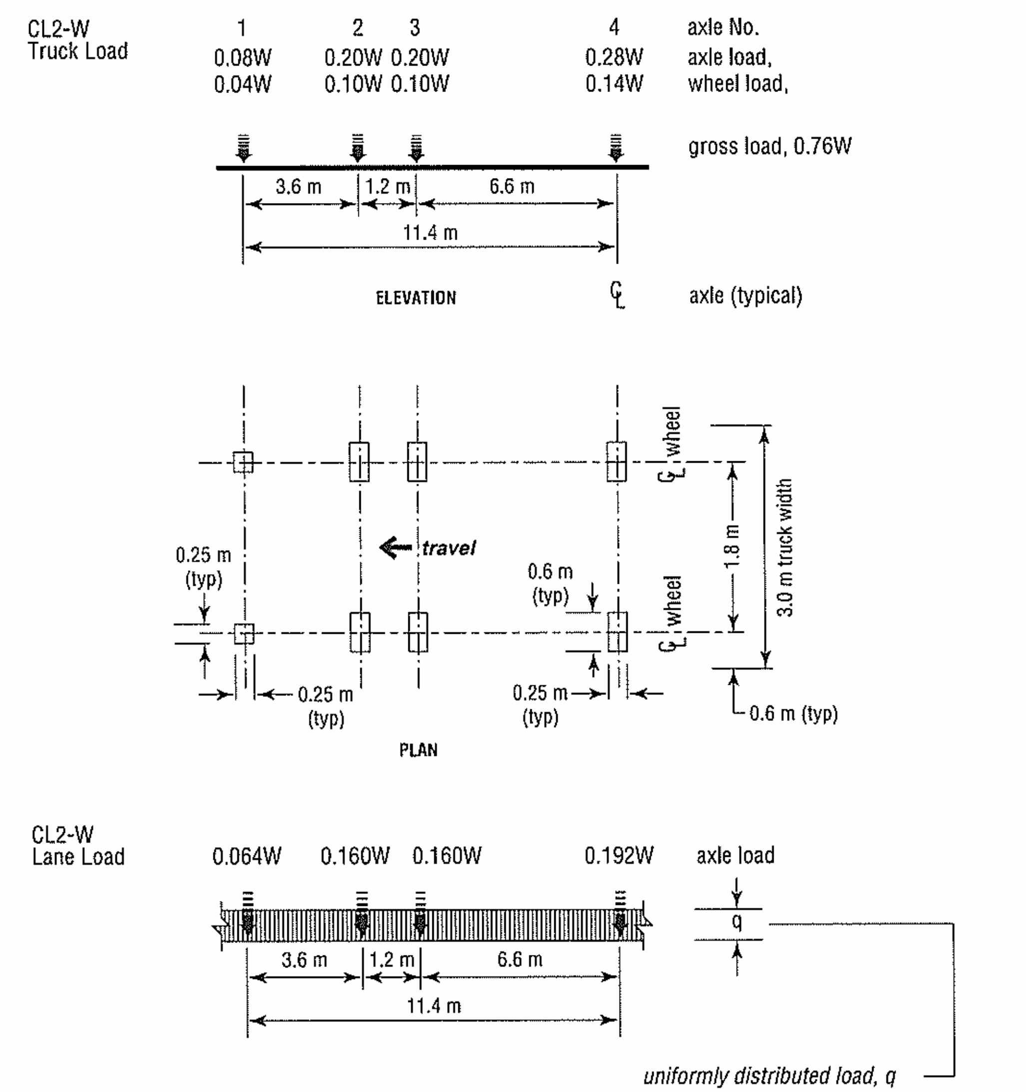
The requirements for ultimate, serviceability and fatigue limit states were checked as per clauses 14.4.1 and 14.18 of the CHBDC.

For concrete and wood components, and for steel components without fatigue prone details or fatigue related defects, only ultimate limit state was investigated. It would include factored combination of forces to find ultimate moments, shear forces etc. and full strengths (concrete strength or steel yield strength) of the materials was used in strength computation.

Level 1 Evaluation Loads with CL1-W Truck



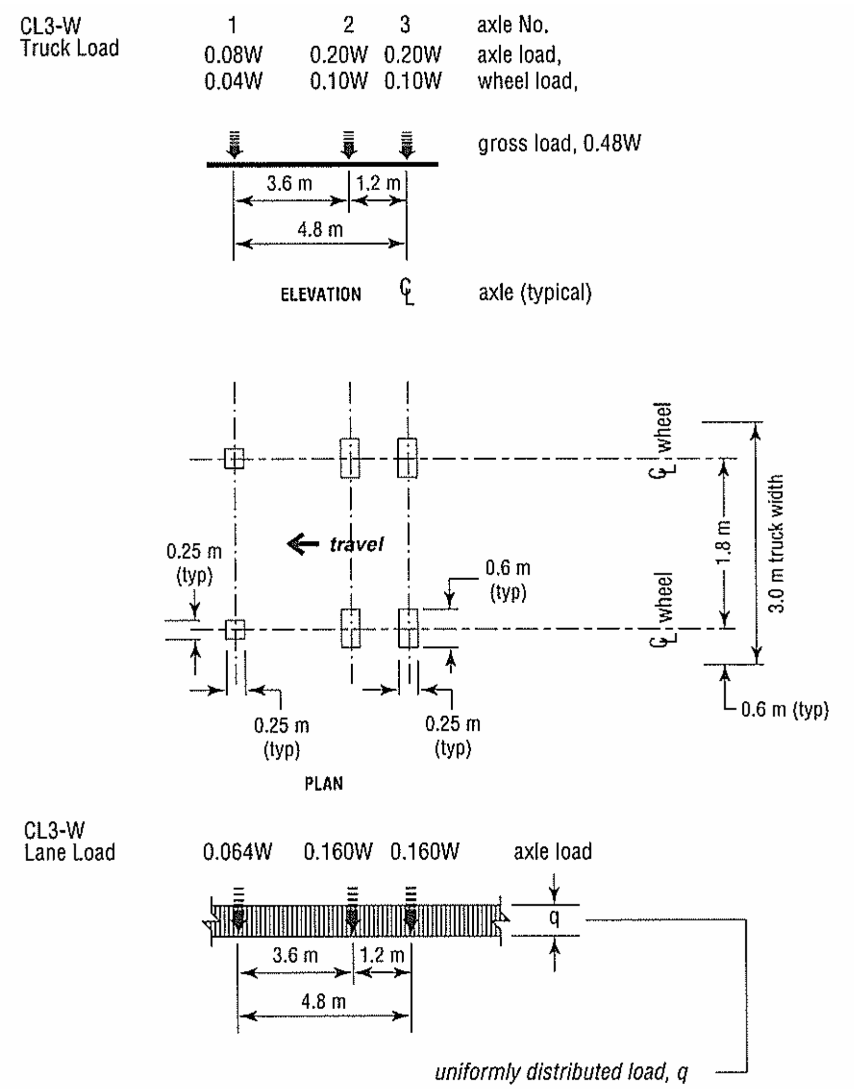
Level 2 Evaluation Loads with CL2-W Truck



Highway Class	A	B	C or D
q, (kN/m)	9	8	7

For definition of Highway Class, see Section 1.

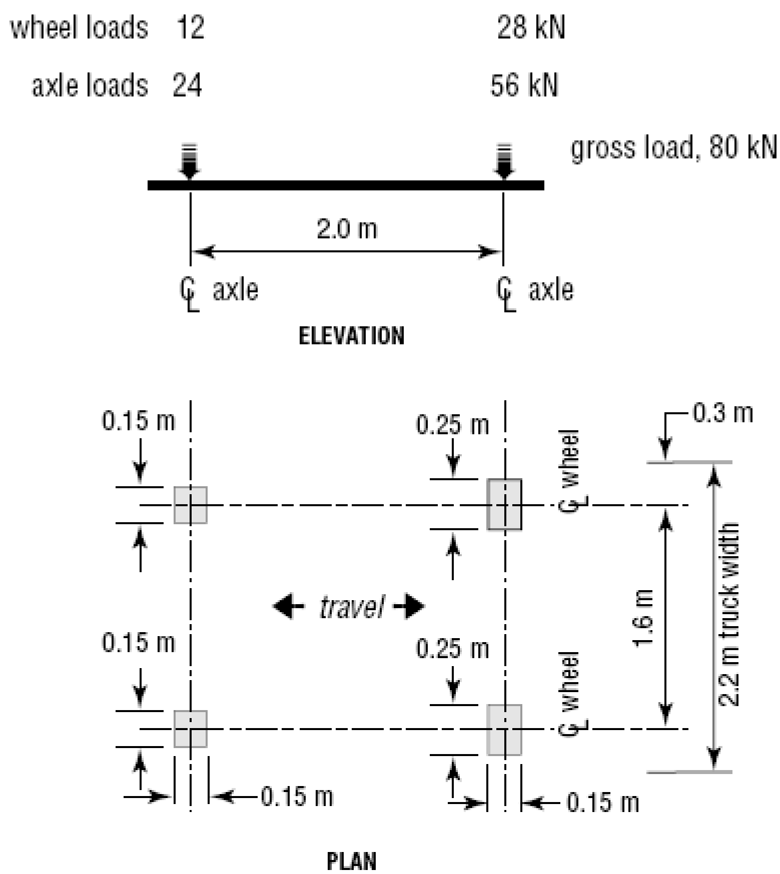
Level 3 Evaluation Loads with CL3-W Truck



Highway Class	A	B	C or D
q, (kN/m)	9	8	7

For definition of Highway Class, see Section 1.

Maintenance Vehicle Load



7. LOAD FACTORS

The load factors were applied to the different types of dead loads and live load in analyses for ultimate limit state evaluation. These load factors are based on the reliability of different types of loads and are therefore tabulated against the "Target Reliability Indices" as below:

Maximum Dead Load Factors, $\propto_D$

	Target Reliability Index, β								
Dead Load Category	2.00	2.25	2.50	2.75	3.00	3.25	3.50	3.75	4.00
D1	1.03	1.04	1.05	1.06	1.07	1.08	1.09	1.10	1.11
D2	1.06	1.08	1.10	1.12	1.14	1.16	1.18	1.20	1.22
D3	1.15	1.20	1.25	1.30	1.35	1.40	1.45	1.50	1.55

Live Load Factors, $\propto_L$

	Target Reliability Index, β						
	2.50	2.75	3.00	3.25	3.50	3.75	4.00
All Spans	1.35	1.42	1.49	1.56	1.63	1.70	1.77

The target reliability index used in the above tables was obtained from CHBDC Table 14.11 (a) as under:

Target Reliability Index, β, for Normal Traffic, PA, PB, and PS Traffic

System Behaviour	**Element Behaviour**	**Inspection Level**		
		INSP1	**INSP2**	**INSP3**
S1	E1	4.00	3.75	3.75
	E2	3.75	3.50	3.25
	E3	3.50	3.25	3.00
S2	E1	3.75	3.50	3.50
	E2	3.50	3.25	3.00
	E3	3.25	3.00	2.75
S3	E1	3.50	3.25	3.25
	E2	3.25	3.00	2.75
	E3	3.00	2.75	2.50

The "S", "E" and "INSP" are the "System Behaviour", "Element Behaviour" and "Inspection Level" respectively where:

System Behaviour

System behaviour considers the effect of any existing deterioration and is classified by one of the following categories:

(a) Category S1, where element failure leads to total collapse. This would include failure of main members with no benefit from continuity or multiple load paths, such as a simply supported girder in a 2-girder system;

(b) Category S2, where element failure probably will not lead to total collapse. This would include main load-carrying members in a multigirder system, or continuous main members in bending;

(c) Category S3, where element failure leads to local failure only. This would include deck slabs, stringers, and bearings in compression.

Element Behaviour

Element behaviour considers the effect of any existing deterioration and is classified by one of the following categories:

(a) Category E1, where the element being considered is subject to sudden loss of capacity with little or no warning. This might include failure by buckling; concrete in shear and/or torsion with less than the minimum reinforcement required by Clauses 8.9.2.2 and 8.9.2.3; bond (pullout) failure; suspension cables; eyebars; bearing stiffeners; over-reinforced concrete beams; connections; concrete beam column compression failure; or steel in tension at net section.

(b) Category E2, where the element being considered is subject to sudden failure with little or no warning but would retain post-failure capacity. This might include concrete in shear and/or torsion with at least the minimum reinforcement required by Clauses 8.9.2.2 and 8.9.2.3; steel plates in compression with post-buckling capacity.

(c) Category E3, where the element being considered is subject to gradual failure with warning of probable failure. This might include steel beams in bending or shear; under-reinforced concrete in bending; decks; or steel in tension at gross section.

Inspection Level

Evaluation was not undertaken without inspection. Inspection levels are classified by one of the following levels:

(a) Level INSP1, where a component is not inspectable. This might include hidden members not accessible for inspection such as interior webs of voided slabs;

(b) Level INSP2, where inspection is to the satisfaction of the evaluator, with the results of each inspection recorded and available to the evaluator;

(c) Level INSP3, where inspection of critical and/or substandard components has been carried

8. ANALYSES

The analyses were performed through structural software such as SAP2000 (Ver. 10.) The structure's geometry, material properties, section properties or member sizes, computed loads and the load factors for desired limit states were input. Final factored forces (Moments, Shears, Axial Loads, etc.) including the effects of "DLA" and distribution of live loads were obtained at the critical locations for various components such as girders, deck, cross beams, etc.

9. RESISTANCE CALCULATION

Following the analyses, resistances of various components for flexure, shear, axial loads, etc. were computed in accordance with the relevant sections of CHBDC for different materials. Appropriate "Resistance Adjustment Factors" (U) were applied to the computed resistances. The "U" values were obtained from CHBDC Table 14.13.2 as follows:

Resistance Category	**Resistance Adjustment Factor, *U***
Structural Steel (ϕ per Clause 10.5.7)	
Plastic Moment	1.00
Yield Moment	1.06
Inelastic LTB Moment	1.04
Elastic LTB Moment	0.96
Compression or tension	1.01
Shear (stocky web)	0.87
Shear (tension field)	0.87
Bolts	1.27
Welds	1.32
Rivets	1.81
Composite - Slab on Steel Girder (ϕ per Clauses 8.4.6 and 10.5.7)	
Bending Moment	0.96
Shear Connectors	0.94
Reinforced Concrete (ϕ per Clause 8.4.6)	
Bending Moment	
$p \leq 0.4\, p_b$	1.06
$0.4\, p_b \leq p \leq 0.7\, p_b$	0.99
Axial Compression	1.11
Shear (> min. stirrups)	0.94
Shear (< min. stirrups)	0.82
Prestressed Concrete (ϕ per Clause 8.4.6)	
Bending Moment	
$\omega_p \leq 0.15$	1.01
$0.15 \leq \omega_p \leq 0.30$	0.94

10. CAPACITY/DEMAND (C/D)

The ratios of resistances to the forces for various bridge components were computed (e.g. $\frac{M_r}{M_f}$, $\frac{V_r}{V_f}$ and $\frac{C_r}{C_f}$) at critical locations. The M_r, V_r and C_r values included the resistance adjustment factors “U”.

11. POSTING

If all the C/D ratios were equal to or greater than 1.0, no posting was recommended. The values of C/D close to 1.0 were also considered sufficient, if in the evaluation engineer’s judgement, there were sufficient factors of safety involved, to ignore the minor deficiency.

Reinforced concrete bridges with C/D ≥ 0.9 were not recommended for posting in accordance with CHBDC 14.17.1. It needs to be mentioned that CHBDC 14.17.1 does not specify the limit of 0.9. It was MRC’s opinion to post the reinforced concrete bridges if C/D was significantly below 1.0.

When a posting was required, additional analyses were performed for CL-2-W and CL-3-W loadings and corresponding factored forces were computed. “Live Load Capacity Factor” (F) was computed as follows:

$$F = \frac{U\phi R - \sum \propto_D D - \sum \propto_A A}{\propto_L L(1+\mathrm{I})}$$

where,

$U\phi R$ is the resistance after adjustment,

$\sum \propto_D D$ is the factored Dead Load Force.

$\sum \propto_A A$ is the factored force due to additional loads including wind, creep, shrinkage, temperature and differential settlement.

$\propto_L L(1+\mathrm{I})$ is factored force due to Live Load including the DLA.

The requirement of posting (single or triple) or consideration for closure was checked in accordance with CHBDC 14.7.2 as below:

F > 1	No posting required
1>F≥0.3	For Eval. 1 (CL-1-W) Triple Posting
F<0.3	For Eval. 1 and F>0.3 for Eval. 3 (CL-3-W) Single Posting
F<0.3	For Eval. 3 Consider closure of bridge

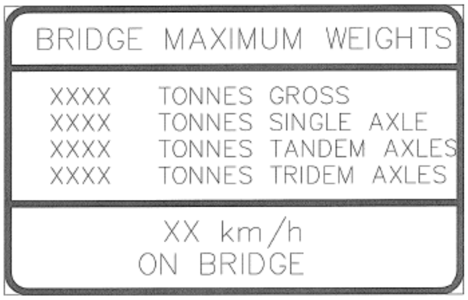
Once the requirement for posting was determined, the axle loads for single, tandem and tridem axle postings were obtained as per CHBDC 14.17.3.3 and regulations set by the Province of Nova Scotia as below:

Gross Weight = 63.7 F

Single Axle = 9.1 F

Tandem Axles = 17.0 F

Tridem Axles = 23.0 F



Sign Size: 8'–0" x 4'–0"

Where,

63.7 is the CL1-W truck weight in tonnes and F is the live load capacity factor for CL1-W loading.

12. CONCLUSIONS

The conclusions were made based on the evaluation results. These would include recommended postings, if required. The recommendations for further inspections and/or testing of specimens might also be included if required as per MRC's opinion or judgement. Recommendation for possible measures which can be adopted in the near future to raise or remove the posting limits might also be made.

13. DISCUSSIONS AND RECOMMENDATIONS

The St. Peter's Canal Bridge is rated as satisfactory for standard CHBDC loading after reducing the "Target Reliability Indices" by 0.25. The application of this provision is a common practice in the Province of Ontario. By adjusting this parameter, MRC recommends that the bridge be inspected at a frequency indicated by the BIM and be re-evaluated within the next five (5) years.

The results of the evaluation indicate that the flexural resistance of the stringers is slightly lower than that required to withstand standard loading (C/D = 0.97). The overall visual inspection indicates no distress in the structure. As the C/D ratio is quite close to 1.0, no posting is being recommended at this time.

APPENDIX
Bridge Evaluation Reports