Assessment of the Accuracy of Bridge Evaluation Methods using CHBDC

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Abstract

This paper presents a comparative study on the common methods in the safety assessment of bridges according to the Canadian Highway Bridge Design Code (CHBDC). The two Live Load Capacity Factor equations outlined in section 14 of the CHBDC are compared and their shortcomings and strengths are discussed. The CHBDC guidelines are also utilized for performing load rating analysis of the main and side spans of Harbour bridge. The comparative study on the Harbour Bridge shows the mean-load method provides a more accurate assessment of the bridge structural reliability while the simplified general method results in an inaccurate and unconservative assessment. A more rigorous analysis of the local traffic data is used for the evaluation of the Harbour bridge main and side spans. It was found that the bridge evaluation using the CL-625 successfully envelop the assessment based on the local traffic data.

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1 Introduction

In bridge engineering, the common practice in the evaluation of bridges is by performing load ratings analysis. The Live Load Capacity Factor (F) outlined in section 14 of the Canadian Highway Bridge Design Code (CHBDC) represents a ratio for the evaluation of the acceptance criteria of the considered live load. Two methods of evaluation are defined in the CHBDC for calculating the live load capacity factor. The general method utilizes the load and resistance factors determined based on target reliability index (target safety level). The second method know as the mean-load method directly accounts for the uncertainties associated with the parameters of load, resistance, and the method of load analysis.

Other evaluation methods such as the reliability analysis provide accurate assessment of the bridge safety. The reliability-based assessment accounts for the uncertainty in the resistance and load parameters by defining the parameters in their detailed probabilistic form. This method have been used by many researchers worldwide for the evaluation of bridges and the calibration of load factors and design codes (Estes (1997); Allen (1992); Nowak (1989); Thoft-christensen (1998); Zonta et al. (2007); Onoufriou and Frangopol (2002); Nowak and Collins (2012); Czarnecki and Nowak (2008); Guan et al. (2012); Estes et al. (2003); Hosni et al. (2006); Liu et al. (2009); Zhou et al. (2013); Estes and Frangopol (1999); Schneider and Vrouwenvelder, (2017); Melchers and Beck, (2018)).

The fundamental theories of the design and evaluation codes are generally based on reliability analysis approach. The load factors used in the evaluation of bridge outlined in the CHBDC are calibrated using the reliability methods. The load rating and reliability analysis methods are associated with several strength and limitations. Estes and Frangopol (2005) studied the advantages and disadvantages of load rating and reliability analysis methods. In their findings, they concluded that the load rating approach does not account for correlation between failure modes nor it provides a measure for the failure probability of a structure. The reliability analysis overcomes both limitations involved in the load rating method. However, the reliability-based assessment of structures is associated with relatively complicated analysis and require the knowledge of the statistical parameters involved in the evaluation. The advantage of the load rating analysis is providing a practical and standardized approach in the evaluation of bridges.

Bridge evaluation based on the guidelines in the CHBDC has also been studied by Au et al. (2005) where load rating on a bridge using the two defined methods of evaluation was performed. Moreover, field data was used in determining the statistical parameters for the live load effects on the bridge and accordingly, load rating using the mean load method was completed. Their results showed a significantly higher live load capacity factor using the field data in comparison to the statistical parameters provided in the CHBDC commentary.

2 Live Load Capacity Factor (F)

The general method for calculating the live load capacity factor (F) is expressed in equation 4.1. In this equation, R_r and U represent the factored resistance and the resistance adjustment factor respectively. D, L, I, and A represent the dead, live, impact, and additional loads respectively. α_D, α_L, and α_A represent the factors for dead, live, and additional loads on the bridge respectively. The factors are defined based on the target reliability index (target safety level) of the members.

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

The dead load effects are generally determined based on the three dead load categories as per the CHBDC. Dead load category one (D1) represents factory-produced components and cast-in-place concrete, excluding decks. Dead load category two (D2) represents cast-in-place concrete decks, wood, field measured bituminous surfacing, and non-structural components. Dead load category three (D3) represents the bituminous surfacing where the nominal thickness is assumed to be 90mm for evaluation (CAN/CSA-S6 2019).

The live load effects for normal traffic in the CHBDC are determined using the idealized live load models (Cl-625 and Cl-625-ONT) shown in Figures 1.a and 1.b for the truck and lane loading respectively. The truck loading generally governs for short and medium span bridges in contrast to long span bridges where the lane loading governs. As seen in the figures, the Ontario truck model (CL-625-ONT) has a slightly modified tandem axles in comparison to the CL-625 truck model.

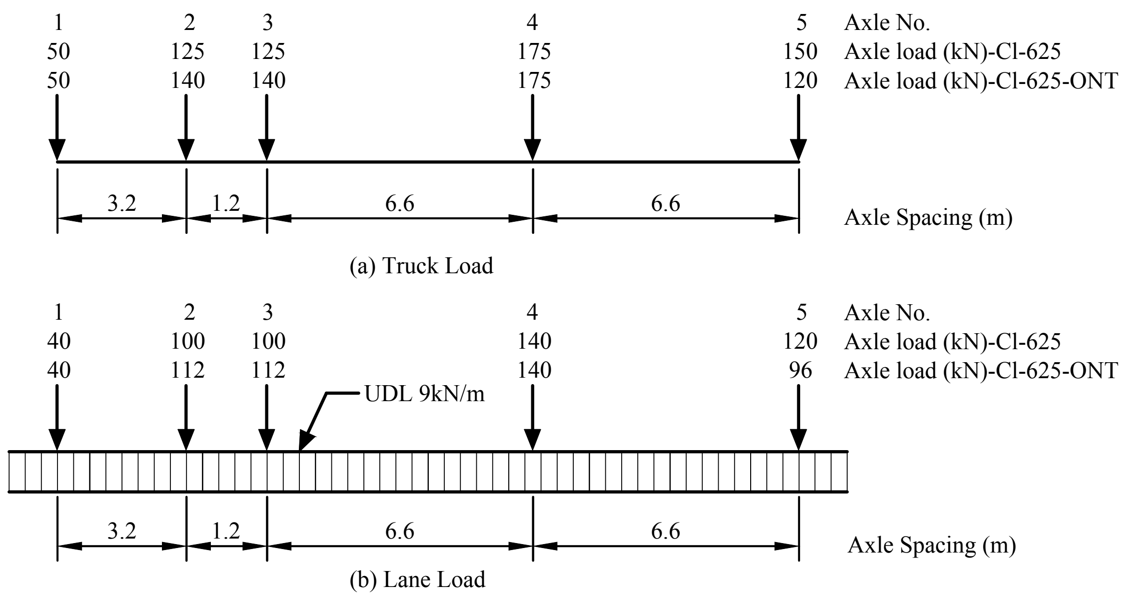


Figure 1. Live load idealized trucks in the CHBDC (CL-625 and CL-625-ONT)

An alternative approach in determining the live load capacity factor (F) is the mean-load method expressed in equation 4.2. The equation is based on the target reliability index as well as the mean values ($\overline{R}, \overline{D}, \overline{L}$), and the coefficients of variation (V_R, V_S) of the resistance and loads. For a variable x, the mean value ($\overline{x}$) is determined based on the bias coefficient (δ_x) using equation 4.3. The coefficient of variation (V_x) is defined as the standard deviation (σ_x) over the mean value as shown in equation 4.4 (Barker and Puckett 2013).

$$F = \frac{\overline{R}\exp\left[-\beta\sqrt{(V_R^2 + V_S^2)}\right] - \sum \overline{D}}{\overline{L}} \tag{2}$$

$$\delta_x = \overline{x}/x \tag{3}$$

$$V_x = \sigma_x/\,\overline{x} \tag{4}$$

In equation 4.2, $\overline{R}$ and V_R represent the resistance's mean and coefficient of variation respectively, $\overline{D}$ and $\overline{L}$ are the means of dead and live loads respectively, and V_S represents the coefficient of variation of the loads. The equations of V_S, S_D, S_L, $\overline{D}$, and $\overline{L}$ are expressed below as per the CAN/CSA-S6 (2019).

$$V_S = \frac{\sqrt{S_D^2 + S_L^2}}{(\sum \overline{D} + \overline{L})} \tag{5}$$

$$S_D = \sqrt{\sum (V_{Di}^2 + V_{ADi}^2)(\delta_{Di}\delta_{ADi}Di)^2} \tag{6}$$

$$S_L = \sqrt{[V_{AL}^2 + V_L^2 + (V_I\delta_I I)^2/(1 + \delta_I I)^2]}[\delta_L\delta_{AL}L(1 + \delta_L I)] \tag{7}$$

$$\sum \overline{D} = \sum \delta_{Di}\delta_{ADi}Di \tag{8}$$

$$\overline{L} = \delta_L\delta_{AL}L(1 + \delta_I I) \tag{9}$$

The evaluation of bridges using the general method provides a practical approach for the safety assessment procedure whereas, the mean-load method requires additional knowledge and understanding of the statistical parameters involved in the analysis. The general method utilizes load factors that are calibrated based on linear regression analysis. As a result, bridge evaluations conducted using the general method can include large errors for bridges with very high or very low dead to live load ratios. This is noted in the CHBDC commentary (CAN/CSA-S6.1 2019). The mean-load method is not subjected to the variation of the dead to live load ratio since the parameters involved in the analysis are defined to account for the uncertainty in the load, resistance, and the method of analysis regardless of the load ratios. Moreover, the mean-load method can be utilized for performing evaluation based on field data such as the local traffic data.

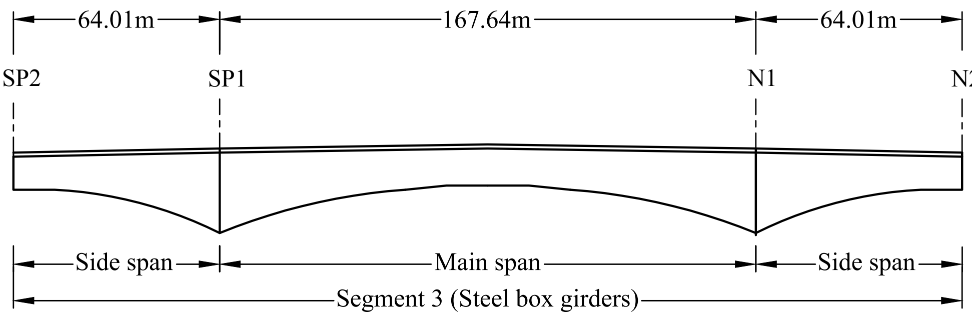
A live load capacity factor (F) of 1.0 or greater indicates that the specified loads can be resisted by the structure's members based on the defined target level of safety. In another word, if F is less than one, the evaluation does not imply failure of the member, it implies that under the specified target level of safety, the defined demand exceeds the capacity. In that case, either posting on the bridge is required to reduce the allowable gross vehicle weight or strengthening of the bridge will be essential to increase the capacity.

3 Application on Main and Side Spans of Harbour Bridge

3.1 Summary of Bridge Segment

Saint John Harbour bridge was built in 1968 consisting of four elements defined as follows: south approach, main and side spans, north approach, and ramps. The focus of this paper is on the main and side spans forming the three-span continuous twin hunched steel box girders shown in Figure 2.a. The elevation view of the bridge segment in Figure 2.b shows the span lengths and the pier numbering.

(a)



(b)

Figure 2. Main and side spans of Harbour Bridge. (a) elevation view (b) dimensions

The cross-section of the main and side spans is shown in Figure 3 comprising a total bridge deck width of approximately 19.81 meters. The length of both side spans is approximately 64.01 meters consisting of a 0.254 meters concrete deck placed over two stringers that are supported by transverse floor beams. The floor beams are bolted to the girders and are spaced approximately 4.58 meters apart. The 6.71 meters box girders depth at the interior supports SP1 and N1 varies along the span length where the depth becomes approximately 2.74 meters at the exterior supports SP2 and N2.

The total length of the main span is 167.64 meters where the orthotropic deck acts as a top flange for the box girders and the transverse floor beams. The orthotropic deck consists of longitudinal U-shaped stiffened ribs that support the top flange steel plate. The box girder depth varies along the span length where the depth ranges from 6.71 meters at the interior piers SP1 and N1 to approximately 3.05 meters at midspan. Transverse floor beams are welded to the girders with a spacing of 4.66 meters. The floor beams are cantilevered on both sides of the box girder to support the orthotropic deck as shown in Figure 3.

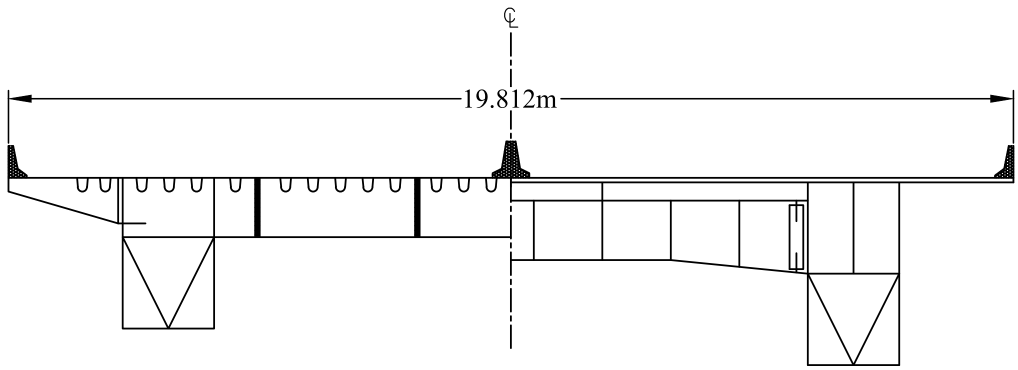


Figure 3. Superstructure cross-section for the main and side spans

3.2 Target Safety Level, Load and Resistance Factors and Coefficients

The required safety level for a member is measured using the target reliability index which is dependent on three factors. First, the system behaviour which is defined for whether the member failure will lead to a total collapse of the structure or it will not. Second, the element behaviour corresponds to whether the failure of the member is sudden, ductile, or the member has post-failure capacity. Third, the inspection level represents how well the condition of the member is known. The inspection level and the system behaviour of the box girders are consistent however the element behaviour of the members vary. Consequently, the target reliability index varies for the box girders in the superstructure as shown in Table 1.

Table 1. Target reliability index for elements in the superstructure

Girder	System Behaviour	Element Behaviour	Inspection Level	Target Reliability Index
Box Girder in Bending - Steel plate in Compression	S2	E2	INSP3	3.00
Box Girder in Bending - Steel plate in Tension	S2	E3	INSP3	2.75
Box Girder in Shear/Torsion	S2	E3	INSP3	2.75

Once the target reliability index is defined, the load factors used in the general method can be determined. Tables 2 and 3 summarize the load factors and the resistance adjustment factors respectively. The general method is based on load factors calibrated based on specified target reliability indices whereas, the mean-load method is based on the bias factors and the coefficients of variation of load and resistance. Table 4 summarizes these coefficients used in determining the live load capacity factor based on the mean load method.

Table 2. Load factors based on the target reliability index (CAN/CSA-S6 2019)

Member Type	Target Reliability Index	Load Factors			
		D1	D2	D3	LL
Box Girder in Bending- Steel plate in Compression	3.00	1.07	1.14	1.35	1.49
Box Girder in Bending- Steel plate in Tension	2.75	1.06	1.12	1.3	1.42
Box Girder in Shear/Torsion	2.75	1.06	1.12	1.3	1.42
Floor Beam / Stringer in Bending or Shear	2.50	1.05	1.10	1.25	1.35

Table 3. Resistance adjustment factors (CAN/CSA-S6 2019)

Resistance category	Resistance adjustment factor U
Steel Box Girder Plastic Moment	1.00
Steel Box Girder Yield Moment	1.06
Steel Box Girder Shear and Torsional shear	1.03

Table 4. Load and resistance bias factors and coefficients of variations based on the CHBDC commentary (CAN/CSA-S6.1 2019)

Parameter	Load				Resistance	
	D1	D2	D3	Cl-625	Yield moment	Shear (tension field)
δ_x	1.03	1.05	1.03	1.35	1.22	1.18
V_x	0.08	0.10	0.30	0.035	0.10	0.10

3.3 Load Rating Based on CHBDC CL-625 Truck and Local Traffic

The three-span continuous bridge segment was analyzed for the hogging and sagging moments failure modes, and for shear failure mode. The dead load effects were determined based on the three outlined dead load categories (D1, D2, and D3) defined in the CHBDC. The live load effects were determined for the idealized truck model in the CHBDC shown in Figure 1. Furthermore, simulation of current traffic data in the province of New Brunswick was applied for calibrating the statistical coefficients used in the mean-load method. The calibration results are summarized in Table 5 (Salah Eddine 2019). The shear and moment resistance of the box girders and the floor beams were determined using the CHBDC. Note that the moment resistance of the box girder and the floor beams were determined using an effective width of the orthotropic deck based on AISC (1963) and FHWA (2012).

Table 5. Bias factors for single and two trucks

	Shear	**Positive Moment**	**Negative Moment**
Bias factor (single truck) – 75 years time period	0.502	0.682	0.513
Bias factor (two trucks) 1month time period	1.00	1.357	1.00

Once the load and resistance parameters are defined, the live load capacity factor is calculated. Figures 4-6 represent the box girder live load capacity factor for the hogging moment, sagging moment, and shear and torsional shear failure modes respectively. These figures show the evaluation results from the general method based on the Cl-625 truck model, the mean-load method based on the Cl-625 truck model, and the mean-load method using the traffic data. As seen in Figure 4, the hogging moment failure mode of box girders is adequate for the Cl-625 loading based on the general method, and the local traffic data based on the mean load method. However, the evaluation based on mean-load method using the Cl-625 loadings shows that the hogging moment falls below the required safety level at approximately 8.4 meters from the interior supports SP1 and N1 towards midspan. For the sagging moment, and the shear and torsional shear in Figures 5 and 6, the box girders are adequate for the loading specified by Cl-625 truck and the local traffic data for both methods of analysis. Note that the highest live load capacity factor was limited to a value of 10.

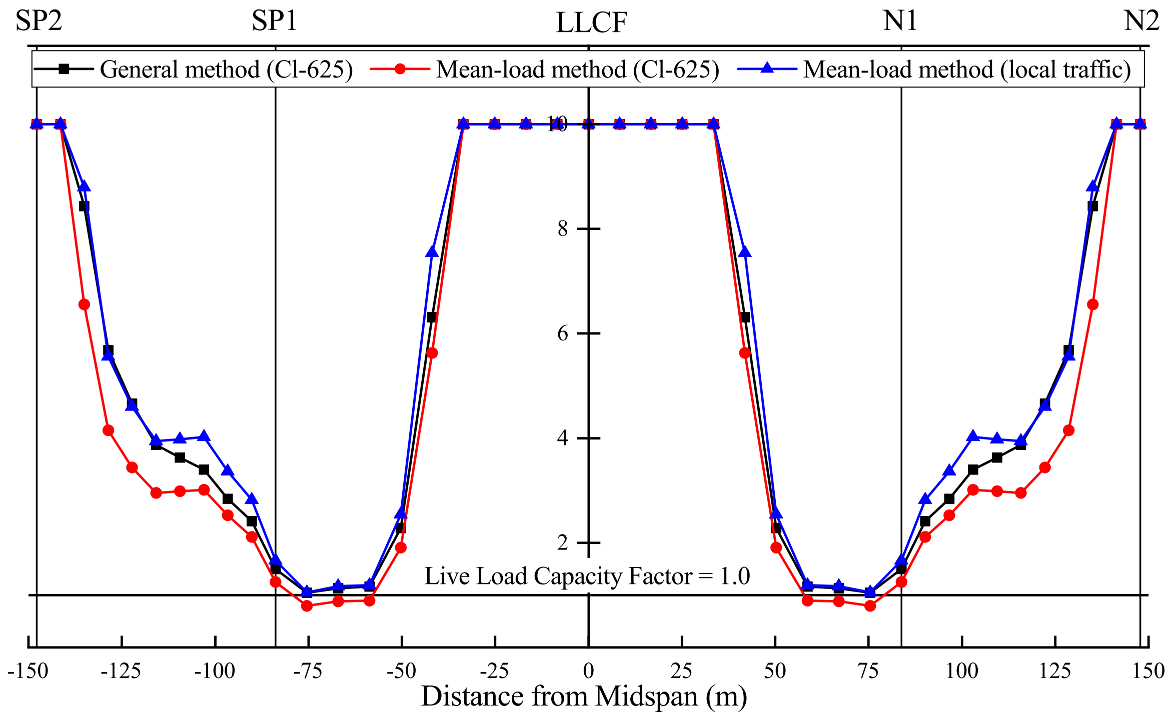


Figure 4. Box girder live load capacity factor for the hogging moment failure mode

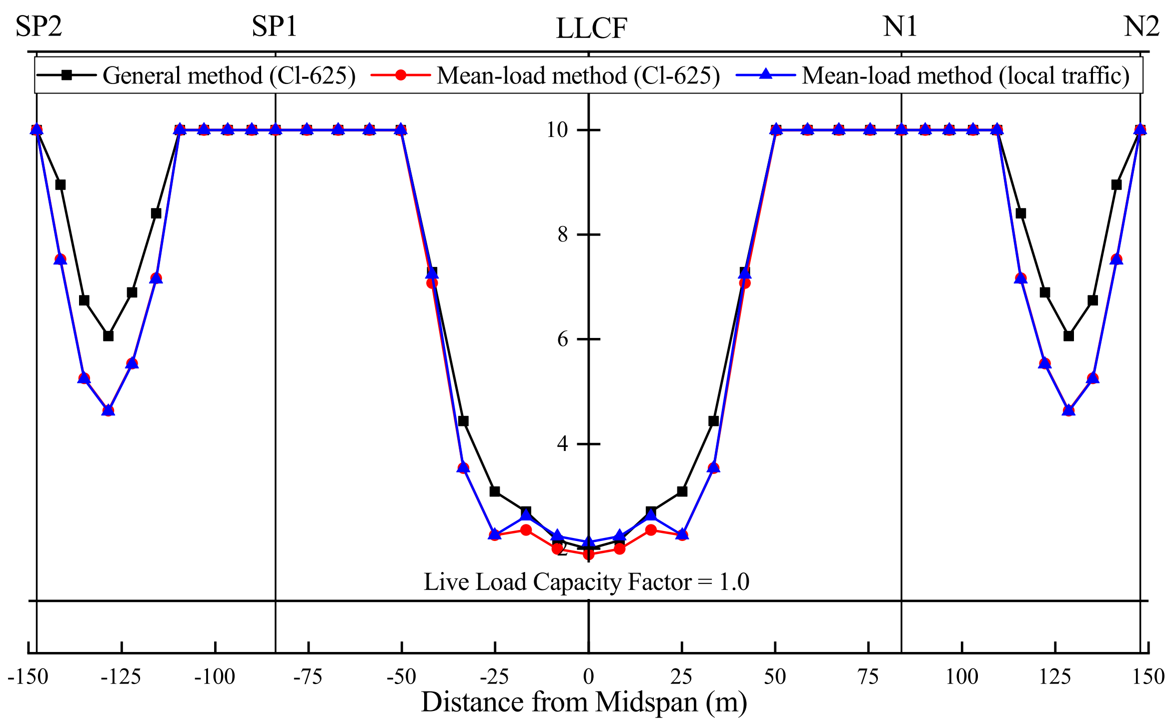


Figure 5. Box girder live load capacity factor for the sagging moment failure mode

(b)

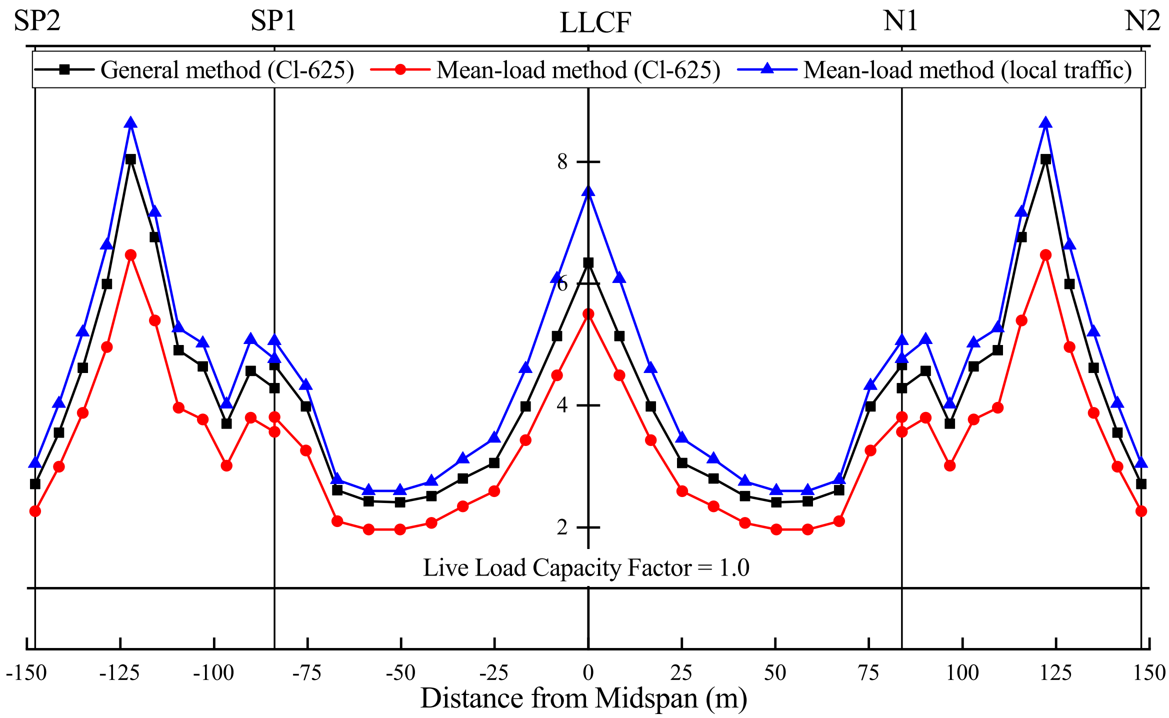


Figure 6. Box girder live load capacity factor for the shear and torsional shear failure mode

The mean-load method is expected to be the more accurate approach in determining the live load capacity factor in comparison to the general method since the factors are calibrated to be more conservative. As seen the figures, the mean-load method estimates lower live load capacity factor in comparison to the general method. This inconsistency in the results is due to the inaccuracy resulted from the calibration of the load factors used in the general method. In Figure 7, an example showing the inconsistency between the results of the live load capacity factor calculated using the mean-load method and the general method. As seen in this figure, the general method does not always result in a conservative estimate of the live load capacity factor. This topic requires further research for the calibration of load factors. At the current state of the CHBDC, for high and low dead to live load ratios, it is recommended to use the mean-load method as it is expected to be the more accurate approach in determining the live load capacity factor. It is noted that since lane loading governs for long span bridges, no dynamic load allowance is included in the evaluation.

General Method

Resistance	R =	100	
Yield Moment	φ =	0.95	
	U =	1.06	

		Ratio		Load
D_1	$\Psi_1 = D_1/R =$	0.4	→	40
D_2	$\Psi_2 = D_2/R =$	0.1	→	10
D_3	$\Psi_3 = D_3/R =$	0.1	→	10
LL	$\Psi_{LL} = LL/R =$	0.2	→	20

β_T	α_{D1}	α_{D2}	α_{D3}	α_{LL}
3	1.07	1.14	1.35	1.49

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

F = **1.11**

Mean-Load Method

	Bias Factors and Coefficients of Variation			Mean and standard deviation	
	δ	V		$\bar{D}, \bar{L}, \bar{R}$	S_D or S_L
D_1	1.03	0.08	→	41.2	3.30
D_2	1.05	0.1	→	10.5	1.05
D_3	1.03	0.3	→	10.3	3.09
LL	1.35	0.035	→	26.46	2.07
M_R (yield)	1.22	0.1	→	122	
				β_T	3

$$F = \frac{\bar{R}\exp[-\beta(V_R^2 + V_S^2] - \sum \bar{D}}{\bar{L}}$$

F = **0.92**

Method of load Distribution for dead and Live Loads		
	δ_{AD}	V_{AD}
D_1	1	0
D_2	1	0
D_3	1	0
	δ_{AL}	V_{AL}
sophisticated	0.98	0.07

Equations for calculating the mean and standard deviation of load

$$\sum \bar{D} = \sum \delta_D \delta_{AD} D \qquad S_D = \left[\sum [(V_D^2 + V_{AD}^2)(\delta_D \delta_{AD} D)^2]\right]^{0.5}$$

$$\bar{L} = \delta_L \delta_{AL} L \qquad S_L = [V_{AL}^2 + V_L^2]^{0.5}[\delta_L \delta_{AL} L]$$

$$V_S = \frac{(S_D^2 + S_L^2)^{0.5}}{(\sum \bar{D} + \bar{L})}$$

Figure 7. Example illustrating the inconsistency between the mean-load method and the general method

4 Conclusion

In this paper, the two methods of evaluation outlined in the Canadian Highway Bridge Design Code (CHBDC) are discussed and their application is illustrated on a case study bridge segment. The results show inconsistency between the general method of evaluation that utilizes load factors in comparison to the mean-load method that applies the statistical coefficients of load, resistance and method of analysis. The mean-load method was utilized for evaluating the bridge segment using local traffic data collected from five sites across the province. The findings indicate higher load carrying capacity for the bridge segment based on the load effects of the local traffic data.

Notations

A	additional loads
Di	dead load category i
$\overline{D}$	mean of dead load
F	live load capacity factor
I	impact factor
L	live load
$\overline{L}$	mean of live load
R_r	resistance
$\overline{R}$	mean of resistance
S_D	standard deviation of dead load
S_L	standard deviation of live load
U	resistance adjustment factor
V_{AD}	coefficient of variation of the method of dead load distribution
V_{AL}	coefficient of variation of the method of live load distribution
V_{Di}	coefficient of variation of dead load category i
V_I	coefficient of variation of impact factor
V_L	coefficient of variation of live load
V_R	coefficient of variation of resistance
V_S	coefficient of variation of load
V_x	coefficient of variation of random variable x
x	random variable
$\bar{x}$	mean of random variable x
α_A	additional load factor
α_D	dead load factor
α_L	live load factor
β	target reliability Index
σ_x	standard deviations of random variable x
δ_{ADi}	bias factor of the method of dead load distribution of category i

δ_{AL}	bias factor of the method of live load distribution
δ_{Di}	bias factor of dead load category i
δ_I	bias factor of impact factor
δ_L	bias factor of live load
δ_x	bias factor of random variable x

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