Bridge evaluation by mean load method per the Canadian Highway Bridge Design Code

Alexander Au, Clifford Lam, Akhilesh C. Agarwal, and Bala Tharmabala

Abstract: The Canadian Highway Bridge Design Code (CHBDC) provides two alternative methods for evaluating the strength of existing bridges. The load and resistance factor method provides a general approach and covers the most extreme load situations that can occur in a general bridge population. The mean load method considers the uncertainties of loads acting on a specific bridge, the method of analysis, and resistance of the structure involved, and thus can provide a more accurate evaluation of individual bridges. Since traffic load represents a major portion of bridge loads, a better evaluation of specific bridges is obtained by using the statistical parameters of traffic loads observed on the structure. However, the overall accuracy depends heavily on capturing the most critical loading conditions during the survey periods. The mean load method is particularly valuable where actual traffic loads are expected to be significantly lower than those used in code calibration and when the potential economic benefits arising from a more realistic evaluation outweigh the extra costs of live load data collection and analysis. This paper demonstrates that the mean load method using site-specific traffic loading information can lead to a significantly higher live load-carrying capacity of a bridge.

Key words: highway bridges, bridge evaluation, reliability, mean load method, bridge testing.

Résumé : Le Code canadien sur le calcul des ponts routiers fournit un choix de deux méthodes pour évaluer la résistance des ponts existants. La méthode du facteur de charge et du coefficient de résistance fournit une approche générale et couvre les situations de charge les plus extrêmes qui peuvent survenir dans la population des ponts en général. La méthode de la charge moyenne tient compte des incertitudes des charges agissant sur un pont spécifique, la méthode d'analyse et la résistance de la structure impliquée, et peut ainsi fournir une évaluation plus précise des ponts individuels. Puisque la charge de roulage représente une portion importante des charges d'un pont, une meilleure évaluation des ponts spécifiques est obtenue en utilisant les paramètres statistiques des charges de roulage observées sur la structure. Cependant, l'exactitude globale dépend grandement sur la saisie des conditions de charge les plus critiques durant les périodes de relevés. La méthode de la charge moyenne est particulièrement valable lorsque les charges de roulage devraient être significativement plus faibles que celles utilisées pour l'étalonnage selon le code et lorsque les avantages économiques potentiels découlant d'une évaluation plus réaliste l'emportent sur les coûts supplémentaires de la collecte et de l'analyse des données de surcharges. Le présent article démontre que la méthode de la charge moyenne utilisant de l'information sur les charges de roulage spécifique à un site peut conduire à une capacité portante des surcharges beaucoup plus élevée d'un pont.

Mots clés : ponts routiers, évaluation des ponts, fiabilité, méthode de la charge moyenne, mise à l'épreuve des ponts.

[Traduit par la Rédaction]

Introduction

The new Canadian Highway Bridge Design Code (CHBDC) (CSA 2000), provides two alternative methods for evaluating the live load capacity of existing bridge structures, which are both based on the probabilistic limit states design (LSD) approach. The general method, referred to as 'load and resistance factor method', uses partial loads and resistance factors, where the factored resistance of a bridge component should exceed the combined factored load effects on the component. The CHBDC specifies a standard set of resistance factors but allows load factors to be varied for evaluation of existing bridges to meet a target reliability level. This target reliability level depends upon the behavior of the specific component as well as the overall bridge and the level of inspection carried out for the evaluation (CSA 2000). These load factors have been developed through a reliability analysis to achieve the target reliability level, represented by a reliability index, under extreme scenarios of normal traffic loads in Canada (Gagnon and Agarwal 2002).

The alternative method, referred to as 'mean load method', does not require the use of load or resistance factors. Instead, the uncertainties associated with the loads and resistances are considered by using the following statistical parameters:

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(*i*) bias coefficient and (*ii*) coefficient of variation (CoV) to describe their variability. The live load capacity factor is determined from a formula that includes the statistical parameters of all the loads and resistance and the target reliability level.

The CHBDC commentary (CSA 2000) provides default statistical parameters that may be used in the mean load method for most bridges. These parameters have been developed to cover the extreme load occurrences for a general bridge population in Canada, and hence may provide an overly conservative load capacity evaluation for some bridges. To address these situations, the CHBDC allows the use of statistical parameters that are derived from field data on loads collected at the specific bridge sites. The use of site-specific data may be particularly beneficial if the uncertainty of a load, such as live load, is expected to be significantly lower than that assumed for the code provisions. However, collection of site-specific field data and its analysis are expensive undertakings and are recommended only in special bridge evaluation projects, where the code parameters are deemed to be too conservative and the economic benefits of a more accurate evaluation are substantial enough to warrant the expenses of field data collection and analysis. The season and periods of measurements need to be selected carefully to capture the most critical loading on a bridge, e.g., for routes commonly used by the forest product industry to transport logs in Northern Ontario, regulatory loads are increased during winter freeze-up season because of greater pavement strength, and the winter season would thus be more critical for traffic loads on bridges.

To get an assessment of the potential benefits of using the mean load method, a bridge was evaluated using (*i*) the statistical data provided in the CHBDC Commentary and (*ii*) statistical parameters on live loads obtained from field measurements at the bridge (Au and Lam 2002). For comparison, the bridge was also evaluated by the load and resistance factor method in accordance with the CHBDC provisions.

Bridge description

The bridge under consideration, shown in Fig. 1, is the eastern half of the twin structure that carries the Queen Elizabeth Way (QEW) over Ford Drive in Oakville, Ontario. It carries three eastbound lanes of traffic into Toronto. Constructed in 1978, it is a simply supported structure with a span length of 39.62 m and has a small skew of 2°25′. As shown in Fig. 2, it consists of three steel-box girders, 1372 mm deep and spaced 5639 mm apart, with a 15.24 m wide and 190 mm thick composite reinforced concrete deck, topped by a 75 mm thick asphalt wearing surface.

Section properties

Figure 3 shows the geometric details of a typical section of the box girder. The specified material properties are (*i*) concrete strength = 27.6 MPa, (*ii*) structural steel yield strength = 350 MPa, and (*iii*) steel reinforcement yield strength = 400 MPa. A modular ratio (E_s/E_c) of 8.0 is assumed.

The composite section properties calculated using provisions of the CHBDC are as follows (in steel units): (*i*) area of composite section = $2.30\text{x}10^5$ mm^2, (*ii*) moment of inertia of composite section = $8.80\text{x}10^{10}$ mm^4, and (*iii*) torsional constant of composite section = $8.40\text{x}10^{10}$ mm^4.

Moment of resistance of composite box-girder section

The load and resistance factor method requires the calculation of the factored moment of resistance of the box-girder section, which is computed using the material resistance factors prescribed in the CHBDC. The mean load method, however, requires the use of the nominal (unfactored) resistance, which is obtained by setting all the material resistance factors to 1.0. Based on the above, the nominal and factored moments of resistance of the composite box-girder section are estimated at 32 120 and 29 980 kN·m, respectively.

Nominal load effects from the CHBDC

Dead load effects

The maximum unfactored dead load moments for an individual girder are as follows:

Steel girder, $D1$ = 1420 kN·m

Concrete slab, barrier and railing, $D2$ = 8490 kN·m

Asphalt wearing surface, $D3$ = 2340 kN·m

where $D1$, $D2$, and $D3$ are the types of dead loads in the structure.

Live load effects

In Ontario, the CHBDC requires the use of CL1-625-ONT, CL2-625-ONT, and CL3-625-ONT truck loadings for the three levels of evaluation. Evaluation level 1 provides the load carrying capacity for vehicle trains, evaluation level 2 for truck combinations with one trailer or semi-trailer, and evaluation level 3 for single unit vehicles. For this bridge, the CL1-625-ONT truck load governs for evaluation level 1. Details of the CL1-625-ONT truck are shown in Fig. 4. The CL2-625-ONT and CL3-625-ONT trucks are sub-configurations, obtained by deleting the last one and two axles from CL1-625-ONT, respectively.

The semi-continuum method (Mufti et al. 1992) was used to analyse the live load effects. Three design trucks were moved simultaneously along longitudinal lines (one truck in each traffic lane) to maximize the live load effect in the middle girder. The maximum unfactored bending moment, M_{625} (or nominal live load effect, L) in girder 2 is found to be 4470 kN·m, which includes the modification factor for multi-lane loading but not the dynamic load allowance (DLA).

Field measurements of live load effects

To obtain the statistics of real live load effects in critical components of the bridge, the latter was converted into a weigh-in-motion scale by instrumenting one of the interior box girders (girder 2). The instrumented bridge was calibrated by running the bridge-testing vehicle of the Ministry of Transportation of Ontario (MTO) with known axle configuration and axle loads. The calibrated system was then used to obtain the live load effects on the bridge due to normal truck traffic.

Bridge instrumentation

Figure 5 shows the locations on the box girder that were instrumented for the live load survey. Electrical strain gauges

Fig. 1. Eastbound QEW at Ford Drive overpass.

Fig. 2. Typical section of the bridge under consideration. All dimensions in millimetres.

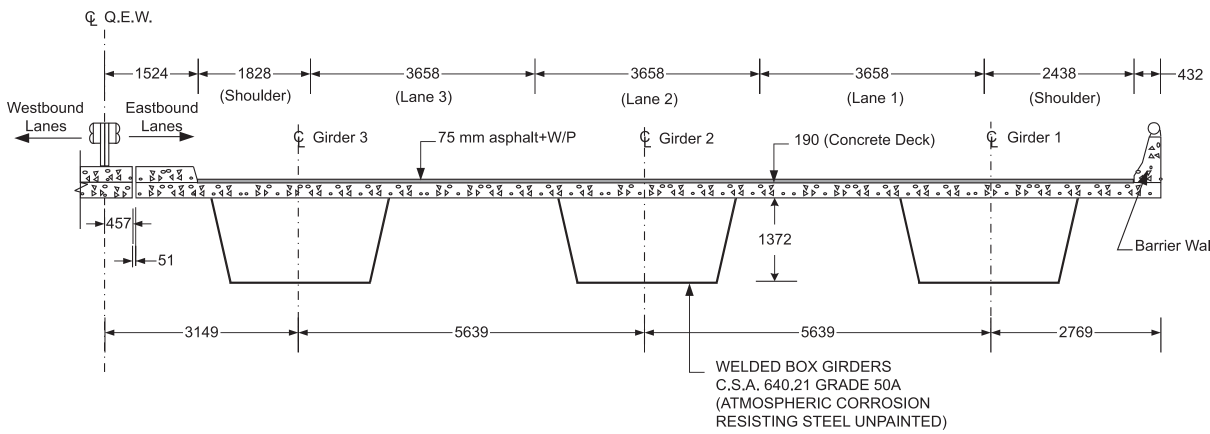


Fig. 3. Typical section of box girder at midspan. All dimensions in millimetres.

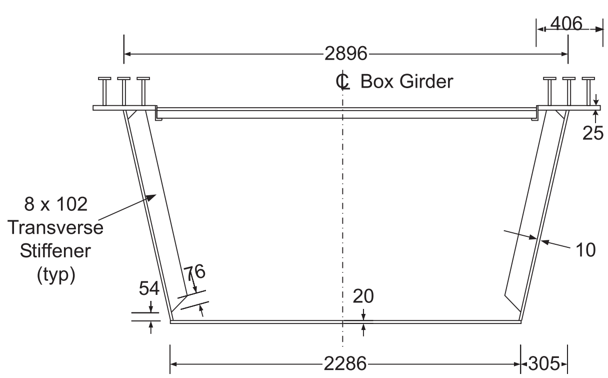
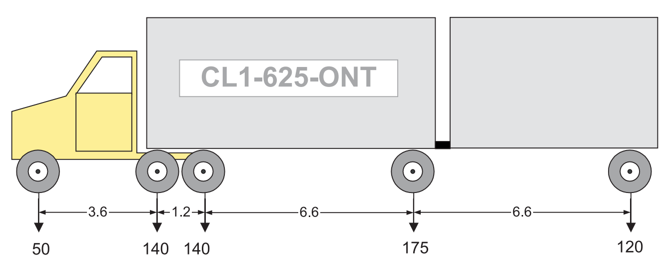


Fig. 4. CL1-625-ONT truck. All dimensions in millimetres. All Axle loads in kilonewton.



were installed inside girder 2 at the two locations shown. At each location, two strain gauges were installed, one on the web and the other on top of the bottom flange (Fig. 6).

Data acquisition

The strain gauge data generated by the highway traffic load are recorded automatically by a high-speed data acquisition and control system (Daytronic System 10). The system has a built-in logic, which analyses the strain data recorded over a user-defined time period corresponding to the passage of a vehicle on the bridge and stores the maximum and minimum strain observed for each individual truck passage. The system can be programmed to process only those truck events that cause strains exceeding a user-specified threshold strain value, thereby precluding cars and other lightweight vehicles from being included in the survey. However, the recorded strain can be due to the effect of multiple vehicles present on the structure at the time the re-

Fig. 5. Bridge instrumentation plan. All dimensions in millimetres.

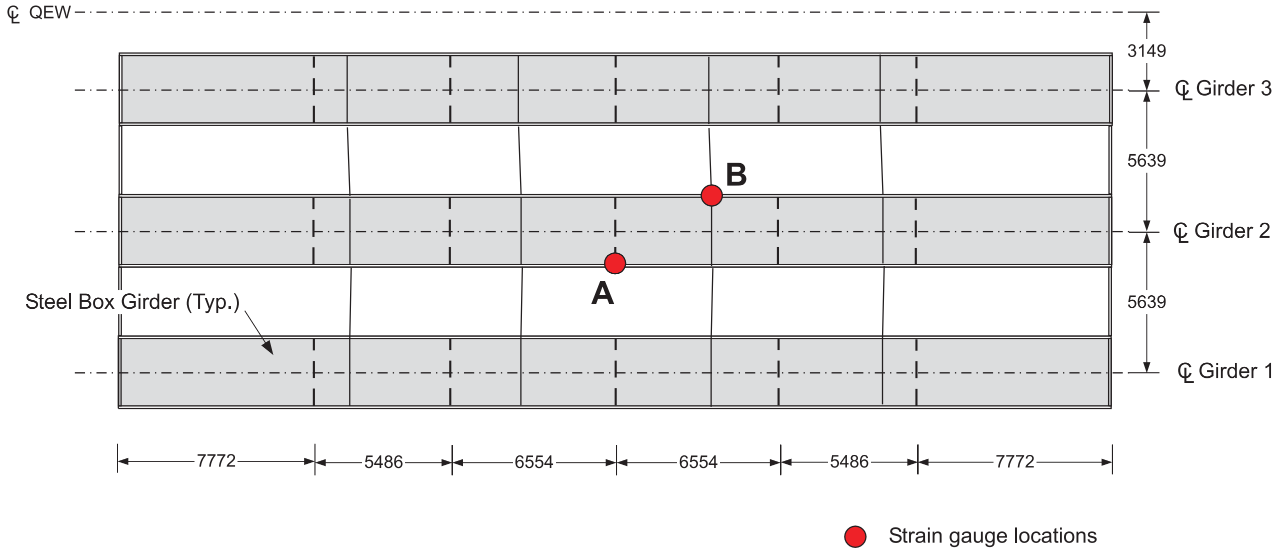


Fig. 6. Strain gauge locations. All dimensions in millimetres.

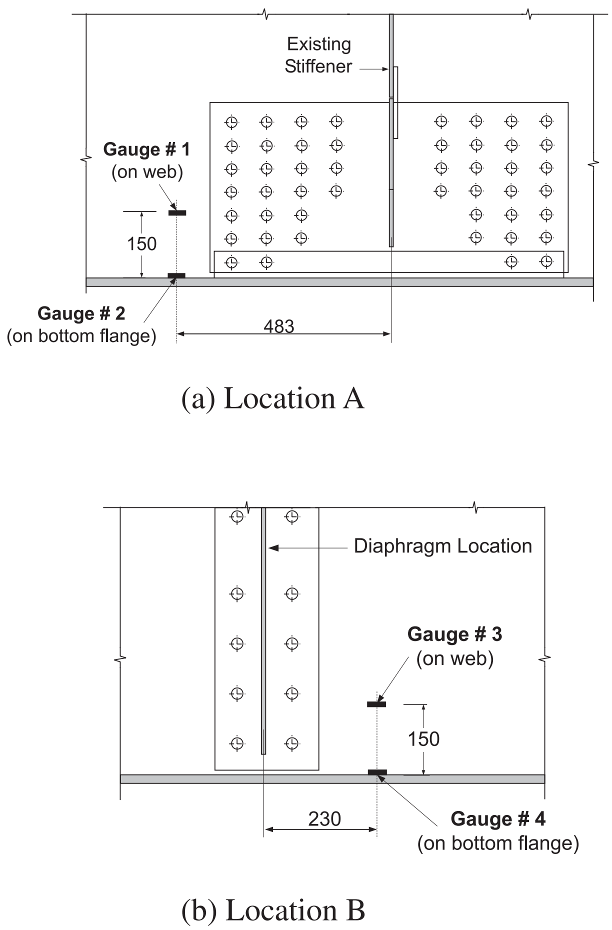
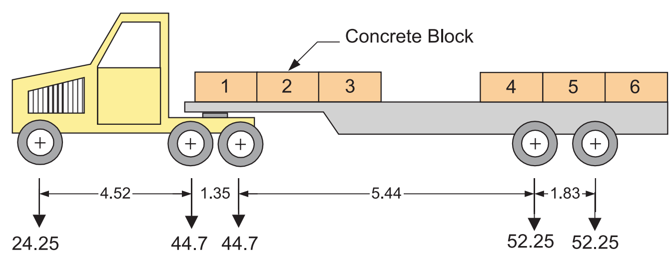


Fig. 7. The test vehicle of Ministry of Transportation of Ontario. All dimensions in millimetres. All wheel loads in kilonewton.



cording was in progress. The recorded peak responses are downloaded at appropriate intervals to provide the raw data for statistical analysis.

The load survey was conducted over a period of 5 d during a typical week in November 2000 and the peak responses from a total of 5102 truck events were recorded.

Calibration of field measurement system

An MTO bridge testing truck, loaded with 24 concrete blocks weighing approximately 1 t each, was used to calibrate the relationship between the theoretical live load effect and the corresponding measured live load response of the instrumented box girder. As shown in Fig. 7, the test truck is a five-axle tractor-semitrailer combination with a gross weight of 436.3 kN. The maximum theoretical bending moment due to the test truck, M_T, at the instrumented location A is calculated as 1680 kN·m (DLA excluded).

Since it was not possible to close the bridge for this calibration exercise, the test truck was run at normal highway speed during the time windows when no other vehicle was on the bridge. To evaluate the repeatability of the test results, seven separate test runs were made with the test truck in the middle traffic lane and the speed of the test truck was recorded for each run. Table 1 shows the measured strains at the four strain gauge locations and the speed of test truck for the seven runs.

The average strains from strain gauges 1 and 2 at location A were used in the calibration process. Since the measured strain data include dynamic load effects, the dynamic load components were first estimated and subtracted from the measured strains to obtain the corresponding static crawl responses. The mean value of the average static strain is calculated as 71.4 με with a coefficient of variation of 0.081. The coefficient of variation reflects variations in measured

Table 1. Strain data in girder 2 for the Ministry of Transportation of Ontario test truck runs.

		Strain gauge 1 (με)			Strain gauge 2 (με)			Dynamic amplification
Test run	Speed (km/h)	Maximum	Minimum	Static	Maximum	Minimum	Static	Average
1	90	96	–13	74	91	–10	71	1.29
2	80	76	–8	63	72	–5	60	1.20
3	100	91	–16	70	89	–12	69	1.30
4	95	95	–17	67	97	–8	70	1.40
5	85	92	–12	75	87	–12	72	1.22
6	85	94	–10	77	87	–7	72	1.22
7	85	109	–8	83	101	–5	77	1.31

static strain due to the randomness in transverse location of the test truck in the lane and the overall accuracy of the measurements. This variation should be accounted for when evaluating the measured strain data from the normal traffic.

Based on the nominal live load effect, M_{625}, of 4470 kN·m, the equivalent static nominal strain, ε_{625}, due to the CL1-625-ONT truck in girder 2 is pro-rated to be 190 με.

Extreme annual live load statistics from field measurements

The extreme annual live load parameters were determined from the field measurements by using a modified procedure based on the Gumbel distribution for extreme loads (Gumbel 1954; Castillo 1988; Agarwal et al. 1997). The same procedure was also used for determining the extreme live load effects from truck survey data during the calibration of the CHBDC (CSA 2002).

The frequency distribution for the measured peak live load effects under normal traffic in girder 2 is given in Fig. 8. These measured strains include the effect of dynamic loads. These maximum strains obtained from a total of 5102 truck events (sample size *n*) were recorded over a period of 5 d. This gives a projected annual population, *N*, of 372 700.

The Ontario truck survey data used in the calibration of the CHBDC (CSA 2002) had a total truck population of 8834, which was used to generate the statistical parameters representing the whole of Ontario. The smaller data size of 5102 events obtained in this study should be a reasonable statistical representation for a local site such as the QEW – Ford Drive overpass. It was also found in Au and Lam (2002) that the size of the sample truck population did not have a major impact on the final evaluation results for this structure e.g., statistical analysis based on single day truck data gave similar evaluation results as live load data samples collected over multiple days.

Analysis of the recorded 5102 truck events gives a mean strain (including dynamic effects) of 73.9 με and a coefficient of variation of 0.250. If the variability observed during the calibration process (coefficient of variation of 0.081) is factored in, the combined coefficient of variation for the sample population is calculated to be 0.263.

The Gumbel distribution is applicable for predicting an extreme distribution from a sample that has a normal distribution (Gumbel 1954). As the distribution of the strain measurements from the survey data is generally not normal, attention is focused on the tail end of the field distribution, which is the region of primary importance in the analysis and is assumed to be part of a certain normal distribution. In the application of this modified procedure based on Gumbel distribution, an intermediate step is therefore needed to determine this theoretical normal distribution, referred to as the representative normal distribution (RND) (Agarwal et al. 1997; CSA 2002). Figure 9 shows the calculated RND from the field measurement distribution and the Gumbel distributions for extreme live load effects for the sample size *n* as well as the projected annual population *N* on the bridge. The statistical parameters for the live load effects calculated from the field survey data are summarized in Table 2.

Target reliability index

The target reliability index, β, represents the required level of safety that must be provided by the structural capacity of bridge components and depends upon the life, safety, and economic consequences of failure of a bridge component (CSA 1981; Allen 1992). For evaluation of existing bridges, the CHBDC allows the target reliability index to be varied depending on overall bridge system behavior, element behavior, and level of inspection. According to the definitions given in CHBDC, system behavior is S2, element behavior is E3, and inspection level is INSP2 for the QEW – Ford Drive overpass. Based on these parameters, the CHBDC gives an annual target reliability index, β_1, of 3.00.

Evaluation by load and resistance factor method

The live load capacity factor, *F*, at the ultimate limit state is given by

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

where *U* is the resistance adjustment factor, R_r is the factored resistance of the component, *D* is the unfactored dead load effect, α_D is the dead load factor, *A* is the unfactored force effect due to other additional loads, α_A is the load factor for force effects due to additional loads, *L* is the unfactored static live load for the evaluation level under consideration, α_L is the live load factor, and *I* is the dynamic load allowance for live load.

According to the CHBDC, for β_1 = 3.00, the load factors are as follows: dead load factors, α_{D1} = 1.07, α_{D2} = 1.14, α_{D3} = 1.35; and live load factor, α_L = 1.49.

Since the critical load effects occur under full truck loads, the dynamic load allowance (*I*) per CHBDC is 0.25.

Fig. 8. Frequency distribution of live load effects at QEW – Ford Drive overpass (5102 readings).

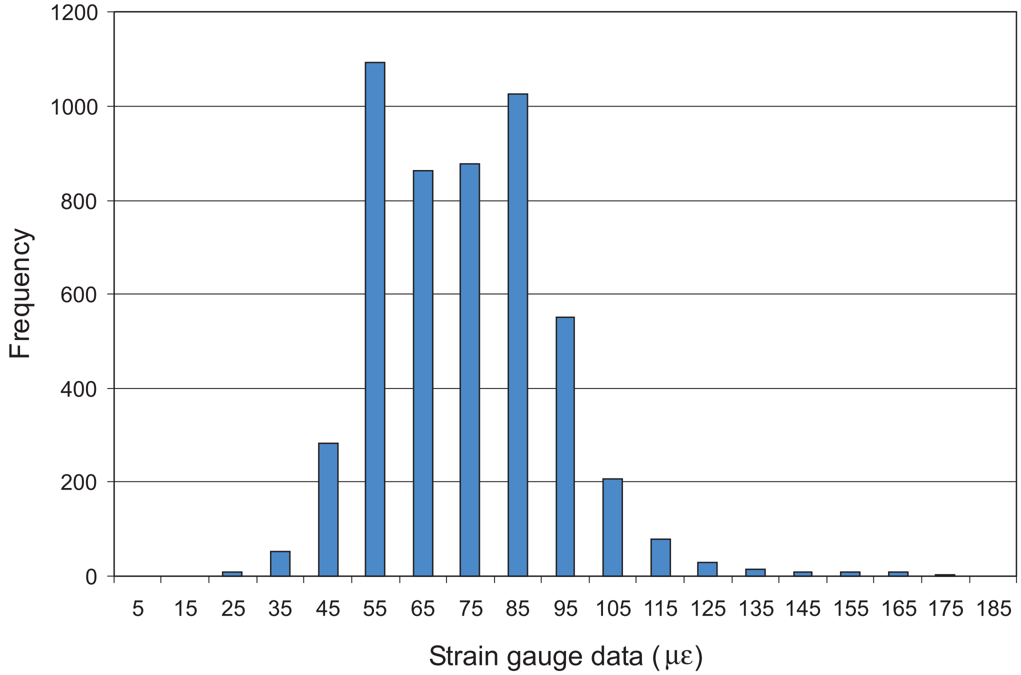


Fig. 9. Analysis of field data by Gumbel distribution at QEW – Ford Drive overpass.

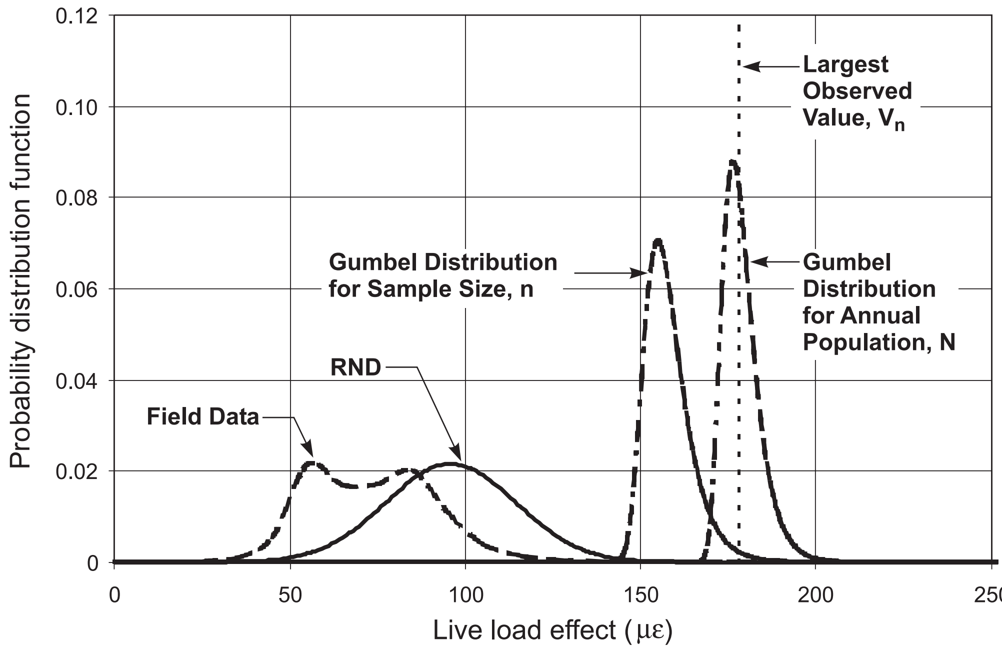


Table 2. Statistical live load parameters calculated from field data.

Statistical live load parameters for annual population (N)	Value
Mean of Gumbel distribution (μ_N)	178.8 με
Standard deviation of Gumbel distribution (S_N)	5.1 με
Bias coefficient [δ_L (1 + δ_I I)]	0.94
Coefficient of variation [V_L^2 + $(V_I\ \delta_I\ I)^2$ / (1 + δ_I $I)^2$]$^{1/2}$	0.028
Number of trucks in annual population (N)	372 700

The resistance adjustment factor (U) for moments in the composite concrete slab on steel girder is 0.96.

For evaluation of the QEW – Ford Drive overpass at the ultimate limit state, only dead loads and live load with dynamic load allowance were considered. The data for unfactored dead and live loads and factored resistance for the girder given earlier was substituted in eq. [1] to obtain a value of 1.73 for the live load capacity factor for the evaluation level 1, $F1$. Since the live load capacity factor for the evaluation level 1 is greater than 1.00, the evaluation for other evaluation levels was not carried out because the live load capacity factor for those evaluation levels would also be greater than 1.00.

Evaluation by mean load method using statistical data from CHBDC commentary

As an alternative to eq. [1], the live load capacity factor, F, at the ultimate limit state can be computed using the mean load method and is given by

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

where $\bar{R}$ is the mean resistance, V_R is the coefficient of variation of resistance, V_S is the coefficient of variation of total load effect including DLA, $\bar{D}$ is the mean dead load effect, and $\bar{L}$ is the mean real live load effect including dynamic amplification.

Statistical parameters for resistance

From the CHBDC, the bias coefficient for the resistance of the composite section, δ_R, is 1.10 and the coefficient of variation, V_R, is 0.10. Therefore, the mean resistance of the composite section, $\bar{R}$, is 35 330 kN·m.

Statistical parameters for load effects

Dead load effects

The total mean dead load effect, $\sum\bar{D}$, and the standard deviation for dead load effect, S_D, are given by

$$[3] \qquad \sum\bar{D} = \delta_{D1}\delta_{AD1}D1 + \delta_{D2}\delta_{AD2}D2 + \delta_{D3}\delta_{AD3}D3$$

where δ_{D1}, δ_{D2}, and δ_{D3} are the bias coefficients for dead load types $D1$, $D2$, and $D3$, respectively; and δ_{AD1}, δ_{AD2}, and δ_{AD3} are the bias coefficients for dead load analysis method for $D1$, $D2$, and $D3$, respectively.

$$[4] \qquad S_D^2 = (V_{D1}^2 + V_{AD1}^2)(\delta_{D1}\delta_{AD1}D1)^2 + (V_{D2}^2 + V_{AD2}^2)(\delta_{D2}\delta_{AD2}D2)^2 + (V_{D3}^2 + V_{AD3}^2)(\delta_{D3}\delta_{AD3}D3)^2$$

where V_{D1}, V_{D2}, and V_{D3} are the coefficients of variation for dead load types $D1$, $D2$, and $D3$, respectively; and V_{AD1}, V_{AD2}, and V_{AD3} are the coefficients of variation for dead load analysis method for $D1$, $D2$, and $D3$, respectively.

Table 3 shows the statistical parameters given by the CHBDC commentary for the various types of dead loads. Based on these parameters, the mean and standard deviation for total dead load effects obtained from eqs. [3] and [4] are 12 790 and 1 150 kN·m, respectively.

Live load effects

The mean live load effect, $\bar{L}$, and standard deviation for live load effect, S_L (including dynamic amplification) are given by

$$[5] \qquad \bar{L} = \delta_L\delta_{AL}L(1+\delta_I I)$$

where δ_L is the bias coefficient for live load, δ_{AL} is the bias coefficient for live load analysis method, L is the nominal live load effect, and δ_I is the bias coefficient for dynamic load allowance.

$$[6] \qquad S_L = \bar{L}\sqrt{V_L^2 + V_{AL}^2 + \frac{(V_I\delta_I I)^2}{(1+\delta_I I)^2}}$$

where V_L is the coefficient of variation for live load, V_{AL} is the coefficient of variation for live load analysis method, and V_I is the coefficient of variation for dynamic load allowance.

The statistical parameters for live load suggested in the CHBDC commentary (CSA 2000, S6.1) are given in Table 4.

The values of L (nominal live load effect, M_{625}) for evaluation level 1 and I given earlier were used to obtain the mean and standard deviation for the live load effects including dynamic amplification from eqs. [5] and [6]; the values are 6510 and 695 kN·m, respectively.

Coefficient of variation of total load effects

The coefficient of variation of total load effects, V_S, is given by

$$[7] \qquad V_S = \frac{\sqrt{S_D^2 + S_L^2}}{\sum\bar{D} + \bar{L}}$$

By substituting the values of the various parameters in eq. [7], the coefficient of variation of total load effects is calculated to be 0.0698.

Live load capacity factor

A live load capacity factor for evaluation level 1, F_1, of 1.80 is obtained by substituting the values of the various parameters into eq. [2]. This result might be expected given CHBDC commentary 14.15(c), whereby the mean load method would give greater accuracy than can be obtained by using single-valued load and resistance factors that are appropriate to cover various situations.

Evaluation by mean load method using field data

Statistical parameters for resistance

The statistical parameters for resistance are the same as specified in the above section.

Statistical parameters for load effects

The statistical parameters for dead load effects remain the same as described above, but those for live load are based on the field measurements obtained from the site survey.

As shown in Table 2, the bias coefficient, $\delta_L(1+\delta_I I)$, and coefficient of variation, $[V_L^2 + (V_I\delta_I I)^2/(1+\delta_I I)^2]^{1/2}$, both of which include dynamic effects, are 0.94 and 0.028, respectively. The mean live load effect, $\bar{L}$ (eq. [5]), and the standard deviation for live load effect, S_L (eq. [6]), are calculated to be 4120 and 311 kN·m, respectively. The coefficient of variation for total load effects, V_S, is obtained from eq. [7] to be 0.071.

Table 3. Statistical parameters for dead loads.

Dead load type	Bias coefficient, δ_D	CoV[1] for dead loads, V_D	Bias coefficient for dead load analysis, δ_{AD}	CoV for dead load analysis, V_{AD}
Steel girder (*D*1)	1.03	0.08	1.00	0.00
Concrete deck, barriers and railings (*D*2)	1.05	0.10	1.00	0.00
Asphalt wearing surface (*D*3)	1.03	0.30	1.00	0.00

[1]Coefficient of variation.

Table 4. Statistical live load parameters from CHBDC commentary.

Statistical live load parameters from CHBDC	Value
Bias coefficient for live load, δ_L	1.35
Coefficient of variation for live load, V_L	0.035
Bias coefficient for live load analysis method, δ_{AL}	0.98
Coefficient of variation for live load analysis method, V_{AL}	0.07
Bias coefficient for dynamic load allowance, δ_I	0.40
Coefficient of variation for the dynamic load allowance, V_I	0.80

Table 5. Summary of live load capacity factor for evaluation level 1.

Evaluation method	Live load factors, F_1
Load and resistance factor method by CHBDC	1.73
Mean load method using statistical parameters from CHBDC commentary	1.80
Mean load method using live load statistics from field measurements	2.83

Live load capacity factor

The values of the various parameters are substituted in eq. [2] to obtain a live load capacity factor for evaluation level 1, F_1, of 2.83.

Comparison of evaluation results

Table 5 summarizes the values of live load capacity factor for evaluation level 1 for girder 2 obtained from the three methods of evaluation that were investigated.

The following observations can be made:

(1) Based on the results of girder 2, the evaluation shows the bridge to be fully capable of carrying maximum normal traffic loads per CHBDC. It should be noted that the evaluation was carried out for one section only and further analysis may be needed to ascertain the load capacity of the entire structure.

(2) The load and resistance factor method using load and resistance factors prescribed in CHBDC and the mean load method using statistical parameters given in CHBDC commentary (CSA 2000) give more or less similar results, with the mean load method providing a slightly higher live load capacity factor.

(3) The mean load method, using live load statistics based on field data, gives the highest load carrying capacity. As noted earlier, the parameters given in CHBDC are generally based on the worst traffic load scenario for highway bridges. Consequently, it will yield evaluation results that may be conservative for some bridges. The following factors should also be noted in evaluating these results.

The QEW – Ford Drive underpass is located east of the by-pass exit for Highway 403 to Highway 401, and hence carries only that portion of the QEW truck traffic that travels to and from downtown Toronto and Mississauga South. This constitutes less than 20% of the truck traffic in the busy sections of the QEW in the Oakville–Burlington area. Most of the trucks going to the industrial areas within Greater Toronto Area (GTA) and those heading to east of GTA generally use the QEW Highway 403 – Highway 401 corridor and miss the bridge under consideration in this paper.

The QEW does not carry the heaviest type of logging trucks, which generally use the resource roads and highways in Northern Ontario, where the trucks are allowed to carry even higher loads during the winter freeze-up period primarily because of pavement considerations. Thus, the QEW, while it may be carrying one of the largest truck traffic volumes, may not be carrying the heaviest of the truck loads at the time of survey. Most of the trucks on the QEW carry general freight and a few may carry heavy steel coils, oil, and gravel.

The data on the bridge were collected for a typical week in November 2000 when economic activities tend to slow down, and the volume of heavy types of trucks (steel trucks, gravel trucks,and construction equipment) is significantly reduced. Therefore, the field data may not represent the true nature of heavy trucks on the QEW.

The code provides conservative scenarios for multi-vehicle presence. The CHBDC provides for simultaneous multiple presence of trucks in various lanes by a multi-lane reduction factor that accounts for (*i*) low probability that critically loaded trucks will be simultaneously present on more than one lane and (*ii*) low probability that the trucks in other lanes will be present simultaneously at the critical location. The provisions represent a simplified treatment of multiple-truck presence, which is easier to carry out in the design process. However, the provisions cater for all types of bridges with all types of lateral load distribution characteristics. These may be overly conservative for bridges with better lateral distribution such as this bridge. A more realistic scenario may be to consider a full truck in the most critical lane and a reduced truck (average loaded truck) in other lanes. A review of Ontario truck weight survey data from 1995 and Quebec weight survey data from 1997 reveals that an average truck weight is approximately 39% of the annual maximum truck.

Furthermore, this being a three lane-bridge, it is likely that within the survey week multiple-truck presence of the nature considered by the code did not occur, and most events in the survey represented mostly single heavy trucks in critical locations. If most events are due to single trucks, it can be seen that the heaviest trucks in the survey caused almost two times the effect due to legally loaded test vehicle.

Conclusions

The case study presented here demonstrates the potential benefits of using the probability-based mean load method for evaluation of existing highway bridges. Improvement in the calculated load carrying capacity is evident if site-specific live load effects based on field measurements are used. However, field testing of this nature is an expensive exercise, and may not be conclusive if the most critical traffic loading periods are missed during the field measurements. The season and periods of measurements need to be selected carefully to obtain the most critical traffic loading on the structure. Nevertheless, for important bridges where the potential economic benefits arising from a more accurate structural evaluation would outweigh the cost of such site-specific live load data collection and analysis, the mean load method provides a very valuable evaluation tool.

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