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Reliability-Based Evaluation Section of the Draft Canadian Highway Bridge Design Code

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Summary

This paper outlines the code provisions of the draft Canadian Highway Bridge Design Code, CHBDC, which is to be published in 1997, for the evaluation of live load capacity of existing bridges. A methodology to assess the adequacy of the existing 3000 provincial bridges, designed to previous codes dating back to the turn of the century, for the current vehicle loads in Ontario based on the probabilistic approach reflected by the draft provisions of CHBDC is discussed.

1. Introduction

The Canadian Highway Bridge design Code, CHBDC, has been under development since 1993 and will be published in 1997. It will replace the current editions of the Ontario Highway Bridge Design Code, OHBDC, [1]; the Canadian Standard, CSA-S6, for Bridge Design, [2]; and the CSA-S6 Supplement No. 1 for Existing Bridge Evaluation [3].

The Evaluation Section of the draft Canadian Highway Bridge Design Code, CHBDC, provides procedures of evaluating an existing bridge to determine if it will carry a particular load or set of loads. The bridge may be evaluated by one of the following methods:

- Ultimate limit state, serviceability limit state, fatigue limit state;
- Mean load method;
- Load testing;
- Other methods approved by the Regulatory Authority.

2. Permanent Loads

The evaluation of the load carrying capacity of existing bridges should consider all permanent loads. Dead load should include the weight of all components of the bridge, fill, utilities and other materials permanently on the bridge. Earth pressure and hydrostatic pressure should be treated as permanent loads.

3. Transitory Loads

3.1 Normal traffic

The traffic load models to be used for the Evaluation Levels 1, 2 and 3 are CL1-W, CL2-W and CL3-W Loading, respectively. They consist of the corresponding Truck given in Figure 1, or the corresponding Lane Load given in Figure 2, whichever gives larger load effects.

The number "W" indicates the gross load in kN of the CL1-W Truck. Corresponding gross loads for the CL2-W Truck and the CL3-W Truck are 0.76 W and 0.48 W, respectively. In general, the value of W is taken as 625 kN.

The uniformly distributed load in a Lane Load occupies a width of 3.0 m in a traffic lane, and is placed transversely concentric with the truck.

3.2 Permit vehicle loads

Vehicles operating under permit are classified as PA, PB, PC or PS.

PA traffic includes the vehicles authorized by permit on an annual basis or for the duration of a specific project to carry an indivisible load, mixed with other traffic without supervision. PB traffic includes bulk haul traffic, or vehicles carrying divisible load authorized by permit programs for many trips, mixed with general traffic. PC traffic includes vehicles authorized by permit to carry an indivisible load on an specified route under supervision and specified travel conditions. PS traffic includes vehicles authorized by permit for a single trip, to carry an indivisible load, mixed with other traffic without supervision.

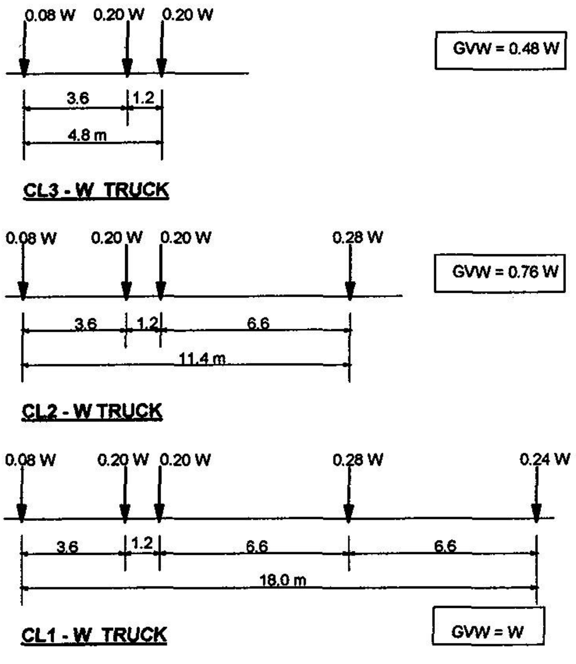
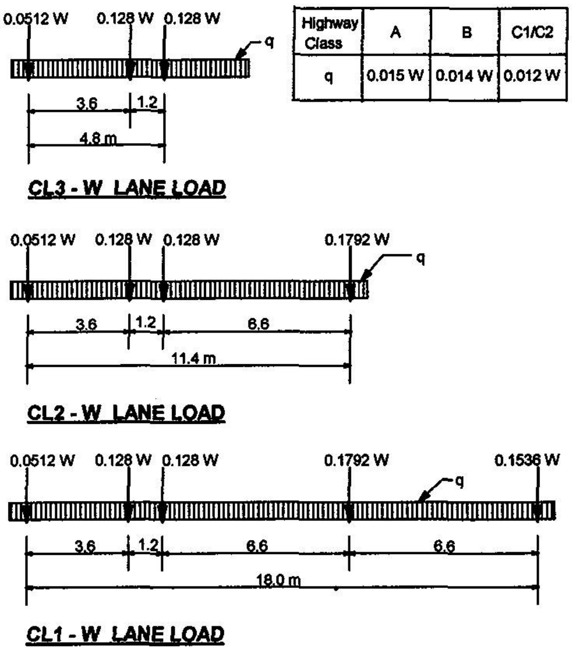


Fig. 1 Truck loading for normal traffic



Highway Class	A	B	C1/C2
q	0.015 W	0.014 W	0.012 W

Fig. 2 Lane loading

4. Target reliability index

The reliability index, β, is taken from Table 1(a) for all evaluation levels for normal traffic and for permit vehicles except PC vehicles for which β is taken from Table 1(b). In both cases, the system behaviour, element behaviour, and inspection level are taken as defined below.

The life safety criteria that forms the basis for the reliability indices presented in this section considers only loss of life resulting directly from the failure of the structure. For structures which indirectly affect life safety or are essential to the local economy or are necessary for the movement of emergency vehicles, a value of β which is 0.25 greater than those given in Table 1 is used.

4.1 System behaviour

- Category S1, where element failure leads to total collapse;
- Category S2, where element failure probably will not lead to total collapse; or
- Category S3, where element failure leads to local failure only.

4.2 Element behaviour

- Category E1, where the element being considered is subject to sudden loss of capacity with little or no warning;
- Category E2, where the element being considered is subject to sudden failure with little or no warning but will retain post-failure capacity; or
- Category E3, where the element being considered is subject to gradual failure with warning of failure probable.

4.3 Inspection level

- Level INSP1, where a component is not inspectable;
- Level INSP2, where inspection is to the satisfaction of the evaluator, with the results of each inspection recorded and available to the evaluator; or
- Level INSP3, where inspection of critical and/or substandard components has been carried out by the evaluator and final evaluation calculations account for all information obtained during this inspection.

System Behaviour	Element Behaviour	Inspection Level		
		INSP1	INSP2	INSP3
S1	E1	3.75	3.5	3.5
	E2	3.5	3.25	3.0
	E3	3.25	3.0	2.75
S2	E1	3.5	3.25	3.25
	E2	3.25	3.0	2.75
	E3	3.0	2.75	2.5
S3	E1	3.25	3.0	3.0
	E2	3.0	2.75	2.5
	E3	2.75	2.5	2.25

Table 1(a) *Target reliability index, β, for CL1-W, CL2-W, CL3-W, PA, PB & PS traffic*

System Behaviour	Element Behaviour	Inspection Level		
		INSP1	INSP2	INSP3
S1	E1	3.25	3.0	3.0
	E2	3.0	2.75	2.5
	E3	2.75	2.5	2.25
S2	E1	3.0	2.75	2.75
	E2	2.75	2.5	2.25
	E3	2.5	2.25	2.0
S3	E1	2.75	2.5	2.5
	E2	2.5	2.25	2.0
	E3	2.25	2.0	2.0

Table 1(b) *Target reliability index, β, for PC vehicles*

5. Load factors

The unfactored loads effects for each element under consideration is multiplied by the appropriate load factors for the value of β determined above.

5.1. Dead loads

When the dead load effect counteracts the effect due to transitory load, the minimum dead load factors are used for all dead load categories at any β value. Otherwise, the maximum dead load factors given in Table 2 are used.

Dead Load Category	Target reliability index, β							
	2.00	2.25	2.50	2.75	3.00	3.25	3.50	3.75
D1	1.04	1.05	1.07	1.08	1.09	1.10	1.12	1.13
D2	1.10	1.13	1.17	1.20	1.22	1.25	1.29	1.33
D3	2.10	2.30	2.50	2.70	2.90	3.10	3.30	3.50
D4	1.08	1.10	1.14	1.16	1.18	1.20	1.24	1.26

Table 2 *Maximum dead load factors, α_D*

Where:

D1 = factory-produced components, excluding wood
D2 = cast-in-place concrete, wood and all non-structural components
D3 = wearing surface
D4 = earth fill, negative skin friction on piles

5.2. Normal traffic

Live load factors for Evaluation Levels 1, 2, and 3 are as given in Table 3.

Type of Analysis	Target reliability index, β						
	2.25	2.50	2.75	3.00	3.25	3.50	3.75
Statically determinate	1.42	1.47	1.51	1.57	1.63	1.69	1.75
Sophisticated	1.48	1.55	1.62	1.69	1.75	1.83	1.90
Simplified	1.36	1.42	1.49	1.55	1.62	1.70	1.78

Table 3 *Live load factors, α_L, for normal traffic*

6. Live load capacity factor

For ultimate limit states the smallest value of the live load capacity factor, F, is calculated using the following equation for all structural components:

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

7. Mean Load Method

As an alternative to Equation (1) the live load capacity factor, F, at the ultimate limit state may be calculated using the following equation:

$$F = \frac{\overline{R}\, \exp[-\beta(V_R^2 + V_S^2)^{0.5}] - \Sigma \overline{D}}{\overline{L}} \tag{2}$$

where:

$$\Sigma \overline{D} = \Sigma \delta_D \delta_{AD} D$$

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

$$\overline{R} = \delta_R R$$

$$V_S = \frac{(S_{\Sigma\overline{D}}^{\,2} + S_{\overline{L}}^{\,2})^{0.5}}{(\Sigma \overline{D} + \overline{L})}$$

$$S_{\Sigma\overline{D}} = (V_D^{\,2} + V_{AD}^{\,2})^{0.5} \delta_D \delta_{AD} D$$

$$S_{\overline{L}} = \left[V_{AL}^{\,2} + V_L^{\,2} + \frac{(V_I \delta_I I)^2}{(1+\delta_I I)^2} \right]^{0.5} \delta_L \delta_{AL} L(1+\delta_I I)$$

Where:

D	nominal (unfactored) dead load effect
$\overline{D}$	mean dead load effect
I	nominal (unfactored) dynamic component of the live load, expressed as a percentage of the nominal static live load effect
L	nominal (unfactored) static live (traffic) load effect
$\overline{L}$	mean static and dynamic live (traffic) load effect
R	nominal unfactored resistance
$\overline{R}$	mean resistance
$S_{\Sigma\overline{D}}$	standard deviation of dead loads force effects
$S_{\overline{L}}$	standard deviation of live load force effects
δ_{AD}, δ_{AL}, δ_D, δ_I, δ_L, δ_R	bias coefficients (ratios of mean to nominal effects) for dead load analysis method, live load analysis method, dead load, dynamic load allowance, live load and resistance respectively.
V_{AD}, V_{AL}, V_D, V_I, V_L, V_R, V_S	coefficients of variation for dead load analysis method, live load analysis method, dead load, dynamic load allowance, live load, resistance, and total load respectively.
β	target reliability index

8. Global assessment of existing bridges in Ontario

Following the principles behind the mean load method described in (8) above, a modified approach was used to assess the adequacy of the existing bridges in Ontario to safely carry the current vehicle loads. The existing bridges in Ontario have been designed for various different design loads, such as H10, H15, H20, HS20, OHBD Loading. The bridges built prior to 1960's were generally designed by the working stress design methods. Bridges built after 1960 have mostly been designed by the load factor design methods or by limit states design approach.

The population was divided in to various families of bridges characterized by the original design load and design method, the structure type and span length. For each family of bridges, the reliability index β was determined for the current vehicle loads. This reliability index was then used to assess the adequacy of the bridges in that family.

To determine the reliability index, the following equation was used,

$$\beta = \frac{\ln(\overline{R}/\overline{S})}{\sqrt{(V_S^2 + V_R^2)}} \qquad (3)$$

where $\overline{S}$ is the real total mean load effect, including the mean dead load effects and the mean largest live load effects including dynamic load allowance. The other notations have the same meanings as given in Section 7.

8.1 Statistics for real loads

For this assessment, only the effects of dead load, traffic loads and the dynamic load allowance were considered. Statistics for various types of dead load effects used in the analysis are given in Table 4.

Dead Load Component	Description	Mean/Nominal Ratio	Coefficient of Variation
D1	Factory-produced components, excluding wood	1.03	0.04
D2	Cast-in-place concrete, wood, and all non-structural components	1.05	0.08
D3	Wearing surfaces	1.10	0.20

Table 4 *Statistics for dead load effects*

The extreme real lifetime live load effects were determined using data for 6287 trucks from the 1995 Commercial Vehicle Survey conducted in Ontario. Real values of mean dynamic load allowance and its coefficient of variation were obtained from the field testing results of a number of bridges in Ontario [4]. Mean value of dynamic load allowance was taken to be 0.20 up to a span length of 10 m with a coefficient of variation of 0.60, and 0.14 for spans greater than 10 m with a coefficient of variation of 0.82.

8.2 Real resistance

Since the evaluation did not address a specific bridge for which actual member sizes could be obtained from the drawings, real resistance for a family of bridges was determined by considering the original design provisions and design method. From the design provisions, minimum required nominal resistance at the ultimate limit states of the critical component was determined. The statistics for the real resistance were obtained by using the bias factors and coefficient of variation for resistance from [5] for structural steel components, and from [6] for the concrete components. These are summarized in Table 5.

Type of Response	Bias Factor, ρ_R	Coefficient of Variation, V_R
Steel Rolled Sections:		
Plastic Moment	1.126	0.081
Moment at First Yield, Axial Tension	1.210	0.077
Steel Welded Section:		
Plastic Moment	1.133	0.096
Moment at First Yield	1.221	0.100
Composite Plastic Moment	1.098	0.096
Reinforced Concrete	1.04	0.090
Prestressed Concrete	1.03	0.080

Table 5 *Bias factor and coefficient of variation for resistance*

8.3 Interpretation of results

The target value of reliability index for main components with ductile behaviour and normal inspection is 3.50. The family of bridges which had a value of reliability index less than 3.50 could be considered somewhat deficient to carry unrestricted traffic. However, with regular inspection and a re-evaluation within five years, reliability index of as low as 2.80 was considered an acceptable threshold for a need to post a bridge. Considering this threshold, it was found that approximately 4 percent of the bridges on the provincial system in Ontario may be potentially deficient and would require a detailed evaluation to establish posting loads. Most of these bridges are old steel trusses or steel girder bridges designed for H 15 or H 20 loads.

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