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SHM Application in Development of New Live Load Distribution factors for Timber Bridges

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ABSTRACT: The Nova Scotia Department of Public Works (NSDPW), based on their observations and research, believe that the simplified method of analysis of timber bridges included in the Canadian Highway Bridge Design Code (CHBDC) yields excessively conservative load ratings for typical timber girder bridges in the province. With over 2000 such bridges in their inventory, this study looks to improve conservative load ratings by developing more realistic live load distribution factors for timber bridges based on load testing and analytical work.
NSDPW engaged SHM Canada to conduct load testing of selected timber bridges and to develop a new regime of realistic live load distribution factors derived from the test data. Six timber bridges of various characteristics were selected for this study to cover as wide a range of bridge configurations as possible in the provincial inventory. The collected displacement and strain measurement data were analyzed and used to calibrate a large number of analytical models and followed by statistical and mathematical formulation of the proposed simplified method. The new method incorporates distribution factors specifically developed for timber bridges, by taking into consideration various parameters such as span length, girder spacing, and mechanical properties of the girders, to offer a fast, reliable, and cost-effective approach for evaluation and management of the province's timber bridge inventory.

KEY WORDS: Timber Bridge, Live load distribution factor, Load testing, Displacement transducer, SMA.

1 INTRODUCTION

The Nova Scotia Department of Public Works (NSDPW) inventory of bridges includes over 2000 timber bridges. These bridges are key components of the province's transportation infrastructure and are critical links on the local and collector roads. The majority of these bridges are short-span structures that have been in service for over 50 years [1].

Because of the size and importance of this inventory, substantial resources are dedicated for the inspection and load rating of these timber bridges. Currently, NSDPW uses the simplified method of analysis given in the latest editions of CAN/CSA-S6, the Canadian Highway Bridge Design Code (CHBDC/ the Code), for the load rating of in-service bridges and for the design of new timber bridges.

Based on their experience and research, NSDPW hypothesizes that load ratings determined through the CHBDC simplified methods are overly conservative for the types of timber bridges commonly found in the province. Comparison of results yielded by the CAN/CSA-S6-14 and -19 simplified methods to the results yielded by the preceding version (S6-06) show that the newer versions are relatively more conservative. This study has shown that even the simplified method in the 2006 edition of the Code yields more conservative load ratings for timber bridges than do the rigorous methods of analysis, [1-4]. Historically, NSDPW has used rigorous methods for analysis of its timber bridges. The application of these methods, however, is time-consuming and resource-intensive and would be too onerous to apply for the evaluation of NSDPW's timber bridge inventory. The overly conservative simplified methods, which have primarily been developed for steel and concrete bridges, also have a significant financial impact on the province. The primary objective of this study, therefore, is to develop an easy to use, and efficient method of evaluation specifically for timber bridges.

SHM Canada Consulting Limited (SHM Canada) was engaged by NSDPW to carry out load testing of six preselected timber bridges, representing a significant portion of the inventory, and to develop new live load distribution factors for timber bridges in the Province of Nova Scotia that would offer a time- and cost-effective alternative to the more sophisticated analysis methods. The accuracy and efficacy of these factors will be critical to the continued safe operation of Nova Scotia's transportation infrastructure and for effective management of resources.

2 LITERATURE REVIEW

The current standard for the design and evaluation of bridges in Canada is CAN/CSA S6:19. Methods of calculation of live load distribution in CHBDC and its predecessor, the Ontario Highway Bridge Design Code (OHBDC), have been revised multiple times since 1983. These methods are categorized as either Simplified Methods of Analysis (SMAs) or Refined Methods of Analysis (RMAs).

RMAs emerged as the increasing availability and enhanced capabilities of computers in the 1980s and 1990s encouraged engineers to attempt more complex forms of analysis and to model bridges with a large number of structural elements. These methods are generally highly accurate but require significant modelling time [1], [5]. More time-efficient simplified methods are therefore required for analysis of large inventories of simpler structures (e.g. the NSDPW inventory of over 2000 timber bridges in Nova Scotia).

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Finite Element Method (FEM), Semi-Continuum Analysis (SCA), Orthotropic Plate Theory, Grillage analogy, Folded plate theory… are some of the recognized RMAs in CHBDC, [1-4], however, only FEM and SCA were used in this study.

The Finite Element Method (FEM) is a powerful tool for analyzing simple to complex structures. The main challenge of this method remains a long modelling time [1] [5].

The semi-continuum analysis (SCA) method is useful for modelling timber bridges as it closely represents their structural configuration with slab on girder elements. The timber deck is modelled as a plate element supported on stringers, which are, in turn, modelled as line elements. The models generated by these methods are close to the reality of the nature of the bridge components and thus lead to more accurate results [1] [3] [4].

SMAs rely on the beam analogy, an equivalent-beam method in use since the 1930s, for longitudinal distribution of loads along the bridge, by calculating the maximum contribution of a single girder (or a unit width of a slab-type bridge) in resisting imposed loads. In all SMAs, transverse distribution of the longitudinal load effects is a result of multiplying a fraction coefficient provided by the formulas in the codes. Span length is a key parameter in deriving simplified method equations, as longer span length results in better transverse load distribution. Tighter spacing between girders leads to effective loads sharing among more number of girders across the width of the bridge, resulting in lower distribution factor values, [1-4], [8], [9].

As the Commentary for the current edition specifies, the CHBDC SMA equation should always produce load effects that are greater than those calculated using RMAs, as the RMAs used to develop the SMA equation are based on the most critical condition permitted by the Code [9]. In general, using RMA result in higher load rating values than those calculated from the three editions of the Code (06, 14, and 19). Using SMA will therefore lead to conservative designs and yield lower load ratings for the existing bridges.

The majority of the constants in the formulation of the CHBDC simplified method were derived from work by Smith, while the recent formulation of SMA in CAN/CSA-S6:19 benefits significantly from the work done by Théoret and Massicotte, [1]. These studies, however, relied primarily on analysis of concrete and steel bridges. Timber bridges differ considerably from the most concrete and steel bridges in their structural configuration, and the mechanical properties of timber are markedly different from those of concrete or steel.

The non-linear behavior of the girders beyond the proportionality limit, and before the point of failure, can result in redistribution of the load effects. The technical committee of the OHBDC, therefore, offered an 8% reduction in the live load moment effect due to the redistribution of the moment effect based on a number of analytical studies of bridges with a concrete slab-on-steel-girder design. This reduction was offered only for the evaluation of existing bridges, and not for the design of new bridges. Although it is recognized as applicable to other bridge types, it has not been investigated extensively for those bridge types [8].

Fanous et al. carried out extensive research on development of live load distribution factors for glued-laminated timber girder bridges with glued-laminated timber deck panels. With a verified numerical model created in ANSYS 11 based on four in-service bridges, more than 100 hypothetical bridge models were produced and live load distribution factor's relation was obtained for various bridge parameters, such as span length, girder spacing, and bridge width, It was shown that the AASHTO LRFD Bridge Design Specifications are conservative compared to the results obtained from field tests and numerical models [10].

An analytical and experimental study of six sawn timber bridges showed that asphalt, which tends to be neglected in both SMAs and RMAs, contributes to better live load sharing between timber stringers. This conclusion was based on the finding that stringers in field tests contributed a 17% smaller value of load sharing than predicted through rigorous analysis using SECAN/SCA. This study also found that analytical results for deflection were 20% larger than values obtained in the field, suggesting a larger modulus of elasticity than assumed in the Code recommendation for that timber species [11].

A more recent study sponsored by NSDPW carried out analysis of timber bridges with multiple editions of CHBDC (CAN/CSA-S6-88 through CAN/CSA-S6-14). This project was aimed at understanding the evolution of the simplified method in CAN/CSA-S6 over time and to compare results of the simplified method with more rigorous analysis using SCA. The study employed SECAN4 software and a scaled-down laboratory mode of the timber bridge. The results showed that the load effects calculated using revised versions of the code had increased over time by up to 30% [1]. NSDPW currently recognizes a rate of increase of 22%. CAN/CSA-S6-06 was found to produce the most accurate results of the versions studied but still yielded more conservative load ratings than those determined from the rigorous analyses. The study recommended the use of the simplified method in CAN/CSA-S6-06 as a first step in evaluation. Where this analysis yields a live load capacity factor (LLCF) falling within the range of 0.7–1.0, as is the case for approximately 60% of bridges in the NSDPW inventory, a more rigorous method may yield a higher load rating [1].

3 INSPECTION AND LOAD TESTING

3.1 Bridge Inspection Program

Prior to field load testing, detailed hands-on inspection of the six test bridges was carried out in accordance with CAN/CSA S6:19 [4]. The primary objective of these inspections was to identify any defects or deficiencies with the potential to affect load testing results.

The inspection program comprised three main components:

- Detailed visual inspection of the bridge superstructure and timber substructure where applicable;
- Non-destructive testing (NDT), including sounding and Stress Wave Timer (SWT) testing; and
- Timber grading.

Detailed measurements were taken with adequate detail to produce elevation and section sketches in AutoCAD format. The overall condition rating of the six bridges as determined during the inspection program is given in Table 1.

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Table 1. Condition Rating of Six Test Bridges

Bridge ID	Overall Current Bridge Condition Rating
HFX 061	Good
COL 098	Fair, near to Good
HFX 334	Fair, near to Good
HFX 322	Good
HFX 325	Fair
HFX 099	Fair

3.2 Load Testing Program

Girder deflection under load from a static truck with a known weight was measured by displacement sensors under each girder of the bridge and by strain gauges installed at select locations. The quality of this data was improved by incorporating an array of strategically placed high-precision sensors in the load testing program.

3.2.1 Data Acquisition System

All displacement sensors and strain transducers were connected to a data logger which acquired data at a speed of 3 Hz. Two 16-channel analog input modules were used to connect displacement transducers and strain transducers. The data acquisition system was housed in the monitoring vehicle located on site and powered by a high-capacity inverter. In order to avoid disruptions in power supply, the system was equipped with a rechargeable 12 V back-up battery.

3.2.2 Displacement Transducers

In selecting the most appropriate type of displacement transducer, three main factors, under-bridge clearance; minimum environmental disturbance; and ease of installation, calibration, and removal of sensors; were considered in the selection of the draw wire displacement transducers. Under-bridge clearance varied between 1.2 m and 3 m, and special adjustable steel cable system was designed to facilitate easy installation.

3.2.3 Strain Transducers

Strain measurements were recorded using strain transducers with a gauge length of 75 mm. The selected strain transducers provide improved accuracy in comparison to bonded foil-type strain gauges. Special mounting brackets were developed to eliminate the effect caused by local variations/imperfections in timber during installation of the transducers.

3.2.4 Installation

For each bridge, draw-wire sensors were installed at mid-span on the soffit of all timber girders. Additional displacement sensors were installed near the supports on bridges where appropriate. Small steel plates were attached to the soffit of each timber girder with wood screws. Matching steel plates with high-power magnets attached to the displacement sensors allowed a quick, secure, and efficient installation and retrieval.

A custom-built sensor installations system, which incorporates a thin aircraft cable, was developed and fabricated to accommodate the varying distance between girder soffits and the stream bed. The draw-wire of the sensor was pulled down approximately 50% of the sensor range and attached to the thin aircraft cable, which was adjusted in the field to make up the remainder of the distance to the stream bed. The aircraft cable was attached to steel hooks that were screwed on to a square timber beam supported on steel stools and ladders resting on the stream bed thus minimizing disturbance to the riverbed. The test setup for the bridge, HFX 099, is shown in Figure 1 below.

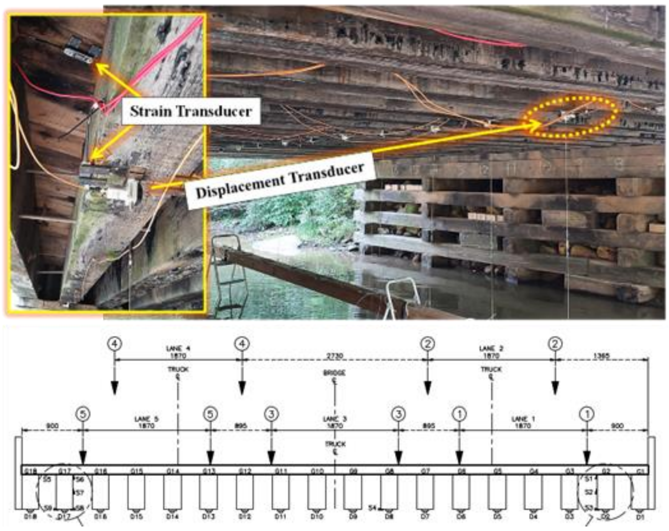

Figure 1. Test setup for HFX 099

Strain transducers were installed on selected timber girders, in close proximity to the draw-wire sensors as possible, in order to correlate strain measurements to the displacement measurements. On select bridges, strain transducers were installed on the deck soffit to determine load sharing characteristics of the timber deck.

3.2.5 Test Vehicle

Bridges were tested using a single-unit tandem axle truck loaded near the legal maximum capacity (25880 kg). The truck was pre-weighed prior to commencing the load testing. A summary of the test vehicle axle loads and axle spacing is presented in Table 2 below for the bridge, HFX 322.

Table 2. Test Vehicle Axle Weight and Spacing

Front Axle Weight (kg)	Twin Rear Axle Weight (kg)	Axle Spacing (mm)	
		First to Second	Second to Third
7930	17950	4150	1360

3.2.6 Load Testing Procedure

After all sensors were installed and tested for functionality, traffic control was implemented to close the bridge to all vehicular traffic. The load testing procedure consisted of the following steps:

- Demarcate travel lanes for each test run.
- Demarcate equally spaced lines at predetermined intervals depending on span length of the bridge.
- Record baseline readings of bridge prior to commencing load testing.
- Move the test truck to align with first travel lane and position centre point of second axles over the first stop as shown in Figure 2.

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- Wait for one minute or longer for bridge to stabilize and then record data for 3 minutes.
- Review collected data to confirm consistency. If data were found to be inconsistent, repeat test before moving to the next load position.
- Move truck to next stop and repeat procedure at each transverse line.
- Collect data at end of each test and determine if any residual deflections remain in girders after removal of test truck from bridge.
- Move test truck to align with next travel Lane and repeat steps above until all travel lanes are completed.
- Once testing was complete in all lanes, confirm consistency of testing data by computing summation of girder deflections for similar stop points of different test lanes.
- Collect data for moving test truck over bridge in order to detect dynamic load effects.

Figure 2. Test Vehicle at COL 098

3.2.7 Load Testing Data Results

Collected data for each bridge was reviewed and processed prior to commencing data analysis. Maximum deflection values for each bridge are presented in Table 3. All bridges were well below the maximum deflection criteria of L/360.

Table 3.Summary of Load Testing Results

Bridge ID	Clear Span (m)	Maximum Recorded Deflection (mm)	Deflection-to-span Ratio
HFX 061	7.9	15.2	1/520
COL 098	6.7	11.3	1/591
HFX 334	5.1	9.4	1/543
HFX 322	6.2	8.5	1/729
HFX 325	3.9	8.8	1/443
HFX 099	8.4	14.1	1/596

4 DATA ANALYSIS AND DEVELOPMENT OF LOAD DISTRIBUTION FACTORS

Following the load testing phase, field-recorded data were processed to be used for validation of the computer models.

4.1 Data Processing and Preliminary Analysis

Data collected during the load testing were processed in this phase. For each bridge, data collected at each position of the test truck were averaged. Initial recorded values from the gauges were then deducted from these averages. In bridges where displacement sensors were installed at the end of the girders, settlement of the abutments was noted. An average settlement of 3 mm was recorded for the tested bridges and was considered in the model calibration process. Data recorded after the truck was moved off the bridges showed that abutments rebounded to their original condition.

4.2 Material Properties of Timber

Material properties of timber are different in three orthogonal directions. Figure 3 shows a typical view of principal axes of wood relative to the direction of the wood grains.

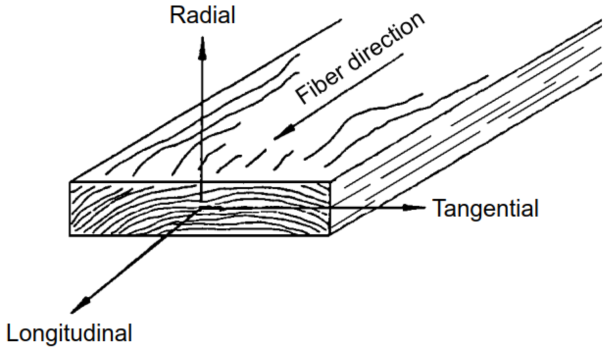

Figure 3. Principal axes of wood material, [12]

Material properties of timber also vary by species. According to the standard drawings for bridges in the province from 1959 and 2001, Douglas fir and hemlock have been extensively used as the material for stringers, and deck elements, respectively. Both are classified as soft [13], [14]. Multiple resources were consulted to determine a range of values for the material properties of these species [1], [11], [12], [15]. The wood material properties used in the initial analysis discussed in this section are presented in Table 4.

Table 4. Material properties of wood species used in Nova Scotia timber bridges.

Property (MPa)	Douglas Fir	Hemlock
Longitudinal (E_L)	1.06+E4	1.09+E4
Radial (E_R)	6.26+E2	4.85+E2
Tangential (E_T)	6.26+E2	4.85+E2
Longitudinal-Radial (G_{LR})	7.53+E2	3.82+E2
Longitudinal-Tangential (G_{LT})	7.53+E2	3.82+E2
Radial-Tangential (G_{RT})	7.42+E1	3.27+E1

Modulus of Elasticity (E)
Modulus of Rigidity (G)

In an initial attempt to validate the girders' mechanical properties in the longitudinal direction, the recorded Strain-Displacement (converted to Moment-Displacement) values were compared with the equivalent results generated by an analytical MATLAB code, developed by Hoseinpour et al. and modified for the current study, [16]. This code uses the typical equations of the Strength of Material to calculate displacement diagrams for the timber beams to be compared and calibrated with the field-recorded displacement data. Strain-Displacement data from slow-speed dynamic load test were the key factors in carrying out the validation process. Figure 4 presents an

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example of the analytical investigation of wood material properties where equivalent moments were determined using the recorded strain values for HFX 099, which has a clear span length of 8.40 m.

The resulting Moment-Displacement diagram was compared with the diagram generated by MATLAB for different span lengths and moduli of elasticity. Analysis showed that the closest relationship between the field data model and the analytical model is achieved by increasing the clear span length of the bridge by 3.5% (to account for the bearing span length) and by using a longitudinal modulus of elasticity of 1.0E4 MPa for the girders.

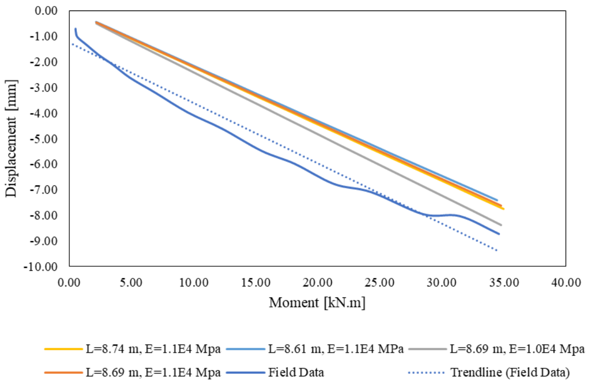

Figure 4. ST 3 on HFX 099

The final step in the process of validating material properties was examination of the values obtained with the 3D numerical model in midas Civil, and the generation of the deflection diagrams for comparison with the field recorded data. Due to the uncertainty associated with wood as an orthotropic material, multiple trials were performed to complete the validation of the wood material properties.

4.3 *Analysis Methods*

Multiple methods were employed in analysis of the bridges under study, including the simplified methods of the codes CAN/CSA S6-06 and S6:19/14. Two different clauses of the SMAs in the newer versions of the Code were used where the method described in C5.6.6, generally applies to slab-on-girder bridges, and the simplified method outlined in C5.6.7, applies specifically to non-skewed timber bridges. Semi-Continuum Method of Analysis (SECAN4 in conjunction with CBridge) and Finite Element Analysis Method (midas Civil Software) were used for rigorous analysis and for evaluation of SMA results, which were then calibrated based on the field test data. The components of the modelled bridges via FEM were assigned appropriate element types, i.e., beam and plate elements for modelling girders and timber decks respectively, and appropriate links were created between them to represent nailed connections. In all analysis methods the timber bridges were assumed to act as a simple span beam.

The CHBDC standard loading truck was used in the analytical process, and the test truck "Test Vehicle" was used in the calibration of the computer models. A 3-D of the finite element model is shown in Figure 5.

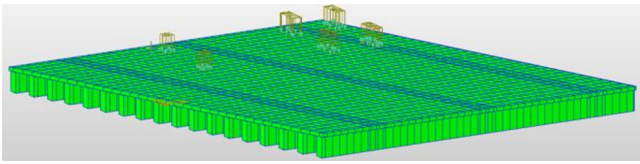

Figure 5. A FEM model of a skew bridge with truck load

The outcome of the final step of the calibration of the FEM models for two of the six bridges under investigation is presented in Figure 6 and Figure 7 below. The figures show the deflection profile across the bridge deck at the test vehicle stop that induced the maximum displacement under reference axle.

The small difference between experimental diagrams and numerical model can be attributed to the effect of other factors (e.g. abutment settlement, condition of the bridges, and presence of asphalt) which were taken into account while developing the distribution equations.

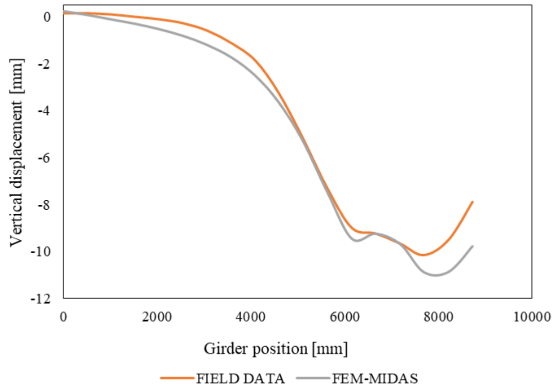

Figure 6. COL 098 – Lane 1: Transverse deflection profile

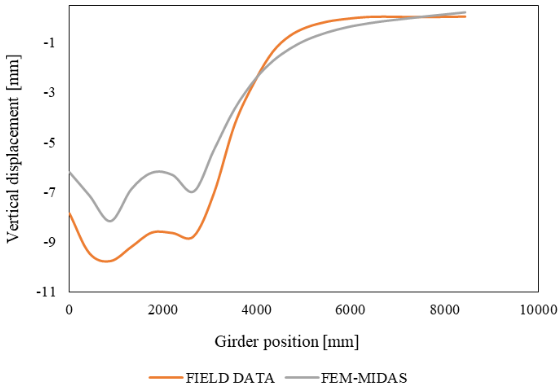

Figure 7. HFX 334 – Lane 5: Transverse deflection profile

4.3.1 *Comparison of Results*

The results obtained by the mentioned methods are summarized in the Table 5 and Table 6, representing moment and shear effects respectively. Multi-lane reduction factor is multiplied in the results obtained by rigorous analysis as this factor is inherent in the code formulation. In both tables, all load factors have been excluded from the results; only net values of the maximum distributed effects are displayed.

4.4 *Loading Characteristics of the Hypothetical Models*

The single unit truck of S6:19 (CL3_625) was selected as the effective live loading on the hypothetical bridges shown in Figure 8, [4]. The selected truck was used to achieve uniform

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loading and to eliminate any effect of additional axle loads. For the majority of timber bridges in Nova Scotia, given their defined range of span lengths, CL3_625 is the governing truck for determining load effects.

CHBDC specifies that the minimum distance between the truck wheels and curb, railing, or barrier wall shall be 0.60 m. Standard drawings for Nova Scotia bridges include a typical barrier width of 0.30 m [13], [14], and are therefore in compliance with the Code. Some of the bridges studied, however, have a barrier measuring only 0.1 m. Therefore, a minimum 0.6 m distance from the edge of the bridge was used to establish the transverse location of the loading truck. To achieve maximum load effect, the truck wheels were also aligned directly over the nearest girder while maintaining a clearance of 0.6 m from the bridge edge.

Table 5. Moment effects comparison (all values are in kN-m).

Bridge	S6-06	S6:19/14, C5.6.7	S6:19/14, C5.6.6	SCA (SECAN)	FEM (MIDAS)
HFX322	41.76	53.89	44.58	46.92	47.95
HFX325	31.56	40.49	36.68	29.80	30.92
HFX334	30.32	38.91	33.51	31.53	30.50
HFX099	70.26	90.31	71.00	60.27	53.53
COL098	51.11	65.44	53.80	47.18	44.46
HFX061	57.86	82.77	58.49	62.56	60.40

Table 6. Shear effects comparison (all values are in kN).

Bridge	S6-06	S6:19/14, C5.6.7	S6:19/14, C5.6.6	SCA (SECAN)	FEM (MIDAS)
HFX322	47.27	59.19	46.61	57.43	55.58* (76.94)
HFX325	52.42	65.65	51.70	64.36	52.75* (78.48)
HFX334	43.72	52.17	41.09	61.41	48.20* (67.14)
HFX099	54.45	69.19	54.49	55.10	61.13* (82.69)
COL098	52.12	65.08	51.25	67.61	56.52* (72.85)
HFX061	50.77	66.59	49.35	80.31	68.34* (91.93)

* These values represent shear effects when wheel load footprints are modelled as patch loads. The values shown in parentheses represent shear effects, when wheel loads are represented as point loads, representing an average increase of 1.35 over the patch load effects.

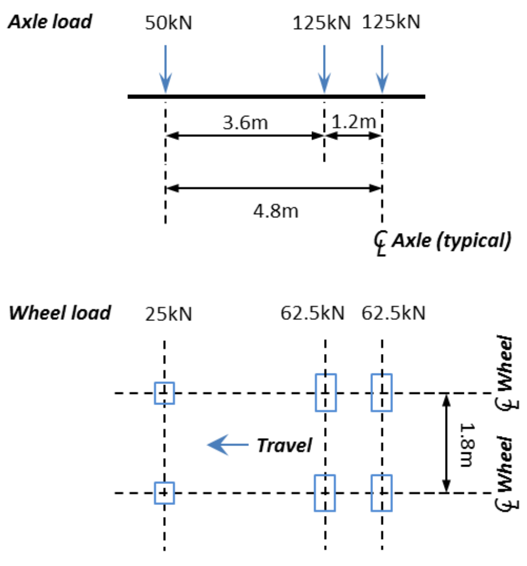

Figure 8. CL3 625 Single unit truck used in the hypothetical models, Source: CHBDC [4]

4.5 *Characteristics of the Hypothetical Models*

Having the FEM models calibrated based on span length and material properties, 101 hypothetical bridges were created and used to generate the mass data required for statistical analysis. Typical values of the parameters of Nova Scotia timber bridge inventory, such as girder spacing and span length were considered when generation the models.

4.6 *Development of Moment Distribution Factor*

Distribution of the moment effect between girders is known to be related to the girders' displacement. Based on the Theory of Timoshenko, however, it is critical to differentiate between displacement imposed on the girder by work moment and displacement imposed by shear effects. Lower maximum distribution factors indicate better distribution of load effects between girders.

4.6.1 *Parametric Analysis of Moment Distribution Factor*

After the maximum distribution factor of each hypothetical bridge was obtained, those values were plotted against variable parameters, and relationships between distributions factors and each parameter was determined. Span length, for instance, which is a key factor in bridge analysis, appeared to have a negative linear relationship with the moment distribution factor, as shown in Figure 9 below.

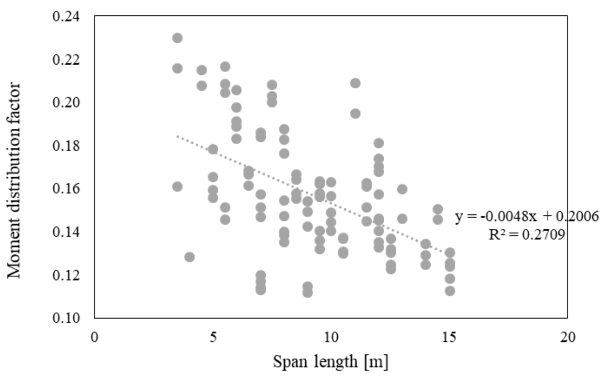

Figure 9. Span length in relation to the distribution factor

Girder spacing was also found to have a positive linear relationship with the distribution factors (Figure 10 below).

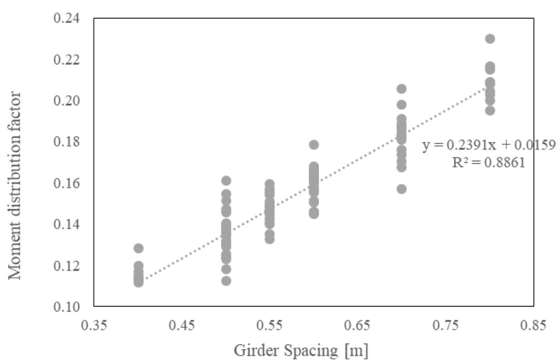

Figure 10. Girder spacing in relation to distribution factors

R-squared values showed that the spacing of girders correlates more strongly with the bridge distribution factor than do other parameters. The number and geometry of girders and

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other parameters, however, did not play an important role in determining distribution factors and were tried to be imported to the equations as modification factors.

4.6.2 *Development of the Moment Distribution Factor Equation*

The Moment distribution factor was formulated using MATLAB; in which the bridge's parameters were combined in different forms, i.e. separate linear combination and multiplication or division of parameters to different powers, to achieve highest correlation. The highest R-squared value (0.96) was found to belong to the simple linear summation of span length and girder spacing shown in Equation 1 below.

$$F_T = S/4.562 - L/394 + 0.051 \quad (1)$$
$$F_{T-min} = S/4.182 + 0.016$$

S: average girder spacing, center to center (metres)
L: bearing span length (metres)

When $F_T > F_{T-min}$: $F_{T-modified} = {F_T}^2/F_{T-min}$
When $F_T \leq F_{T-min}$: $F_{T-modified} = F_{T-min}$

In cases where F_T is larger than F_{T-min}, a correction is required to account for the effect of different girder sizes. Therefore, the value obtained by the above equation shall be multiplied by the ratio of F_T/F_{T-min} as shown in the step 3 of Equation 1. Based on calibration process described in Section 4.2, the recommended bearing span length for timber bridges is 1.035 times of the measured clear span length.

4.6.3 *Effect of Skew Angle (ψ) on Distribution Factors*

Skew angle and span length are two important parameters used to determine skew effect in CHBDC. Bridge width, which was the dominant factor in the past editions of the code, plays an insignificant role in the current edition (S6:19).

Maximum moment effect is reduced in skewed bridges due to the decrease in the effective span length. Shear effect is subject to increase in the obtuse corner of skewed bridges, as a result of decreasing stiffness of the girders at the points on the axle line toward the acute edge and a corresponding reduction in load carrying of the far girders, [9].

The Mohr circle concept was employed to generate deck material properties for bridges with different skew angles. Analysis of the hypothetical skewed models also demonstrated that shear effects are increased, and moment effects are decreased in skewed bridges. The results for skewed bridges were close to those obtained by multiplying the CHBDC skew equation with the results of non-skewed bridges. The CHBDC skew effect equation should therefore be used to magnify shear effects and reduce moment effects. The shear modification coefficient determined from the CHBDC skew effect equation is denoted as F_S in relevant formulations.

$$F_S = 1.2 - (2.0/(\varepsilon + 10) \quad (2)$$

ε = (L/S) tan ψ for ψ = ≤ 45
L = Bearing span length
S = Girder spacing

4.6.4 *Effect of Girder Dimension on Distribution Factors*

Mechanical properties of different girder sizes for each bridge configuration showed low correlation with the obtained moment distribution factors and made it difficult to find a simple formulation to determine the effect of girder dimension. Given its partial correlation with the distribution factors, the effect of the mechanical properties of the girders is reflected in the original expression with F_{T-min} and the proposed increase in the moment distribution factors.

4.6.5 *Effect of Multiple Traveling Lanes on the Distribution Factors*

Nova Scotia timber bridges are typically limited to two travelling lanes. Deflection diagrams showed that the effect of a single truck on one side of a wide bridge has an uplift effect on the far girders at the opposite side. Having three trucks on the specified lanes of the timber bridges would therefore lower distribution factor values; therefore, only two-lane bridges were considered in the modelling process. To account for multilane effect, the Number of Lanes factor, F_L, was included in Equation 4, with the values displayed in Table 7 below.

Table 8. Number of Lanes factor (F_L)

Single Lane Bridge	Two-Lane Bridge
1.00	1.21

The Code's statistical factor to account for multi-lane effects is still applicable and is presented in the current research as well. Per sections 3 and 14 of the Code, respectively, the modification factor is 0.90 for design of a two-lane bridge, and 0.85 for evaluation. This factor is denoted by R_L in Equation 3.

4.6.6 *Effect of Bridge Condition on the Distribution Factors*

Live load testing results in conjunction with inspection findings and analytical modelling suggest that the bridge condition may affect the load distribution factor. In some cases, field-validated loading responses of the bridge structures differed from those expected based on the calibrated FE model. In order to align the FE model with recorded field data, the condition rating of the bridges under investigation was considered (See Section 3.1). The values provided in Table 9 below were assigned as the factor F_C to reflect the overall condition of the bridges in the load distribution formulation, Equation 4.

Table 9. Bridge Condition factor (F_C)

Good	Fair to Good	Fair
1.00	1.12	1.20

None of the bridges included in the current study were rated in lower than Fair condition. Local decays in bridges in Poor condition has the potential to significantly affect the load distribution factor. No F_C factor, therefore, is provided for evaluation of bridges rated at Poor condition and it has to be concluded based on professional judgment.

4.6.7 *Effect of Asphalt Surface on the Distribution Factors*

Asphalt pavement of varying thickness was found on four of the six bridges in the current study. Thicker asphalt would result in better load distribution and therefore lower load distribution factors. The effect of asphalt surface is designated as F_A in Equation 3 and Table 10. Linear interpolation shall be used to find an equivalent factor for different asphalt thickness.

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Table 10. Asphalt surface factor (F_A)

90 mm	150 mm	250 mm
1.08	1.15	1.25

A gravel surface, in contrast to an asphalt surface, is inconsistent in thickness/distribution over the bridge deck and has low elastic properties. Furthermore, only one of the bridges under investigation had a gravel surface, and it appeared to be loose and uneven near the edge of the bridge. Therefore, the contribution of a gravel surface to the load distribution, could not be evaluated in this study, and F_A, shall be assumed as 1.0.

4.6.8 *Generalized Moment Distribution Factor Equation*

Equation 3 below presents the overall distribution factors for moment effects. This equation includes coefficients that account for the effects of other parameters, as discussed earlier in this chapter.

$$F_M = F_{T-modified} F_L F_C\, R_L / F_A F_S \tag{3}$$

$F_{T-modified}$ = Moment distribution factor
F_S = Skew angle coefficient
F_L = Multiple lane effect factor
R_L = Statistical multilane coefficient
F_C = Bridge condition factor
F_A = Asphalt effect factor

4.7 *Shear Distribution Factor*

Shear distribution factor was obtained by dividing the maximum contribution of the girders by the maximum shear effect of the same loading on a single beam with the same span length, where the single beam was analyzed using CBridge. In order to produce maximum shear in a simple beam, the heavy truck axles must be located as near as possible to the supports where, due to the minimum deflection of the girder, the girder closest to the wheel load absorbs much of the shear effect. Shear distribution does not, therefore, follow the same pattern as the moment distribution, and a separate distribution factor is required.

4.7.1 *Parametric Analysis of Shear Distribution Factor*

Like the moment distribution factor, shear distribution factors showed a stronger correlation with girder spacing than with span length. In general, weaker correlations (i.e. R-squared values) were established between the shear distribution factor and variable parameters of the timber bridges than between the moment distribution factor and those parameters.

4.7.2 *Development of the Shear Distribution Factor Equation*

The shear distribution factor in the 2019 version of the Code is only a function of girder spacing, magnified by a modification factor. Using this factor in the current study yielded an R-squared value well below the expected value.

MATLAB was used to combine the effects of different parameters and to develop a formulation for the shear distribution. Multiple trials, however, failed to yield correlations (i.e. acceptable R-squared values) that would suggest sufficient accuracy for these formulas.

A method similar to that used for determination of the moment distribution factor, was therefore applied to generate a modification coefficient to account for the shear distribution factor. As shown in Equation 4, the shear distribution factor was obtained by modifying the moment distribution factor, the use of which can produce shear effects having an acceptable correlation with an R-Squared value of 0.71.

$$F_v = {F_S}^2 F_M / (1.35\, \gamma_v\, \gamma_{lv}) \tag{4}$$

Where, F_S is the skew angle factor, and the shear modification factor γ_v is obtained from the following equation:

$$\gamma_v = S/2.532 - L/134 + 0.247 \tag{5}$$

The correction factor, γ_{lv}, of the shear effect for two-lane bridges is equal to 1.14. See Equation 1 and 3 for the notations.

In an attempt to create maximum load effects, the longitudinal wheel line of the loading truck was aligned over the girder closest to the bridge edge, while respecting the minimum 600 mm clearance. This was the governing case specifically for determining the shear effects.

Considering the low potential for transverse load distribution at the end of the bridges' span it would be safe to assume that the loaded girder absorbs the entire wheel load on the support.

On the other hand, analyzing the standard truck of CL-625 on a short span bridge has the potential of eliminating the effect of the heavier axle load in the shear analysis as it remains off the bridge span in the critical shear loading case. As a result, there is always a possibility to disregard the heavier axle loads with wide spacing in the shear analysis, where they have no potential to be transversally distributed when acting on the support. Therefore, the minimum unfactored shear effect shall be taken as the largest of the wheel load of the loading truck divided by the patch load correction of 1.35.

4.8 *Comparison of the results of the developed SMA to the loading tests and CHBDC results*

Table 11 presents distribution factors obtained from live load tests, different versions of the Code, and the developed SMA in this study to validate the results of newly developed formulations.

Table 11. Comparison of the moment distribution factor derived using new formulation with other methods

Source	HFX322	HFX325	HFX334	HFX099	COL098	HFX061
(1)	0.1722	0.2563	0.1758	0.2167	0.2142	0.1938
(2)	0.1367	0.1986	0.1296	0.1460	0.1503	0.1347
(3)	0.1281	0.1708	0.1172	0.1444	0.1428	0.1292
(4)	0.1282	0.2117	0.1309	0.1668	0.1440	0.1270
(4-1) %	-25.6	-17.4	-25.5	-22.8	-32.8	-34.5
(4-2) %	-4.5	17	6.1	10.9	-3.6	-5.7
(4-3) %	0.1	23.9	11.7	15.5	0.8	-1.7
(5)	0.1273	0.2099	0.1309	0.1631	0.1624	0.1374
(5-1) %	-26.1	-18.1	-25.5	-24.5	-24.1	-29.1
(5-2) %	-5.1	16	6.1	8.4	8.7	2
(5-3) %	-0.6	22.9	11.7	13	13.7	6.3

(1) S6-14/ S6:19 C5.6.7
(2) S6-14/ S6:19 C5.6.6
(3) S6-06
(4) Live Load Test
(5) Developed SMA

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This comparison established a close relationship between the live load test results and the results of the developed SMA here. Compared to the results obtained with CHBDC S6-14 and S6:19, C5.6.7, both live load tests and the new SMA show a decrease of between 17.4% and 34.5% in moment distribution factors across all timber bridges studied, which is consistent with NSDPW's hypothesis that the Code-specified analysis method for timber bridges leads to more conservative design and evaluation criteria.

5 CONCLUSION

Results of this study show that the current SMAs in the Canadian bridge codes are overly conservative, and the congruence of analytical and load test results supports the hypothesis that the newly developed equations are sufficiently accurate for the simplified analysis of typical timber bridges in Nova Scotia. The equations presented in this report can be used for both design and evaluation purposes, using appropriate factors and subject to the following parametric limitations:

- Span range: 3 m to 15 m.
- Girder spacing: 350 mm to 800 mm.
- Girder width: 150 mm to 300 mm.
- Girder depth: 250 mm to 800 mm.
- Girder spacing: Uniform or with less than 10% variation.
- Skew angle: Less than 45°

Bridges with characteristics outside the above ranges will require additional study.

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[Note: Nova Scotia Department of Public Works (NSDPW) was formerly known as Nova Scotia Transportation and Infrastructure Renewal (NSTIR)]