Ministry Contract No. EN1128A003

Evaluation of timber deck systems to develop rational engineering design and analysis of existing systems, Phase 2

Final Report

by

Ricardo O. Foschi, P.Eng
3806 West 36th Avenue
Vancouver, BC

February 15, 2011

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1. Summary

The province of British Columbia is encouraging the expanded use of wood in structures, and the forest industry is particularly interested in the utilization of timbers in bridge applications.

The **objective** of this study is an evaluation of the performance of wood decks as applied to British Columbia bridges, and an assessment vis-à-vis provisions in the CSA-S6-00 Bridge Code guidelines. As this Bridge Code is a Limit States Design document, the evaluation is carried out using a full reliability analysis of timber decks in British Columbia bridges, independently from S6 recommendations. The bridges analyzed here consist of a timber deck over two main steel girders. The deck consists of timber ties, one (or two) wood plank layers on top of the ties, and guard rails on either side of the deck. The work reported here corresponds to Phase 2, complementary to the work reported on March 30, 2010, under Ministry Contract EN0987A007.

The reliability analysis uses the following input items:

1) statistics for actual truck loadings in British Columbia,
2) a detailed structural analysis of the timber deck system,
3) timber strength and stiffness data from experimental tests on relevant BC species and grades,
4) definition of performance requirements and
5) software to calculate the structural reliability of the system, including guidelines for using the software for a generic truck.

This report includes items 1), statistical representation of actual truck loadings, 2) the development of a detailed structural analysis model, 4) and 5) the assessment of the structural reliability for different bridge construction configurations.

Test data, under Item 3, were not experimentally obtained as part of this work. The timber strength and stiffness data were taken from previous, existing information, and only in reference to Douglas fir timbers, in Select Structural (SS), No.1 and No.2 grades. These data apply only to bending properties. Data for shear strength and compression perpendicular to the grain were taken from information in the literature and were not grade-specific.

Reliability levels are estimated for three failure modes: 1) bending failure of one tie, 2) shear failure of one tie and 3) failure in compression perpendicular to the grain at the tie supports provided by the steel girders. For each of the three timber grades, a total of 8 bridge configurations/truck loadings were analyzed. Furthermore, the work assessed the influence of the nailing pattern used between the planks and the ties.

This Report includes software for the structural analysis of the deck system, DECK, developed specially for this project.

As discussed in the Conclusions, BC bridges were found to be sufficiently reliable regarding bending strength requirements. Checks using S6 design equations, when they imply the contrary, probably reflect gaps in the calibration for that Code. Of the three limit states considered, shear strength appears to be the weakest or controlling mode. The limit state of compression perpendicular to the grain appears to be the less likely mode of failure.

2. Truck loading data and their statistical representation.

Truck loading information was taken from three previous reports submitted by Buckland and Taylor Ltd. to the Ministry of Forests (1/2003, 11/2003 and 10/2004). These reports analyzed data based on scale surveys of logging truck vehicles in British Columbia.

The weight surveys were conducted by the Forest Engineering Research Institute of Canada (FERIC). The data were obtained in two main phases. Phase I (1/2003) contained information on 1) logging trucks generally conforming to the description of L75 loading and operating in the Interior region of BC, and 2) off-highway logging trucks (L150-L165) operating in Coastal regions. The data also included axle weight distributions (including side to side variations). Phase II used a much more extensive scale weight database for logging trucks operating off-highway in either the Interior or the Coastal regions of BC, including trucks which conformed to highway regulations. These data were provided by forestry companies on a confidential basis, with hidden company names and operating locations. A Phase III (10/2004) added additional data for the Coastal region.

For the purpose of the present study, the data were **analyzed first** obtaining Cumulative Distribution Functions (CDF) for the total truck weight GVW, in each of the five following groups: L75 and L150-165 from Phase I, Interior, Coastal and Highway from Phase II. The additional coastal data from Phase III were added to the coastal information from Phase II. All the coastal off-highway data corresponded to 5-axle trucks (as shown in Figures 1 through 3). All interior and highway data corresponded to 7-axle trucks (tri-axle drive/tri-axle trailer).

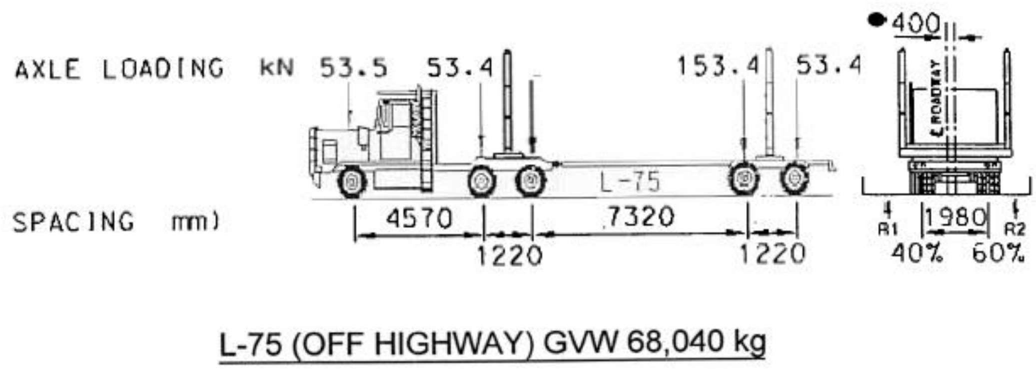


Figure 1

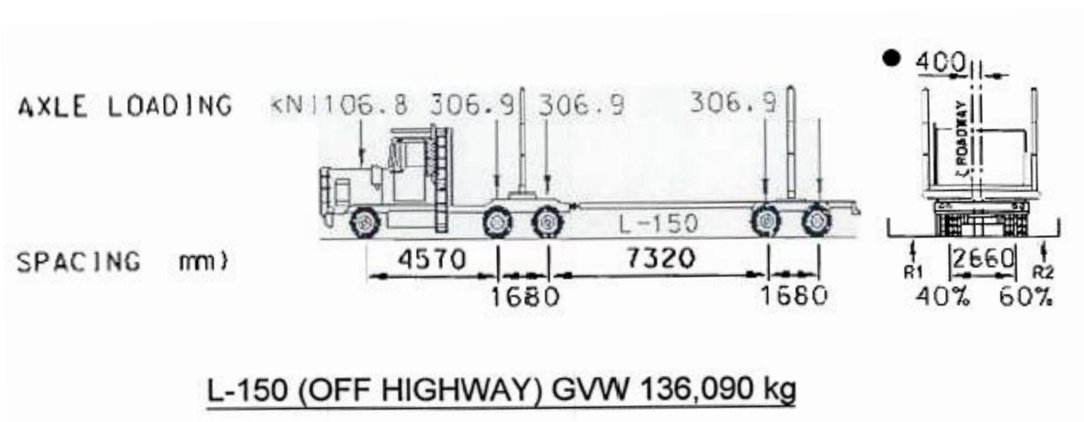


Figure 2

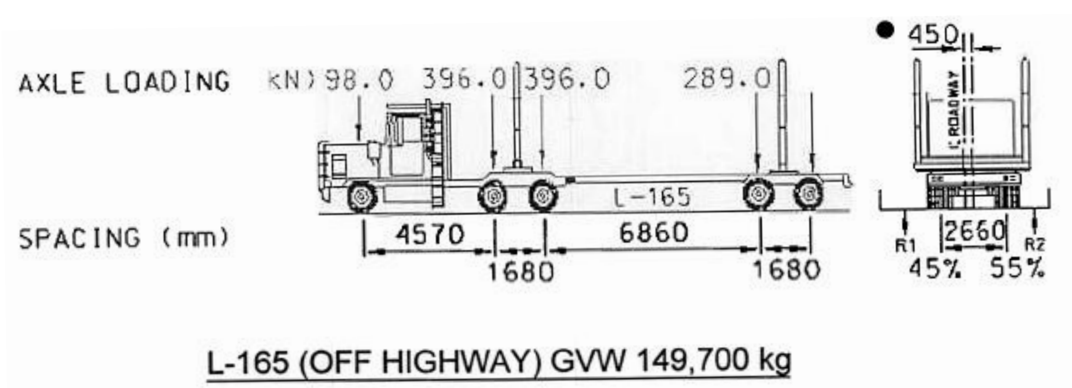


Figure 3

The number N of samples in each of the four truck load groups, collected over a period of time, were, respectively,

N = 123 for L75 (Phase I)
N = 78 for L150-165 (Phase I)
N = 82036 for Interior BC (Phase II)
N = 14055 for Coastal BC (Phase II + additional from Phase III)
N = 117 for Highway BC (Phase II)

Figure 4 shows the data CDFs corresponding to each of the five groups. This Figure implies that the sample from the L75 group in Phase I (corresponding to the Interior region) is quite consistent with the more extensive sample for the Interior BC Region from Phase II.

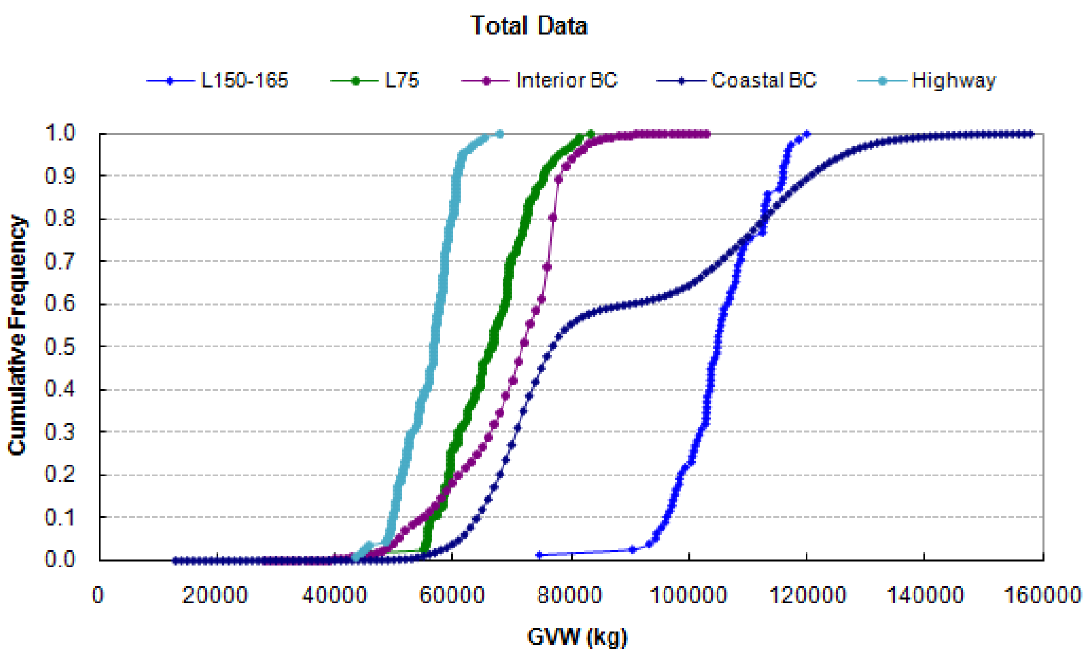
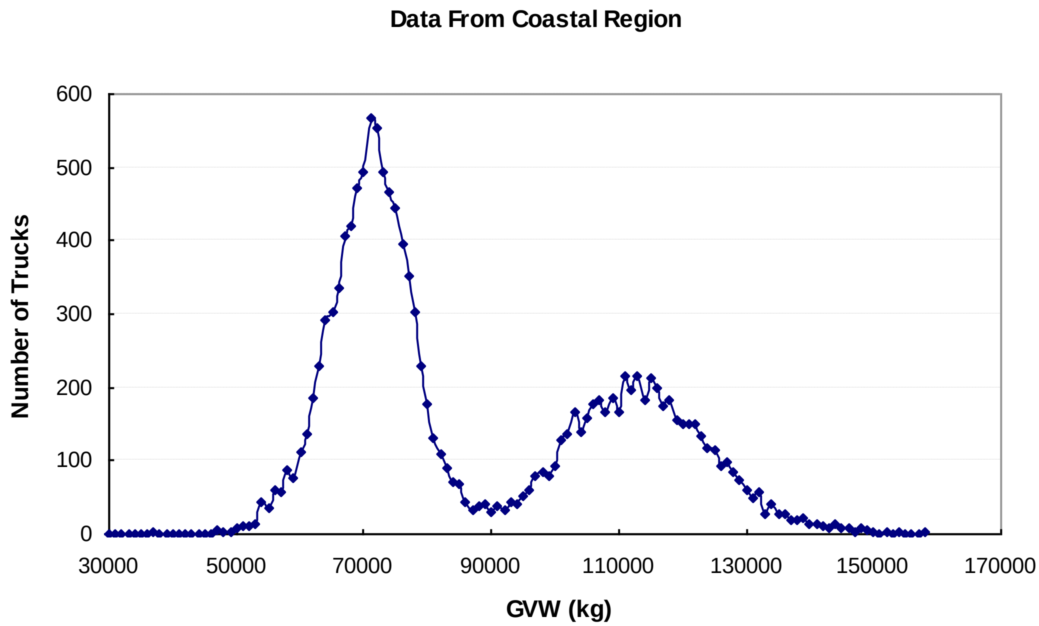


Figure 4. Cumulative Distribution Functions for GVW

The CDF for trucks meeting highway regulations, with the lighter GVW, is also shown in Figure 4, as well as the CDF for the heavier L150-165 data in Phase I. The Coastal region data appear to contain two components, one more consistent with L75 loading and another more with the L150-165 data. This is clearly shown in Figure 5, following. The first graph in Figure 5 is a histogram of the Coastal BC data, clearly showing two clusters. The second graph in Figure 5 is the corresponding CDF, also shown in Figure 4.



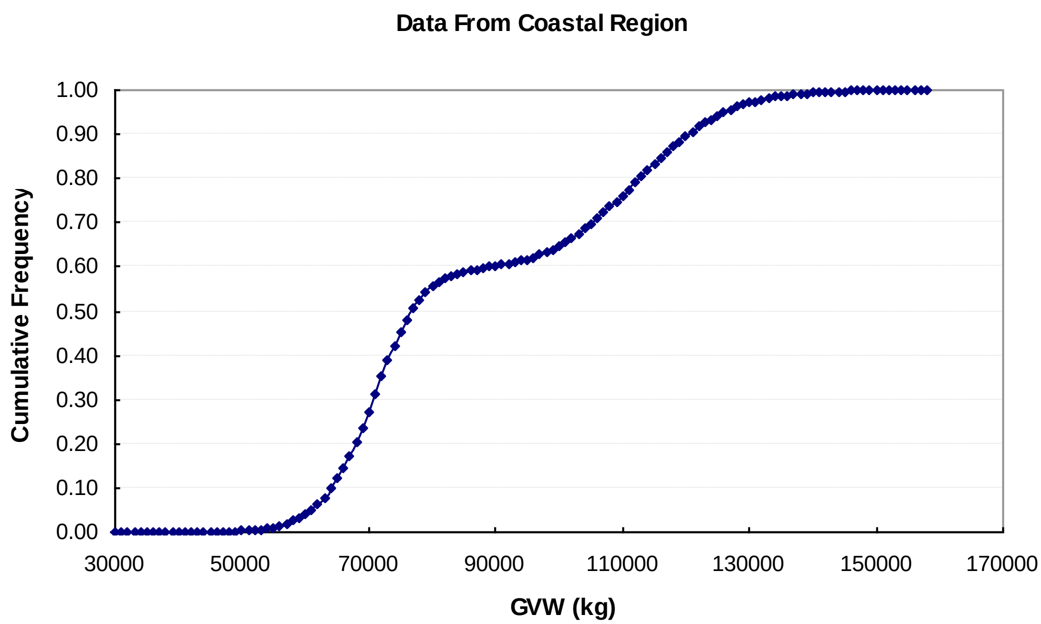


Figure 5. Coastal BC Data

For reliability studies, all the CDF data need to be given a mathematical representation. In each of the five cases, three distribution functions were considered: Normal, Lognormal and Gumbel (Extreme type I). The corresponding goodness of fit obtained with each one is shown, for each of the first four data groups, in Figures 6, 7, 8 and 9.

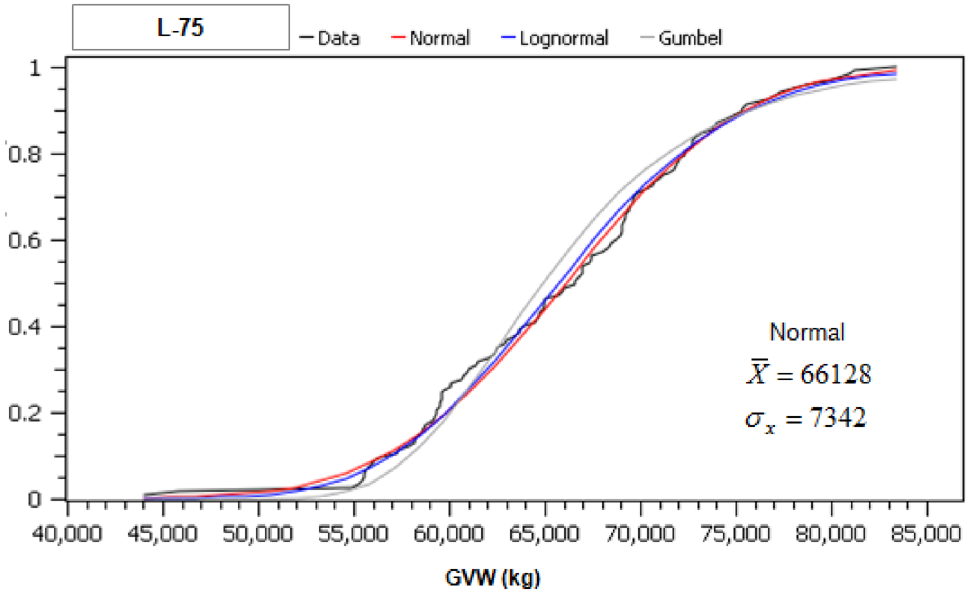


Figure 6. Representation of L75 data

It appears from Figure 6 that, for the **L75** data, a good fit is achieved with any one of the three distributions. For simplicity, the data can be represented with a *Normal distribution, with a mean value of 66,128 Kg , a standard deviation of 7,342 Kg and a corresponding coefficient of variation of 0.111 (11.1%).*

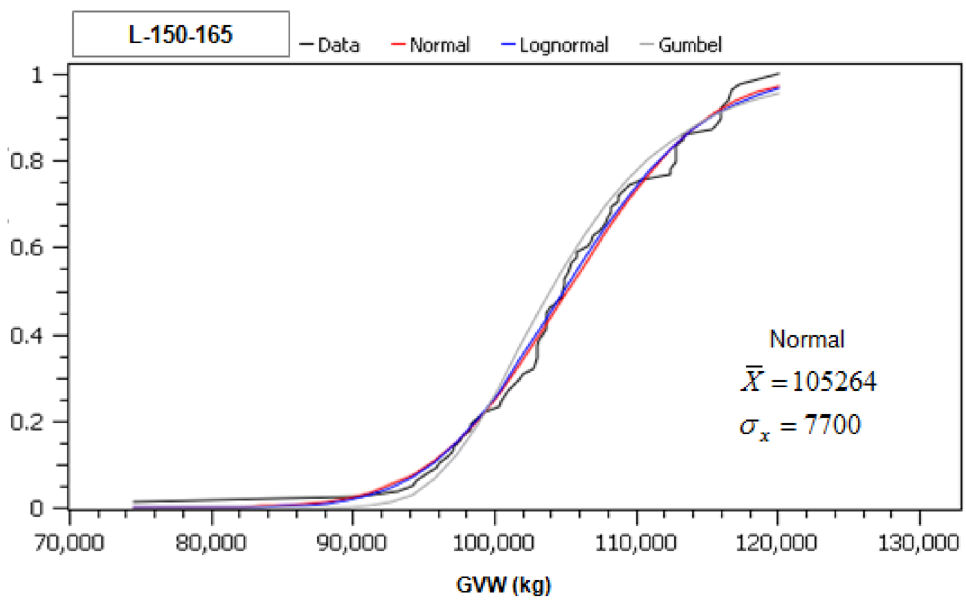


Figure 7. Representation of L150-165 data

Figure 7 shows the results for the heavier **L150-165** category. It appears, again, that a good fit is achieved with any of the three distributions and that, for simplicity, the data can be represented by a *Normal distribution, with a mean value of 105,264 Kg , a standard deviation of 7,700 Kg and a corresponding coefficient of variation of 0.073 (7.3%).*

The sample size in either Figure 6 or 7 is relatively small. On the other hand, Figures 8 and 9 correspond, respectively, to the much more extensive sample size in the Interior and Coastal BC categories.

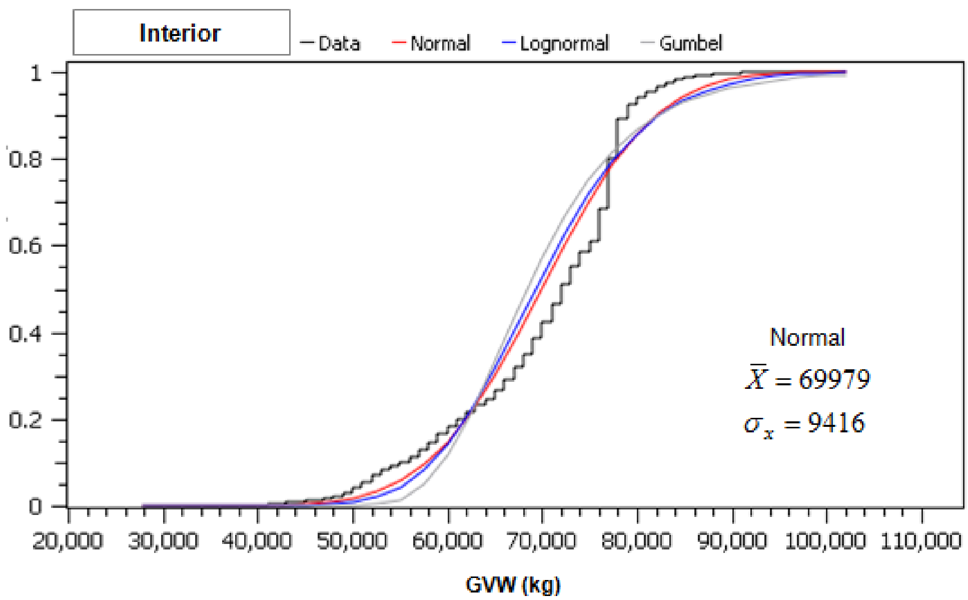


Figure 8. Representation of Interior BC data

Still, the goodness of fit achieved with either of the three distributions is quite similar, as shown in Figure 8, and the data for the **Interior BC** are reasonably represented by a *Normal distribution, with a mean of 69,979 Kg , a standard deviation of 9,416 Kg and a corresponding coefficient of variation of 0.135 (13.5%).*

Figure 8 is consistent with the L75 data in Figure 6, but the database for Figure 8 is much more extensive, with the consequence of a somewhat higher mean value and coefficient of variation.

Finally, Figure 9 shows the results for the Coastal BC data.

Given the clustering shown in Figure 5, the goodness of fit when using a single distribution for the entire range is not as good as in the previous cases. Still, a reasonably good representation can be achieved for **Coastal BC** with a *Normal distribution, with a mean of 88,106 Kg, a standard deviation of 22,783 Kg and a corresponding coefficient of variation of 0.259 (25.9%).*

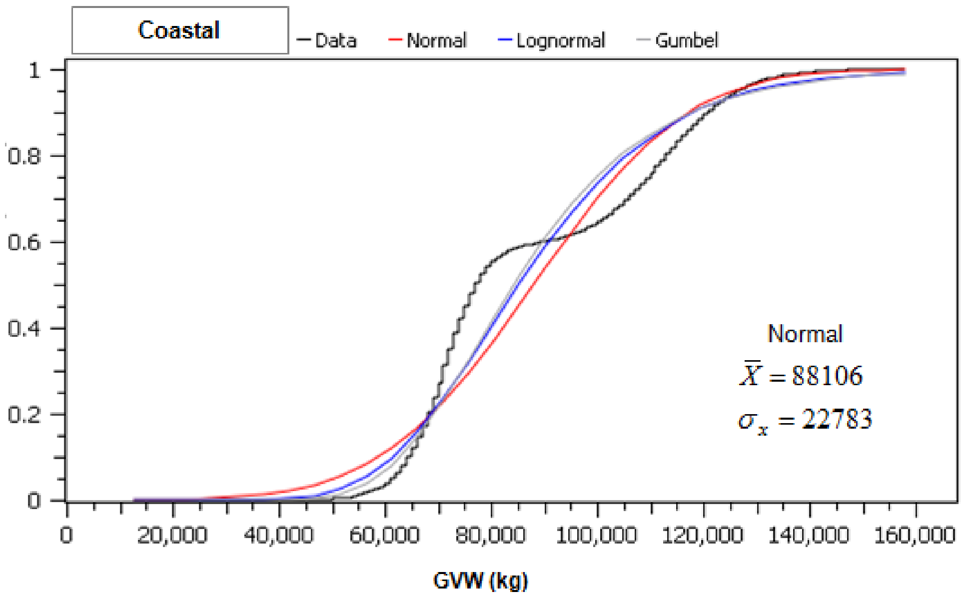


Figure 9. Representation of Coastal BC data

A **second analysis** was carried out. It consisted of considering only four groups: one with all the available off-highway Interior data, a second with all available off-highway Coastal data for lighter trucks, a third for all off-highway Coastal data for heavier trucks (GVW > 90,000 Kg) and a fourth for all highway-legal data. The corresponding CDF distributions were obtained and are shown in Figures 10,11 and 12 and 13.

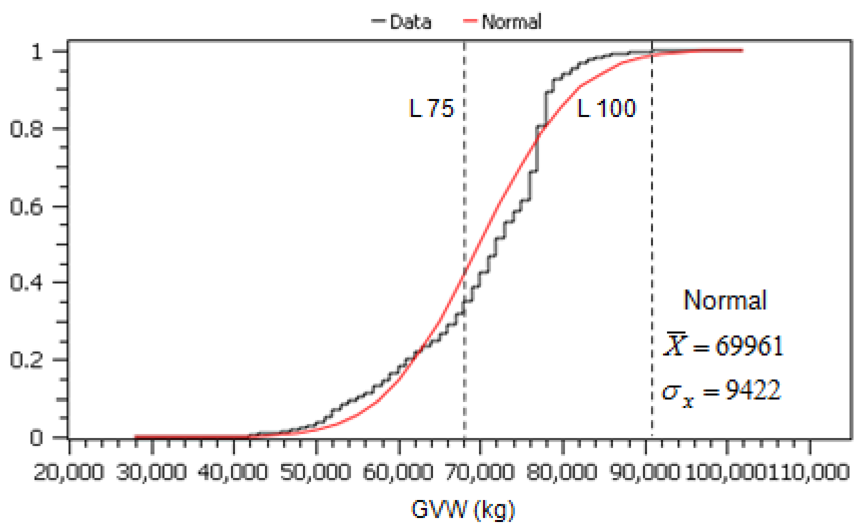


Figure 10. Interior data, 7- axle vehicles, Normal distribution

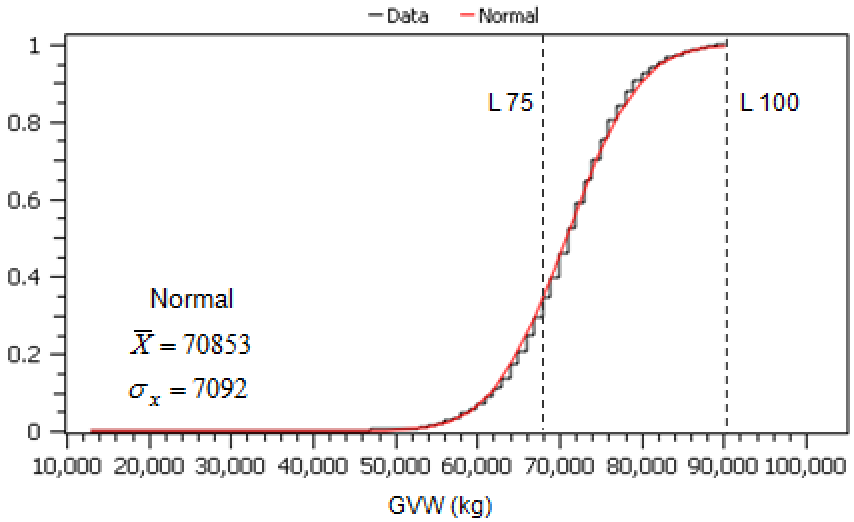


Figure 11. Coastal data, Lighter 7-axle vehicles, Normal distribution

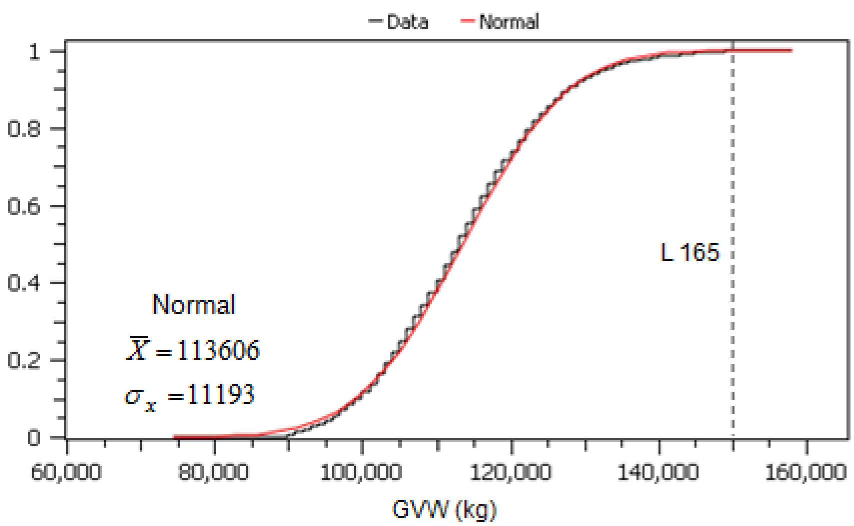


Figure 12. Coastal data, Heavier 5-axle vehicles, Normal distribution

It is apparent from a comparison of Figures 10 and 11 that the statistics for Interior and for the lighter Coastal trucks are very similar and that, for these cases, the L75 truck weight is exceeded with a probability of approximately 60%. On the other hand, the L100 truck weight is exceeded only with a small probability and can be considered an upper bound for these data.

Figure 12 corresponds to the heavier trucks in the Coast, or the second cluster in Figure 5. Figure 12 shows a very good representation using a Normal distribution, and that the L165 truck weight is exceeded with only a small probability and can be considered an upper bound for the data. Finally, Figure 13 shows the distribution and the Normal representation for highway-legal truck data.

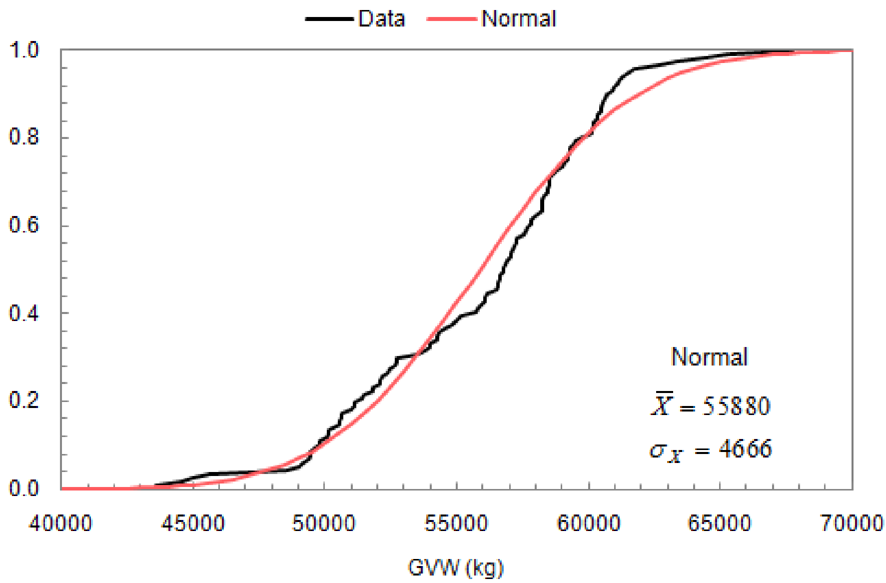


Figure 13. Highway data, 7-axle vehicles, Normal distribution representation

In what follows, reliability analyses will use the Normal representations shown for the four cases in Figures 10, 11, 12 and 13.

Data from Phase I also allowed the determination of the axle weight distribution. The ratios of axle group to total load are quite consistent across the databases for L75 or L150-165, allowing the simplified calculation of the axle loads as a mean ratio multiplier of the random total truck load.

Table 1 shows the mean ratios (in %) for the Steer, Drive and Trailer trains, both for the left side and the right side of the truck survey data. The steer includes one axle, while the drive and trailer groups include, depending on the case, two axles or three axles.

Table 1. Weight distribution, mean values (% of GVW)

L-75	Left				Right			
	Steer	Drive	Trailer	Total	Steer	Drive	Trailer	Total
	5.3%	24.3%	22.0%	51.6%	5.1%	23.0%	20.3%	48.4%

L-150-165	Left				Right			
	Steer	Drive	Trailer	Total	Steer	Drive	Trailer	Total
	5.2%	23.2%	21.2%	49.6%	5.4%	24.7%	20.3%	50.4%

Average	*5.3%*	*23.8%*	*21.6%*	*50.6%*	*5.2%*	*23.8%*	*20.3%*	*49.4%*

As shown in Table 1, it is possible to simplify the analysis by discounting the differences between the left and the right (unbalanced loads) and also the differences between the two types of trucks. Overall average values of the distribution coefficients are shown in the last line of Table 1. Previous analyses of wood decks have adopted an unbalanced distribution of 60%-40%, or a reduced unbalance of 55%-45%. The reliability analysis in this project utilizes the distribution coefficients obtained from the data shown in Table 1.

The left and right axle loads will be applied through tire footprints (or load patches). The width of the footprints will be as detailed in Phase I for L75 and L150-165 trucks, while the length of the footprints, in the longitudinal bridge direction, would be dependent on tire pressure and are assumed to be 0.30m for 7-axle trucks and 0.40m for heavier, 5-axle trucks.

For the structural and reliability analysis, the position of the complete truck will be determined by the random location coordinates x-y of a corner of the load patch for one of the steering tires, as shown in Figure 14. This random location will take into account the range of possible positions of the truck from side-to-side and along the bridge. The 5-axle truck includes a total of 10 load patches (2 for the steer, 4 each for the drive and trailer units). The 7-axle truck includes a total of 14 load patches (2 for the steer, 6 for each of the drive and trailer units). Given x and y, the position of each load patch is automatically determined by the separation distances between the axles and the tire groups. Further details on load patch specification are given in Section 7 of this Report.

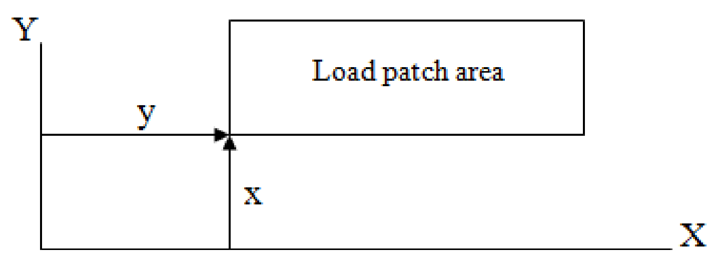


Figure 14. Load patch and coordinate system for its location

3. The structural analysis model

The wood deck system consists of timber ties, perpendicular to the main steel girders, and wooden planks running perpendicular to the ties. Up to two sets of planks may be used: deck planks resting on the ties, and running planks for the road surface.

Figures 15 and 16 show a schematic of the system. The ties are supported by steel girders and span a distance Δ, with cantilever sections of length Δ_C. The tie spacing is S, and their cross-sectional dimensions are B and H.

The deck and running planks have thickness T1 and T2. There are mechanical fasteners between the planks and between the deck planks and the ties. These fasteners may also be used with every other tie.

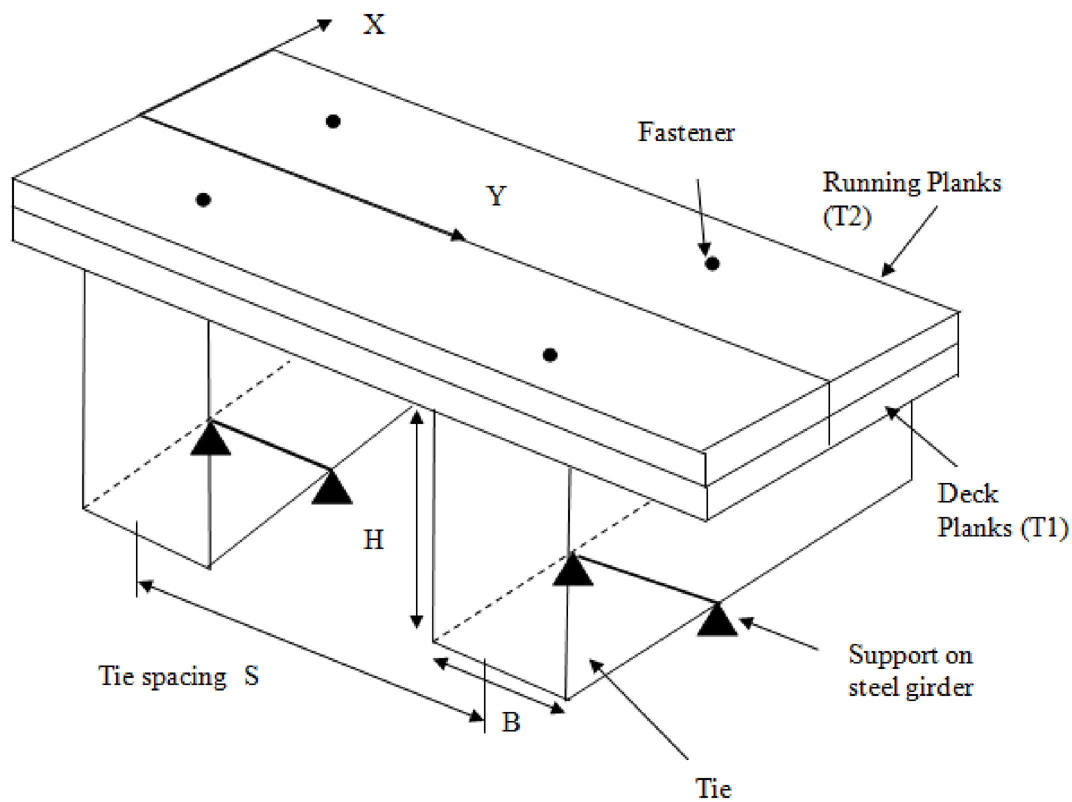


Figure 15. Tie spans and girder supports

The structure is modeled as beams (the ties), with a perpendicular plate of up to two layers (the planks). Under the transverse loads from the truck load patches, the deflection of the ties and the planks should also account for the influence of shear deformation. The planks form a plate assumed with bending stiffness in one direction only (Y), with no stiffness in the perpendicular direction (X) and no torsional stiffness.

The modulus of elasticity E for each of the ties varies randomly between ties, but obeys the same probability distribution. In general, the bending strength of a tie is positively correlated with its modulus of elasticity. Similarly, the modulus of elasticity E for the planks are random variables obeying corresponding probability distributions.

Each of the beams is modeled with a sequence of elements of length L, as shown in Figure 16. Within each element, the deflection w(x) is modeled with a cubic polynomial and, thus, the assumption includes four degrees of freedom per element: the deflection w_1 and rotation θ_1 at node 1 of the element, and the deflection w_2 and rotation θ_2 at node 2 .

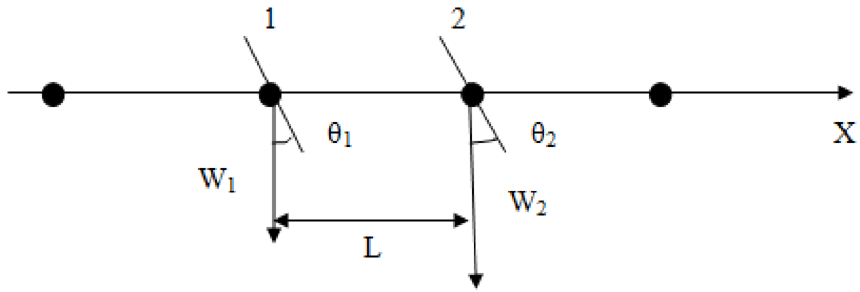


Figure 16. Beam elements and degrees of freedom

Each tie is subdivided into 6 elements, corresponding to 7 nodes. Each of the cantilever sections Δ_C contains one element, with 4 elements assigned to the main span Δ. With 2 degrees of freedom per node, the total of degrees of freedom (unknowns) for the system is

$$N_{DOF} = 14\, N_T \tag{1}$$

in which N_T is the number of ties included in the system. As the spacing of the ties is approximately S = 0.40m in BC bridges, and since the model must accommodate a nominal truck length of around 20m, the number of ties is adopted to be a maximum of N_T = 60, and the maximum number of unknowns (degrees of freedom) for the problem is, therefore, N_{DOF} = 840.

The cubic polynomial for the beam deflections within an element is

$$w(x) = \theta_1 \cdot \left(x + \frac{x^3}{L^2} - 2 \cdot \frac{x^2}{L} \right) + \omega_1 \cdot \left(1 - 3 \cdot \frac{x^2}{L^2} + 2 \cdot \frac{x^3}{L^3} \right) + \theta_2 \cdot \left(\frac{x^3}{L^2} - \frac{x^2}{L} \right) + \omega_2 \cdot \left(3 \cdot \frac{x^2}{L^2} - 2 \cdot \frac{x^3}{L^3} \right) \tag{2}$$

in which *L* is the element length and *x* varies, within the element, from 0 to *L*. On the other hand, the deflections *w*(*x*,*y*) of the planks need to match the deflections of the beams they join, and, between these beams, the plank deflection is also assumed to be a cubic polynomial:

$$\begin{aligned} w(x,y) = {} & \theta_1 \cdot \left(x + \frac{x^3}{L^2} - 2 \cdot \frac{x^2}{L} \right) \cdot \left(1 - 3 \cdot \frac{y^2}{s^2} + 2 \cdot \frac{y^3}{s^3} \right) + \omega_1 \cdot \left(1 - 3 \cdot \frac{x^2}{L^2} + 2 \cdot \frac{x^3}{L^3} \right) \cdot \left(1 - 3 \cdot \frac{y^2}{s^2} + 2 \cdot \frac{y^3}{s^3} \right) \\ & + \theta_2 \cdot \left(\frac{x^3}{L^2} - \frac{x^2}{L} \right) \cdot \left(1 - 3 \cdot \frac{y^2}{s^2} + 2 \cdot \frac{y^3}{s^3} \right) + \omega_2 \cdot \left(3 \cdot \frac{x^2}{L^2} - 2 \cdot \frac{x^3}{L^3} \right) \cdot \left(1 - 3 \cdot \frac{y^2}{s^2} + 2 \cdot \frac{y^3}{s^3} \right) \\ & + \theta_3 \cdot \left(x + \frac{x^3}{L^2} - 2 \cdot \frac{x^2}{L} \right) \cdot \left(3 \cdot \frac{y^2}{s^2} - 2 \cdot \frac{y^3}{s^3} \right) + \omega_3 \cdot \left(1 - 3 \cdot \frac{x^2}{L^2} + 2 \cdot \frac{x^3}{L^3} \right) \cdot \left(3 \cdot \frac{y^2}{s^2} - 2 \cdot \frac{y^3}{s^3} \right) \\ & + \theta_4 \cdot \left(\frac{x^3}{L^2} - \frac{x^2}{L} \right) \cdot \left(3 \cdot \frac{y^2}{s^2} - 2 \cdot \frac{y^3}{s^3} \right) + \omega_4 \cdot \left(3 \cdot \frac{x^2}{L^2} - 2 \cdot \frac{x^3}{L^3} \right) \cdot \left(3 \cdot \frac{y^2}{s^2} - 2 \cdot \frac{y^3}{s^3} \right) \end{aligned} \tag{3}$$

in which s is the spacing between ties. With this assumption, the deflections in the planks are only functions of the degrees of freedom of the beam elements they join, and no additional degrees of freedom are introduced.

Using these polynomial shape functions, the stiffness matrices corresponding to the tie (beam elements) and to the plates (planks) are obtained.

For each element of each tie, the stiffness matrix **K** is a 4x4 matrix as follows:

$$\mathbf{K} = 2\,\frac{E\,I}{L\,(1+g)}\cdot\begin{bmatrix} 2\cdot(1+0.25\,g) & \frac{3}{L} & (1-0.5\,g) & -\frac{3}{L} \\ & \frac{6}{L^2} & \frac{3}{L} & -\frac{6}{L^2} \\ & & 2\cdot(1+0.25\,g) & -\frac{3}{L} \\ sym & & & \frac{6}{L^2} \end{bmatrix} \qquad [4]$$

in which E is the modulus of elasticity for the wood in the tie, and I is the moment of inertia of the cross-section: $B\,H^3/12$. The constant g introduces the contribution from shear deformation, and it is related to the shear modulus G:

$$g_{beam} = 1.2\,\frac{E}{G}\left(\frac{H}{L}\right)^2 \qquad [5]$$

It is seen that shear deformations need to be taken into account, since the ratio E/G for wood is large (approximately 17) and the ties are relatively deep in relation to their span. Similarly, the assumed $w(x,y)$ for the deflections within one layer of the planks, between the corresponding beam elements, is used to calculate the stiffness matrix (8x8),

$$\mathbf{K} = \frac{E\,h^3}{420\cdot s^3\cdot(1+g)}\begin{bmatrix} 4\,L^3 & 22\,L^2 & -3\,L^3 & 13\,L^2 & -4\,L^3 & -22\,L^2 & 3\,L^3 & -13\,L^2 \\ & 156\,L & -13\,L^2 & 54\,L & -22\,L^2 & -156\,L & 13\,L^2 & -54\,L \\ & & 4\,L^3 & -22\,L^2 & 3\,L^3 & 13\,L^2 & -4\,L^3 & 22\,L^2 \\ & & & 156\,L & -13\,L^2 & -54\,L & 22\,L^2 & -156\,L \\ & & & & 4\,L^3 & 22\,L^2 & -3\,L^3 & 13\,L^2 \\ & & & & & 156\,L & -13\,L^2 & 54\,L \\ & & & & & & 4\,L^3 & -22\,L^2 \\ sym & & & & & & & 156\,L \end{bmatrix} \qquad [6]$$

in which E is the modulus of elasticity for the wood in the plank, h is the plank thickness (T1 or T2), s is the tie spacing, L is the beam element length, and g is the parameter that introduces the influence of shear deformations and the shear modulus G:

$$g_{plank} = 1.2 \frac{E}{G} \left(\frac{h}{s} \right)^2 \qquad [7]$$

The matrices above, plus the one corresponding to the performance of the mechanical fasteners between ties and planks, and between the two layers of planks, are arranged into a global stiffness matrix **K** **.** The matrices contributed by the fasteners are not shown here for brevity. If the vector of unknowns is **a** , and the vector of load actions is **R**, then the system of equations

$$\mathbf{K\,a} = \mathbf{R} \qquad [8]$$

is solved for **a,** after the appropriate support conditions have been introduced. The load vector **R** is obtained so as to be consistent with the position of the truck on the deck and with the deflection function used for the planks. The assembly of the global matrix **K** and the global load vector **R** was completed and programmed into the accompanying software DECK, so that the vector **a** (all the ties deflections and rotations) can be determined for any position of the truck on the deck and any distribution of modulus of elasticity *E* for the ties. Knowing the deflected shape of each of the ties, the maximum bending stress is calculated for each tie, and the overall maximum S_{bmax} for the deck is thus determined.

Similarly, the shear stresses $\tau(x)$ are obtained along the tie coordinate x and, following Foschi and Barrett (1976), the equivalent Weibull shear stress τ^* is calculated according to:

$$\int_V \tau^k \, dx = \tau^* \qquad [9]$$

in which V indicates the volume of the tie. The distribution of the shear stresses τ is assumed to be parabolic over the depth H of the tie. This procedure for shear stresses is also specified in the S6 Bridge Code. The Weibull shear stress τ^* is calculated for each tie and the overall maximum T_{max} for the deck is obtained. T_{max} is then used for comparisons against the benchmark shear strength of a unit volume under uniform shear (Foschi and Barrett, 1976). This shear formulation introduces the known size dependence of shear strength in wood.

The analysis also computes the support reactions for the ties bearing on the steel girders. The overall maximum reaction R_{max} is obtained and used in a comparison with the compression perpendicular to the grain capacity of the tie.

The structural analysis only considers the static loading from the truck. The dynamic effects are taken into account by using a dynamic amplification factor as specified in the Bridge Code S6.

4. Structural Analysis, sample results

4.1 Example 1, one load patch, GVW = 1000 kN, symmetric loading.

The program DECK was run for the following example:
50 ties, spaced 0.4m o.c.
Tie dimensions 0.20m x 0.30m
Cantilever span Δ_C = 0.9m, main span Δ = 3.0m
All ties with the same E = 10,000 x 10^3 kN/m^2
1 plank, thickness = 0.10m and E = 10,000 x 10^3 kN/m^2 (DECK can accept two planks, but this example considers only one)
Nail stiffness tie/plank = 1,500 kN/m , nail spacing 0.4m, nails at all tie/plank intersections
Applied load: One patch, 0.40m x 1.60m, applied at x = 2.025m and y = 0.4m
The x-y location of the patch puts it on a symmetric position across the width of the deck.

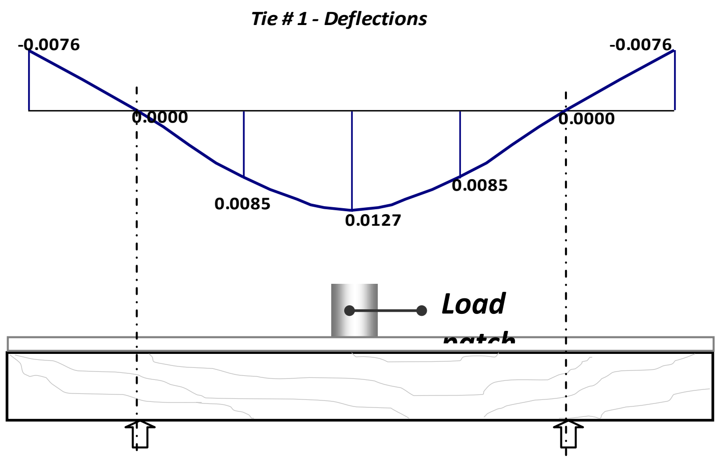


Figure 17. Deflections, Example 1.

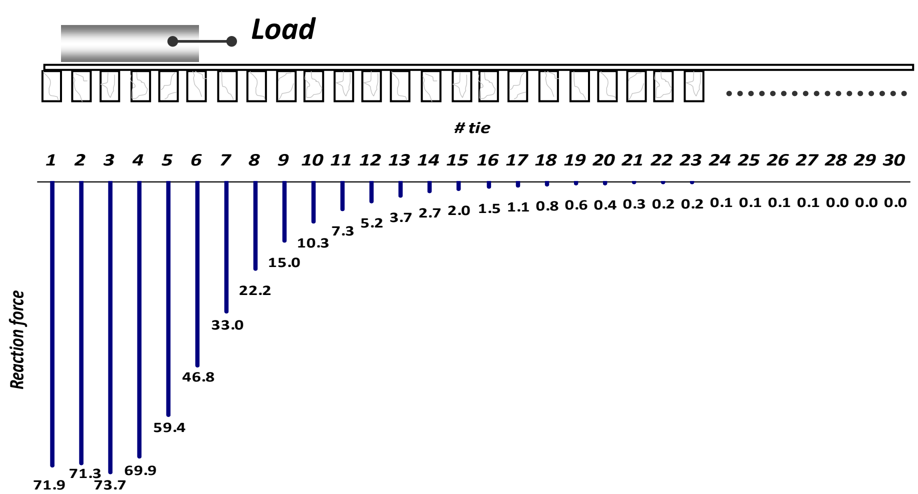


Figure 18. Support reactions, Example 1.

Figure 17 indicates that the deflections from the DECK solution are symmetric, as corresponds to the imposed patch loading. Further, the reactions in Figure 18 show that the load spreads over several ties (the patch spans from tie #2 to tie #6), with significant load sharing.

4.2 Example No. 2, 5-axle truck, GVW = 100000kg (1000kN), non symmetric loading.

The program DECK was run for the following second example:

50 ties, spaced 0.4m o.c.
Tie dimensions 0.20m x 0.30m
Cantilever span Δ_C = 0.64m, main span Δ = 3.6m
All ties with the same E = 10,000 x 10^3 kN/m^2
1 plank, thickness = 0.10m and E = 10,000 x 10^3 kN/m^2
Nail stiffness tie/plank = 1,500 kN/m , nail spacing 0.4m, nails at all tie/plank intersections
Applied load: 5-axle truck, 10 load patches. The dimensions of the patches for the steering axle are 0.33m x 0.40m, while those for the drive and the trailer are 0.77m x 0.40m. Referring to Figure 14, the coordinates of the front wheel patch are x = 1.47m and y = 0.40m. This x-y location puts the truck in an non-symmetric position across the width of the deck.
The length of the deck segment with 50 ties @ 0.406m o.c. is 19.89m. The coordinates x and y may be changed within the following allowable limits: 0.0m < x < 1.57m (these limits are given by the truck touching the curbs) and 0.0m < y < 3.69m, for the length of the truck to be contained within the segment.
Figure 19 shows the un-symmetric deflection of one tie (#1), and Figure 20 the distribution of the reactions both for the less loaded and the more loaded bridge edge. The sum of all the reactions equals 1000 kN, the total GVW of the truck.

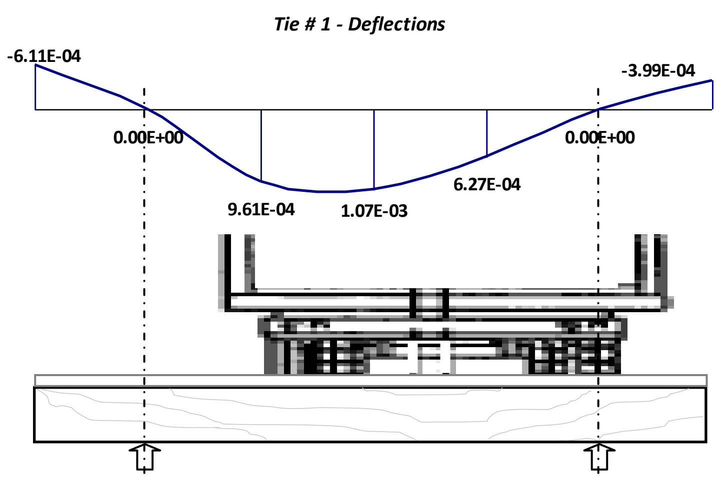


Figure 19. 5-axle truck, non-symmetric

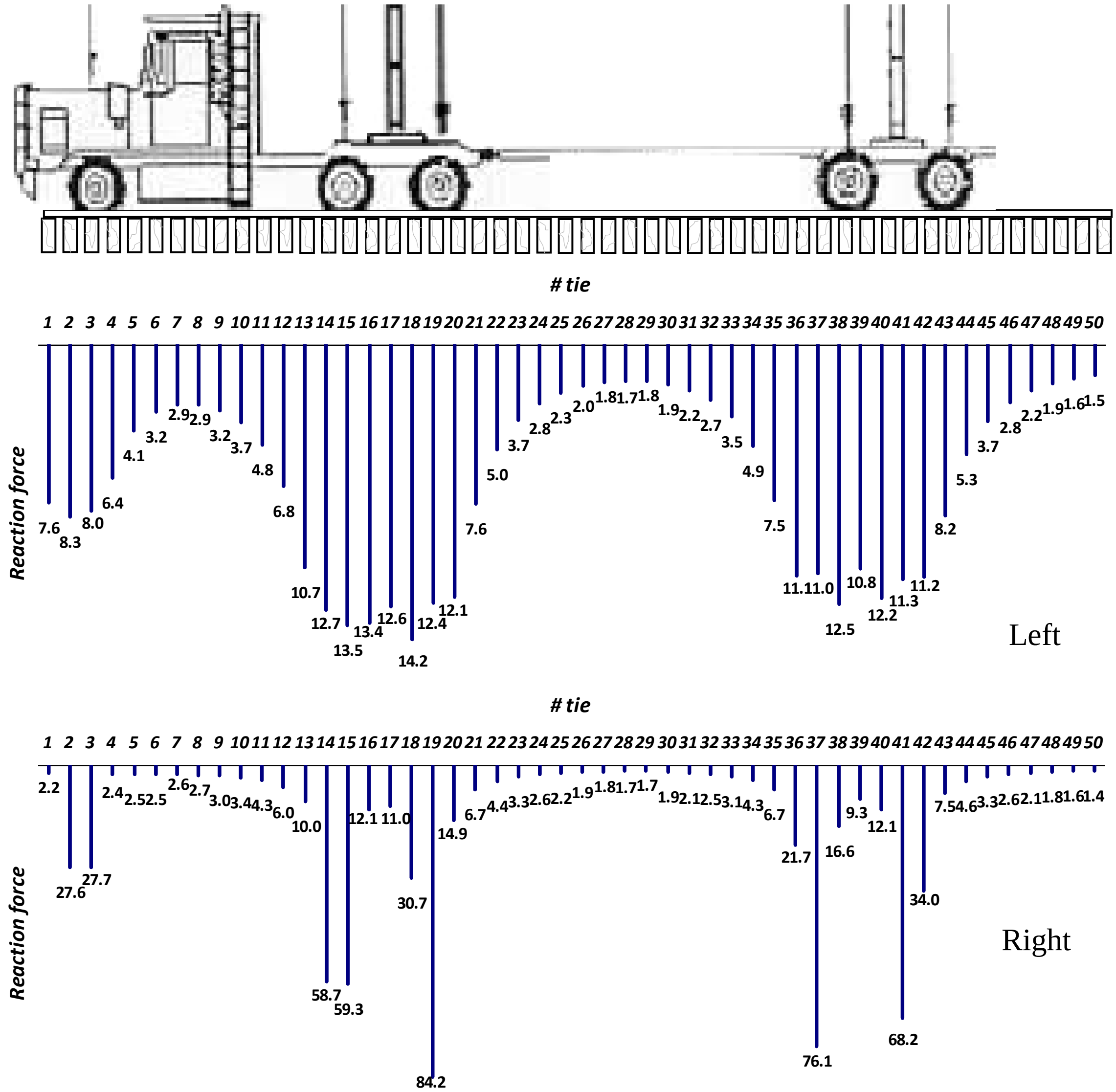


Figure 20. 5-axle truck, GVW = 100000kg (1000kN), reactions for non-symmetric loading

Figure 20 shows that, as expected, the maximum reactions correspond approximately to the location of the axles. Furthermore, the analysis produced the following results:

Overall maximum bending stress S_{bmax} = 8.82 MPa = 8820 kN/m^2
Overall maximum reaction R_{max} = 84.2 kN
Overall maximum tie deflection w_{max} = 0.002m

The software DECK operates very quickly, even for this case of 10 patches and 50 ties (700 unknowns). The accuracy of the analysis has also been validated for trucks with 7 axles (or 14 load patches). The speed of the software is a requirement when implemented in the reliability analysis, as these calculations require repeated calls to the structural evaluation.

DECK produces an output file, the name of which can be chosen arbitrarily. Appendix A, at the end of this Report, shows an example of output file named DECK OUT. It contains the deflections at each of the 7 nodes, the left and right reactions, the maximum bending stress and the Weibull shear stress τ^*. Finally, a summary is shown for the results over the total number of ties. The Appendix shows a summary for 50 ties, although individual tie results, for brevity, are only shown for up to tie No. 3.

5. Reliability Analysis

The reliability analysis included consideration of 3 failure modes involving 56 different random variables (when using 50 ties). When a maximum of 60 ties were used, the number of random variables was 66.

5.1 Random variables

The 56 variables, for 50 ties, were:

X(1) – X(50) the modulus of elasticity *E* for the ties. These were different variables but assumed to obey the same probability distribution. This was justified on the assumption that all ties would come from the same stock. The probability distribution chosen was a Lognormal.

X(51) the bending strength for the ties. This variable was assumed to obey a 2-parameter Weibull distribution, based on experience from testing dimension lumber in bending.

X(52) coordinate X for the location of the truck, assumed to be uniform between limits controlled by the distance between curbs and the overall width of the truck.

X(53) coordinate Y for the location of the truck along the bridge, also assumed to be uniform between the limits controlled by the length of the deck segment considered and the length of the truck.

X(54) the GVW of the truck, as the ratio between the actual GVW and 1000kN, the load used for the structural analysis. This variable, from Section 2 of this report, is taken to be Normally distributed.

X(55) the shear strength of the wood in the tie, given for a unit volume ($1m^3$) under uniform shear. Following Foschi and Barrett (1976), this variable follows a 2-parameter Weibull distribution. For Douglas fir, the scale parameter of this distribution is m = 2,540 kN/m^2, with a shape parameter k = 5.3.

X(56) the compression perpendicular strength of the wood in the tie (following published recommendations by Blass and Gorlacher (2004)). From these data, this variable is assumed Lognormally distributed, with a mean of 3,000 kN/m^2 and a coefficient of variation of 20%.

5.2 Performance functions

Three limit states or performance functions *G* were considered:

1. Bending failure:

$$G = X(51) - (X(54)/1000.0)\ f_i\ S_{b\max} \qquad [10]$$

in which S_{bmax} is the maximum overall bending stress from the DECK structural analysis, using a GVW of 1000kN, and f_i is an impact coefficient.

2. Shear failure:

$$G = X(55) - (X(54)/1000.0)\ f_i\ \mathrm{T}_{\max} \qquad [11]$$

in which T_{max} is the maximum Weibull stress computed according to Eq.[9] using the results from the DECK structural analysis.

3. Failure in compression perpendicular:

$$G = X(56)\,A - (X(54)/1000.0)\ f_i\ R_{\max} \qquad [12]$$

in which R_{max} is the maximum overall support reaction from the DECK structural analysis and *A* is the area of contact at the support (the product of tie width and girder flange width).

5.2 Calculating the reliability index β for each limit state (or failure mode)

The calculation of the reliability index β (and associated probability of failure) was carried out with an update of the general software RELAN (Foschi, 2010), into which the three performance functions from Section 5.1 were implemented. To arrive at the results presented here, RELAN carried out the calculation of β, first with FORM (First Order Reliability Method) and then, whenever not quickly converging, FORM was supplemented by Importance Sampling Simulation, with a sample size of 20000. For each mode, the FORM algorithm finds out the combination of the variables most likely to result in failure. As a consequence, the FORM method finds out automatically the coordinates x and y (variables X(52) and X(53)) giving the worst position for the truck.

As shown by the description of the random variables and performance functions, the reliability software can work with a nonlinear combination of multiple variables, each with a different type of probability distribution. This capability is one of the differences between the present method and the one adopted in the calibration of the S6 Code.

5.3 Reliability results

Results were obtained for eight scenarios, each corresponding to a different combination of truck loading, bridge configuration and tie spacing. The eight scenarios are shown in Table 2 below.

Table 2. Scenarios considered for reliability analysis

Scenario	Truck data	No. of axles	Tie Spans (m) (*)	Tie dimensions (mm)	Tie spacing (mm)
1	Interior	7	4.30 / 3.00	200 x 250	406
2	Interior	7	4.88 / 3.60	200 x 250	406
3	Coastal	5	4.88 / 3.60	250 x 300	406
4	Highway	7	4.30 / 3.00	200 x 250	406
5	Highway	7	4.88 / 3.60	200 x 250	406
6	Interior	7	4.30 / 3.00	200 x 300	406
7	Interior	7	4.88 / 3.60	200 x 300	406
8	Coastal	5	4.88 / 3.60	250 x 300	305
9	Coastal	5	4.88 / 3.60	250 x 300	406 (#)

(*) Total tie length / main span
(#) Nails used every other tie.
Impact coefficient: 1.20
Only one plank: thickness = 0.10m (100mm x 300mm), E = 10,000.0E+03 kN/m^2
Nails tie/plank : stiffness = 2,600 kN/m , spacing 0.30m o.c

Table 3. Bending Strength Characteristics, Douglas fir timbers

Douglas fir Grade	Mean MOE (MPa)	COV MOE (%)	5% MOR (MPa)
Select Structural (SS)	13,600	15.0	32.6
No.1	13,000	15.0	25.3
No.2	13,000	19.0	23.8

The values shown in Table 3 are consistent with the limited data range available for timbers. A testing report by Borg Madsen (1982) shows average modulus of elasticity for 8” x 12” Douglas fir to be (with a small sample size) 1.97 x10^6 psi and 1.65 x 10^6 psi for, respectively, Select Structural and No. 1 grades. These values are equivalent to, respectively, 13,586 kN/m^2 and 12,379 kN/m^2. The same report, for the same timbers, also shows 5^{th}-percentiles for the bending strength, with 4,730 psi for Select Structural and 3,670 psi for No. 1. These values are equivalent to, respectively, 32.62 and 25.3 MPa. Properties for Douglas fir No.2 were obtained from more recent testing done at the University of British Columbia on 12” x 12” timbers. Bending strength of these timbers were obtained only for the lower tail of the corresponding distribution, using a proof-loading procedure, with results as shown in Figure 21.

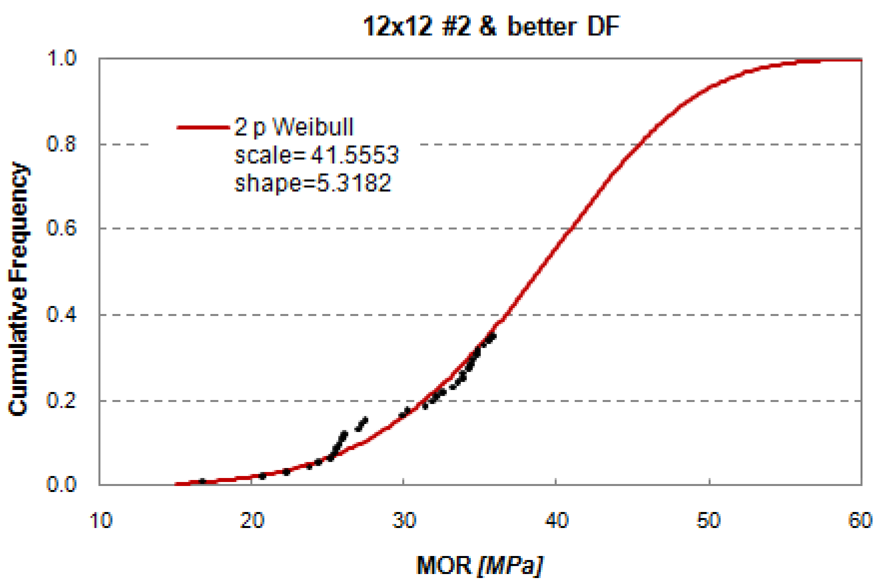


Figure 21. Test results, bending strength of No.2 Douglas fir, 12”x12”

Results for the reliability index β are listed in Table 4, for all nine scenarios described in Table 2. For each scenario, results are given for three different grades for Douglas fir timbers, for a total of 27 cases. Reliability indices for shear and compression perpendicular are based on properties estimated from the literature, and do not discriminate as to the grade of the timber.

Table 4. Reliability Results

Scenario 1	Bending	Shear	Compression perpendicular
DF SS	3.5	3.0	3.9
DF No.1	3.1	3.0	3.9
DF No.2	3.0	3.0	3.9

Scenario 2	Bending	Shear	Compression perpendicular
DF SS	3.5	3.2	3.5
DF No.1	3.1	3.2	3.5
DF No.2	3.0	3.2	3.5

Scenario 3	Bending	Shear	Compression perpendicular
DF SS	3.3	2.4	3.0
DF No.1	2.8	2.4	3.0
DF No.2	2.6	2.4	3.0

Scenario 4	Bending	Shear	Compression perpendicular
DF SS	4.1	3.4	4.2
DF No.1	3.5	3.4	4.2
DF No.2	3.4	3.4	4.2

Scenario 5	Bending	Shear	Compression perpendicular
DF SS	3.9	3.5	4.6
DF No.1	3.5	3.5	4.6
DF No.2	3.4	3.5	4.6

Scenario 6	Bending	Shear	Compression perpendicular
DF SS	3.6	2.7	3.2
DF No.1	3.2	2.7	3.2
DF No.2	3.1	2.7	3.2

Scenario 7	Bending	Shear	Compression perpendicular
DF SS	3.6	2.9	3.4
DF No.1	3.2	2.9	3.4
DF No.2	3.1	2.9	3.4

Scenario 8	Bending	Shear	Compression perpendicular
DF SS	3.6	2.7	3.7
DF No.1	3.2	2.7	3.7
DF No.2	3.1	2.7	3.7

Scenario 9	Bending	Shear	Compression perpendicular
DF SS	3.2	2.4	3.0
DF No.1	2.8	2.4	3.0
DF No.2	2.6	2.4	3.0

6. Guidelines for Implementation of the Methodology with Alternative Vehicle Configurations in the Software DECK

The methodology presented in this report, and implemented in the software DECK, is completely general. The results presented in the previous sections correspond to the two default truck configurations taken from the three previous reports submitted by Buckland and Taylor Ltd. to the Ministry of Forests (1/2003, 11/2003 and 10/2004), which were also used for the truck weight statistics. The structural analysis program DECK can be run in two modes: 1) reading data from a pre-existing data file or 2) entering the data manually while creating a data file for later use. The data file could be given an arbitrary name.

When changing a vehicle configuration, apart from a possibly different number of axles, the only data that need to be changed are the distance between axles and the distribution of total weight between the load patches or tire footprints. The required information is shown schematically in Figure 22.

A point O, which could be a corner of one of the front wheels or load patches, is used as the origin of a local system of coordinates X, Y to refer the position of the other load patches, given by the points P (typical) corresponding to each one. The position of point

O, in the global coordinates X, Y shown in Figure 15, is used to fix the position of the entire truck (as per Figure 14).

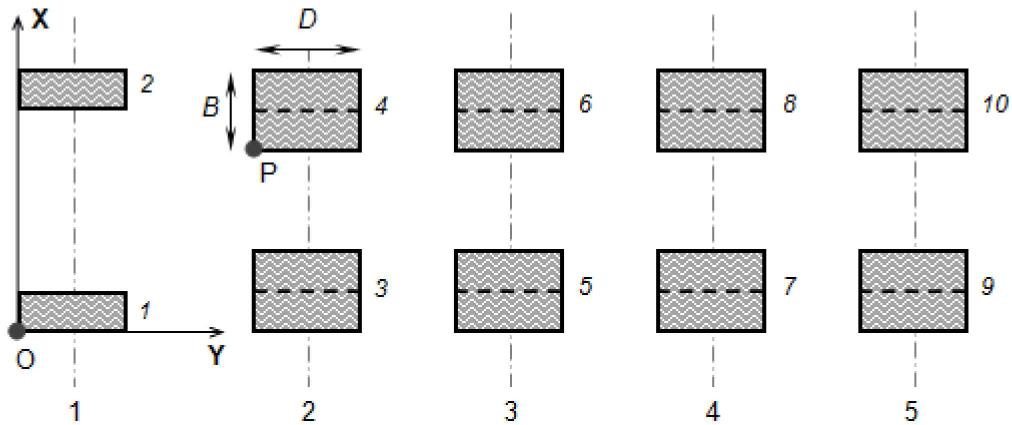


Figure 22. Data for alternative vehicle configurations

DECK asks for the coordinates of each point P, or the distances X and Y from the chosen point O. These coordinates are asked in turn for each patch during the manual construction of the data file.

In addition to the total GVW, the following additional data are required for each patch: 1) the dimensions of the footprint, B and D shown in Figure 22, and 2) the percentage of the total vehicle weight (GVW) corresponding to each particular patch.

7. Conclusions

This project has focused on the development of a structural analysis for the wood deck, coupling it with a reliability analysis under either heavier, coastal truck loads, or lighter, interior trucks, or highway-legal vehicles.

The reliability assessment considered three limit states (or failure modes) for the ties: failure by bending stresses, failure related to shear, and failure related to compression perpendicular to the grain stresses due to bearing of the ties on the support girders

The reliability assessment corresponding to nine different bridge scenarios was made independently from recommendations in the Canadian Highway Bridge Code S6.

The data on bending stiffness and strength for Douglas fir timber, Select Structural or No.1 grades, were consistent with the limited timber data collected by Borg Madsen, at UBC, in 1982. Data for No.2 grade were obtained from more recent experimental testing at UBC using 12” x 12” timbers, and were considered to provide a good approximation for the tie sizes considered in this study. Shear strength incorporated size effects as detailed by Foschi and Barrett in 1976, a procedure which is already used in the Canadian Code CSA-086 for wood structures. Compression perpendicular to the grain data were taken from the literature (Blass, 2004). These data show wide scatter depending on the testing configuration. Nevertheless, the values for compression perpendicular used in this report are a reasonable lower bound from the test results.

It can be concluded from Table 4 that the bending reliability indices associated with the BC bridge configurations studied, under realistic (measured) truck loads, are satisfactory and consistent with the aims of S6, particularly since the consequence of one tie failing does not imply the collapse of the entire deck. Although the comprehensive method used here for estimating reliability differs substantially from the simplified approach adopted in S6 (this makes it rather difficult to compare reliability results), BC bridges appear to be sufficiently reliable, and S6 design checks that imply the contrary probably reflect gaps in the calibration procedure for that Code. The lowest reliability levels in bending occur for Scenario 3, grades No.1 and No.2, corresponding to the heavier coastal trucks, with 5 axles, a longer main span of 3.6m, and ties with dimensions 250 x 300mm spaced 0.406m. A reduction in tie spacing to 0.305m (Scenario 8) is sufficient to bring the reliability levels up to the levels for the other configurations considered.

Of the three limit states, shear strength appears to be the less reliable. However, the Weibull model used for shear strength could be conservative (it is based on full brittle behavior), so that a calculated β could be lower than the actual one. Table 4 also show that, as far as shear is concerned, the heavier, 5-axle truck configuration is more demanding than the 7-axle trucks. Although the calculated reliabilities for the shear mode are considered to be reasonable, the lower reliabilities in shear indicate that a new timber testing program should include an assessment of shear strength and the monitoring of end cracks which would affect shear capacity.

Compression perpendicular to the grain appears to be the less likely mode of failure and does not control the performance of the bridges.

Finally, scenarios 1 through 8 assumed that the planks were nailed to every tie. Scenario 9, on the other hand, considered Scenario 3 but with the planks nailed only every other tie. The results for Scenario 9 indicate that such reduced nailing pattern does not result in a reduction in reliability.

The methodology, based on the general analysis program DECK, can be applied to any truck configuration after entering the appropriate number of axles, the distances between axles, the total vehicle weight and the distribution of this weight among all the load patches.

8. References

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Ricardo O. Foschi, P.Eng
Professor Emeritus
Civil Engineering
University of British Columbia
Vancouver, BC

Appendix

The output file can be given any arbitrary name, entered at the time of running the program DECK. In the following example, the file is called DECK OUT.

Example of output file DECK OUT, produced by the structural analysis DECK:

```
TIE #  1
 DEFLECTIONS:

   -.61082E-03    .00000E+00    .96066E-03    .10670E-02
    .62656E-03    .00000E+00   -.39921E-03

 REACTION 1=    .76162E+01   REACTION 2=    .21983E+01


 SMAX=    .36821E+04   TMAX=    .22753E+03


 TIE #  2
 DEFLECTIONS:

   -.59912E-03    .00000E+00    .11235E-02    .10630E-02
    .62091E-03    .00000E+00   -.39725E-03

 REACTION 1=    .82832E+01   REACTION 2=    .27617E+02


 SMAX=    .51703E+04   TMAX=    .34751E+03


 TIE #  3
 DEFLECTIONS:

   -.55836E-03    .00000E+00    .10696E-02    .10098E-02
    .59565E-03    .00000E+00   -.37932E-03

 REACTION 1=    .79770E+01   REACTION 2=    .27672E+02


 SMAX=    .49405E+04   TMAX=    .33409E+03
```

OVERALL MAX. DEFLECTION = .20414E-02
OVERALL MAX. BENDING STRESS = .88239E+04
OVERALL MAX. SHEAR WEIBULL STRESS = .84194E+03
OVERALL MAX. REACTION = .84234E+02
SUM OF REACTIONS, EQUILIBRIUM CHECK = .10000E+04