Ministry Contract No. EN0987A007

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

Final Report

by

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March 30, 2010

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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 wood decks as applied to British Columbia bridges, and an assessment vis-à-vis 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 reliability analysis requires the following inputs:

1) statistics for actual truck loadings in British Columbia,
2) a detailed structural modeling of the timber deck system,
3) timber strength and stiffness data from experimental tests on relevant BC species, and
4) software to calculate the structural reliability.

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

Test data under Item 3 are not included in this Report, as an experimental program is planned to start after April 1, 2010. In the absence of such data, this Report includes bridge reliability assessments for different levels of timber strength and stiffness, thus providing information that can be used when actual data become available.

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.

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

As discussed in the Conclusions, BC bridges are sufficiently reliable regarding bending strength requirements. Checks using S6 design equations, when implying the contrary, probably reflect gaps in the calibration for that Code. Of the three limit states considered, shear strength appears to be the controlling mode. Compression perpendicular to the grain stresses appear to be the less likely mode of failure.

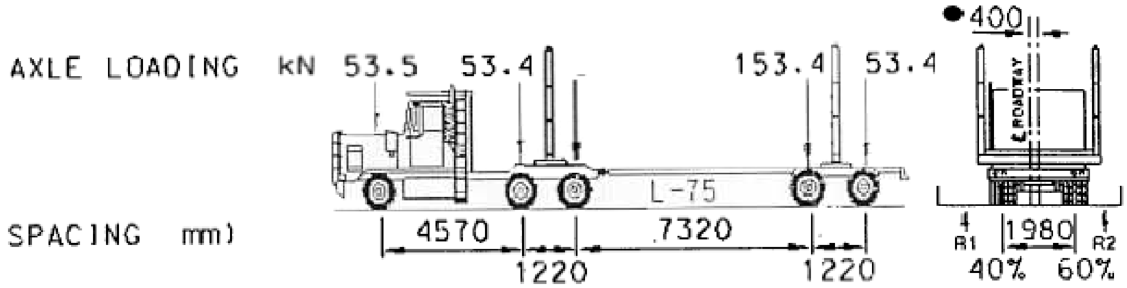
2. Truck loading data and their statistical representation.

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 amount of weight scale data for logging trucks operating in either the Interior or the Coastal regions of BC. 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.

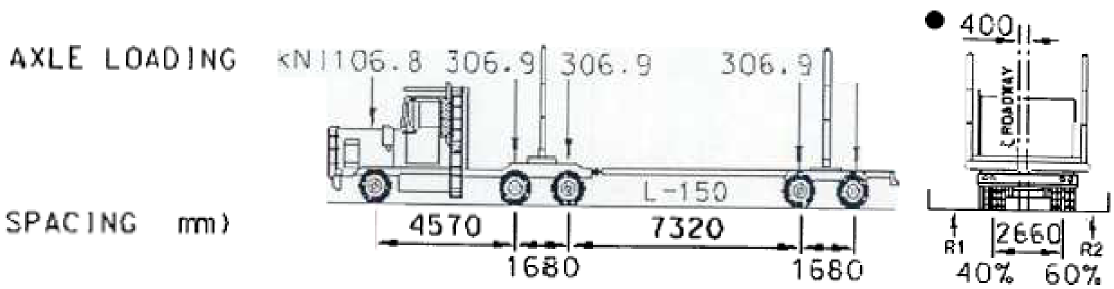
For the purpose of the present study, the data were **analyzed first** obtaining Cumulative Distribution Functions (CDF) for the total truck weight GVW, in each of the four following groups: L75 and L150-165 from Phase I, Interior and Coastal from Phase II. The additional coastal data from Phase III 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 data corresponded to 7-axle trucks (tri-axle drive/tri-axle trailer).



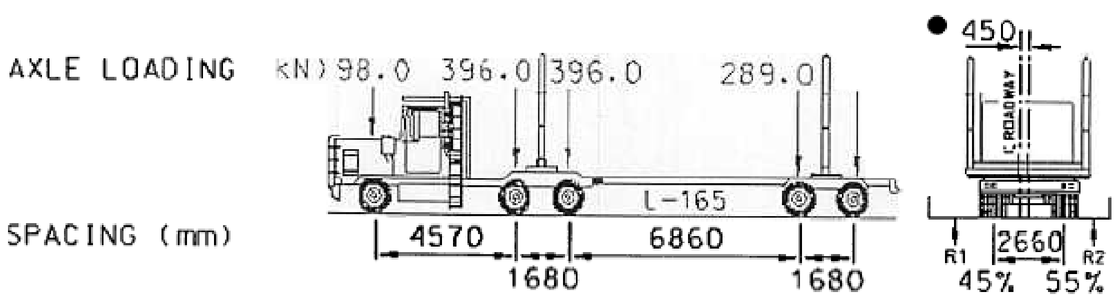
L-75 (OFF HIGHWAY) GVW 68,040 kg

Figure 1



L-150 (OFF HIGHWAY) GVW 136,090 kg

Figure 2



L-165 (OFF HIGHWAY) GVW 149,700 kg

Figure 3

The number of samples N 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)

Figure 4 includes the CDFs corresponding to each of the four groups. This Figure shows 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.

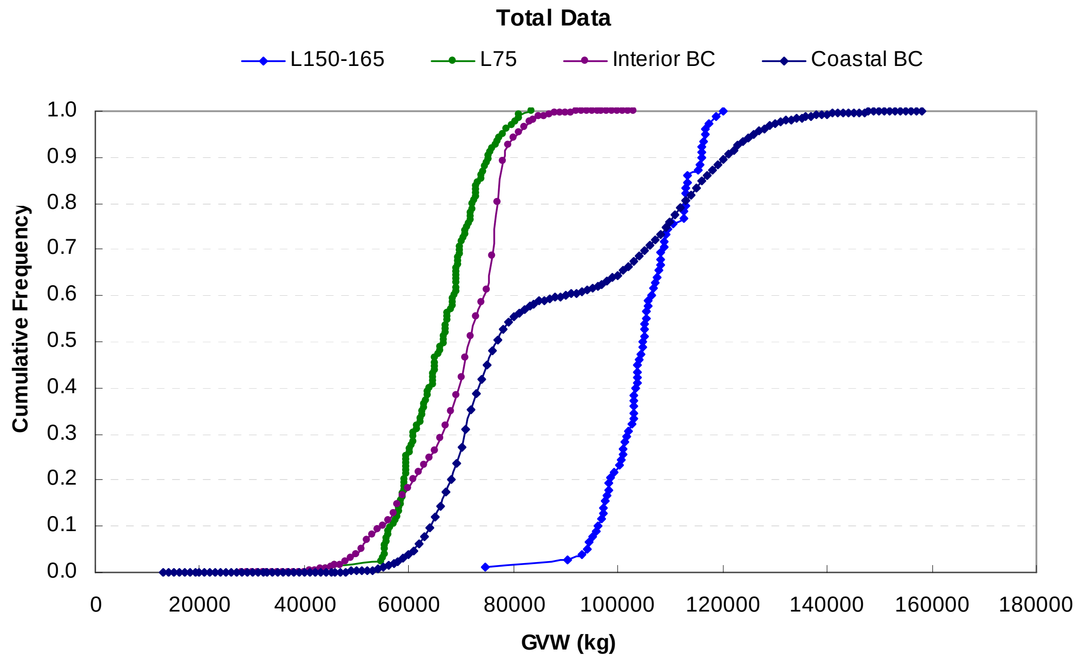
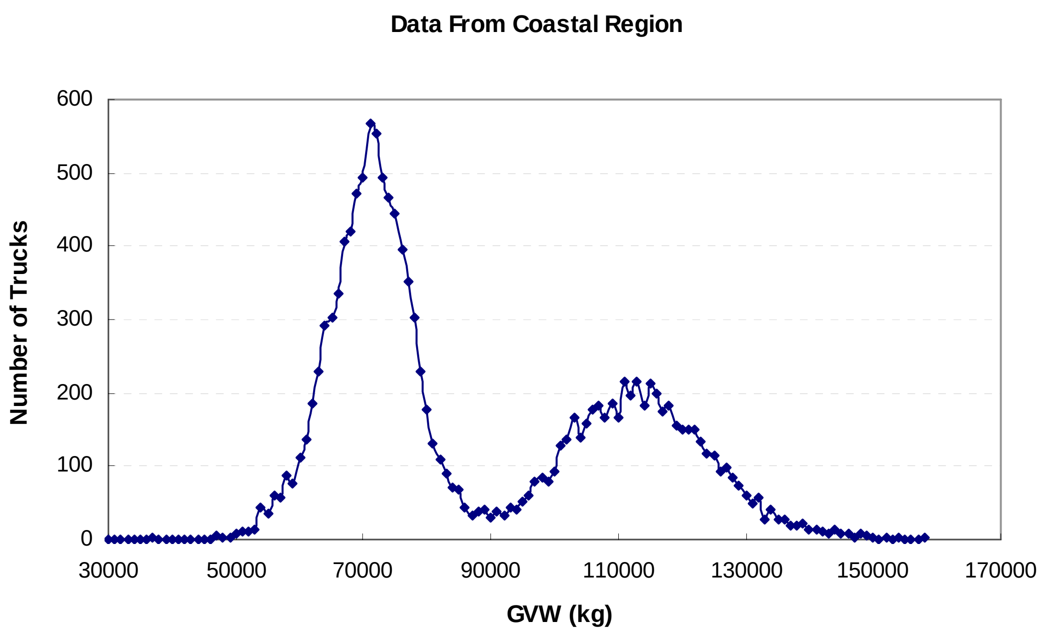
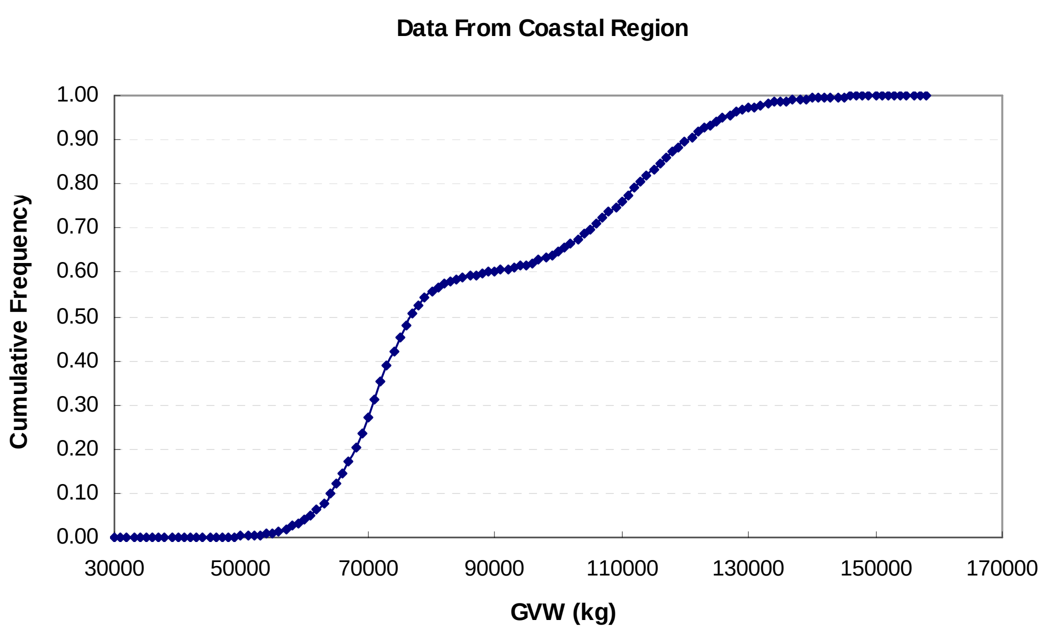


Figure 4. Cumulative Distribution Functions for GVW

Figure 4 also shows the CDF from the L150-165 data in Phase I. The Coastal region data appear to contain two components, one consistent with L75 loading and another more consistent 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, the CDF data need to be given a mathematical representation. In each of the four 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 four data groups, in Figures 6, 7, 8 and 9.

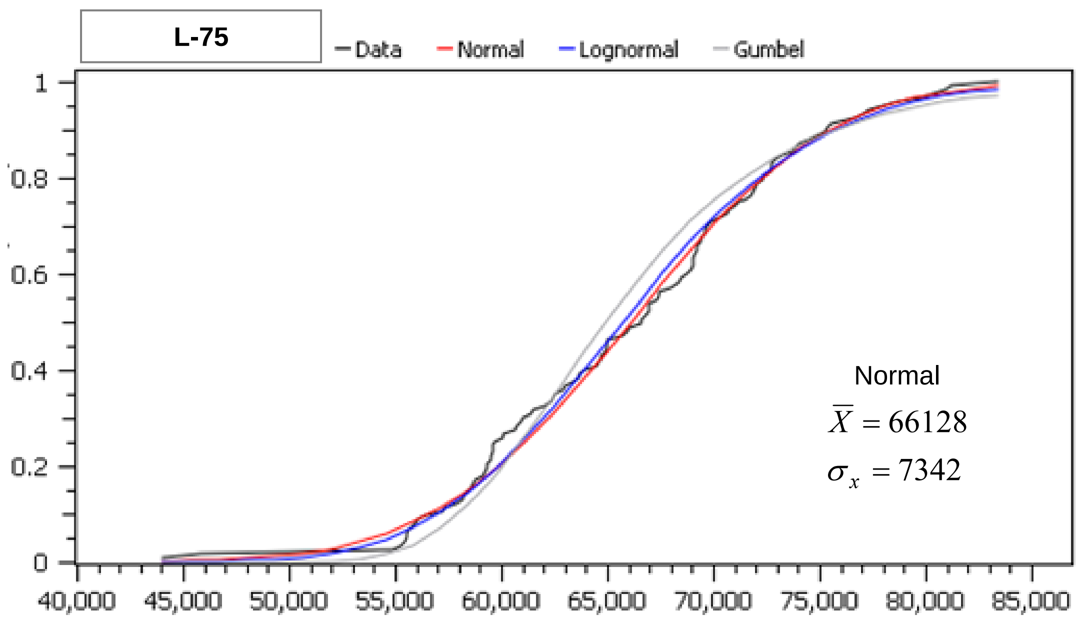


Figure 6. Representation of L75 data

It appears from Figure 6 that, for the **L75** data, a good fit is achieved with any 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 shows the results for the **L150-165** category. It appears, again, that a good fit is achieved with any of the three distributions and, 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%).*

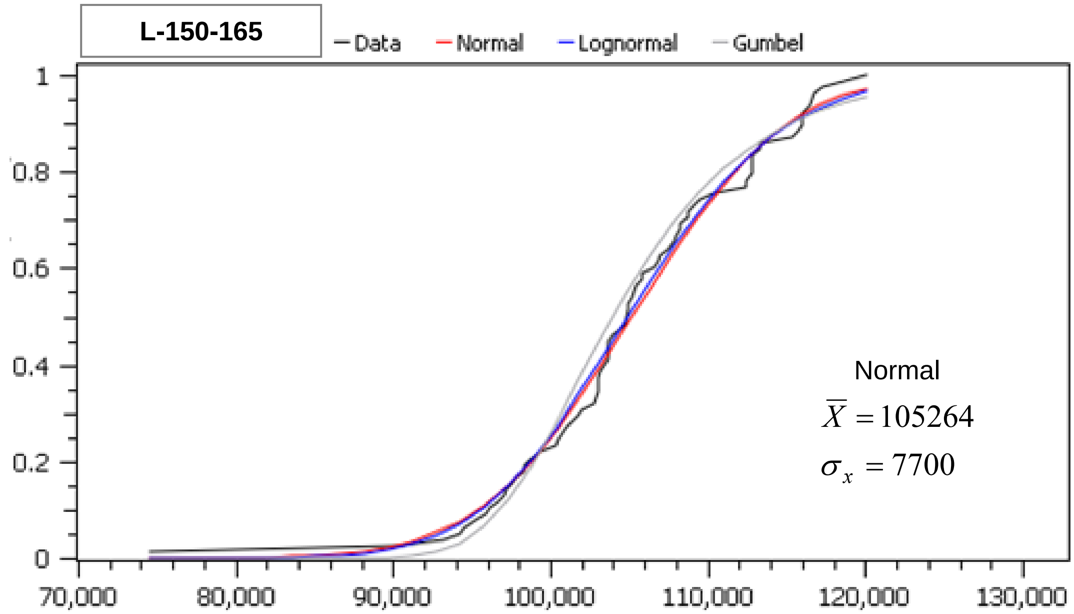


Figure 7. Representation of L150-165 data

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.

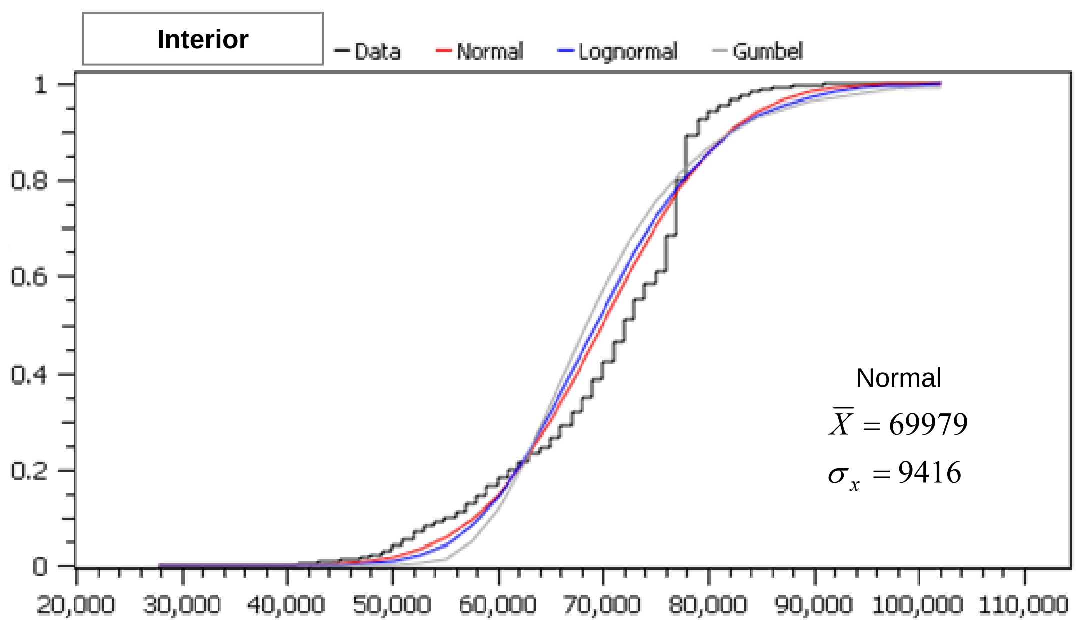


Figure 8. Representation of Interior BC data

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.

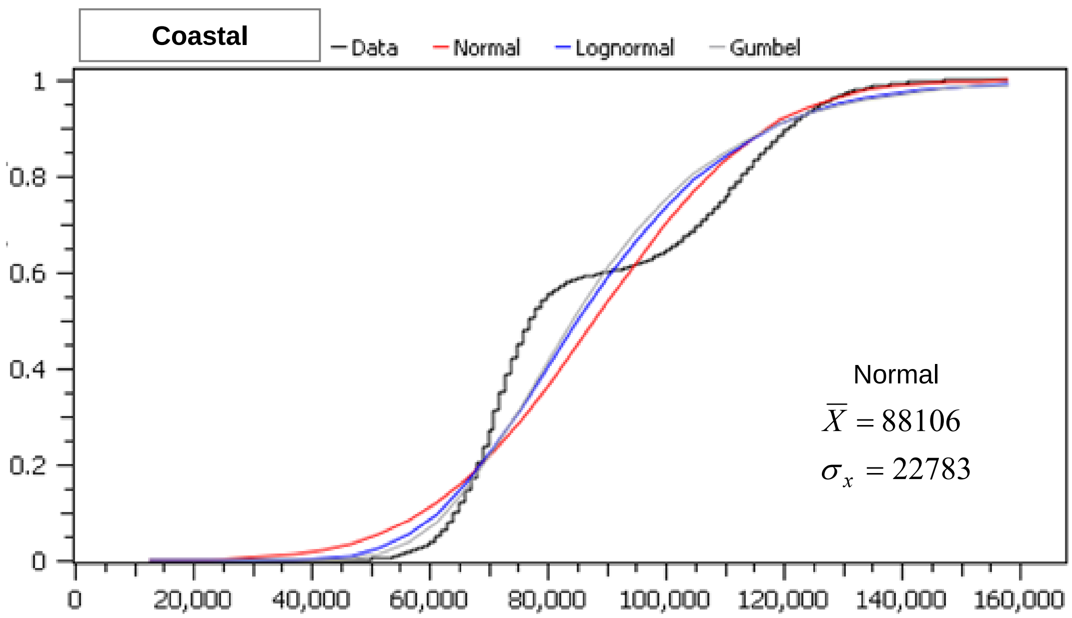


Figure 9. Representation of Coastal BC data

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%).*

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

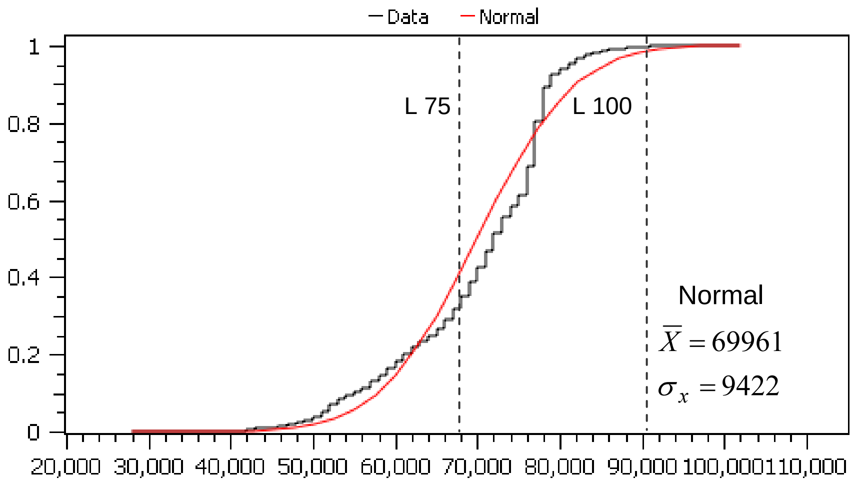


Figure 10. Interior data, 7- axle vehicles, Normal distribution representation

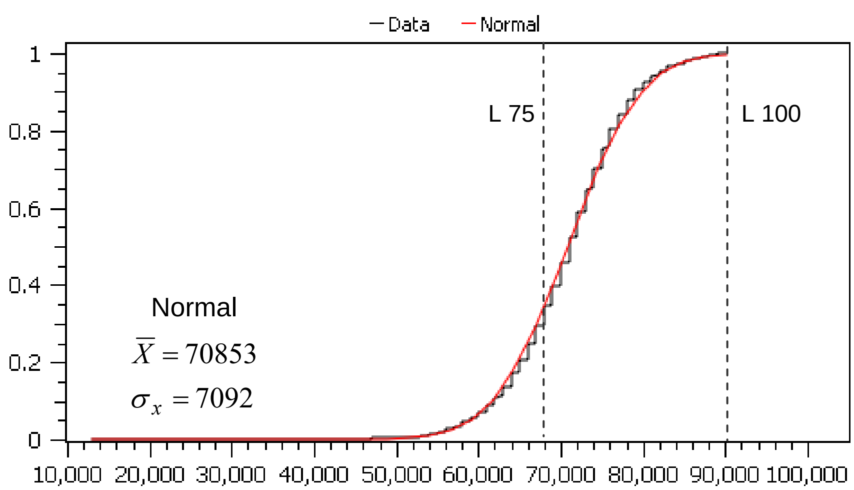


Figure 11. Coastal data, Lighter 7-axle vehicles, Normal distribution representation

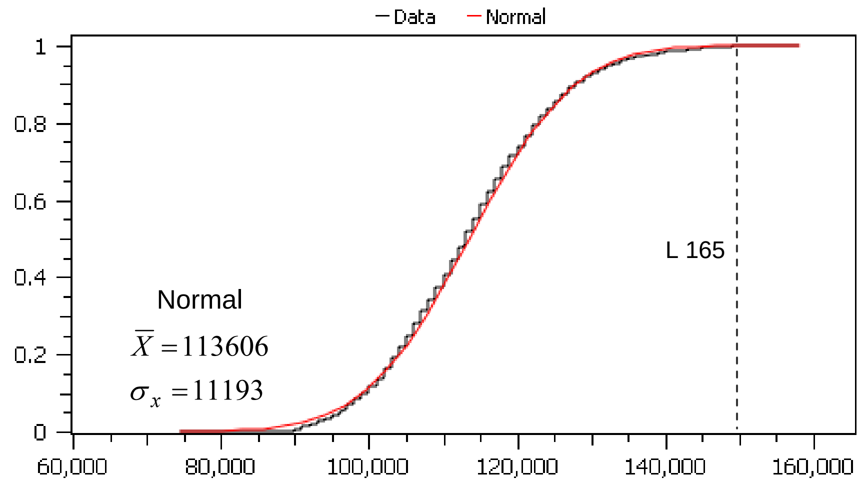


Figure 12. Coastal data, Heavy 5-axle vehicles, Normal distribution representation

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.

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

The 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)

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

	Left				Right			
L-150-165	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 x-y of a corner of the load patch for one of the steering tires, as shown in Figure 13. 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.

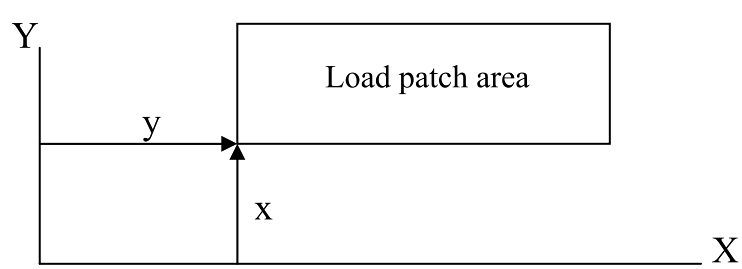


Figure 13. Load patch and coordinate system for its location

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 14 and 15 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.

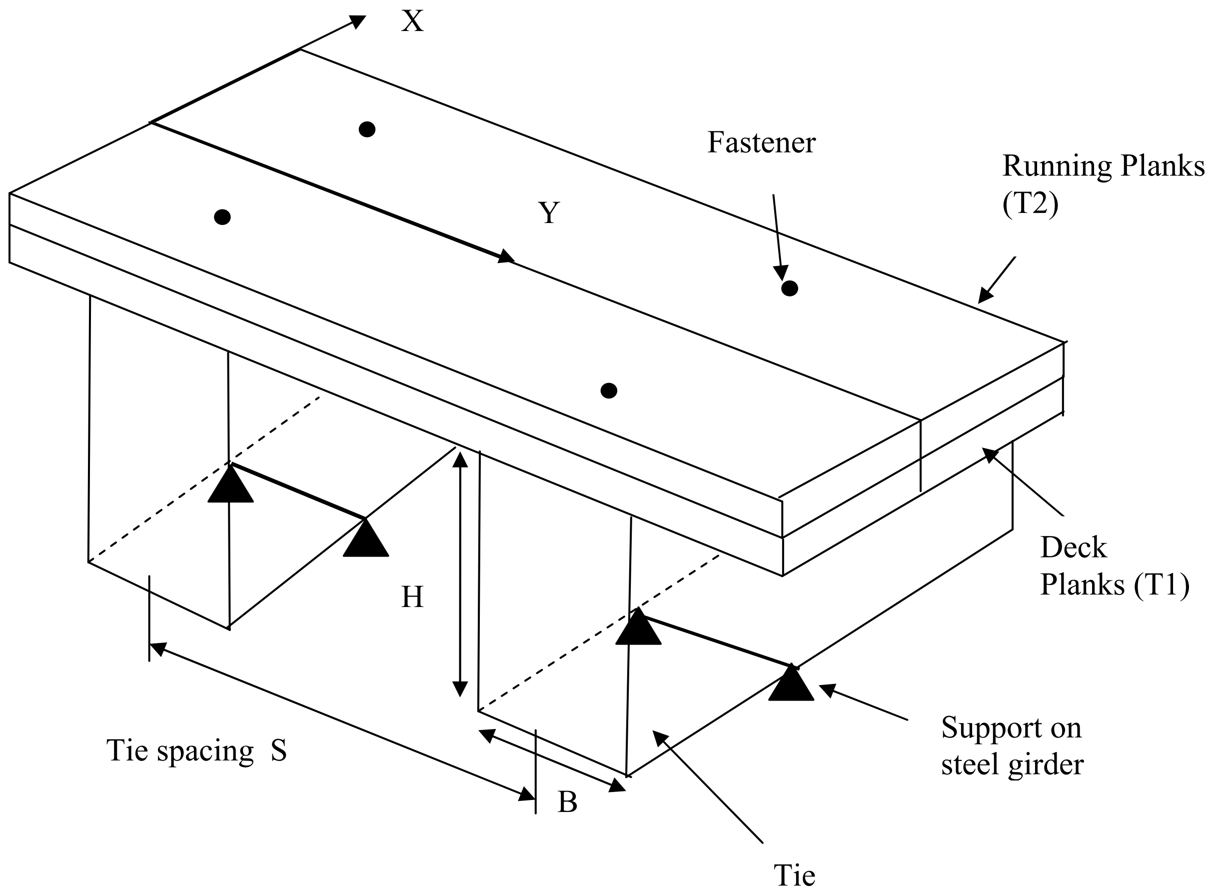


Figure 14. Schematic arrangement of planks and ties in the wood deck

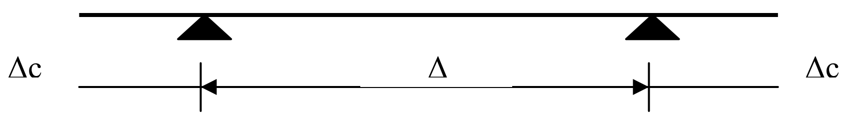


Figure 15. Tie spans and girder supports

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 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 obey 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 .

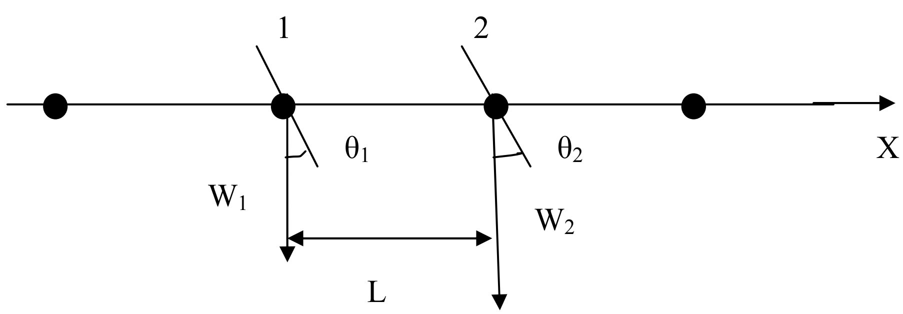


Figure 16. Beam elements and degrees of freedom

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

$$N_{DOF} = 14\ N_T \qquad [1]$$

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 account for a nominal truck length of around 20m, the number of ties is adopted to be $N_T = 50$, and the number of unknowns (degrees of freedom) for the problem is, therefore, $N_{DOF} = 700$.

The cubic polynomial for the beam deflections within an element is

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

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:

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

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:

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

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), [6]

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

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 $\boldsymbol{K}$. The matrices contributed by the fasteners are not shown here for brevity. If the vector of unknowns is $\boldsymbol{a}$, and the vector of load actions is $\boldsymbol{R}$, then the system of equations

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

is solved for $\boldsymbol{a}$, after the appropriate support conditions have been introduced. The load vector $\boldsymbol{R}$ 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 $\boldsymbol{K}$ and the global load vector $\boldsymbol{R}$ was completed and programmed into the accompanying software DECK, so that the vector $\boldsymbol{a}$ (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:

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

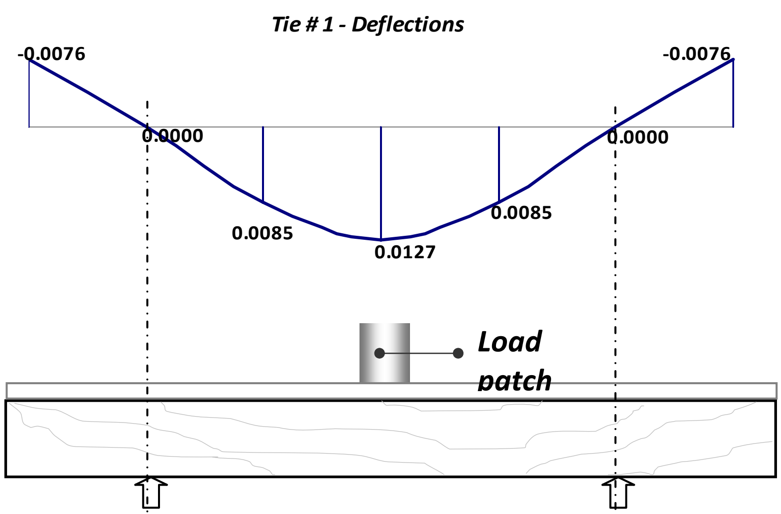
in which V indicates the volume of the tie. The distribution of the shear stresses τ is assumed to be parabolic over the depth H of the tie. This procedure for shear stresses is also specified in the S6 Bridge Code. The Weibull shear stress τ^* is calculated for each tie and the overall maximum T_{max} 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.

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
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.



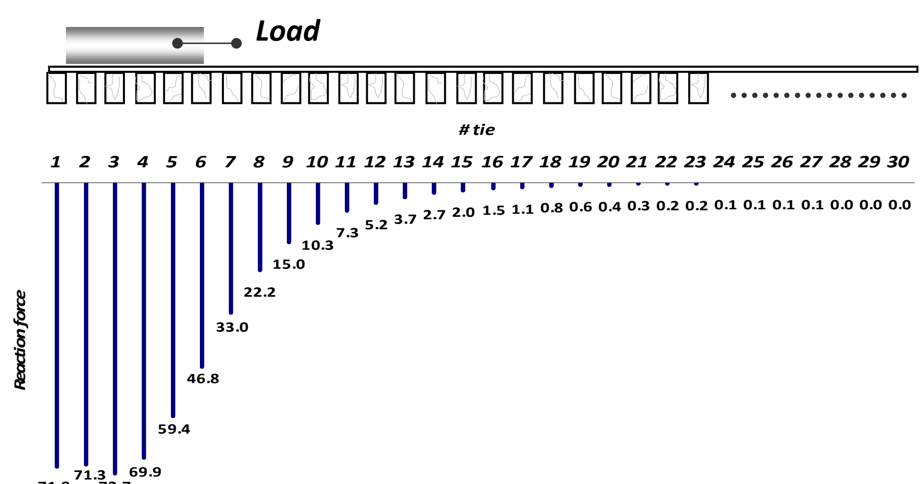


Figure 17. Deflections and support reactions, Example 1.

Figure 17 indicates that the deflections from the DECK solution are symmetric, as corresponds to the imposed patch loading. Further, the reactions 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), not symmetric.

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 = 0.64m, 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
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 13, the coordinates of the front wheel patch are x = 1.47m and y = 0.40m. This x-y location puts the truck in an un-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 18 shows the un-symmetric deflection of one tie (#1), and Figure 19 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.

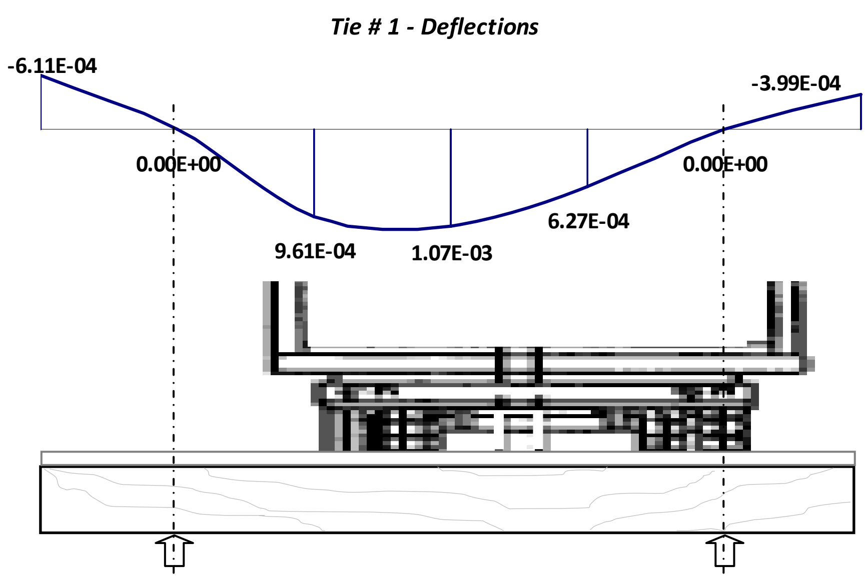


Figure 18
5-axle truck,
un-symmetric

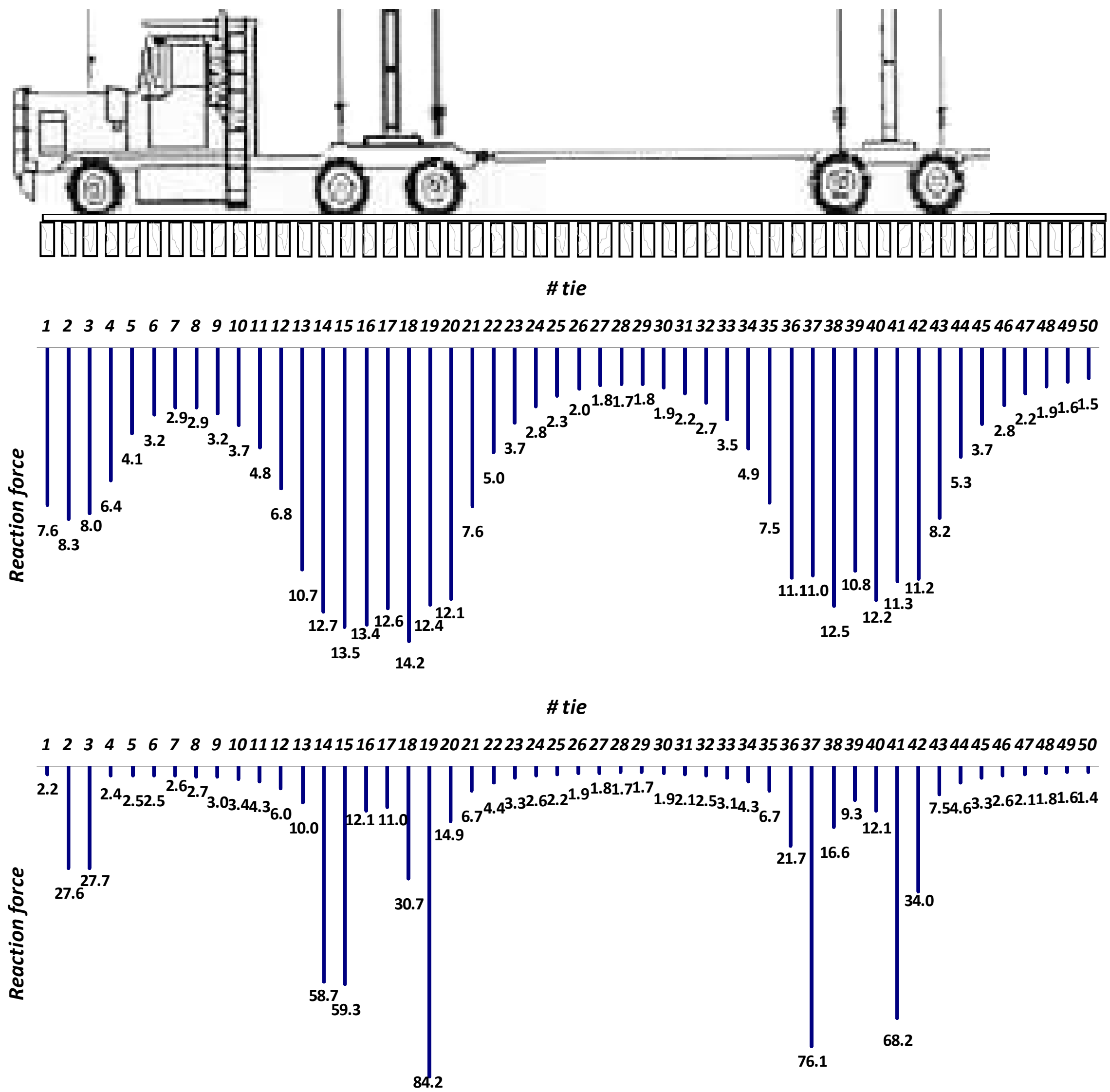


Figure 19. 5-axle truck, GVW = 100000kg (1000kN),
reactions for un-symmetric loading

Figure 19 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, DECK RESULTS, a sample of which is included as an Appendix at the end of this Report. For each tie, DECK RESULTS 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 up to tie No. 3.

5. Reliability Analysis

The reliability analysis included consideration of 3 failure modes involving 56 different random variables.

5.1 Random variables

The 56 variables 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, to be used with 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 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 structural analysis.

2. 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 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. 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, to arrive at the results presented here.

For each mode, the FORM algorithm finds out the combination of the variables most likely to result in failure. For example, the methods finds out automatically the coordinates x and y giving the worst position of the truck.

As shown by the description of the random variables, 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 cases. Four of these used the Heavy Coastal, 5-axle truck data. The other four used the Interior 7-axle truck data.

For the **Coastal, heavy trucks**, the following deterministic or fixed parameters were used:

50 ties
5 axles
Tie dimensions: 0.25m x 0.30m
Tie spacing: 0.406m
Tie main span: 3.60m
Tie cantilever span: 0.64m
Impact coefficient: 1.20
1 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

For the **Interior trucks**, the following deterministic or fixed parameters were used:

50 ties
7 axles
Tie dimensions: 0.25m x 0.30m
Tie spacing: 0.406m
Tie main span: 3.0m
Tie cantilever span: 0.65m
Impact coefficient: 1.20
1 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

The impact coefficient f_i =1.2 is consistent with experimental and theoretical dynamic studies in BC wood bridges (Horyna et al., 2001).

For either the Coastal or the Interior case, *four categories of wood material properties* were considered and these are detailed in Table 2.

The values shown in Table 2 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 11,379 kN/m^2. The same report, for the same timbers, also shows 5th-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. Pending new data from the planned testing of timbers following this project, the values in Table 2 are used to estimate the range in the reliability indices β for different levels of mechanical properties.

Table 2. Assumed wood mechanical properties

Category	Modulus of Elasticity E (kN/m^2) Lognormal		Bending Strength (kN/m^2) 2-P Weibull distribution		
	Mean	Cov (%)	Scale m	Shape k	5th Percentile (MPa)
1	10,000 x 10^3	15.0	50,000	6.0	30.48
2	12,000 x 10^3	15.0	50,000	6.0	30.48
3	12,000 x 10^3	15.0	65,000	6.0	39.62
4	14,000 x 10^3	15.0	80,000	6.0	48.76

Results for the reliability index β are listed in Tables 3 and 4.

Table 3. Reliability Indices β, Heavy Coastal Trucks, 5-axles

Category	Bending	Shear	Compression perpendicular
1	3.30	2.50	3.90
2	3.18	2.34	3.95
3	3.50	2.40	4.40
4	3.90	2.30	4.20

Table 4. Reliability Indices β, Interior Trucks, 7-axles

Category	Bending	Shear	Compression perpendicular
1	3.90	3.02	5.93
2	3.80	2.91	5.85
3	4.19	2.93	5.88
4	4.40	2.90	> 6.00

6. Conclusions

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

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 was made independently from recommendations from the Canadian Highway Bridge Code S6. Furthermore, lacking comprehensive data on timber strength, the analysis was done for four assumed categories of timber stiffness and strength, to provide background information on reliability when more definite timber data become available.

The timber data chosen are consistent with available experimental information. Thus, the bending strength and stiffness categories were chosen to be consistent with the limited timber data collected by Borg Madsen, at UBC, in 1982. 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), but the 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 Tables 3 and 4 that the reliability indices associated with the BC bridge configurations studied, under realistic (measured) truck loads, are satisfactory, since the consequence of one tie failing does not imply the collapse of the entire deck. The reliabilities in the bending mode are consistent with the aims of S6, 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). On bending strength alone, BC bridges appear to be sufficiently reliable, and S6 design checks implying the contrary probably reflect gaps in the calibration procedure for that Code. The results from Tables 3 and 4 show that, in general, stiffer (higher MOE) and correspondingly stronger ties (higher MOR) lead to higher reliability.

Of the three limit states, shear strength appears to be the controlling mode. 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. Tables 3 and 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 reasonable, the lower reliabilities in shear indicate that the new 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.

7. References

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Appendix

Example of output file DECK RESULTS, 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
```