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Connection Design for Post and Beam Construction
Performance of Bolted Connection in Western Hemlock

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EXECUTIVE SUMMARY

Connection design provisions in Canadian Code on Engineering Design in Wood has recently been modified to consider 4 failure modes in bolted connection design loaded axially in the parallel to grain direction. The aim for the change is to provide more realistic failure mode identification while achieving more efficient and economic designs. There is little information on the performance of connection design with the new provisions in terms of structural reliability and there is no information on the robustness of the new design provision to identify failure mode.

This study aims to establish procedures that can evaluate the structural reliability of some bolted connections in post and beam construction. Three connection configurations made with 130 x130 mm No.1 and No.2 Western Hemlock were evaluated in tension parallel to grain. Steel-Wood-Steel (SWS) and Wood-Steel-Wood (WSW) connections with 12.7 mm diameter bolts and 12.7 mm thick steel plates were considered with 2 rows of two bolts (SWS and WSW) and 1 row of 2 bolts (SWS). The test data established the failure mode and the capacity of the connection. Structural reliability analysis were conducted to evaluate the performance of these connections based on snow load conditions in Vancouver, The results indicate that the Canadian design provisions for such connections was very conservative and failure mode was not properly identified in some cases. There is a need to conduct further study to allow proper recognition of the performance of bolted connections in Western Hemlock.

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1. INTRODUCTION

UBC has been involved in studying the behaviour of heavy timber connections for many years. The scope of work ranged from studying the behaviour of screw, tube and bolted connections under monotonic and reverse cyclic loading. Also we studied reinforcement techniques for bolted connections to achieve improved performance.

This study is a new project in support of the wood first initiative intended to develop and extend knowledge from the past so that we can quantify the performance of heavy timber connections in terms of structural reliability. This information is needed to support changes in the Canadian deign code Engineering Design in Wood (CAN3-O86.1). The connection design provisions in CAN3-O86.1 were recently modified to consider 4 distant failure modes in the design of bolted connection loaded in tension in the parallel to grain direction. The code also specifies minimum distances for row spacing, fastener spacing in a row, loaded end distance, unloaded edge distance as a function of bolt diameter (d) as 3d, 4d, maximum of (5d or 50 mm), and maximum of (1.5d or half of row spacing), respectively.

The identified failure modes were row shear, group tear out, net tension, and bolt yielding. In general these changes were positive allowing the designers to specify more economical and rational solution compared to before. Furthermore a designer can in theory detail bolted connections to achieve a certain failure mode for example to achieve ductile behaviour by bolt yielding. To be able to control the failure mode is an important concept in timber engineering especially in the provision of lateral resistance against earthquake forces. In the case of BC, the ability to provide economical and safe design solutions is also critical for the success of government wood first and midrise initiatives,

There is however little information on the performance of connection design with the new provisions in terms of structural reliability and there is limited information on the robustness of the new design provision to identify failure mode. The objective of this study is to develop a database focusing on BC Western Hemlock (*Tsuga heterophylla*) on a few connection configurations and evaluate the performance of the connection in terms of structural reliability to ascertain the robustness of the new CAN3-O86.1 design provisions.

2. MATERIAL AND METHODS

2.1 Materials

As there are many available species and wood products and test configurations, the potential evaluation matrix is orders of magnitude beyond the resource available for the scope of the current project. Here three test configurations were chosen and studied in detail. 130 mm x 130 mm No. 1 and No.2 (mostly No. 2) Western Hemlock members were chosen. The material were air dried to average moisture connect of ~13.5%. Steel-Wood-Steel (SWS) and Wood-Steel-Wood (WSW) connections with 12.7 mm diameter bolts and 12.7 mm thick steel plates were considered. In the case of the SWS connection, 2 rows of two bolts with 2 steel plates (2R2) and 1 row of two bolts with 2 steel plates (1R2) were considered. In the case of WSW connection 2 rows of two bolts with 1 steel plate (2R1) were tested. In all cases 5d was chosen as the row spacing, fastener spacing in a row, and loaded end distance. The edge distance was chosen as 2.7d. Even though the spacing was larger than the minimum distances specified in the code they were deemed reasonable for the cross section of the member studied. Figures 1 to 3 show the schematics of the test configurations.

CNC machine was used to predrill the holes for the bolts with a diameter of 12.7 mm and precision of ±0.1mm. The specimens were conditioned at 20^{o}C and 65% relative humidity climate chamber after the holes were drilled to prevent the specimens from cracking. The steel plates were also predrilled with a diameter of 13.5 mm and precision of ±0.1mm.

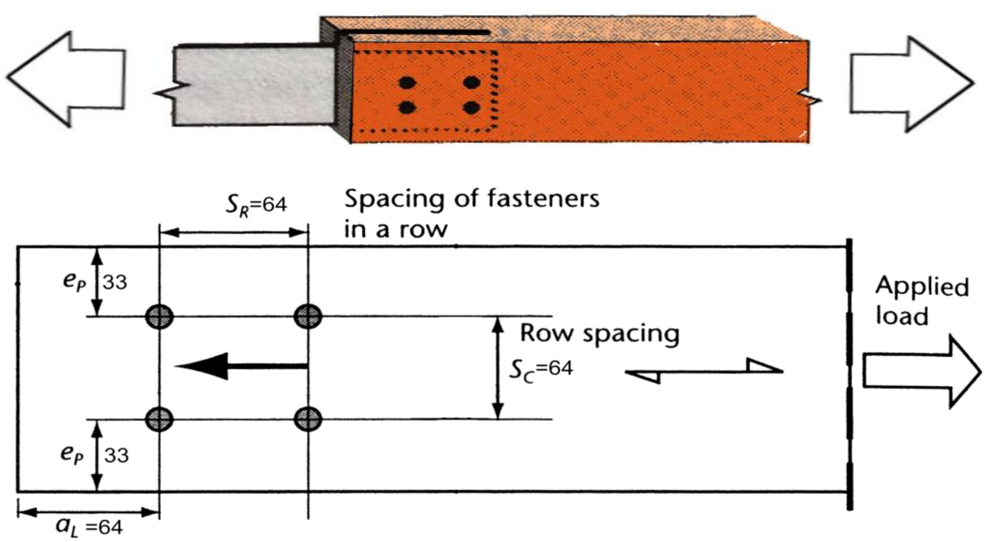

Figure 1. Schematics of the test configurations for WSW 2R1 group

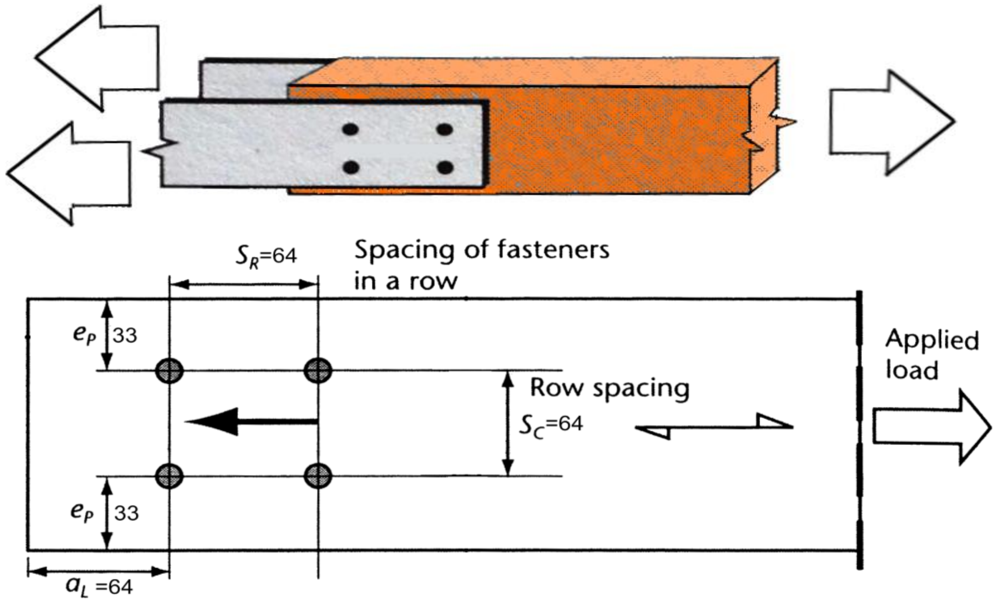

Figure 2. Schematics of the test configurations for SWS 2R2 group

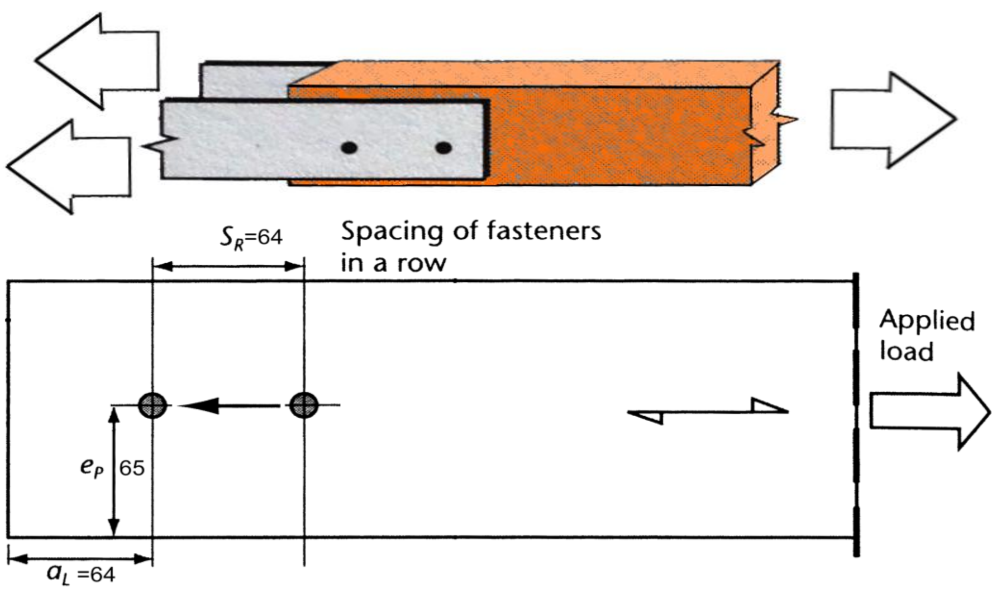

Figure 3. Schematics of the test configurations for SWS 1R2 group

2.2 Test Procedure

The specimens were tested on the MTS Universal test machine in the UBC Timber Engineering and Applied Mechanics Laboratory. The machine was displacement controlled at a test rate of 1.4 mm/min for the connection tests to reach the peak loads in approximately 10 minutes, not less than 5 and not more than 20 minutes according to ASTM D5652-95 Standard Test Methods for Bolted Connection in Wood and Wood-Based Products. Two linear voltage displacement transducers were installed on two sides of the connection only at the bottom end of the specimen to monitor the displacement between the wood member and the steel connectors. A view the test assembly is presented in Photo 1. The specimens were loaded until failure to obtain the peak load.

Photo 1. Test specimens being loaded

3. TEST RESULTS

Table 1 presents the summary results of the tension capacity of each group. Detailed test results are shown in Tables 2 to 4. In all but one case row shear failure was observed. The case that did not fail in row shear was a wood slope of grain type failure which occurred in SWS 1R2 with a capacity of 61 kN. It should be noted that it was not the lowest in the group. Photos 2 to 3 show the failure modes.

Table 1. Summary results of the tension capacity

Group	WSW 2R1		SWS 2R2		SWS 1R2	
	Moisture Content (%)	Capacity (kN)	Moisture Content (%)	Capacity (kN)	Moisture Content (%)	Capacity (kN)
Average	13.70	144.67	13.40	169.23	14.10	76.05
Stdev	1.15	14.57	1.14	28.36	0.70	16.69
COV	8.4%	10.1%	8.5%	16.8%	5.0%	22.0%
Maximum	15.4	167	16.8	242	15.3	122
Minimum	11.6	113	11.8	130	12.7	58
Count	24	24	22	22	22	22
5th%tile		116.00		130.75		58.15

* XRY: X - number of rows; Y –number of steel plate.

Table 2. Detail results of the tension capacity for WSW-2R1 group

WSW- 2R1	Grade	*MC %*	Max. Load (KN)	Failure Mode
2r1-03	2	13.20	130.000	Top shear
2r1-05	2	15.40	113.000	Top shear
2r1-06	2	14.10	142.000	Top shear
2r1-07	2	12.40	157.000	Bottom shear
2r1-08	2	12.50	127.000	Top shear
2r1-09	1	15.40	132.000	Bottom shear
2r1-10	2	12.80	156.000	Top shear
2r1-11	2	11.60	158.000	Top shear
2r1-12	1	11.70	134.000	Top shear
2r1-14	2	13.60	165.000	Top shear
2r1-15	2	15.20	125.000	Bottom shear
2r1-16	2	14.00	162.000	Top shear
2r1-17	2	14.00	143.000	Top shear
2r1-18	2	14.70	129.000	Top shear
2r1-19	1	13.60	152.000	Top shear
2r1-20	1	14.50	151.000	Top shear
2r1-21	2	14.00	153.000	Bottom shear
2r1-22	1	12.80	150.000	Bottom shear
2r1-23	1	12.30	148.000	Top shear
2r1-24	2	14.40	148.000	Top shear
2r1-25	2	13.70	125.000	Top shear
2r1-26	1	15.20	157.000	Top shear
2r1-27	2	12.90	167.000	Bottom shear
2r1-28	2	14.90	148.000	Top shear
Average		13.70	144.67	
Stdev		1.15	14.57	
COV		8.38%	10.07%	
Maximum		15.40	167.00	
Minimum		11.60	113.00	
Count		24	24	
5th%tile			116	

Table 3. Detail results of the tension capacity for SWS-2R2 group

WSW- 2R1	Grade	*MC %*	Max. Load (KN)	Failure Mode
2r2-03	2	12.80	183.000	Bottom shear
2r2-04	2	11.80	140.000	Top shear
2r2-05	2	13.70	166.000	B &T shear
2r2-07	2	13.30	140.000	B &T shear
2r2-08	2	14.30	171.000	Top shear
2r2-09	1	14.80	162.000	Top shear
2r2-10	2	13.20	160.000	Bottom shear
2r2-11	2	14.00	147.000	Bottom shear
2r2-12	1	13.60	153.000	Top shear
2r2-14	2	14.00	162.000	Top shear
2r2-15	1	12.60	171.000	B &T shear
2r2-17	2	12.70	154.000	B &T shear
2r2-18	2	12.10	185.000	Bottom shear
2r2-19	2	13.20	242.000	Bottom shear
2r2-20	2	16.80	130.000	Bottom shear
2r2-21	2	14.50	135.000	Top shear
2r2-22	2	14.10	199.000	Top shear
2r2-23	2	13.60	171.000	Top shear
2r2-24	2	12.00	166.000	B &T shear
2r2-25	2	12.10	196.000	Top shear
2r2-27	2	12.80	159.000	Bottom shear
2r2-28	2	12.70	231.000	Top shear
Average		13.40	169.23	
Stdev		1.14	28.36	
COV		8.50%	16.76%	
Maximum		16.80	242.00	
Minimum		11.80	130.00	
Count		22	22	
5th%tile			130.75	

Table 4. Detail results of the tension capacity for SWS-1R2 group

SWS-1R2	Grade	***MC %***	***SG***	Max. Load (KN)	Failure Mode
1r2-01	1	12.70		67.0	Bottom shear
1r2-02	2	14.40		94.0	Top shear
1r2-03	2	13.20		68.0	Top shear
1r2-04	1	14.60		64.0	Bottom shear
1r2-05	1	13.80		71.0	Bottom shear
1r2-06	2	13.40		98.0	Top shear
1r2-07	2	13.50		73.0	Bottom shear
1r2-08	1	13.40		88.0	Bottom shear
1r2-09	2	14.80		62.0	Top shear
1r2-10	2	14.00		90.0	Top shear
1r2-11	1	14.00		68.0	Top shear
1r2-12	2	13.00		61.0	Woo SOG
1r2-14	2	14.50		87.0	Top shear
1r2-15	2	14.50		101.0	Bottom shear
1r2-16	1	14.10		59.0	Top shear
1r2-17	1	14.50		75.0	Top shear
1r2-18	1	15.30		64.0	Bottom shear
1r2-20	1	14.80		70.0	B &T shear
1r2-22	1	14.90		72.0	Top shear
1r2-23	2	14.80		58.0	Top shear
1r2-26	1	14.60		122.0	Top shear
1r2-28	1	13.50		61.0	Bottom shear
Average		14.10		76.05	
Stdev		0.70		16.69	
COV		4.99%		21.95%	
Maximum		15.30		122.00	
Minimum		12.70		58.00	
Count		22		22	
5th%tile				58.15	

Photo 2. Failure modes in 2R2 group

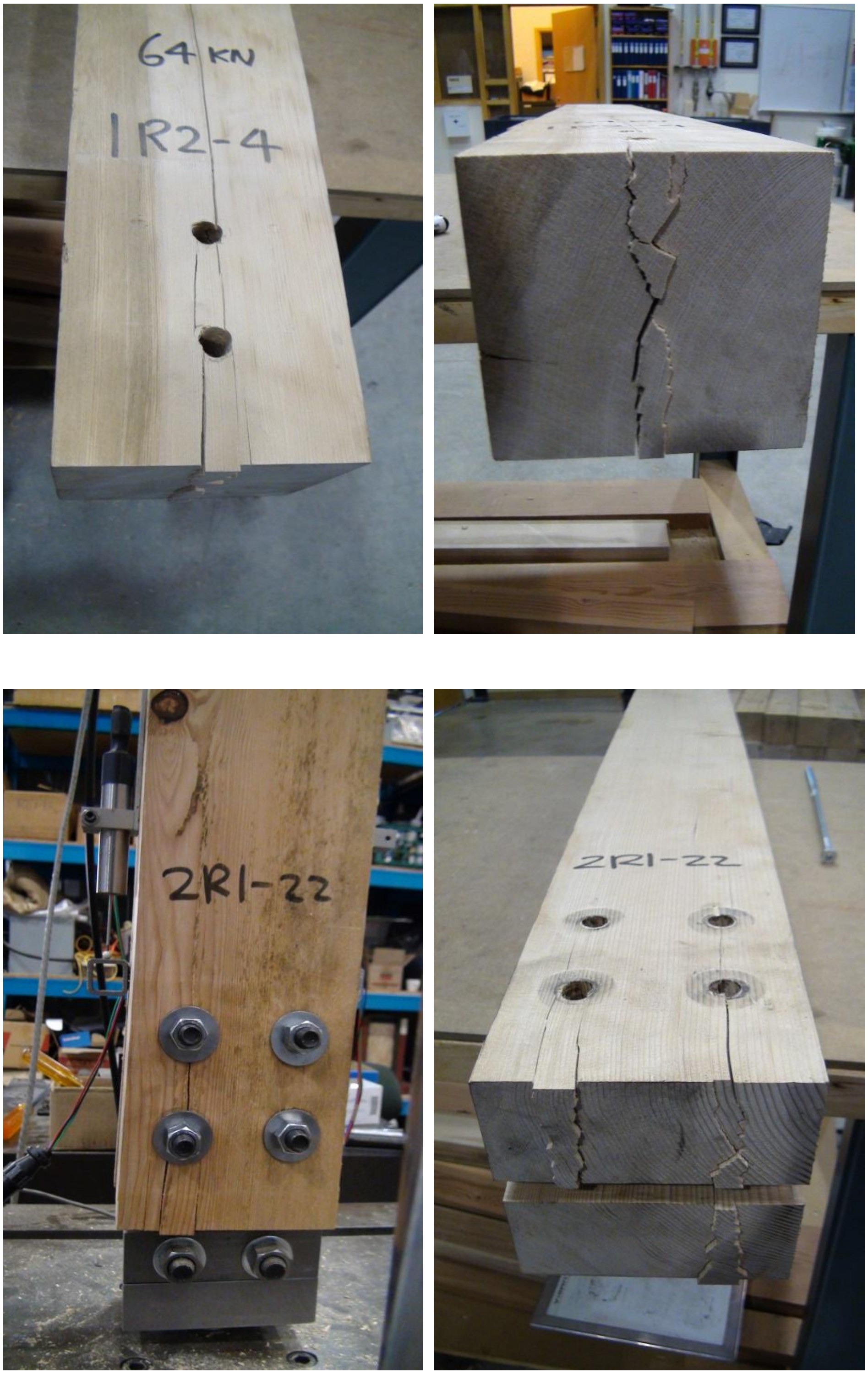

Photo 3. Failure modes in 2R1 and 1R2 groups

In the three cases, probability distributions were fitted to the test data the resulting cumulative distributions were plotted in Figures 4 to 6. The distributions and their parameters deemed to provide the most reasonable fit to the data is shown in Tables 5.

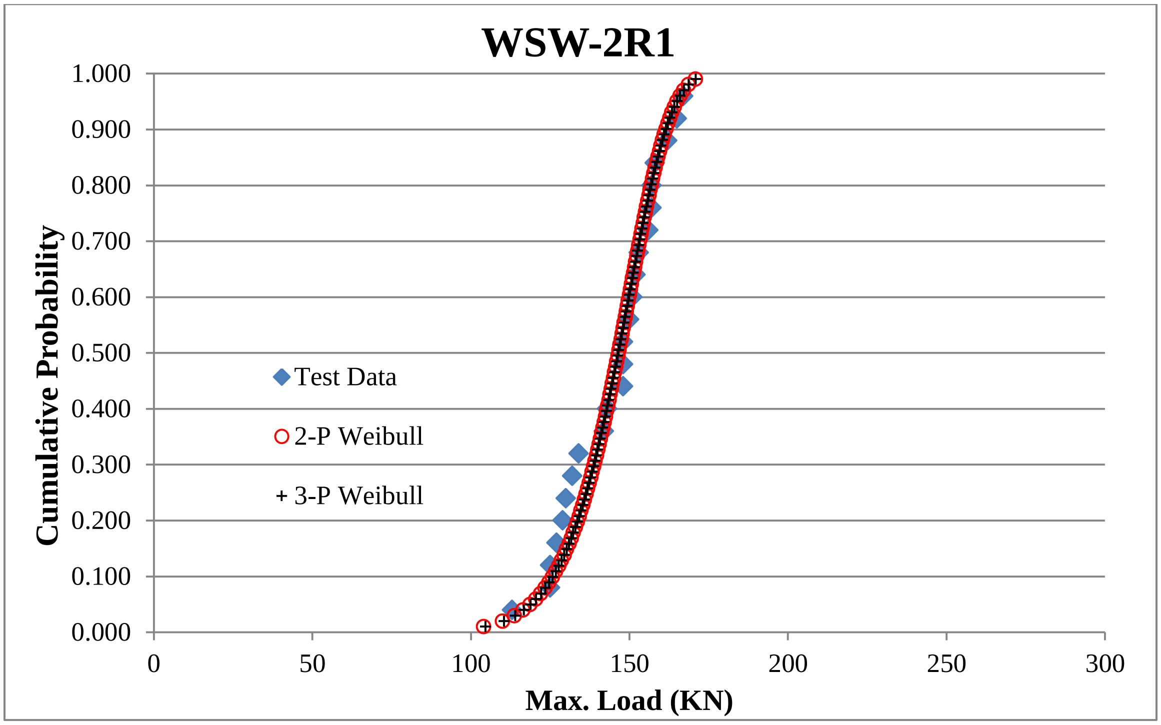

Figure 4. Data fitting for WSW-2R1group with grade No.1+No.2

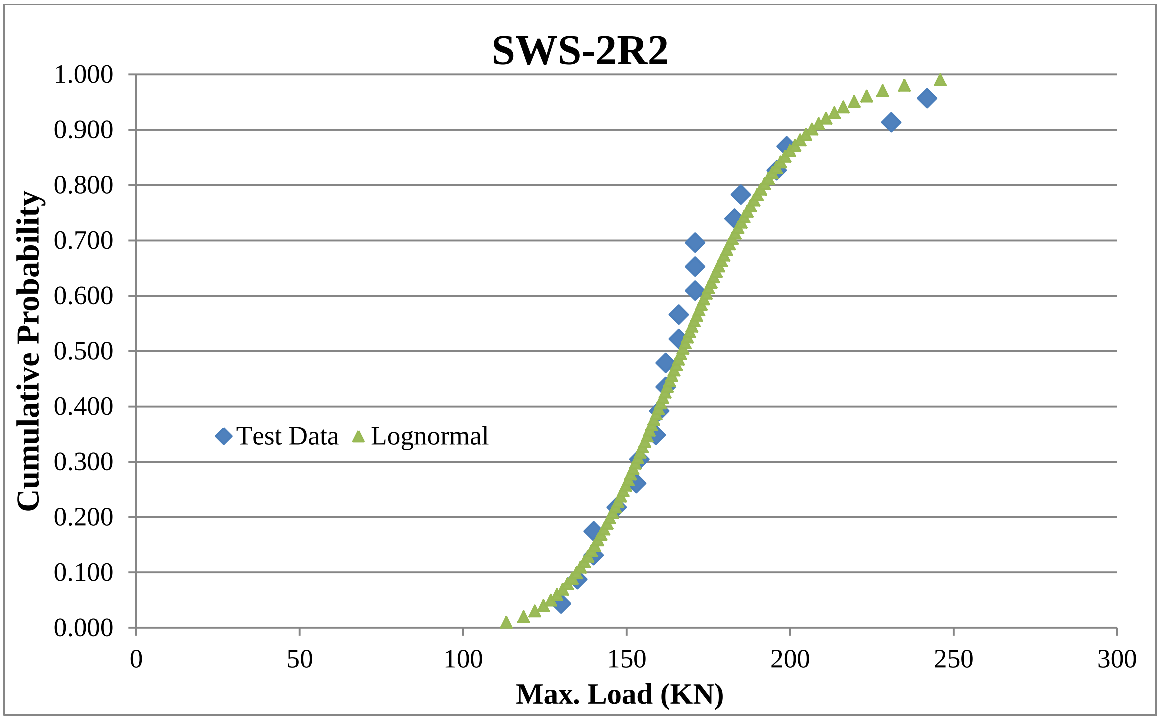

Figure 5. Data fitting for SWS-2R2 group with grade No.1+No.2

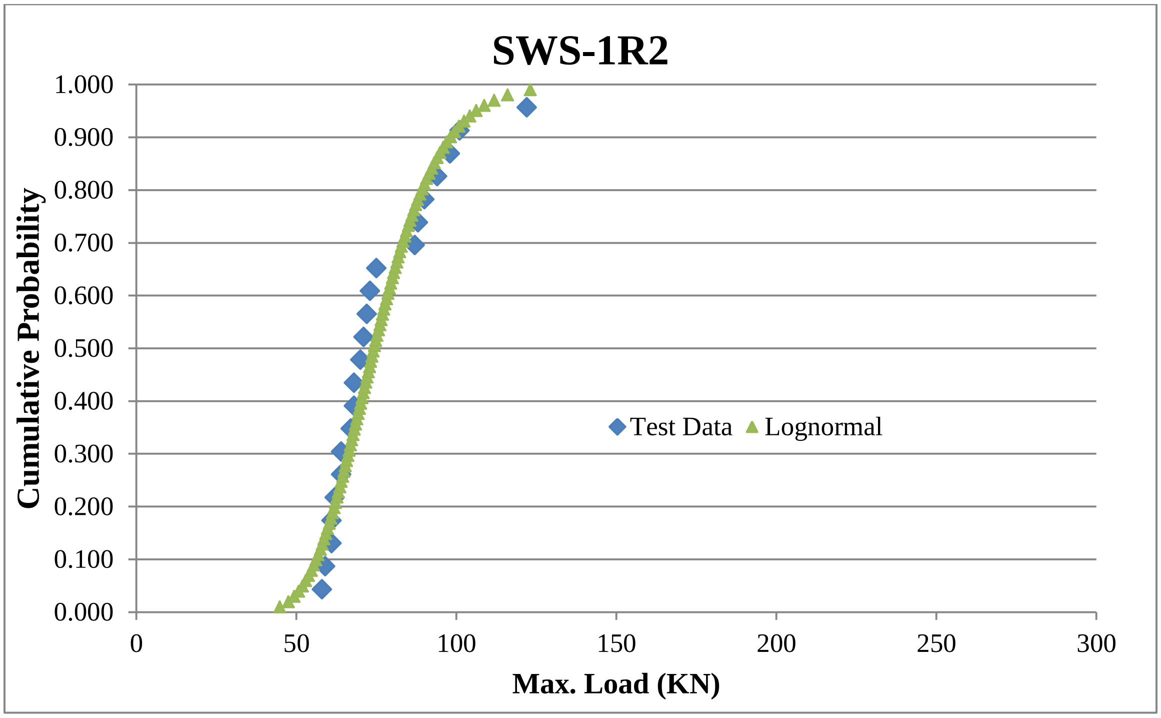

Figure 6. Data fitting for SWS-2R1group with grade No.1+No.2

Table 5. The distributions and parameters used for the data fit

	WSW 2R1		SWS 2R2	SWS 1R2
	2-P Weibull	3-P Weibull	Lognormal	Lognormal
Scale	150.9493	137.1187	-	-
Shape	12.3708	11.1972	-	-
Location	-	13.7754	-	-
Average	-	-	169.2273	76.04545
COV	-	-	0.1676	0.2195

4. RELIABILIY ANALYSIS

4.1. Concept

The reliability of a structural component can be defined as the probability that it will achieve a predetermined level of performance in service. The probabilistic nature of the problem arises from the fact that randomness exists in many intervening variables that could influence the behaviour of the structural component of interest. In the case of bolted connection such random variables could include the parameters that influence the strength properties of the connection and the parameters that influence the load demands. The uncertainties associated with these parameters need to be quantified and described in statistical terms to allow the evaluation of the reliability of the structural component.

A performance function, G, for a given design condition is given as:

$$G(X) = G(X_1, X_2, X_3,.., X_N) = \text{Capacity} - \text{Demand} \qquad (1)$$

Where the vector $X = X_1, X_2, X_3,\ldots, X_N$ contains N random variables that are associated with the problem. If $G>0$, the capacity is greater than the demand and the design situation is safe. If $G<0$, the capacity is less than the demand and the design situation fails. If $G=0$, a limit state exists. For a given situation, the objective is to establish the probability of failure associated with the random variable vector X.

Following the First order or Second Order Reliability Methods (FORM/SORM), a reliability index, β, can be calculated for a given G. The probability of failure, P_f, can then be estimated from β with some basic assumptions and using the standard normal probability distribution function $\Phi(\bullet)$ as:

$$P_f = \Phi(-\beta) \qquad (2)$$

Detailed discussions of the approach to reliability based design of wood structures can be found in Foschi et al. (1989).

In the Canadian limit state design code CSA 086-09 Engineering design in wood, a design equation takes on the following form:

$$\alpha_D \boldsymbol{E}(D_n) + \alpha_Q \boldsymbol{E}(Q_n) = \phi R_o \quad (3)$$

Where α_D =1.25 and α_Q =1.5 are load factors associated with dead and live loads, respectively; $E(D_n)$ and $E(Q_n)$ are effect of nominal dead and live loads on the structural component of interest, respectively; R_o is the design resistance of the structural component of interest under testing condition term of loading of say 15 minutes (clauses 10.4.4.3 to 10.4.4.6 in CAN3O86.1). In wood failure modes the design provisions in the Canadian code is set to correspond with a standard load term of 3 months. To convert back to the case of test duration of say 15 minutes a factor of 1.25 should be applied to increase the design value. For bolt yielding failure mode there is no need to make adjustment except for wood embedment strength calculation. ϕ is the performance factor of the design set at a prescribed target performance level or probability of failure for the various mode of failure.

Since there four design provisions for bolted connection in tension parallel to grain corresponding to the failure modes of row shear, group tear out, net tension, and bolt yielding (clauses 10.4.4.3 to 10.4.4.6 in CAN3O86.1), equation 3 should also have four corresponding versions.

Although the design values in equation 3 appear as deterministic values, in real life, the effect of loads and the resistance of the structural component are both random. A performance function can therefore be written as:

$$G = R - (E(D) + E(Q)) \quad (4)$$

Where R is the random resistance of the structural component of interest, E(D) and E(Q) are effect of random dead and live loads on the structural component of interest, respectively. The performance function can be linked with the design equation and rewritten as:

$$G = R - \phi \frac{R_o\,(d\,\gamma + q)}{\alpha_D \gamma + \alpha_Q} \quad (5)$$

Where:

$\gamma = D_n / Q_n$ is the nominal dead load to nominal live load ratio typically set as 0.25 for timber structures;

d = D / Dn is the dead load to nominal dead load ratio;

$q = Q / Q_n$ is the live load to nominal live load ratio.

Thus R, d, and q are the random variables in the problem. Given the statistical information of these variables, the failure function can be studied with respect to different ϕ values to establish the β vs ϕ relationship for the design. So for a target β level, the associated ϕ level can be established so that consistent design and safety level can be established for different structural components.

Now the statistical parameters for the random variables associated with the loads d and q have previously been established and reported by Foschi et al. (1989). For the random resistance R, its statistical parameters need to be established from the test data base Tables 5. In this study we used these statistical parameters to study the performance of three configurations of bolted Western Hemlock wood connection considering the effect of snow and dead loads on the structural components.

4.2. Evaluation of the Reliability

Reliability analyses were performed under dead and snow load conditions for Vancouver following the procedures outlined by Foschi et al. (1989). The statistical distributions and parameters for the dead and snow load for Vancouver were described in detail by Foschi et al. (1989) where the snow loads are considered on a 30-year return period.

Table 6 shows the code predicted connection capacity and failure mode compared to the test results. The Canadian CANO86.1 design provisions for such connections were very conservative and failure mode was not properly identified in some cases.

Table 6. Code predicted connection capacity and failure mode

Group	1R2	2R2	2R1
Predicted connection strength (kN)	29.72	52.83	34.86
Predicted mode of failure	RS	GT	RS
5^{th}% strength in test (kN)	58.15	130.75	116.00
Mode of failure in test	RS	RS	RS

Table 7 shows the summary results of the β values for the various cases of bolted connections for each design provision. Considering the governing failure mode from the design provisions in the Canadian Code, *i.e.* RS, GT and GT for 1R2, 2R2 and 2R1, respectively, the associated β values are in the range of 5.30-8.62. Foschi et al. (1989)

stated that the target β values should be in the range of 2.8 to be comparable to steel design.

Table 7. Summary results of the β values

Test Group	β (dead load + live load)		β (dead load + snow load)	
	Row shear	Group tear-out	Row shear	Group tear-out
SWS 1R2	5.68	N/A	5.30	N/A
SWS 2R2	6.43	6.84	5.97	6.36
WSW 2R1	8.62	8.23	7.98	7.61

The large difference in the calculated β values from the target indicates that the lack of consistency in the code in terms of how safety is treated for the connection configurations studied. This is an indication that the design for these connections is too conservative thus reducing their economic competitiveness.

Row shear was the only failure mode observed. Although this may be attributed to the fact that the row spacing and fastener spacing in a row are greater than the minimum values specified in the code, the code predicted failure mode should more accurately reflect material behaviour. The influence of spacing, bolt/specimen geometry and properties on the failure mode in a bolted connection is clearly not fully understood. As it is now, some engineers may assume that the code provisions would allow control of failure mode but in fact this might be true only in limited cases. This could be a dangerous assumption in some critical situations.

5. REFERENCES

CSA (Canadian Standards Association) Standard (2010). CSA O86-09: Engineering Design in Wood. Canadian Standards Association, Mississauga, Canada.

Foschi R.O., B.R. Folz, and F.Z.Yao (1989) Reliability-based design of wood structures. Structural Research Series, report No. 34. Department of Civil Engineering, University of British Columbia. Vancouver, Canada. ISBN 0-88865-356-5.

ASTM D5652-95, Standard Test Methods for Bolted Connection in Wood and Wood-Based Products, West Conshohocken, PA.

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