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Evaluation of Bolted Connections In Douglas Fir and Western Hemlock

by

Chao (Tom) Zhang
Zheng Li
George Lee
Frank Lam

Timber Engineering and Applied Mechanics (TEAM) Laboratory
Department of Wood Science
Faculty of Forestry
University of British Columbia
Vancouver, B. C., Canada

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Forestry Innovation Investment Ltd.
1200 – 1130 West Pender Street
Vancouver, B.C. V6E 4A4
Canada

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EXECUTIVE SUMMARY

This project investigated the tensile capacity of five types of bolted wood connections loaded along the grain and evaluated the associated design provisions in CSA-O86. Combined with the three configurations in the previous study, the eight types of connections included different species, bolt grades, bolt diameters, number of bolt rows, row spacings, and steel plate configurations.

The un-factored resistance of the connection according to CSA-O86 was in the range of 27-61% of the fifth-percentile capacity of the tested connections. The dominant failure mode for all eight configurations was row shear. The code predicted failure mode was in agreement with the observed row shear failure mode in five configurations but in the remaining three cases the code predicted group tear-out. Albeit significantly under predicting the capacity of the bolted connections, the influence of wood species on the capacity of the connection was accounted for in the code prediction. Furthermore, the provisions in CSA-O86 do not consider bolt grade or bolt diameter under row shear. The test results showed that the change of bolt grade did not affect the load significantly, but the increase of bolt diameter from 12.5 mm to 15.9 mm (½" to 5/8") contributed to a 15% increase of peak load. For row shear, the code treated each row independently: the load for multiple rows was the sum of the load for each row. In the test the double row connection was 207% of the single row for 5d but only 165% for 3d spacing. The code did not consider row spacing for row shear, but the 5d spacing had 25% higher load than 3d. The difference between the steel-wood-steel and wood-steel-wood configurations was also found to be smaller than the code predictions.

The reliability indices for all the configurations were in the range of 5.68-8.62 for a performance factor of 0.7, much higher than the target reliability index of 2.8 based on steel design, especially for the two wood-steel-wood connections and the larger bolt diameter case. Some modifications to the CSA-O86 design provisions would be appropriate to reflect the proper failure mode and provide more consistent and economic predictions of the load capacity.

Reported by:

Chao (Tom) Zhang
Research Engineer

George Lee
Wood Engineering Scientist

Li Zheng
Graduate Research Assistant

Reviewed by:

Frank C. F. Lam, Ph.D., P. Eng.
Professor

1 INTRODUCTION

The recent change of the connection design provision in Engineering Design in Wood (CSA O86-09) included the following failure modes: bolt yielding (BY), row shear (RS), group tear-out (GT), net tension (NT), and splitting (SP). For load applied parallel to grain, only the first four types of modes were considered. This approach was expected to reflect more realistic behavior of the connection and help engineers to control the failure mode. Few works have been done to evaluate these design code provisions under different configurations. Thus a comprehensive database is needed to develop better understanding of this type of connections.

In this project five configurations of bolted connections have been considered with on two species: Western Hemlock (*Tsuga heterophylla*, referred as Hemlock in the report) and Douglas Fir (*Pseudotsuga menziesii,* referred as D-Fir), 140 specimens in total. The configurations differed in species, row spacing, bolt grade, bolt diameter, bolt numbers, etc. In combination with results from a previous test program (Zhang et al. 2013), the results provide an assessment of the effectiveness of the design code provisions, as well as the behavior of the connections.

2 TEST SETUP

2.1 Material

Five configurations of bolted connection were designed: three of Hemlock and two of D-Fir (sawn lumber). Each configuration consisted of 28 specimens. All specimens had three-member, double shear connections, identical on the two ends, in the form of either steel-wood-steel (SWS) or wood-steel-wood (WSW), as shown in Figure 1. The connections met the requirements specified in CSA O86-09 for wood connections.

The characteristics of the five groups are presented in Table 1. Several factors differed between the groups: species, bolt grade, bolt diameter, and row spacing. The last three groups were previous tests reported in *TEAM Report 2012-07 Connection Design for Post and Beam Construction Performance of Bolted Connection in Western Hemlock* (Zhang et al. 2013).

The Hemlock and D-Fir timber (No.2 or better) had two different cross sections: 130 mm by 130 mm and 105 mm by 105 mm. The length of the specimens was 770 mm.

The steel bolts were SAE J429 Grade 5 (G5) and Grade 2 (G2). The G5 bolts, for diameters 12.5 mm (1/2″) and 15.9 mm (5/8″), had a minimum tensile strength of 827 MPa (120,000 psi), and minimum yield strength 634 MPa (92,000 psi). The G2 bolts had a minimum tensile strength of 510 MPa (74,000 psi), and minimum yield strength 393 MPa (57,000 psi). The steel plates were 12.5 mm thick cold rolled mild steel.

The bolt holes on the wood were drilled by a SCM Record 110 AL TVN Prisma – 5 Axis CNC Machining Centre. The holes on the steel plate were drilled by a drill press. The tolerance of the holes on the wood and steel plates was 0.031 (1/32) mm.

Table 1 Specimen configuration

Group	Species	Type	Cross-Section (mm×mm)	Bolt grade	Bolt diameter (mm)	n_R	S_C	n_B	S_R	a_L
Hs.3d	H	SWS	130×130	G5	12.7	2	3d	4	5d	5d
Hw.3d	H	WSW	130×130	G5	12.7	2	3d	4	5d	5d
H5/8	H	SWS	105×105	G5	15.9	1	N	2	4d	5d
D5	D	SWS	105×105	G5	12.7	1	N	2	5d	5d
D2	D	SWS	105×105	G2	12.7	1	N	2	5d	5d
Hs.5d	H	SWS	130×130	G5	12.7	2	5d	4	5d	5d
Hw.5d	H	WSW	130×130	G5	12.7	2	5d	4	5d	5d
H1/2	H	SWS	130×130	G5	12.7	1	N	2	5d	5d

n_R: number of rows; S_C: row spacing; n_B: number of bolts in one connection; S_R: spacing of bolts in a row; a_L: end distance. *H*: hemlock; *D*: D-fir. *G5*: SAE J429 Grade 5; *G2*: SAE J429 Grade 2.

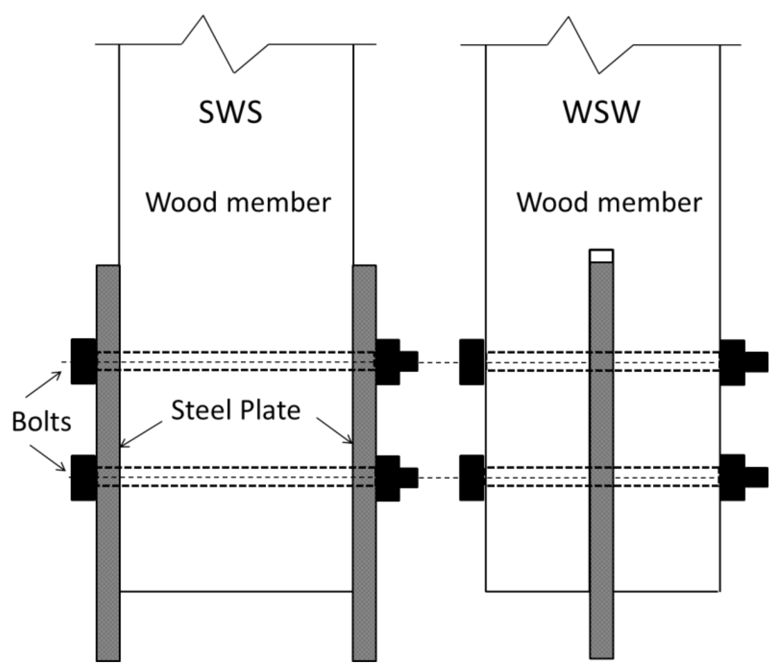

Figure 1 SWS and WSW configurations

2.2 Test setup

The tests were conducted in accordance with ASTM D5652-95 (2013) "Standard Test Methods for Bolted Connections in Wood and Wood-Based Products". The testing system consisted of a material testing system (MTS810), two linear voltage displacement transducers, a data acquisition unit and a computer. The accuracy of the equipment was ±0.05%. The MTS810 had a capacity of ±250 kN and a stroke of 76.2 mm. The two transducers were mounted on the wood surface at the bottom to measure the wood displacement with reference to the steel plates. A testing setup is shown in Figure 1.

The specimen was loaded in tension at a rate of 1.4 mm/min until its failure (when the load dropped below 80% of the peak load). Moisture content was measured by a Delmhorst Moisture Meter before testing.

Figure 2 Test setup

3 RESULTS

3.1 General

3.1.1 Moisture content

The moisture content of each group is shown in Table 2.

Table 2 Moisture content

Moisture content (%)	D2	D5	H5/8	Hs.3d	Hw.3d	H1/2	Hw.5d	Hs.5d
Average	11.3	11.6	12.9	10.8	11.0	10.8	14.2	13.3
Stdev	0.7	0.8	1.2	0.6	1.0	0.6	0.8	1.1
Cov	6%	7%	9%	6%	9%	6%	5%	8%

Note: the last three groups were previous tests in Zhang et al. 2013.

3.1.2 Load

The test results and the predictions according to CSA-O86-09 are shown in Table 3 and Figure 2. To compare with the test results, the predicted load was calculated without considering the resistance factor ϕ. And a load duration factor K_D=1.25 was used to convert to the case of test load duration (10-15 min). The code prediction was about 2-4 times the test results (non-parametric 5th percentile) and the difference was greater in higher strength connections.

Table 3 Summary of results

Peak Load (kN)	D2	D5	H5/8	Hs.3d	Hw.3d	H1/2	Hw.5d	Hs.5d
Average	68.26	74.92	106.73	135.91	129.89	74.52	144.76	165.92
Stdev	11.36	12.48	18.1	20.2	14.9	15.84	16.46	26.61
Cov	17%	17%	17%	15%	11%	21%	11%	16%
Max	98.98	101.86	143.05	174.82	154.35	122.32	167.19	230.81
Min	45.85	53.66	57.66	89.99	101.54	57.74	99.9	111.3
5th percentile	49.04	54.70	65.96	95.78	102.54	57.91	106.27	119.72
Predicted	30.00	30.00	30.00	41.28	27.86	29.72	34.86	52.83
5th percentile/ Predicted	1.63	1.82	2.20	2.32	3.68	1.95	3.05	2.27
Predicted/5th percentile	61%	55%	45%	43%	27%	51%	33%	44%
Failure mode	RS	RS	RS	RS	RS	RS	RS	RS
Predicted FM	RS	RS	RS	GT	GT	RS	RS	GT

Note: the last three groups were previous tests in Zhang et al. 2013.

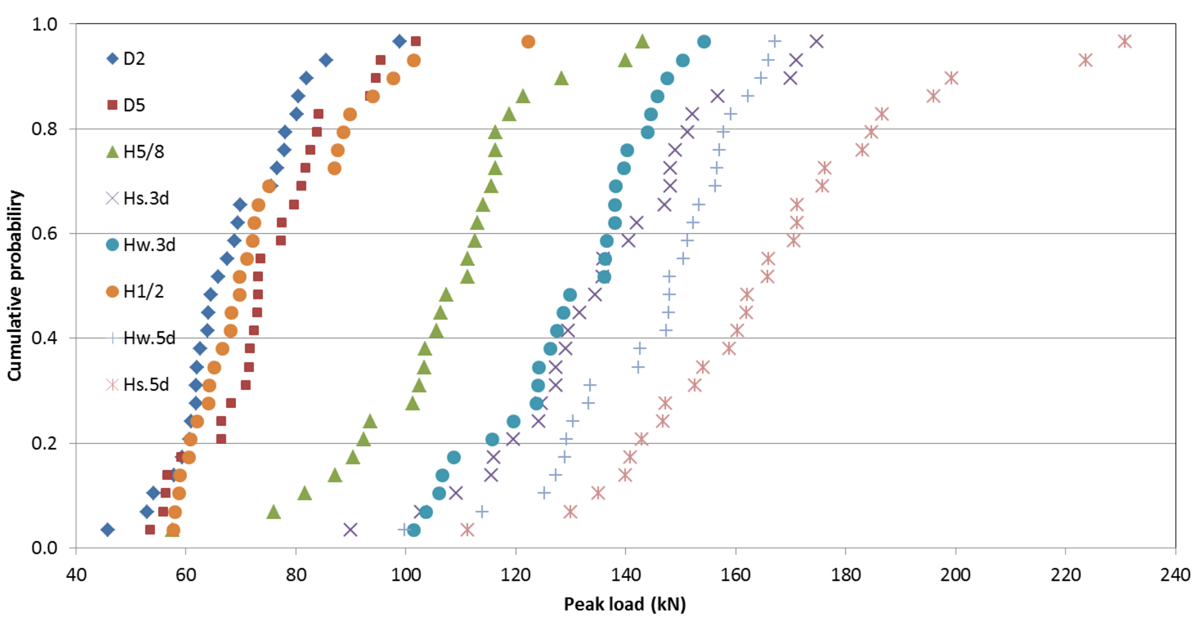

Figure 3 Test results

The proportional limit load and yield load were calculated according to ASTM D5652, as shown in Figure 3: the proportional limit load was the load at the end of the linear range of the load-deformation curve; the yield load was determined by the intersection of the load-deformation curve and the offset of the linear fit to the curve by 5% of bolt diameter. For those cases where these two curves did not intersect, the peak load was considered as the yield load. The average proportional limit load and yield load for each configuration are shown in Table 4. The proportional limit load was within the 60-80% range of the peak load. The yield load was above 78% of the peak load, and in five cases above 90%.

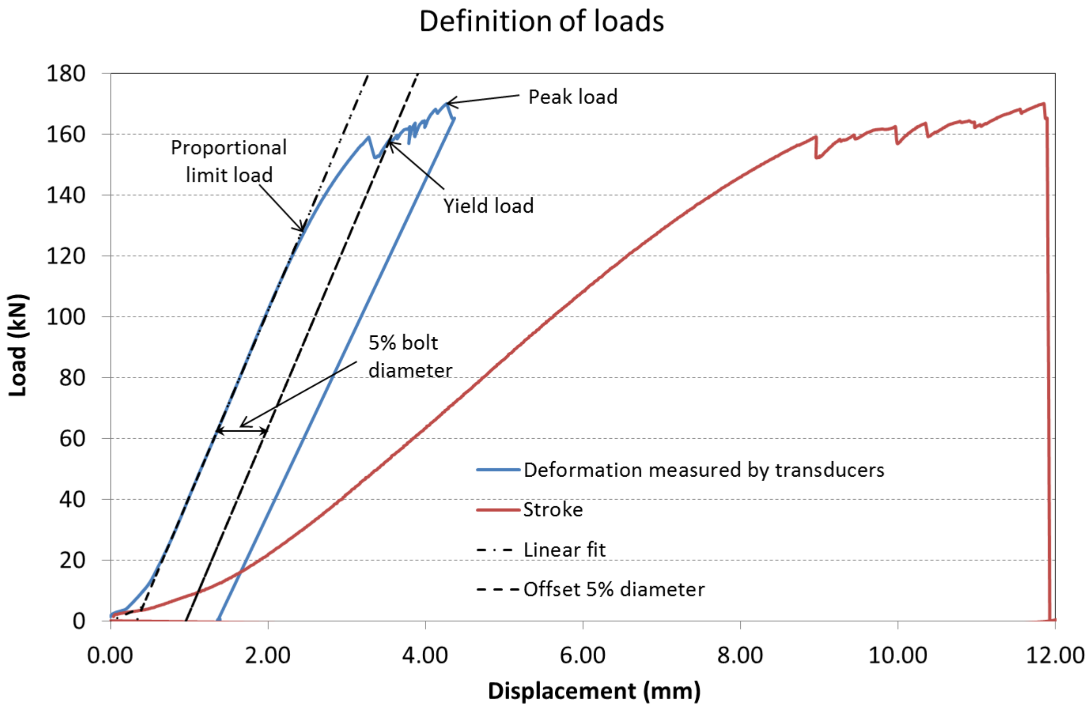

Figure 4 Definition of loads according to ASTM D5652

Table 4 Average proportional limit load, yield load, and peak load for each configuration

Load (kN)		D2	D5	H5/8	Hs.3d	Hw.3d	H1/2	Hw.5d	Hs.5d
Prop. limit load	Average	41.44	49.20	83.75	94.43	81.99	53.93	94.71	104.72
	Stdev	4.06	3.71	13.58	15.17	8.98	14.33	6.48	43.80
	Cov	10%	8%	16%	16%	11%	27%	7%	42%
Yield load	Average	56.61	67.84	105.08	132.42	127.03	64.77	134.58	129.09
	Stdev	4.15	6.45	15.65	20.01	14.42	7.87	12.15	12.87
	Cov	7%	10%	15%	15%	11%	12%	9%	10%
Peak load	Average	68.26	74.92	106.73	135.91	129.89	74.52	144.76	165.92
	Stdev	11.36	12.48	18.10	20.20	14.90	15.84	16.46	26.61
	Cov	17%	17%	17%	15%	11%	21%	11%	16%
Prop. limit load/peak load		61%	66%	78%	69%	63%	72%	65%	63%
Yield load/peak load		83%	90%	98%	97%	98%	87%	93%	78%

Note: the last three groups were previous tests in Zhang et al. 2013.

3.1.3 Failure mode

Most connections had brittle failure, except that several specimens with Grade 2 bolts exhibited some ductile behavior. Three examples of load-deformation curves from three different configurations are shown in Figure 5: the D2 specimen was ductile failure, where a large deformation between the proportional limit load and peak load occurred; the H5/8 specimen reached the peak shortly after the linear range; the Hs.3d reached its peak with not much deviation from the linear section of the curve. It is to be noted that not all D2 specimens showed similar ductile behavior as in Figure 5, some of them were brittle.

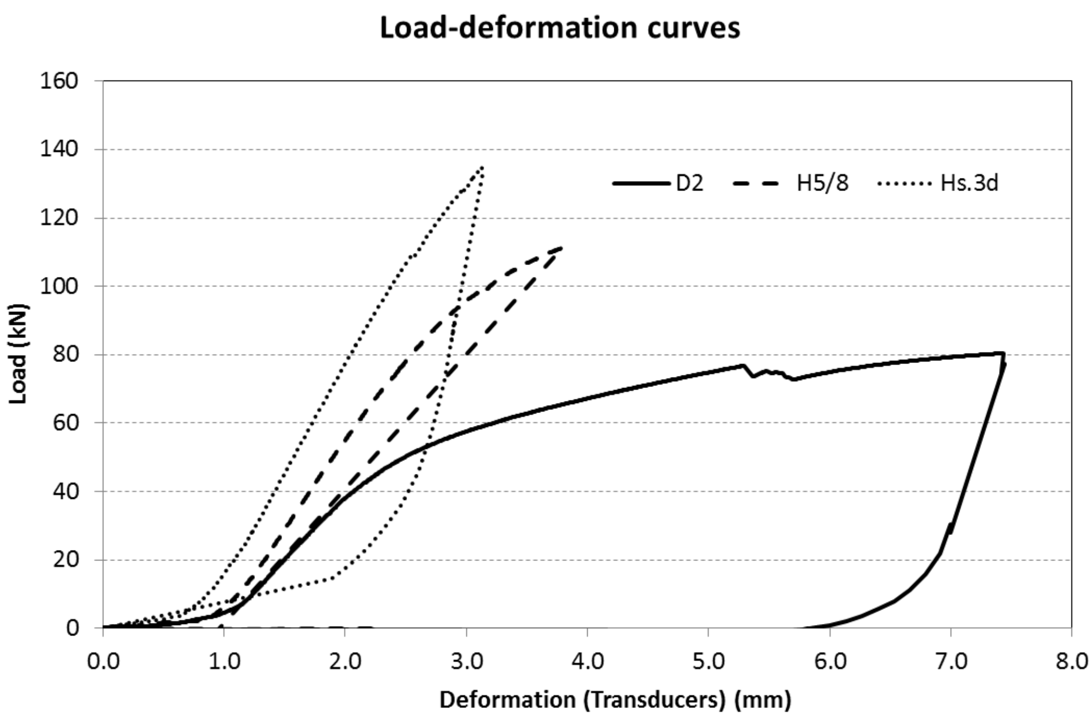

Figure 5 Examples of load-deformation relationships

Inspection of the failed specimens indicated that the apparent dominant failure mode in all the configurations was row shear (Figure 6): the shear failure occurred longitudinally on the two shear planes near the bolts. It should be noted that the stress state in these connections is very complicated; hence, observed apparent failure mode of row shear do not necessarily mean that only shear strength governs. Nevertheless in CSA-O86 the longitudinal shear strength f_v of the wood was used as the governing strength of this failure mode. The code specifies the f_v for D-Fir and Hemlock was 1.5 MPa and 1.2 MPa for all SS, No.1, and No.2 grades.

In all 140 specimens, only three specimens with 3d row spacing had group tear-out, as shown in Figure 7. Both shear and tension failures were involved here. CSA-O86 provisions for this failure mode indicate that both the tension strength parallel to grain and longitudinal shear strength determined the critical failure load. Two specimens had slope of grain failure induced by the row shear. The bolts were bent in various degrees but did not act as the controlling factor for the ultimate failure of specimens.

Figure 6 Row shear failures

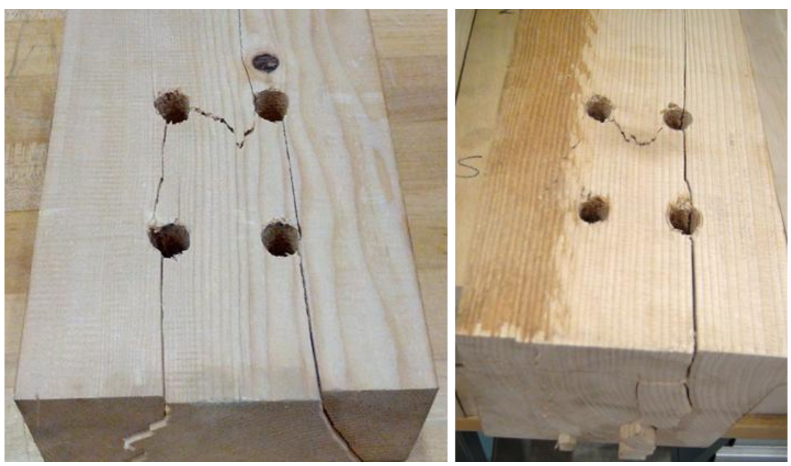

Figure 7 Group tear-out failures

3.2 Code comparison

The code predictions were plotted against the experimental results in Figure 6 (data in Table 3). Since row shear was the only dominant failure mode, the critical load for row shear in each case according to the code is also shown. The code predictions were found to be conservative. The ratio between the CSA-O86 values and the test results was in the range of 0.27-0.61, with an average of 0.45. This ratio for WSW configurations was much lower than the rest.

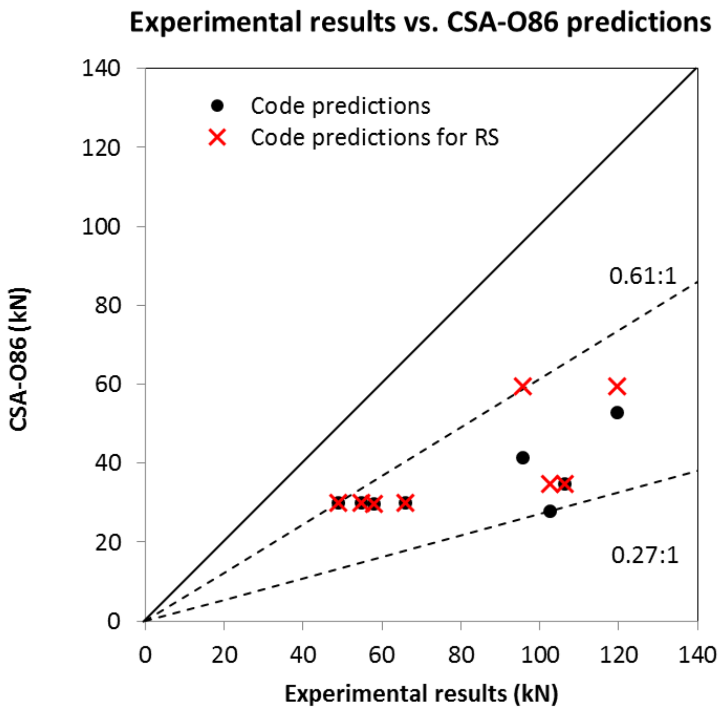

Figure 8 Code prediction and test results

3.3 Comparison by configurations

The eight configurations had differences in bolt grade, wood species, bolt diameter, row number, row spacing, and SWS/WSW designs. Each factor was examined and the effectiveness of the code to consider the factors evaluated.

Bolt grade: D2 and D5 had the same configurations except the bolt grade. D2 used SAE J429 Grade 2 bolts, and D5 SAE J429 Grade 5 bolts. The minimum tensile strength for Grade 5 was 827 MPa (120,000 psi), and minimum yield strength 634 MPa (92,000 psi). The minimum tensile strength for Grade 2 was 510 MPa (74,000 psi), and minimum yield strength 393 MPa (57,000 psi). This did not make significant difference in the peak load, although in average D5 was slightly higher. The low grade bolt provided more ductility in the connection: the average deformation between the proportional limit load and peak load for D2 was 2.72 mm, for D5 only 1.89 mm. The grade 2 bolts were bent more than the grade 5 bolts due to its lower strength.

The code predicted the failure mode correctly in both cases. The predicted load was about 55-61% of the test result. The code only considered the effect of bolt grade in the bolt yielding failure mode, therefore the prediction was identical for the two configurations.

Species: H1/2 and D2 had the same bolt configuration and grade. The predicted failure mode was correct in both cases and the predicted load was in the 51-61% range of the test result. Since H1/2 had a larger cross section than other single row configurations D2, D4, and H5/8, its test result and code prediction was adjusted proportionally to the size of 105 mm × 105 mm from 130 mm × 130 mm. Even though the code predictions were significantly lower than the test results, the influence of species was accounted for: the H1/2 was expected to be 20% lower than D5 in the code, and the result was found to be 15%.

Table 5 Single row configurations

Peak Load (kN)	D2	D5	H5/8	H1/2
5th percentile	49.04	54.70	52.77**	46.77*
Predicted	30.00	30.00	24.00**	24.00*
5th percentile/ Predicted	1.63	1.82	2.20	1.95
Predicted/5th percentile	61%	55%	45%	51%
Failure mode	RS	RS	RS	RS
Predicted FM	RS	RS	RS	RS

Note: * the results of H1/2 was adjusted to size 105 × 105 mm; ** the results of D5/8 was adjusted to end distance of 63.5 mm.

Bolt diameter: H5/8 had a larger bolt diameter than H1/2: the end distance was 79.4 mm and 63.5 mm respectively. The test result and prediction of H5/8 were adjusted to end distance of 63.5 mm in Table 5. The code predicted the correct failure mode. The test peak load in H5/8 was 13% higher than that of H1/2. Since the code did not consider bolt diameter in the row shear, its prediction was the same for the two cases.

Row number: Compared to H1/2, another row of bolts was added to Hs.3d and Hs.5d. This additional row increased the peak load by 65% in Hs.3d and 107% in Hs.5d. The code did not predict the correct failure mode: the dominant failure mode in Hs.3d and Hs.5d was row shear rather than group tear-out. The predicted load was 28% of the test result in both configurations.

Table 6 Double row configurations compared

Peak Load (kN)	*H1/2*	Hs.5d	Hs.3d	Hw.5d	Hw.3d
Row number	*1*	2	2	2	2
5th percentile	*57.91*	119.72	95.78	106.27	102.54
Predicted	*29.72*	52.83	41.28	34.86	27.86
5th percentile/ Predicted	*1.95*	2.27	2.32	3.05	3.68
Predicted/5th percentile	*51%*	44%	43%	33%	27%
Failure mode	*RS*	RS	RS	RS	RS
Predicted FM	*RS*	GT	GT	RS	GT

Note: the cross sections were all 130 mm × 130 mm in the test

Row spacing: Hs.5d had a wider row spacing than Hs.3d and the peak load increased 25%. The code predicted group tear-out but the test showed row shear in all but three specimens. This indicated that the code underestimated the critical load in the group tear-out mode in relative to row shear.

If only the prediction in row shear mode was considered in the two cases, the row shear failure load was identical: 59.44 kN. The code treated each row of bolts independently and simply multiplied the load under single row by the number of rows, thus the predicted load for both cases was two times the predicted load for H1/2. For Hs.5d, the actual load for Hs.5d was 207% of H1/2, while for Hs.3d, the actual load was 165% of H1/2. Although the code did not consider the effect of row spacing for row shear failure mode, the test showed it actually existed and its effect should not be ignored: changing row spacing from 3d to 5d had a peak load increment of 25% while the failure mode remained to be row shear.

SWS vs. WSW: For 5d spacing, the peak load of SWS connection was 15% higher than WSW; but for 3d spacing, the two types of connections had almost the strength. However, the code predicted 50% higher load in SWS than WSW in both row spacings. The failure mode for Hw.3d was predicted to be group tear-out, but actually it was row shear. For Hw.5d, the predicted failure mode was correct.

The predicted load in row shear mode for WSW was 34.86 kN, which did not consider row spacing either. This was 40% lower than that predicted for SWS. The test showed the difference between WSW and SWS was much smaller than the code expected: in the 3d row spacing cases, this difference was statistically insignificant. This indicates that the code may have underestimated the strength of WSW configurations, especially when the row spacing is small.

4 RELIABILITY ANALYSIS

The reliability of a structural component can be defined as the probability that it will achieve a predetermined level of performance in service. 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. A performance function, G, for a given design condition is given as:

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

Where the vector $X = X_1, X_2, X_3,\ldots, X_N$ are random variables associated within 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. 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:

$$P_f = \Phi(-\beta) \tag{2}$$

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:

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

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 10-15 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 are set to correspond with a standard load term of 3 months. To convert back to the case of test duration of 10-15 minutes a factor of 1.25 has been applied to increase the design value (Section 3.1.2). 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 versions corresponding to the four failure modes. The performance function can be written based on Equation (3):

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

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:

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

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, and the fitted lognormal parameters are shown in Tables 7.

Table 7 Fitted lognormal parameters

Specimen	D2	D5	H5/8	Hs.3d	Hw.3d	H1/2	Hs.5d	Hw.5d
Average	68.27	74.94	106.36	135.97	129.81	74.16	166.02	144.50
Stdev	11.63	13.11	20.40	21.66	15.68	14.62	27.53	17.55
Fitting error	2.0%	2.4%	11.4%	1.9%	2.3%	8.3%	1.4%	4.4%

Reliability analyses were performed under dead load, live load and snow load following the procedures outlined by Foschi et al. (1989). The statistical distributions and parameters for the dead load, live load, 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.

Tables 8 shows the summary results of the β values for each design provision. Considering the governing failure mode from the design provisions in the Canadian Code, *i.e.* Row Shear (RS) and Group Tear-out (GT), for ϕ=0.7 the associated β values were in the range of 5.68 to 8.62 for the combination of dead load and live load. The β values were in the range of 5.27 to 7.98 for the combination of dead load and snow load. Foschi et al. (1989) stated that the target β values should be in the range of 2.8 to be comparable to steel design. This shows that the design approach in the code is very conservative.

Table 8. Summary results of the β values (ϕ=0.7)

Test Group	β (dead load + live load)		β (dead load + snow load)	
	Row shear[#]	Group tear-out	Row shear[#]	Group tear-out
D2	5.68	N/A	5.27	N/A
D5	5.94	N/A	5.52	N/A
H5/8	6.88	N/A	6.44	N/A
Hs.3d	5.82	7.10*	5.39	6.60*
Hw.3d	8.19	9.13*	7.56	8.46*
H1/2	5.68	N/A	5.30	N/A
Hs.5d	6.43	6.84	5.97	6.36
Hw.5d	8.62	8.23*	7.98	7.61*

[#]: dominant failure mode; * where predicted failure mode did not match actual failure mode

The reliability index β and performance factor ϕ relations are shown in Figures 9 and 10. For a specific ϕ the large range of reliability index β was obtained for different configurations, which indicated the lack of consistency of the code. Three cases had much higher reliability indices: the two wood-steel-wood cases, and the larger bolt diameter case. That means current design provisions lead to excessive material consumption for such connections. For row shear failure mode, including the effect of bolt diameter and re-examining the design procedure for wood-steel-wood would be necessary to resolve such problems.

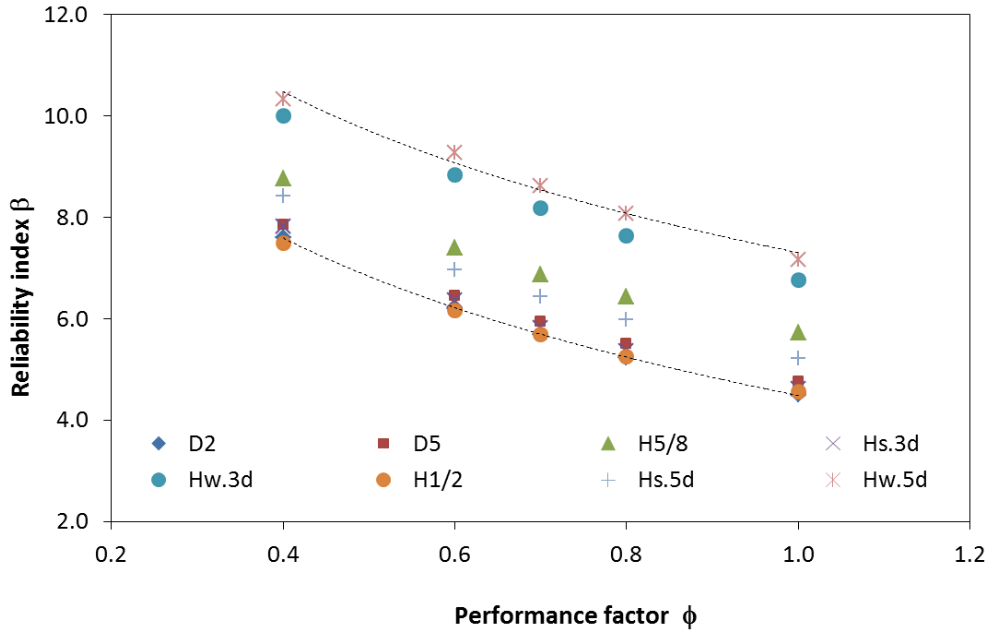

Figure 9 ϕ–β Relation for dead + live load

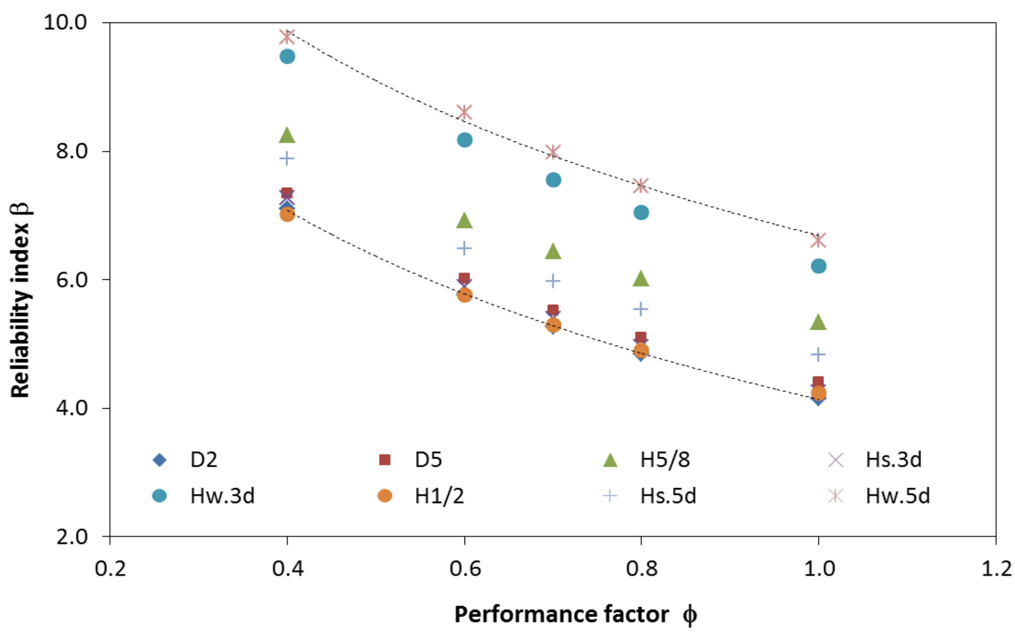

Figure 10 ϕ–β Relation for dead + snow load

Besides the reliability analysis above, Finite Element Analysis was conducted on the tested connections using ANSYS® by Mendler and Lam (2014). The splitting force (tension perpendicular to grain) was found to be important for the failure in row shear.

Thus considering the longitudinal shear alone was not sufficient to address the complicated stress states at such failures.

5 CONCLUSION

This project tested five configurations of bolted wood connections, in addition to the three tested previously. CSA-O86 was used to predict the peak load and failure mode for comparison:

1. The predicted peak load was in the range of 27-61% of the test results (5^{th} percentile). The predicted failure mode was row shear in five cases and group tear-out in three cases, but the actual dominant failure mode was row shear in all configurations. This indicated that the code had underestimated the critical load for group tear-out in relative to row shear.
2. The change of wood member from Hemlock to D-Fir was accounted for in the code prediction: 20% difference was expected and the actual was 15%.
3. The code did not consider bolt grade and bolt diameter in row shear failure mode. The test showed that while bolt grade did not affect the ultimate load significantly, the increase of bolt diameter did bring an increase of peak load by 13%.
4. For row shear, the code treated the load of multiple rows of bolts as the sum of the load calculated for each row. Thus the load for Hs.5d and Hs.3d, with two rows of bolts, were expected to be double of H1/2. Actually, Hs.5d was 207% of H1/2 while Hs.3d was 165%. Thus this assumption needs to be re-examined for narrower row spacings.
5. The code did not consider the change of row spacing as long as the failure mode was row shear. Thus it predicted the same critical load in row shear for 3d and 5d spacing in Hs.3d and Hs.5d. The actual load for Hs.5d was 25% higher than Hs.3d and both were row shear failure. The effect of row spacing to the row shear failure mode was significant.
6. According to the code, Hs.5d and Hs.3d were expected to have about 50% higher loads than Hw.5d and Hw.3d respectively. The test showed that: Hs.5d was 15% higher than Hw.5d, and Hs.3d almost the same as Hw.3d. Thus the difference between the SWS and WSW was much smaller than the code predicted. And this difference seemed to diminish as the spacing was reduced.
7. The reliability analysis found the reliability index for all cases tested was much higher than the value comparable for steel design. Among them, the wood-steel-wood connections and the large bolt diameter connection were much higher than the rest. It would be necessary to re-examine the design procedures for such connections in order to utilize the materials more efficiently.

6 REFERENCES

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