Edge Spacing of Pegs in Mortise and Tenon Joints

Final Report

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Introduction

Perpendicular to grain resistance is considered one of the weakest mechanical properties of wood due to the possibility of splitting. In most wood frame construction applications, perpendicular to grain loading is to be avoided. When these types of connections are created, the current wood design criteria in the United States imposes spacing restrictions which are meant to limit splitting potential. However, most of the connections which are detailed in the *National Design Specification for Wood Construction (NDS)* use metal dowels in wood connections (AF&PA 2005).

The use of perpendicular to grain connections is common in timber frame construction, where the standard fastener uses a wood dowel or peg rather than a metal dowel. Many timber frame joints cannot conform to the NDS spacing limitations, and previous practice, including research of mortise and tenon joints, found that acceptable joint strength was achieved at lower spacing intervals. In fact, Goldstein (1999) states that many pegged mortise and tenon connections fall outside of the spacing requirements of the NDS.

Current NDS Spacing

Splitting of the mortise member loaded by fasteners perpendicular to grain was one of the identified failure modes from previous timber frame joint studies, such as Schmidt and Daniels (1999), and Miller (2004). The current approach of the NDS in the United States is to limit splitting behavior by restricting the edge distance of fasteners. The loaded edge distance for dowels loaded perpendicular to grain is 4.0 times the diameter (4D). Listing distances as multiples of diameter of the bolt provides equivalent scaling. Moss (1997) provides a review of the research done to develop the current spacing restriction. Reference to work by Mettem and Page (1992) is made for perpendicular to grain loading, but little explanation of the choice of loaded edge distances is provided.

van der Put Model for Splitting Perpendicular to Grain

The current perpendicular to grain provisions for both the CSA 086 and Eurocode 5 are based upon a fracture mechanics model developed by van der Put (1990) to solve the case of notched beams, which is theoretically similar to the solution of a wooden member with a hole loaded perpendicular to grain. Simplifying assumptions included the use of superposition, neglecting normal forces in the member, and stable crack propagation. Crack propagation was initiated when the loss of potential energy due to cracking was equal to the required energy for crack formation. An additional requirement was that the fracture energy, G_f, associated with crack propagation, must be of sufficient magnitude to propagate the crack in both the length and width directions of the beam (van der Put and Leijten 2000). The relationship for splitting failure, derived by van der Put and Leijten (2000) in Equation 1.

$$\frac{V_f}{b\sqrt{h}} = \sqrt{GG_f}\sqrt{\frac{\alpha}{0.6(1-\alpha)}} \tag{1}$$

where,

α = h_e / h = location of the dowel with respect to the loaded edge and the beam height
b = beam width, mm.
h_e = loaded edge distance, mm.
h = beam height, mm.
G = shear modulus of the material, N/mm^2
G_f = fracture energy, N/mm
V_f = the maximum shear force at fracture, N

The fracture energy term, G_f, as referred to by van der Put and Leijten (2000), assumes a combined (mixed-mode) mode I and mode II interaction. This interaction was derived empirically by Petersson (1995), based on the relationship between tension perpendicular to grain stress and shear stress. However, a simplifying assumption of G_f based only on mode I fracture, is a reasonably accurate approximation (Schoenmakers 2006). An alternate form of Equation 1 is shown in Equation 2 (van der Put and Leijten 2000):

$$\frac{V_f}{b\sqrt{h}} = C_1\sqrt{\frac{\alpha}{(1-\alpha)}} \quad \text{with} \quad C_1 = \sqrt{\frac{GG_f}{0.6}} \tag{2}$$

Ehlbeck et al. (1989) and Ballerini (1999) conducted connection tests of nails and dowels loaded perpendicular-to-grain in simply supported beams and determined the apparent fracture parameter, C_1, through calibration (van der Put and Leijten 2000). Successful calibration was also achieved by Reshke (1999) who tested simply supported and cantilevered spruce glulam beams with steel-timber-steel bolted connections loaded perpendicular-to-grain, and Reffold et al. (1999) who tested punched metal plate connections loaded perpendicular-to-grain. This calibration determined the C_1 characteristic lower bound for loaded edge distance less than 70% of the depth of the member, defined as 2/3 of the lower bound, to be 10 N/mm$^{1.5}$ (van der Put and Leijten 2000). Substitution of this value into Equation 2, gives Equation 3.

$$\frac{V_f}{b\sqrt{h}} = 10\sqrt{\frac{\alpha}{(1-\alpha)}} \tag{3}$$

CSA 086

Section 10.4.4.7 provides the perpendicular to grain splitting resistance, QS_{ri}, for the CSA 086 design code (Equation 4). The QS_i term in Equation 5 has the same form as Equation 3 with the square root over depth term in the denominator removed, and the width (thickness) term brought to the right side of the equation (CSA 2009). The codes have an implicit assumption of 14.0

N/mm$^{1.5}$ for C_I. With the assumption of C_I = 14.0 N/mm$^{1.5}$, no material properties are included in the splitting equations in the CSA 086, or the Eurocode 5 design. In CSA 086, the equation for QS_i contains no terms related to the species of wood used in the connection (CSA 2009). Therefore, it is assumed that, according to CSA 086, all wood materials – regardless of species, density, grain orientation – have the same splitting resistance.

$$QS_{ri} = \phi_w QS_i \left(K_D K_{SF} K_T\right) \quad (4)$$

$$QS_i = 14t\sqrt{\frac{d_e}{1-\frac{d_e}{d}}} \quad (5)$$

Where,
t = member thickness, mm
d_e = effective member depth (loaded edge distance), mm
d = member depth, mm
f_w = resistance factor for brittle failures (0.7)
K_D = load duration factor (1.15 for short term loads)
K_{SF} = service condition factor (1.0 for dry use)
K_T = adjustment for pressure preservative treatment (1.0 for untreated lumber)

Eurocode 5 (EC5) Splitting Capacity

A manifestation of Equation 3 appears in the European design code (Eurocode 5 or EC5) as a specific check of the splitting capacity for perpendicular-to-grain connections in softwoods (ENV 2005-1-1, 2004).

$$F_{90,Rk} = 14bw\sqrt{\frac{h_e}{\left(1-\frac{h_e}{h}\right)}} \quad (6)$$

Where:
- $F_{90,Rk}$ = the characteristic splitting capacity, N
- w = modification factor for fastener type (1.0 for all but metal plate connectors)
- h_e = loaded edge distance of the most distant fastener, mm
- h = member height, mm
- b = member width, mm

Accounting for material type, load duration, and moisture content effects, the following equation (Section 2.4.3 of Eurocode 5) relates the characteristic value ($F_{90.Rk}$) to the design value ($F_{90,Rd}$) (ENV 2005-1-1, 2004):

$$F_{90,Rd} = k_{\mathrm{mod}} \frac{F_{90,Rk}}{\gamma_M} \qquad (6)$$

Where,

$F_{90,Rd}$ = design splitting capacity, N
k_{mod} = partial factor for material properties = 1.3 for connections
γ_M = modification factor considering load duration and service moisture content (= 0.9 for solid wood/LVL under a short term load (less than one week) with moisture content not exceeding 20%)

Fracture Mechanics Models for Predicting Splitting Perpendicular to Grain

Several researchers have applied principles of fracture mechanics to predict the capacity resistance achieved before splitting occurs, as fracture mechanics pertain especially to brittle failure modes such as splitting. While a variety of different fracture models using various theories and assumptions exist, two of the most common fracture mechanics concepts applied to the splitting perpendicular to grain are the previously mentioned van der Put model (van der Put and Leijten 2000) and the Jensen model (Jensen 2003). The Jensen model is described and several research studies that have used these models for splitting perpendicular to grain are presented.

A quasi-non-linear fracture mechanics model (Jensen et al. 2003) was based upon beam-on-elastic foundation (BEF) theory. A complete model derivation using the conventional stress method, finite element solution, and experimental validation was provided by Jensen et al. (2003). The model was based on an elastic Timoshenko beam with finite length, with support provided by linear elastic springs connected to a stiff foundation. The foundation stiffness of these springs model the perpendicular to grain strength and fracture performance of the wood. While this performance is non-linear, it is "represented by a linear response that is equivalent in terms of peak tensile stress perpendicular to grain, f_t, and fracture energy dissipations, G_f" (Jensen et al. 2003). Failure criterion for the beam was defined as when the maximum stress is equal to f_t of the wood (Jensen et al. 2003).

The model equations for failure load of a single dowel loading a beam perpendicular to grain are as follows (Jensen et al. 2003):

$$P_p = \mu P_{P,LEFM} = \mu b \sqrt{\frac{20}{3} G G_f h_e} \qquad (4)$$

$$\text{with } \mu = \sqrt{\frac{2\varsigma + 1}{\varsigma + 1}} \qquad \text{and } \varsigma = \frac{5}{\sqrt{3}} \frac{G}{E} \sqrt{\frac{2EG_f}{h_e f_t^{\,2}}}$$

where,

b = beam width, mm.

h_e = distance from the closest dowel to the loaded edge, mm.

G = shear modulus of the material, N/mm^2

G_f = fracture energy perpendicular to grain (mode I), N/mm

E = modulus of elasticity of the material, N/mm^2

f_t = tensile strength perpependicular-to-grain, N/mm^2

$P_{P,LEFM}$ = the failure load as a LEFM solution, N

P_P = the failure load, N

Finite element model (FEM) analysis of a ‘symmetrical beam with one or more dowels’ was performed by Jensen et al. (2003) in order to determine the accuracy of the derivation based on BEF theory. Good correlation was shown between experimental failure loads of laminated veneer lumber (LVL) plate and beam specimens, and theoretical predictions. The theory was also applied to structural glued laminated timber (glulam) beams with relatively low span to beam depth ratios, and plate tests, performed by Yasumura (2001), Quenneville and Mohammad (2001), and Kasim and Quenneville (2002). The glulam plates showed good agreement between theoretical predictions and experimental results. The capacity of the glulam beams was shown to be over-predicted for larger loaded edge distances (Jensen et al. 2003).

Snow et al. (2004a) analyzed the accuracy of the van der Put model for predicting the splitting capacity of perpendicular to grain, single-dowel connections in laminated strand lumber (LSL). Comparisons between experimental results of connections loaded to ultimate strength, and model predictions, concluded that model predictions were relatively accurate. However, LSL connection members typically failed in bending, rather than by splitting propagating from the connection. Additional work from Snow et al. (2004b) extended experimental analysis of perpendicular to grain single-dowel connections to include laminated veneer lumber (LVL) and parallel strand lumber (PSL). Both materials exhibited splitting behavior at failure; analysis of the van der Put model was not included in the scope of the project. It should be noted that the experimental connection configurations of Snow et al. (2004a) and Snow et al. (2004b) both consisted of a 3/4 in. diameter dowel loading the main member, of 3-1/2 in. depth, along the neutral axis. The loaded edge distance of this configuration, 2.33D, is considered unsatisfactory by current NDS requirements of a minimum 4D loaded edge distance for perpendicular to grain connections.

The models proposed by van der Put and Jensen represent current European work on connections causing splitting of wood perpendicular to grain. The van der Put model relies upon the dimensions of the connection (height, loaded edge height), as well as the shear modulus and fracture energy. The Jensen model uses the shear modulus, fracture energy, modulus of elasticity and tension strength perpendicular to grain. The simplification of the van der Put model that is currently used in the European design literature assumes a constant value of shear modulus and fracture energy for softwoods and is only dependent upon the dimensions of the connection.

The fracture energy and properties of hardwoods have been found by other authors, including the *Wood Handbook* (USDA 2011) and are available for use in these models. Considering the use of these equations for timber-frame joints with peg connections, the precedent set by the TFEC standard (TFEC 2010) of using the *Wood Handbook* properties seems most appropriate.

Previous Timber Frame Joint Testing

Schmidt and Daniels (1999) measured a variety of pegged mortise and tenon connections using different species and dimensions. Mortise and tenon joint species included southern yellow pine (*Pinus spp.*), recycled Douglas-fir (*Psuedotsuga menziesii*), and red oak (*Quercus rubra*). A range of peg diameters (0.75 inch to 1.25 inch), end distance (1.5D to 2D), and loaded edge distance (2D to 4D) were tested using a universal testing machine with a displacement rate of 0.0005 in/sec. Miller (2004) used a mobile test frame with a hand pump to evaluate yellow poplar (*Liriodendron tulipifera*) joints with various loaded edge distances from 1.5D to 3D.

Joint failures included peg failures, mortise splitting, and tenon splitting. Shear/bending was the dominant peg failure mode. Southern yellow pine joints with 1.25 inch diameter octagonal oak pegs did not reach 5% offset yield, but rather failed due to the development of a single flexural peg hinge within the tenon. A peg bearing failure identified as Mode I_d was discovered from a review of previous research. Joint stiffness was not determined. Specific gravity was the major factor affecting peg bending yield strength with a positive correlation of 0.73. Peg shear yield strength increased with specific gravity. (Schmidt and Daniels 1999)

Purpose of Project

The purpose of this project was to examine the perpendicular splitting behavior of pegged mortise and tenon connections. The focus of this work is the splitting of the mortise member only. To prevent failures related to the tenon or peg, these elements were replaced with steel components. Results of this work will be compared against previous testing of mortise and tenon pegged joints to check if these modifications affects the connection. Another change to the experimental methods was the reuse of beams to allow testing on both faces. If the splitting failure is only engaging wood fibers between the peg and loaded edge, then the wood fibers on the other side of the beam should remain unloaded. Experimental results will be compared with CSA 086 provisions for splitting perpendicular to grain, previous testing of joints by Schmidt and Daniels (1999), and van der Put and Jensen fracture models.

Methods and Materials

The test protocol (Table 1) was developed to encompass the common range of species in timber framing as well as the range of edge spacing previously tested by Schmidt and Daniels (1999) and Miller (2004). A total of 44 tests were conducted on 22 individual specimens. The doubling of the sample size was accomplished by turning each beam over and drilling a second set of holes for testing. When possible, different hole sizes were used to make sure that the maximum distance between the holes was maintained (i.e, a 3.0 inch edge distance was used on one side, and a 1.0 inch edge distance was used on the other).

Table 1. Experimental Testing of Timber Frame Mortises

Mortise Species	Edge Distance, in	Number of Samples
Douglas fir (Pseudotsuga menziesii)	1.0”	4
	1.5”	4
	2.0”	4
	2.5”	4
	3.0”	4
Eastern White Pine (*Pinus strobus*)	1.5”	4
	2.0”	4
	2.5”	4
	3.0”	1
White Oak (*Quercus alba*)	1.5”	3*
	2.0”	3
	2.5”	2
	3.0”	3*

* One sample of the White Oak 1.5” and two samples of the White Oak 3.0” used ¾” diameter bolts due to difficulties installing 1” diameter bolts

Test specimens were 6x8 nominal timbers 6 feet long with a 6 inch long by 2 inch wide through mortise centered at the midspan of the beam. A steel tenon (Figure 1) was constructed of solid steel 6 inches wide by 2 inches long by 15 inches long. The tenon had 2 – 1-1/16 inch diameter holes spaced 3 inches apart. Instead of standard wooden pegs, the connection used 1 inch diameter Grade 8 bolts. In three instances, difficulty in installing the 1 inch diameter bolts led to the installation of ¾ inch diameter bolts for the White Oak Samples.

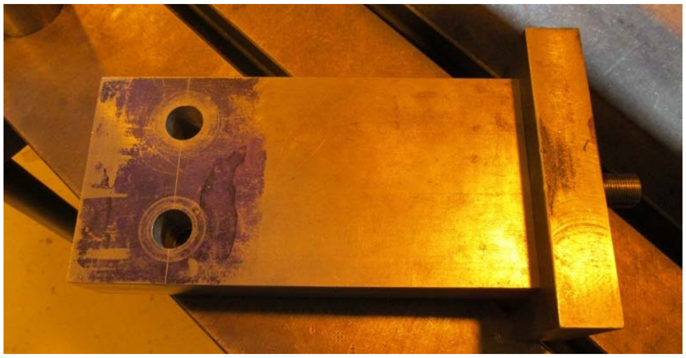

Figure 1. Steel Tenon Used for Mortise Member Testing

Figure 2 is a photograph of the testing setup with directional arrows showing the transfer of load from the crosshead to the test frame. An MTS universal testing machine with a 50,000 lb. capacity was used for all testing. The crosshead and tenon member had an upward displacement rate of 0.01 in/min. Holddowns were placed 4.5 feet apart and were attached to the support table by threaded rods to restrain the mortise member. Load and displacement of the tenon member were measured by the integrated load cell and LVDT within the MTS universal testing machine crosshead. A separate LVDT was placed at the edge of the mortise member to measure the movement of the mortise member. Load and displacement were continually monitored throughout the test. Specimens were loaded until failure occurred and at least a 10% drop in load occurred. From the data collected, the yield load and maximum load were found. Initial linear stiffness of the joint was calculated as the load divided by the difference in displacement of the crosshead and displacement of the mortise member.

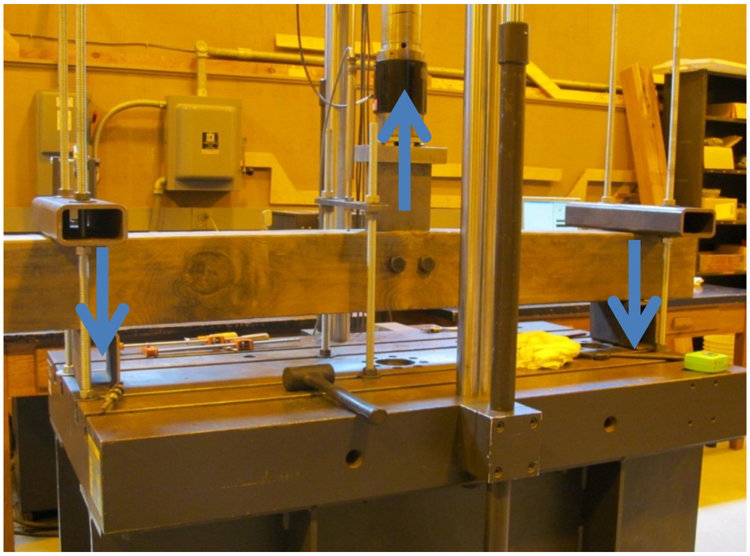

Figure 2. Loading of Mortise Specimens Showing Steel Tenon and Holddowns

Results

The average and coefficient of variation (COV) values for the specific gravity, yield load, maximum load and stiffness are given in Table 2. Specific gravity values are listed for each species rather than individual edge distances to avoid repetition. In general, as the edge distance increased, the yield load, maximum load and stiffness increased. Interestingly, many of the values decreased at the 3.0 inch edge distance. Many of the yield load values were similar or only slightly less than the maximum load, demonstrating the brittle nature of these joints. The ¾ inch diameter bolts used in some of the White Oak specimens did not affect the average results.

Table 2. Experimental Testing Results of Mortise Members

Species	Edge Distance, in	Yield Load, lbs (COV)	Maximum Load, lbs (COV)	Stiffness, lb/in (COV)
Douglas fir, SG = 0.436 (6.6% COV)	1.0	2410 (29.9%)	2570 (28.5%)	41,500 (58.0%)
	1.5	3100 (15.1%)	3790 (22.5%)	53,500 (9.7%)
	2.0	4780 (19.4%)	5350 (10.6%)	102,000 (27.0%)
	2.5	4700 (35.0%)	5270 (37.3%)	89,600 (25.0%)
	3.0	7380 (27.0%)	7920 (28.8%)	72,200 (18.0%)
Eastern White Pine, SG= 0.331 (7.7% COV)	1.5	2480 (27.5%)	2920 (38.7%)	39,300 (31.9%)
	2.0	2940 (45.0%)	3360 (35.8%)	63,700 (30.0%)
	2.5	4030 (48.3%)	4200 (43.0%)	62,300 (45.3%)
	3.0	4200 (59.2%)	4550 (58.7%)	60,700 (23.2%)
White Oak, SG=0.696 (8.3% COV)	1.0	8000	8000	102,000
	1.5	7900 (23.6%)	8700 (18.7%)	90,500 (4.4%)
	2.0	10,300 (25.8%)	11,700 (32.7%)	95,700 (21.0%)
	2.5	13,900 (0.1%)	16,000 (1.2%)	134,000 (2.8%)
	3.0	9900 (12.4%)	10,400 (16.3%)	70,000 (43.8%)

Figure 3 is a graph of a typical load-displacement curve. After an initial settlement loading, the load and displacement increase linearly until yielding, which was accompanied by a small crack or splitting between the two holes. As the load continues to increase, the maximum load is achieved and is accompanied by a large crack.

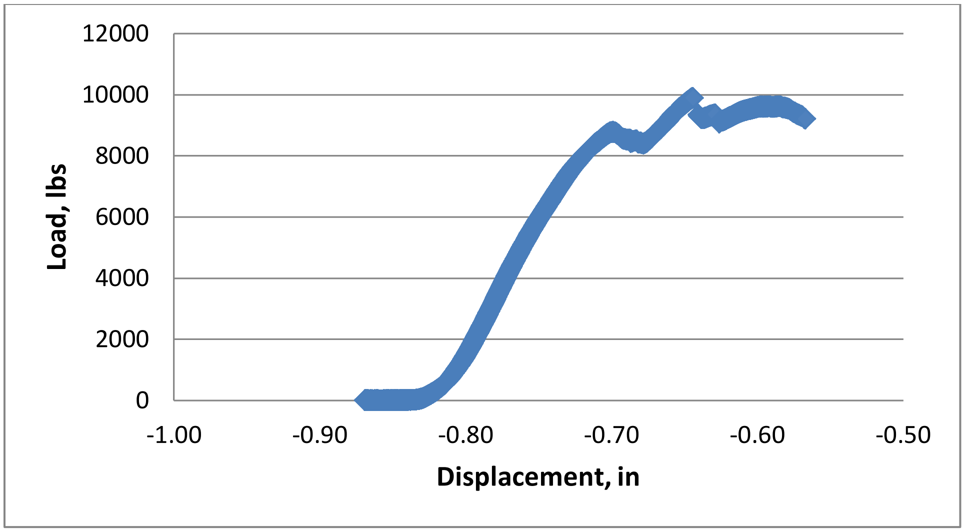

Figure 3. Load-Displacement Curve of a Mortise Specimen Showing Brittle Behavior

Four different failures of the specimens were identified. All failures were brittle. The most common failure was a splitting of the beam at or near the centerline of the bolts (Figure 4). Some of the specimens with larger loaded edge distances (2.5*D* to 3*D*) experienced splitting in conjunction with a bending failure (Figure 5) on one side of the split. Another failure observed in only a few beams was a shear crack (Figure 6), which extended from one bolt to the loaded edge at a 45 degree angle. The shear crack occurred near a large knot on the top surface of the mortise member. This crack was often associated with a knot near the mortise. Finally, one specimen, the third white oak beam with a 2*D* loaded edge distance, experienced a split in line with the previously tested holes on the specimen. The split (Figure 7) traveled along the entire length of the specimen and acted to reduce the section, thereby lowering the stiffness value of this sample.

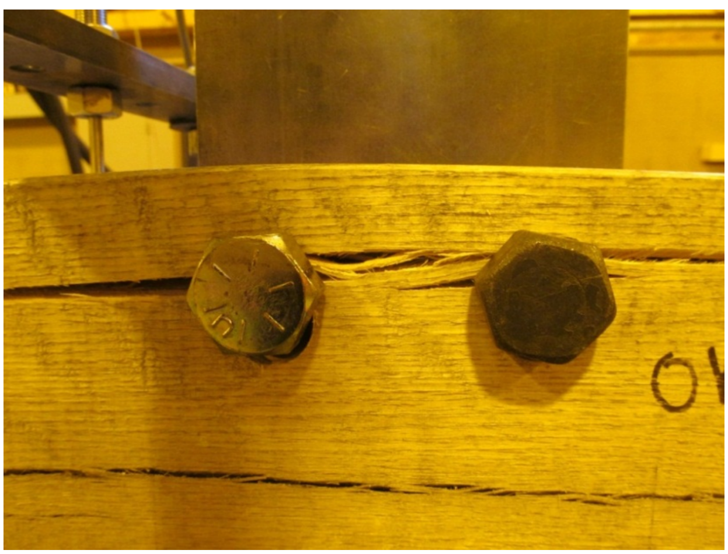

Figure 4. Splitting Failure Observed for Majority of Samples

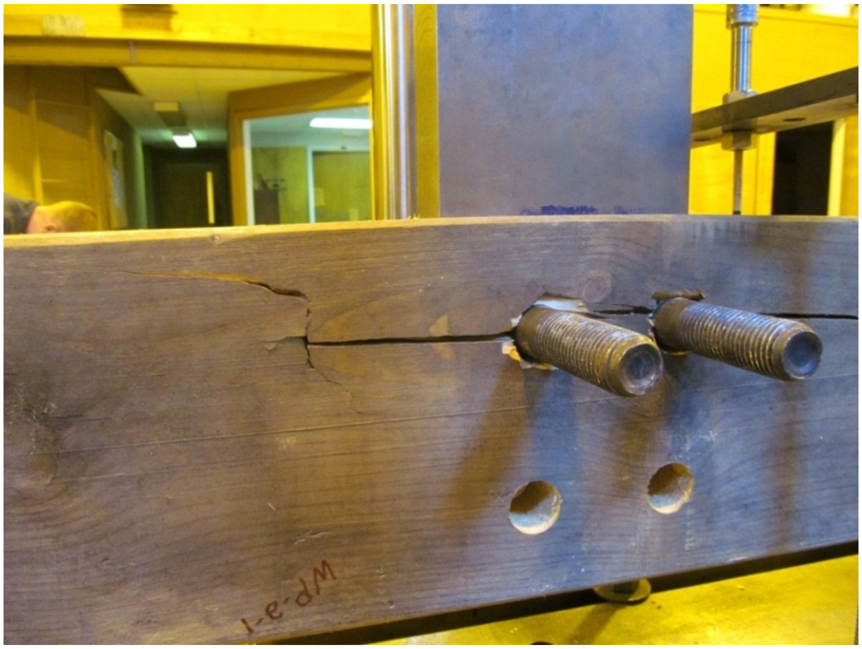

Figure 5. Splitting and Bending Failure Observed for Larger Loaded Edge Distances

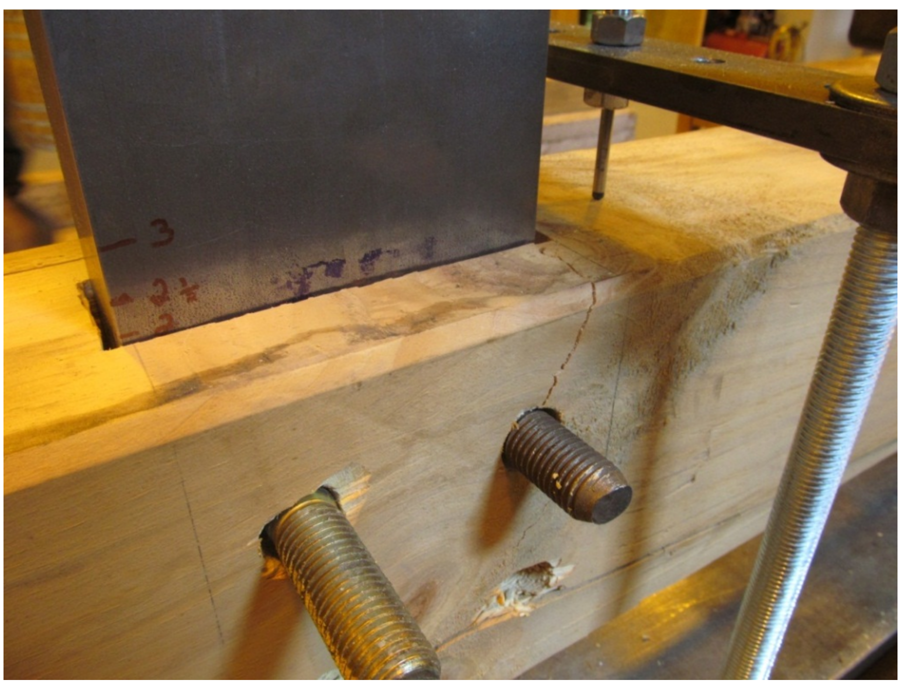

Figure 6. Shear Crack Failure At Bolt Hole

Figure 7. Split Failure Extending Through Previously Tested Bolt Holes

The experimental values were statistically compared to examine differences in the species, edge distance, and specific gravity of the mortise. The factors of species and specific gravity were used to determine if the difference in experimental values was due to characteristics of a species (i.e., wood anatomy) versus a change in specific gravity. The p-values from an analysis of variance (ANOVA) test are shown in Table 3. For the yield load and maximum load, both the specific gravity and edge spacing were significantly different ($p < 0.05$). For the stiffness, only the edge distance was significantly different ($p < 0.05$), while the specific gravity was not significantly different ($p = 0.4645$). The species factor was not significant for any of the experimental measures, indicating wood anatomy was not found to result in the different mechanical properties.

Table 3. Statistical Results of Edge Spacing and Specific Gravity Factors (α=0.05)

Factor	Yield Load	Maximum Load	Stiffness
Species	0.3981	0.3259	0.0635
Edge Spacing	0.0001	0.0003	0.0004
Specific Gravity	0.0056	0.0063	0.4645
Interaction	0.1889	0.0318	0.0238

Interaction of the edge distance and specific gravity terms, however, were significant for the maximum load and the stiffness. Investigation of the interaction plots shows a consistent trend for the maximum load; however, for the stiffness, the 2.0 and 3.0 edge distances decreased as specific gravity increased, while the other edge distances did not. For the yield load and maximum load, the specific gravity showed a positive trend (greater specific gravity, greater load).

For the edge distance values, a Tukey's Honestly Significant Difference (HSD) test was conducted. The Tukey's HSD is able to identify sub-groups, denoted by letters, of significant or not significant factor levels within the edge distance term (Table 4). For the yield load, maximum load, and stiffness, values for the three largest edge distances (3.0, 2.5, 2.0) were NOT significantly different. For the yield load and maximum load, the values at 2.0 and 1.5 were not significantly different, and the values at 1.5 and 1.0 were not significantly different. The stiffness values at 3.0, 1.5 and 1.0 were not significantly different, which is the basis for the previous comments about some of the stiffness terms at the 3.0 loaded edge distance being less than the proceeding loaded edge distance. The edge distance terms that were considered not significantly different may only require future testing of fewer levels – say 1.0, 2.0, 3.0 edge distances – to adequately assess the change in load due to edge distance. This finding may help to optimize future mortise and tenon testing.

Table 4. Tukey's HSD Results for the Edge Distance Factor

Edge Distance	Yield Load	Maximum Load	Stiffness
3.0	A	A	A B
2.5	A	A	A
2.0	A B	A B	A
1.5	B C	B C	B
1.0	C	C	B

Experimental Results Compared to CSA 086 / EC-5

The calculated values of QS_{ri} from Equation 4 are shown in Table 5 for different edge distance and diameter of peg. The width value, t, in Equation 4 is equal to the mortise width minus the section removed for the tenon. These values are insensitive to mechanical properties of the wood, or changes in species or specific gravity. The appendix section shows the values used and the calculation of the QS_{ri} values.

Table 5. QS_{ri} Results From CSA 086 For Comparison to Experimental Values

Edge Distance, *D*	Diameter of Peg (*D*), in	QS_{ri}, lbs
1.5	0.75	1310
3	0.75	2040
1	1	1220
1.5	1	1550
2	1	1880
2.5	1	2200
3	1	2540

Figure 8 is a graph of the maximum load for each wood species and the CSA 086 values from Table 5. Each species is highlighted in a different color and shape. Note the white oak values that reflect the previous discussion on specific gravity, while the Douglas fir and eastern white pine values are similar. The line represents the CSA 086 values, which do increase with an increase in loaded edge distance. The CSA 086 values appear lower on the chart, making them conservative compared to the average values for the white oak, Douglas fir and eastern white pine tested.

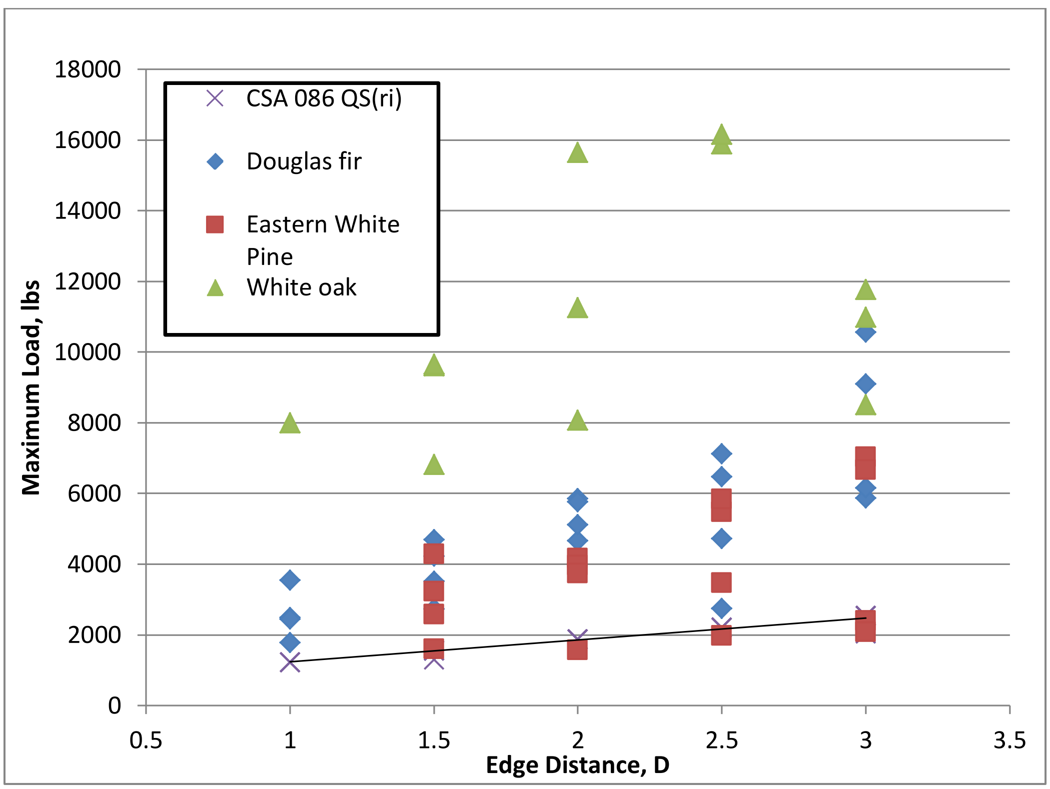

Figure 8. Comparison of the Experimental Maximum Load and CSA 086 Values

As the specific gravity increases, the CSA 086 values become even more conservative. For instance, the CSA 086 curve in Figure 8 has two of the eastern white pine load values above and below the curve, while the curve has all Douglas fir values above the curve. All white oak values are approximately three times the CSA 086 curve. The van der Put model, which the CSA 086 was derived from, was focused on softwood materials, particularly spruce, with specific gravity values similar in the range of 0.4 to 0.5, which is similar the Douglas fir values. Incorporating a specific gravity term in the QS_{ri} equation for timber frame mortise members may improve the prediction. However, the current CSA 086 values can serve as a lower bound for Douglas fir, eastern white pine, and white oak mortise members.

Experimental Results Compared to Schmidt and Daniels

Figures 9, 10, 11 are graphs of the maximum load and edge distance for the experimental results, and data from Schmidt and Daniels (1999) for each species. Work by Schmidt and Daniels (1999) was used for comparison due to the similarities in specific gravity with the experimental results. Figure 9 is a comparison of Douglas fir values (average SG = 0.436) with Recycled Douglas fir (average SG = 0.505). Figure 10 is a comparison of eastern white pine values (average SG = 0.331) with southern yellow pine (average SG = 0.55). Figure 11 is a comparison of red oak values (average SG = 0.696) with red oak values (average SG = 0.65).

The values from Schmidt and Daniels (1999) are all within the range of the experimental values tested. Statistical tests could not compare the experimental data values to Schmidt and Daniels (1999) because of the small number of samples. It should be noted that some of the Schmidt and Daniels (1999) data used other variables not considered in this work, such as changes in peg diameter and peg species. A full listing of the Schmidt and Daniels (1999) data used is shown in the Appendix. The testing of both sides of the mortise members did not seem to affect the maximum load values.

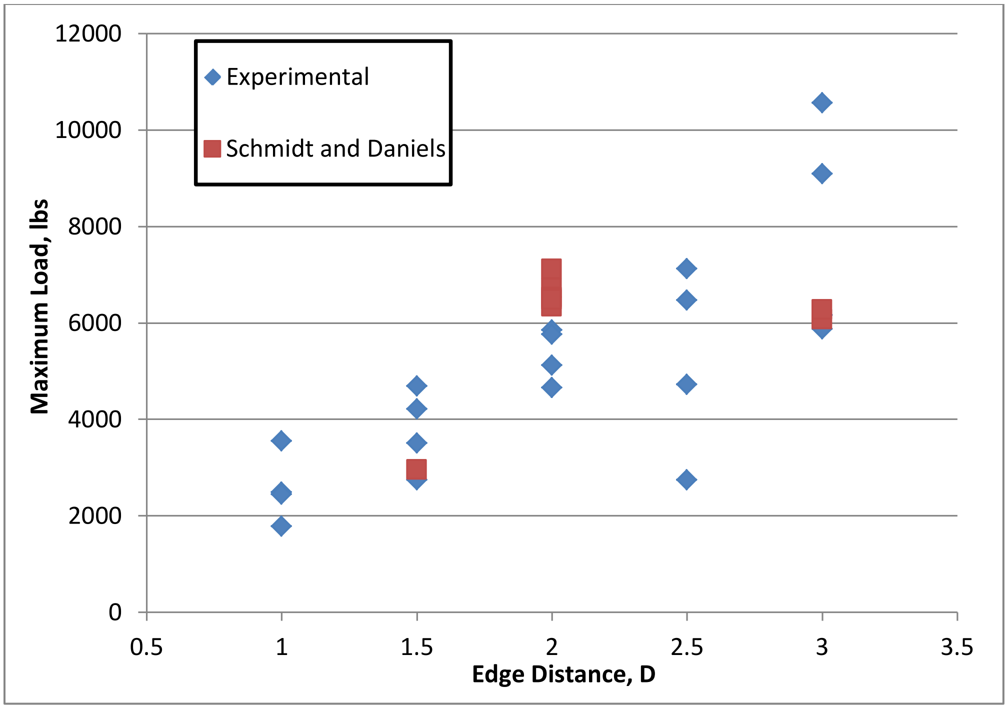

Figure 9. Comparison of Douglas fir Experimental Values with Recycled Douglas fir Values from Schmidt and Daniels (1999)

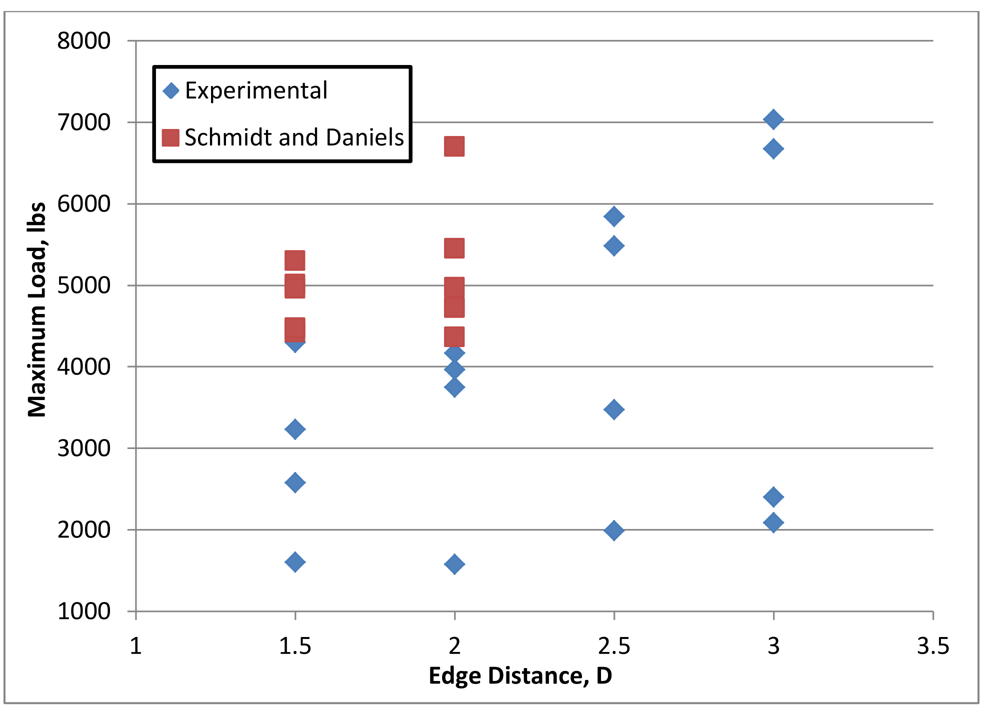

Figure 10. Comparison of Eastern White Pine Experimental Values with Southern Yellow Pine Values from Schmidt and Daniels (1999)

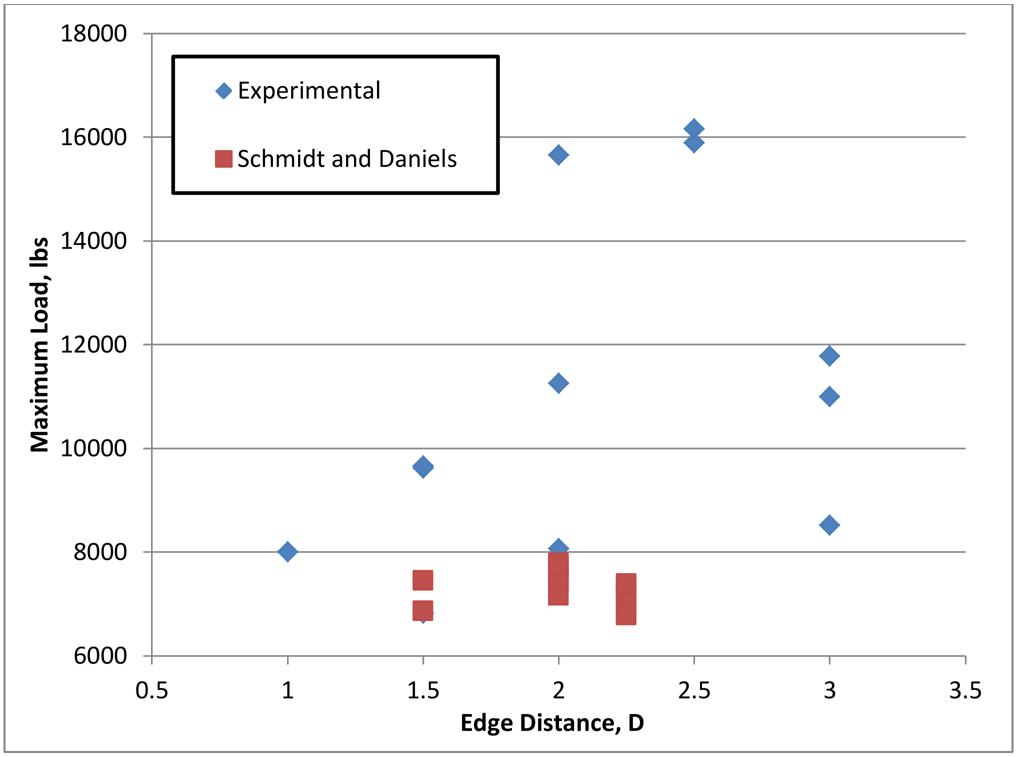

Figure 11. Comparison of White Oak Maximum Load with Red Oak Values from Schmidt and Daniels (1999)

Figures 12, 13 and 14 are graphs of the stiffness and loaded edge distance for the experimental results and data from Schmidt and Daniels (1999) for each species. Similar to the maximum load values, the stiffness terms from Schmidt and Daniels (1999) overlapped the experimental results, indicating similar values from the two test methods.

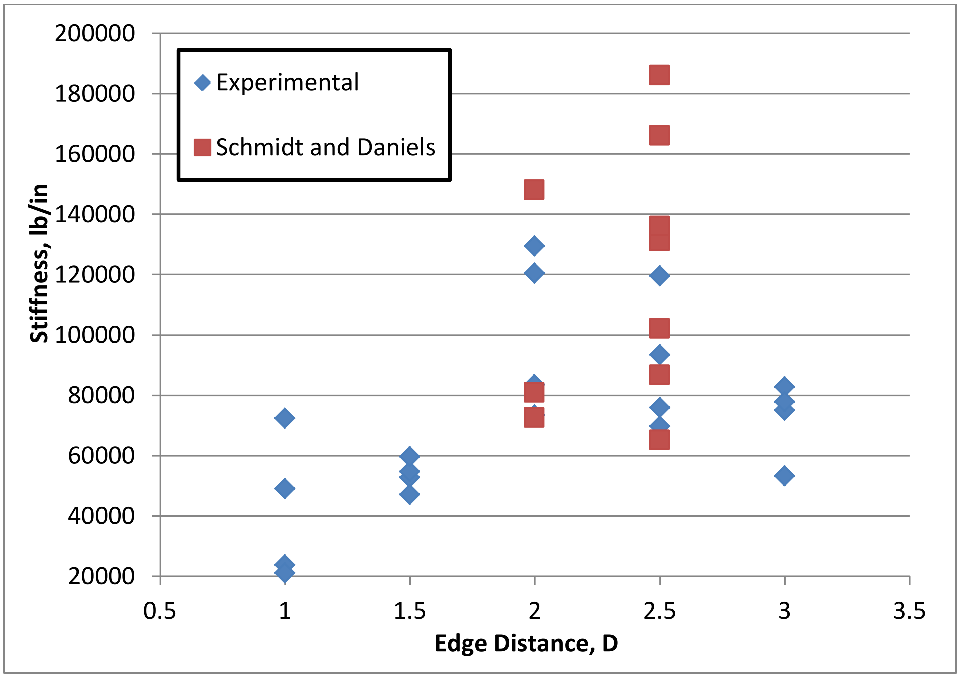

Figure 12. Comparison of Douglas fir Stiffness Values with Recycled Douglas fir from Schmidt and Daniels

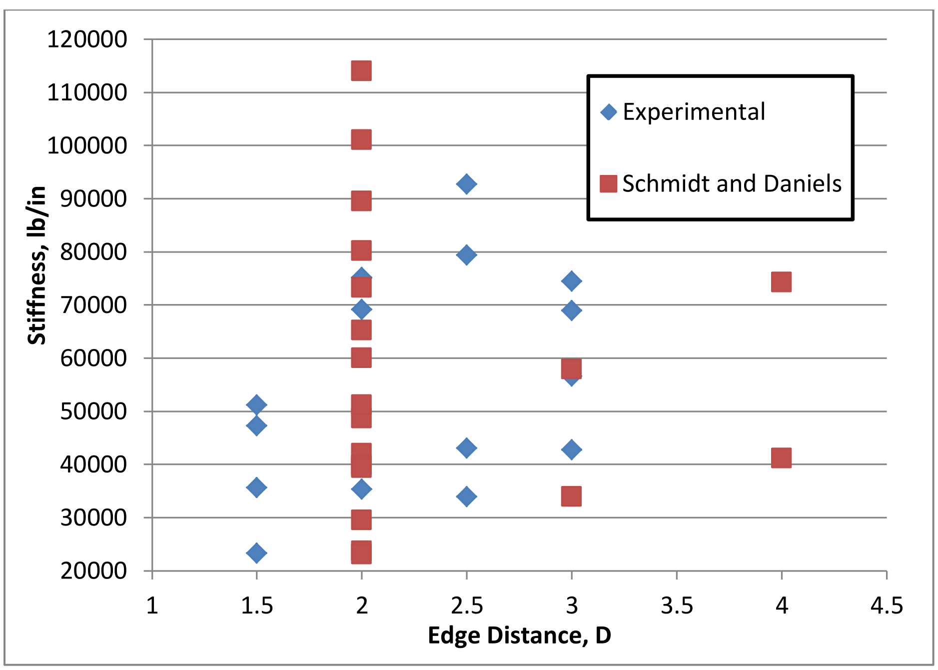

Figure 13. Comparison of Eastern White Pine Stiffness Values with Southern Yellow Pine Values from Schmidt and Daniels (1999)

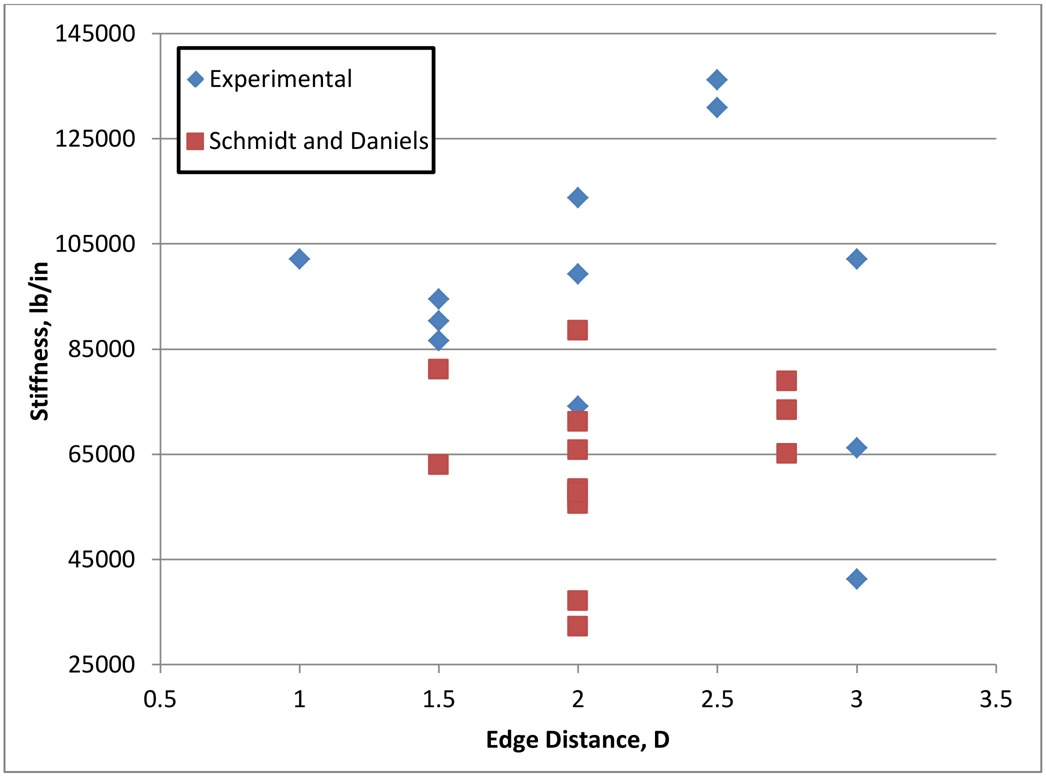

Figure 14. Comparison of White Oak Stiffness Values with Red Oak Values from Schmidt and Daniels (1999)

Experimental Results Compared to Other Fracture Models

In addition to comparisons of the CSA 086 design values and previous research by Schmidt and Daniels (1999), comparisons of the experimental results were made with fracture mechanics predictions from the van der Put model and the Jensen model. To create the fracture mechanics model predictions, the mechanical properties of the mortise members were needed. Samples of the mortise members were taken after testing, and there are future plans to measure the modulus of elasticity, tension perpendicular to grain, and the fracture energy. Unfortunately, it does not appear that shear modulus can be examined for these specimens. Shear modulus testing requires either a long beam for torsion or repeated bending tests, or a large, thin square plate for anti-clastic loading.

Using properties in the *Wood Handbook*, sets of mechanical properties for the Douglas fir, eastern white pine and white oak are shown in Table 6. Values for modulus of elasticity and tension strength perpendicular to grain were given at green and 12% moisture content values. Using the formulas in the *Wood Handbook* and the moisture content of the individual beams measured after testing, the modulus of elasticity and tension strength perpendicular to grain were estimated for each individual beam (See Appendix). Shear modulus is presented as a ratio of the modulus of elasticity after moisture content was adjusted. The fracture energy values were not found in the *Wood Handbook*. A value of 4.37 lb/in was measured by Finkenbinder (2007) for southern pine material. This value was used for all three species to gain an estimate of the fracture capacity.

Table 6. Mechanical Properties of Mortise Members for van der Put and Jensen Fracture Models

Mechanical Property	Douglas fir (Coastal)	Eastern White Pine	White Oak
Modulus of Elasticity (E)	1.56×10^6 psi [Green] 1.95×10^6 psi [12% MC]	0.99×10^6 psi [Green] 1.24×10^6 psi [12% MC]	1.25×10^6 psi [Green] 1.78×10^6 psi [12% MC]
Ratio of Elastic Modulus to Shear Modulus (G), G_{LR}:E_L	1.0 : 0.064	1.0 : 0.052[1]	1.0 : 0.086
Tension Perpendicular to Grain Strength (f_{tperp})	300 psi [Green] 340 psi [12% MC]	250 psi [Green] 310 psi [12% MC]	770 psi [Green] 800 psi [12% MC]
Fracture Energy (G_f)	Used Eastern White Pine Value	4.37 lb/in	Used Eastern White Pine Value

[1] Since no Eastern White Pine value was available, Western White Pine was used.

Figures 15, 16, and 17 are graphs comparing the maximum load experimental results with values from the van der Put and Jensen models. For all three wood species, the van der Put values were less than the Jensen values. Both the van der Put and Jensen models produced similar slopes (van der Put slope = 1907.5 lbs/inch, Jensen slope = 2324.5 lbs/in). As seen in the CSA 086 values, the van der Put and Jensen models do not account for specific gravity. These models were calibrated for softwood timbers and did not include an adjustment for specific gravity. For the Douglas fir and eastern white pine, the values for the van der Put and Jensen models were greater than all experimental data. For the white oak, the values of the van der Put and Jensen models were close to the average experimental values. Finkenbinder (2007) also found that the van der Put model overestimated LVL bolted connections. Improvements to the mechanical property estimates may lower the value of the models. However, these models are not seen as a great improvement over the CSA 086 model used.

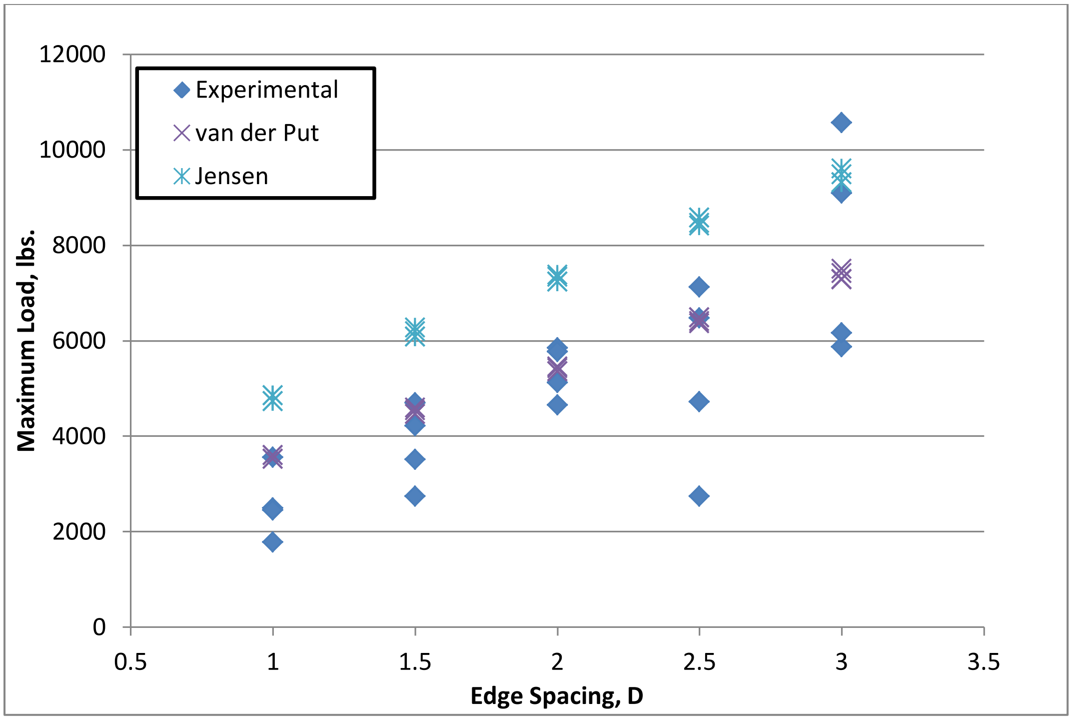

Figure 15. Comparison of Douglas fir Maximum Load Values with Fracture Mechanics Models

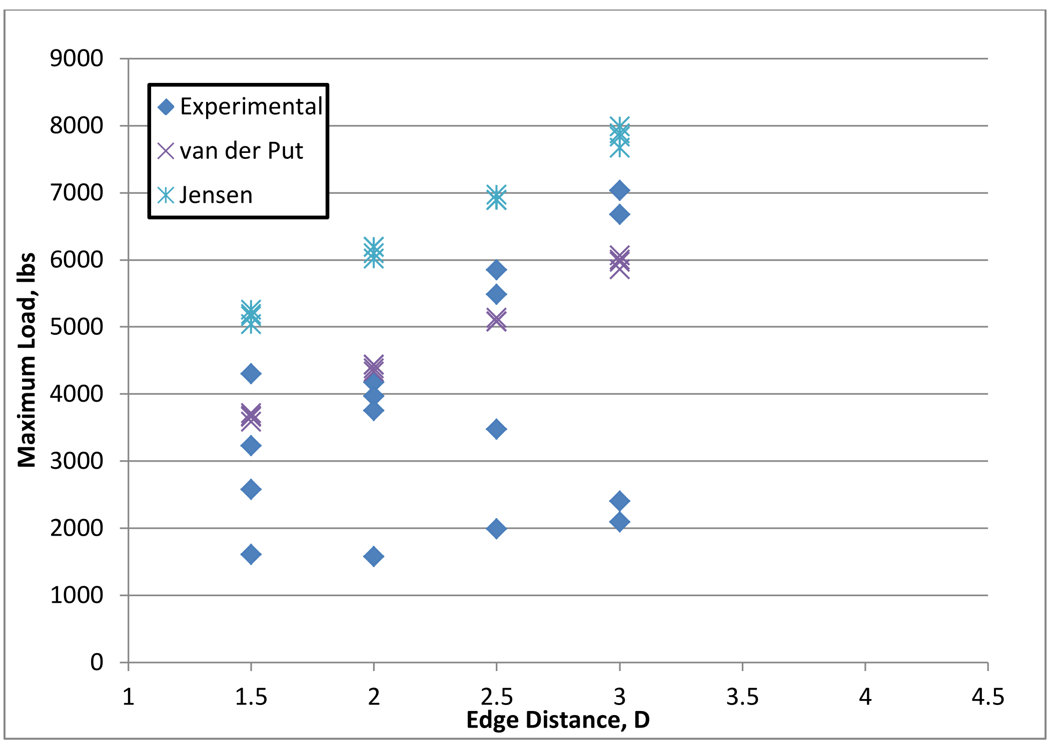

Figure 16. Comparison of Eastern White Pine Maximum Load Values with Fracture Mechanics Models

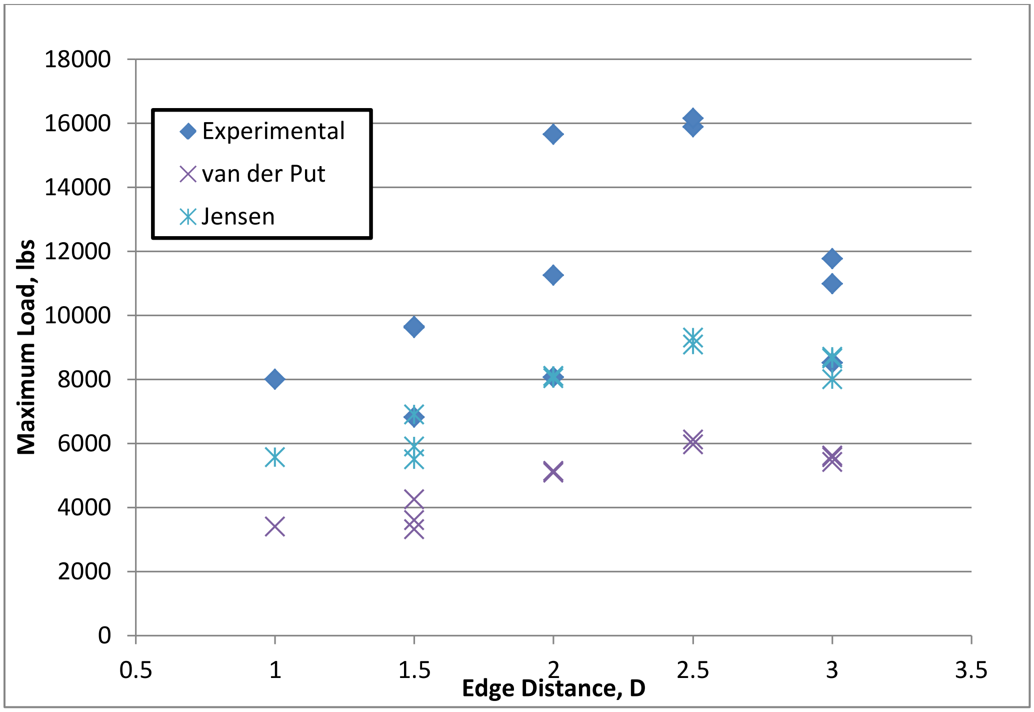

Figure 17. Comparison of Douglas fir Maximum Load Values with Fracture Mechanics Models

Conclusions

This project measured the splitting resistance of mortise members in perpendicular to grain pegged mortise and tenon joints. A novel test apparatus has a steel tenon and Grade 8 bolts to restrict failures to the mortise member. Also, the timbers were tested on both sides of the mortise to increase the possible test repetitions. Yield load, maximum load, and stiffness values increased as edge distance increased. As specific gravity increased, the yield load and maximum load increased. No damage was noted from testing the joints on both sides. CSA 086 values, using the width as the remaining cross-sectional width of the mortise, represented a lower bound for all maximum load values. The CSA 086 values increased with edge distance, but not with specific gravity. The CSA 086 estimation could be used as a baseline estimate of splitting resistance for the wood species of connections tested. Comparing the experimental data to Schmidt and Daniels (1999), the maximum load and stiffness values were in the same range, indicating that no difference between the metal/wood test and testing the joints on both sides was detected. The van der Put and Jensen models were greater than the Douglas fir and eastern white pine values, and were similar to the white oak values. Refinement of the models by mechanical properties may be helpful, but the CSA 086 model in current form seemed to be a better prediction of splitting resistance.

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