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MOMENT CAPACITY AND STIFFNESS OF WEB-TO-CHORD PLATED TRUSS JOINTS

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ABSTRACT: Tests of a truss sensitive to web-to-chord rotational stiffness show that variability in stiffness of that joint can be large, resulting in significant differences in performance. To better identify properties of typical web joints, joints of two wood members fastened together with metal connector plates in a T-shape were tested under a moment loading. Tests were done with various lumber sizes and species, plate sizes and orientations, and with and without the presence of a gap between the wood members to identify the effect of these parameters on strength and stiffness. Measured strength and stiffnesses are reported. Comparisons are made with expected strength based on equations available from design standards and expected stiffness based on equations using axial stiffnesses. Rotational stiffness variation was large, with some joints showing much lower stiffness than may be predicted based on prior research.

KEYWORDS: Rotational stiffness, Truss, Metal Connector Plate

1 INTRODUCTION

Many design specifications for metal-plate-connected wood trusses give little guidance for rotational stiffness modelling of joints, with that guidance typically either missing [1] or simplified to recognizing fully rigid or fully free (pinned) models [2,3]. Further information is given in general language or in commentary language [1,3], but it is difficult to use without further knowledge, such as testing or complex analysis specific to the joint under consideration. The Truss Plate Institute's TPI 1 specification [1] removed guidelines specifying that joints between chord members and web members shall be modelled as pinned in its 2002 edition in recognition that more accurate modelling of rotational stiffness of joints is possible. Some truss designers have modelled web-to-chord joints as fully rigid, especially when they occur on the perimeter of a truss. This application of rigid modelling of perimeter webs to chords can be desirable with respect to economic benefit as the end panel of the truss chord shows reduced positive moment (near the middle of the chord panel) with such modelling. However, such modelling can be unsafe in under-predicting moment on the chord when the web-to-chord joint is more flexible than presumed in the analysis. Tests were conducted to investigate the rotational stiffness of web-to-chord joints.

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2 TRUSS TEST

Tests of a pair of sample trusses were done to illustrate the potential impact of variation in web-to-chord joint rotational stiffness.

2.1 MATERIALS

A parallel chord flat truss with untriangulated end panels was selected for testing as such a design produced a large difference in overall truss deflection with variation in rotational stiffness of the joints between the end vertical web and the adjacent chords. This permitted evaluation to be done based upon overall truss deflection, an easily measured parameter. Two trusses were designed with identical design parameters except that one truss (Truss 1) was designed with pinned perimeter web connections to the adjacent chords, while the other (Truss 2) was designed with semi-rigid perimeter webs (meaning the connection of the web to the chord joint was modelled with a single zero-length having a rotational stiffness, Kr, set equal to 1029 kip-in/rad (116 kN-m/rad). The trusses were designed to support an imposed load consisting of a uniform load across the top chord. See Figures 1 and 2 for the plating resulting from these designs. All plates were Alpine Wave plates [4].

The lumber used for the trusses was 2x4 (38 x 89 mm) Southern Pine #2. The weights. moisture content and flatwise bending stiffness were measured for each individual board prior to truss fabrication and used to determine the wood specific gravity and modulus of elasticity, shown in Table 1. These MOE values were used in the structural matrix analysis to predict the deflection for the test trusses.

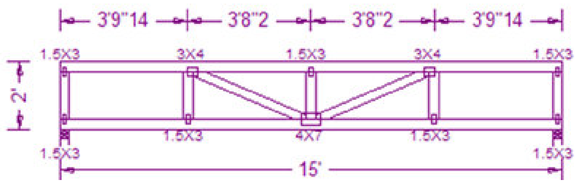

Figure 1: *Truss 1 with plate sizes from pinned analysis including 1.5x3-inch plates on end verticals.*

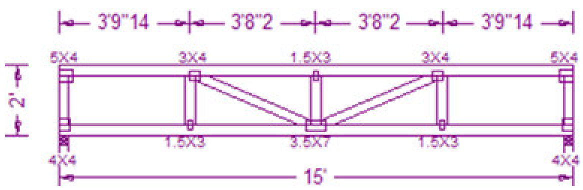

Figure 2: *Truss 2 with plate sizes from semi-rigid analysis, including 5x4-inch (at top chord) and 4x4-inch (at bottom chord) plates on end verticals.*

The trusses were built with no gaps between wood members except that 1/8-inch gaps between the leftmost perimeter web on each truss and each adjacent chord were specifically included so as to see if this affected the resulting truss deflection. This tolerance is permitted between webs and chords per standard truss quality tolerances [1]. All plates were embedded using a hydraulic press.

Table 1: *Wood properties*

Board No.	Moisture Content	Specific Gravity	MOE psi x 10^6	Use (Truss /Member)
1	17	0.644	2.216	1/top chord
2	17	0.697	2.672	1/bottom chd
3	17	0.655	2.409	2/bottom chd
4	17	0.662	2.262	2/top chord
5	18	0.608	1.810	1+2/verticals
6	18	0.604	2.123	1+2/diagonals.

2.2 LOADING METHOD & MEAUREMENTS

The trusses were loaded using four hydraulic cylinders centered on the truss and located at 2 ft o.c. from each other, with each cylinder evenly distributing load through two bearing pads at 1 ft o.c. This resulted in load being applied only to the middle two panels of the truss (between the second and fourth panel points on the top chord), rather than the entire length of the truss as was initially presumed for design purposes. This change from the original design loading was made to permit better assurance of uniform loading from each cylinder, as the light loading for these trusses resulted in very low loads on a per cylinder basis. Load was measured using load cells under each truss reaction.

Deflections were measured at the top chord panel points with displacement transducers for subsequent comparison with the deflection predicted by the structural analysis. Each truss was loaded and unloaded three times in an identical manner with deflection measured in each load cycle to verify that measurements were repeatable. The total load on the truss attained during these cycles were approximately 1000 lb for Truss 1 and 1600 lb for Truss 2.

2.3 RESULTS

The measured deflection values from each of the three loading cycles were plotted against load and found to be linearly correlated to load across nearly the full range of test loads that were applied, as shown in Figures 3 and 4 for Truss 1.

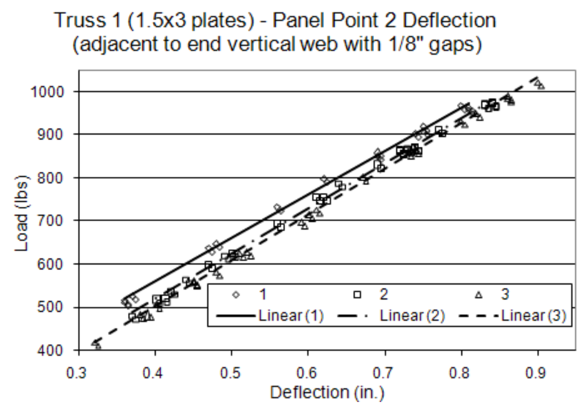

Figure 3: *Load-deflection plot of 2nd panel point, Truss 1.*

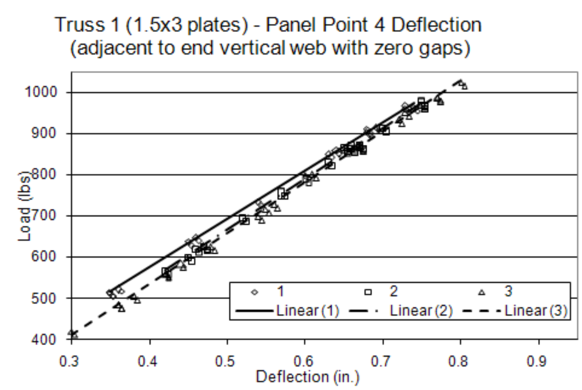

Figure 4: *Load-deflection plot of 4th panel point, Truss 1.*

The stiffnesses, found as the slopes of the load-deflection plots, are shown in Table 2 at the second and fourth top chord joints for each individual run (loading) and the average of the last two runs. Stiffness was found to be repeatable within 2% between the last two load cycles of each truss. For this reason, the average of the last two load cycles for each truss was used as the stiffness for further comparison.

***Table 2:** Measured truss stiffnesses (lb/in)*

		Truss Stiffness	
Truss	Run	Joint 2 (by 1/8 in..gaps)	Joint 4 (by no gaps)
#1 – 1.5x3	1	1004	1176
Plates @	2	1048	1229
End	3	1058	1239
Verticals	Avg 2+3	1053	1232
#2 – 5x4/4x4	1	1824	1929
plates @	2	1777	1875
End	3	1746	1849
Verticals	Avg 2+3	1761	1862

The stiffness at the fourth joint was 17% and 6% higher for Truss 1 and Truss 2, respectively, than the stiffness at the second joint, due to the 1/8 in. joint gaps at the vertical web near joint 2.

2.4 DISCUSSION

A 1/8-in gap at perimeter web joints caused a higher deflection (lower stiffness) at the adjacent panel point, relative to the opposite end of the truss where no such gap was present. This deflection was higher by 17 percent for Truss 1, which had 1.5x3 plates on the perimeter webs, but only 6 percent for Truss 2, which had 5x4 and 4x4 plates on the perimeter webs. This indicates that rotational stiffness is increased to a greater degree by wood-to-wood gaps when small plates are on the joint. This may also have been affected the 1.5x3 plates having their weakest cross-section (the slots) over the joint while the 5x4 and 4x4 plates had their strongest cross-section (the steel between rows of slots) over the joints, as the Wave plate has slots aligned in rows.

Test results are shown in Table 3 in terms of the average deflection at a total truss design load of 595 lb from the second and third load cycles, for ease of comparison to predicted deflections from the structural analysis for various presumptions of joint stiffness for the end-vertical-to-chord connections. These predictions from structural analyses, termed "Calc" in Table 3, with the stiffness of the spring used to model each web-to-chord connection for the end vertical web shown. The 595 lb truss design load was selected for comparison as it was the maximum allowable reaction for the initial truss design with the initial presumption of joint rotational stiffness (Kr).

The last line for each truss in Table 3 shows the rotational stiffness at the perimeter web to chord joints that matches up with the measured truss deflections for the case of zero joint gap: 213 kip-in/rad (24 kN-m/rad) for Truss 1 and 1164 kip-in/rad (132 kN-m/rad) for Truss 2. Since the effect of the gap was relatively small on the deflection of Truss 2, no effort was made to identify the stiffness required to match the truss deflection adjacent to the end vertical web with the 1/8 inch gap on this truss.

***Table 3: Truss** deflection at PP 2 & 4 at 595 lb total load from test measurements and from structural analyses using various joint rotational stiffnesses (Kr).*

	Truss Deflection (in.)	
Truss	Joint 2 (by 1/8 in..gaps)	Joint 4 (by no gaps)
Truss 1 (1.5x3 plates): Test	0.57	0.48
	(Average = 0.52)	
Calc with Kr=0 (pinned)	0.74	0.74
Calc w/ Kr=97 kip-in/rad (Kr=11 kN-m/rad)	0.57	0.57
Calc w/Kr=116 kip-in/rad (Kr=13 kN-m/rad)	0.52	0.52
Calc w/Kr=213 kip-in/rad (Kr=24 kN-m/rad)	0.48	0.48
Calc w/Kr = ∞ (rigid)	0.24	0.24
Truss 2(5x4/4x4 plate): Test	0.34	0.32
Calc with Kr=0 (pinned)	0.77	0.77
Calc w/Kr=1164 kip-in/rad (Kr=132 kN-m/rad)	0.32	032
Calc w/Kr = ∞ (rigid)	0.25	0.25

An effort was made to match the stiffness for the joint with the 1/8 inch gap for Truss 1 as the variation in deflection due to the gap was significant, as well as to compare the results to the analysis presuming a pinned joint. As shown in Table 3, the structural analysis predicted a deflection based on a pinned analysis (Kr=0) that was 42% higher than the average measured deflection for Truss 1 (this is the average of a 54% and a 30% over-prediction for deflection adjacent to the web without gaps and deflection adjacent to the web with 1/8-in gaps, respectively), thus the effect of the joint stiffness is significant for this truss. To match the average measured deflection of the truss with 1.5x3 plates (Truss 1), a rotational stiffness for the perimeter webs of 116 kip-in/rad (13 kN-m/rad) on average, or 97 kip-in/rad (11 kN-m/rad) to match the deflection occurring with a 1/8 inch web-to-chord gap, or 213 kip-in/rad (24 kN-m/rad) to match the deflection occurring adjacent to the web with no web-to-chord gap, should be used. This shows that the rotational stiffness of a tightly built web-to-chord joint with 1.5x3 plates is about twice the stiffness of the same joint with a 1/8 inch wood-to-wood gap.

The difference in joint stiffness for the variations in joint configuration (plate size and wood-to-wood gap) examined in this truss is significant with respect to its effect on the stiffness of the adjacent truss chord. Using joint stiffnesses matching those reported above as giving best estimates of joint behaviour, the effect of the joint stiffness on the strength of the truss can be identified by structural analysis. For example, analyses for the truss using the test load distribution and standard grade E

values gives a maximum allowable truss load, as limited by chord bending stresses in the open panels, of 480 lb if a pinned joint is presumed, 600 lb if a stiffness matching the least stiffness from the truss tests is used, and 704 lb if a rigid joint is used. There is both potential economy and hazard here, depending on the truss designer's assumptions. If the joint rotational stiffness is presumed to be zero, the chord capacity is underestimated by 20 percent. If the joint rotational stiffness is presumed rigid, the chord capacity is overestimated by 17 percent.

With respect to the effect on wood capacity of the adjacent chord panel, it is desirable to not overestimate joint rotational stiffness. For this reason, if there is variation possible in the joint rotational stiffness, it is recommended that joint stiffness be underestimated. As the 1/8 inch gap is defined as acceptable in truss quality standards [1] for web-to-chord joints, it is recommended that joint rotational stiffness assignment consider the effects of such a gap. For small plates, this can reduce the stiffness by an order of magnitude.

Due to the potential variation possible with various joint configurations, including effects of plate rotation and lumber density, further testing was desired to identify such effects.

3 JOINT TEST

Measurement of rotational stiffness from web-to-chord joints at a much lower cost than possible from truss tests was desired so as to permit testing of a wide variety of joint configurations was desired. For that purpose, a series of joint tests were conducted using Southern Pine (S. Pine) and Spruce-Pine-Fir (SPF) lumber, which are assigned average specific gravities for the species group overall of 0.55 and 0.42 [5], various truss plate sizes, two truss plate orientations, and with and without the presence of 1/8 inch gaps between the web and chord material.

3.1 MATERIALS & METHODS

Web-to-chord joints in a T-shape were fabricated, with each joint consisting of a top member and a bottom member, joined by a pair of metal connector plates. The top member of the T-joint was a 13.5 inch long 2x4 of S. Pine or SPF species groups and was bolted into a specially designed steel jig to hold it vertical on the bed of a universal testing machine. This results in the bottom member of the T-joint positioned horizontally, as shown in Figure 5. The bottom member was 18 inches long. Joints were tested with bottom members of 2x3 and 2x4 sizes of SPF lumber and of 2x4 and 2x6 sizes of S. Pine lumber. Joints were tested with Wave metal connector plates of sizes 1.5x3, 1.5x4, 2x3, 2x4, 3x3, and 3x4. (Size format of plates is WidthxLength using units of inches, where width is plate dimension perpendicular to slot length and length is plate dimension parallel to slot length.) All plates were oriented so the slot lengths were parallel to the length of the bottom member of the T-joint, except some joints with 3x3, 3x4 and 4x4 plates had the plates rotated 90 degrees from this position. A joint gap of 1/8 inches between the two wood members was also evaluated in one set of joints using 2x4 S. Pine lumber. A list of the various joint configurations considered in shown in Table 4. All plates were hydraulically pressed into the lumber. Three replicates were tested for each orientation.

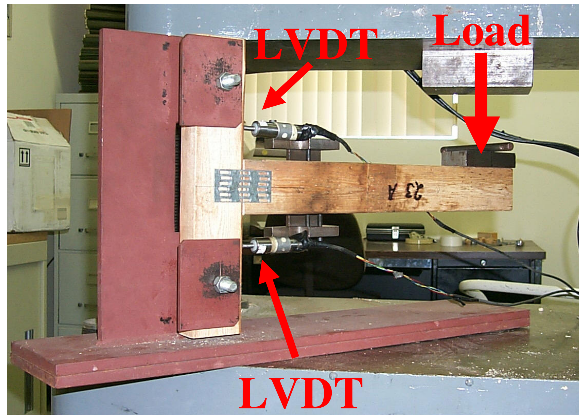

Figure 5: *T-shaped joint in test fixture.*

The testing machine head loaded the end of the bottom piece of the T-joint in a direction perpendicular to the length of the piece, as shown in Figure 5. Load was applied through a steel pin oriented perpendicular to the length of the wood members and bearing on the center of a four inch long steel plate positioned at the end of the bottom member. This resulted in the load being centered two inches from the end of the bottom wood member and 16 inches from the joint between the top and bottom wood members. Load was applied by a constant downward movement of the machine crosshead at a rate between 0.05 and 0.1 in/min. Displacement transducers (LVDTs) measured the deflection between the top and bottom pieces on either side of the bottom member of the T-joint, as shown in Figure 1. The LVDT positions were each one inch beyond the edges of the bottom member of the T-joint. Load and deflection were recorded at intervals of ten lb and the specimen was loaded until failure occurred.

Due to the position of the load 16 inches from the joint, the moment resulting from the load was the test load times 16 inches. Rotation of the specimen was calculated from the average measured deflections and the distance between these locations of measurement as shown in Equation (1) below:

$$\text{Rotation (rads)} = \arctan\{(D1+D2)/W\} \qquad (1)$$

where: $D1$ is outward deflection from the top LVDT, $D2$ is inward deflection from the bottom LVDT, and W is the width of the bottom member plus 2 inches.

The moments and rotations were calculated from the recorded loads and deflections, plotted and used to determine the rotational stiffness, or moment per unit of rotation, for each joint. The rotational stiffness was

nonlinear, thus stiffness was identified between two sets of points on the load-deflection plot: 1) as the slope between the first two points on the plot, which represents an initial stiffness; and 2) as the slope between the initial point and the point at 50 percent of the ultimate load. This latter value is expected to be lower than the initial stiffness, but may be the most appropriate value to use for design purposes given that this may be the maximum nominal design load for the joint.

Table 4: *T-joint test series*

JOINT ID	TOP MEMBER	PLATE (R=Rotated)	BOTTOM MEMBER
JOINTS WITHOUT WOOD-TO-WOOD GAPS			
1-3	SPF 2x3	1.5x3	SPF 2x3
4-6	SPF 2x3	1.5x4	SPF 2x3
7-9	SPF 2x3	2x3	SPF 2x3
10-12	SPF 2x3	2x4	SPF 2x3
13-15	SPF 2x3	3x3	SPF 2x3
16-18	SPF 2x3	3x4	SPF 2x3
19-21	SPF 2x3	3x3(R)	SPF 2x3
22-24	SP 2x4	1.5x3	SP 2x6
25-27	SP 2x4	1.5x4	SP 2x6
28-30	SP 2x4	2x3	SP 2x6
31-33	SP 2x4	2x4	SP 2x6
34-36	SP 2x4	3x3	SP 2x6
37-39	SP 2x4	3x4	SP 2x6
40-42	SP 2x4	4x4	SP 2x6
43-45	SP 2x4	3x3(R)	SP 2x6
46-48	SP 2x4	3x4(R)	SP 2x6
49-51	SP 2x4	4x4(R)	SP 2x6
52-54	SPF 2x4	1.5x3	SPF 2x4
55-57	SPF 2x4	1.5x4	SPF 2x4
58-60	SPF 2x4	2x3	SPF 2x4
61-63	SPF 2x4	2x4	SPF 2x4
64-66	SPF 2x4	3x3	SPF 2x4
67-69	SPF 2x4	3x4	SPF 2x4
70-72	SPF 2x4	4x4	SPF 2x4
73-75	SPF 2x4	3x3(R)	SPF 2x4
76-78	SPF 2x4	3x4(R)	SPF 2x4
79-81	SPF 2x4	4x4(R)	SPF 2x4
82-84	SP 2x4	1.5x3	SP 2x4
85-87	SP 2x4	1.5x4	SP 2x4
88-90	SP 2x4	2x3	SP 2x4
91-93	SP 2x4	2x4	SP 2x4
94-96	SP 2x4	3x3	SP 2x4
97-99	SP 2x4	3x4	SP 2x4
100-102	SP 2x4	4x4	SP 2x4
103-105	SP 2x4	3x3(R)	SP 2x4
106-108	SP 2x4	3x4(R)	SP 2x4
109-101	SP 2x4	4x4(R)	SP 2x4
JOINTS WITH 1/8" GAPS			
113-114	SP 2x4	1.5x3	SP 2x4
115-117	SP 2x4	1.5x4	SP 2x4
118-120	SP 2x4	2x3	SP 2x4
121-123	SP 2x4	2x4	SP 2x4
124-126	SP 2x4	3x3	SP 2x4
127-129	SP 2x4	3x4	SP 2x4
130-132	SP 2x4	4x4	SP 2x4

3.2 RESULTS

Table 5 shows average bending stiffnesses, determined from the data per equation 1, average specific gravity (basis of oven-dry volume, adjusted from measurements in air-dry condition at 17-18% MC) of the top member, initial joint stiffness and the secant stiffness from the initial load to 50 percent of the ultimate load, and the failure moment.

Table 5: *T-joint test series results*

JOINT ID	Average SG (OD) Top Mbr	JOINT STIFFNESS, kip-in/rad (kN-m/rad) Initial	To 50%Ult	Avg. Ult., in-lb
JOINTS WITHOUT WOOD-TO-WOOD GAPS				
1-3	0.415	122 (14)	108 (12)	2091
4-6	0.409	121 (14)	126 (14)	2549
7-9	0.420	168 (19)	168 (19)	2923
10-12	0.399	207 (23)	156 (18)	3605
13-15	0.365	252 (28)	172 (19)	3749
16-18	0.368	280 (32)	245 (28)	4821
19-21	0.368	336 (38)	222 (25)	3264
22-24	0.530	252 (28)	423 (48)	4939
25-27	0.505	354 (40)	601 (68)	6725
28-30	0.462	493 (56)	606 (68)	6256
31-33	0.446	620 (70)	622 (70)	7792
34-36	0.448	880 (99)	857 (97)	7189
37-39	0.431	1560 (176)	1040 (117)	9744
40-42	0.554	920 (104)	1530 (173)	8736
43-45	0.533	574 (65)	558 (63)	7440
46-48	0.503	978 (110)	755 (85)	6709
49-51	0.495	800 (90)	1312 (148)	10315
52-54	0.428	147 (17)	200 (23)	2843
55-57	0.409	154 (17)	163 (18)	3264
58-60	0.426	246 (28)	212 (24)	3333
61-63	0.415	323 (36)	302 (34)	4453
64-66	0.391	616 (70)	469 (53)	4645
67-69	0.371	733 (83)	593 (67)	5728
70-72	0.369	792 (89)	856 (97)	7493
73-75	0.371	851 (96)	474 (54)	4805
76-78	0.388	851 (96)	750 (85)	5824
79-81	0.371	763 (86)	816 (92)	8448
82-84	0.528	235 (27)	243 (27)	3323
85-87	0.518	284 (32)	277 (31)	3973
88-90	0.510	305 (34)	322 (36)	3712
91-93	0.493	528 (60)	406 (46)	5184
94-96	0.528	675 (76)	572 (65)	5477
97-99	0.621	851 (96)	603 (68)	7413
100-102	0.600	909 (103)	967 (109)	9264
103-105	0.567	763 (86)	477 (54)	5883
106-108	0.532	821 (93)	678 (77)	6069
109-101	0.509	880 (99)	1127 (127)	8032
JOINTS WITH 1/8" GAPS				
113-114	0.512	117 (13)	56 (6)	2944
115-117	0.516	154 (17)	58 (7)	3856
118-120	0.533	270 (31)	87 (10)	3707
121-123	0.551	293 (33)	84 (10)	5200
124-126	0.479	606 (68)	329 (37)	4379
127-129	0.499	939 (106)	549 (62)	6027
130-132	0.509	909 (103)	934 (106)	8069

Failure modes were primarily tooth withdrawal. Steel failure also occurred in some specimens, but was only the predominant failure mode for the 1.5x3 plates in SPF 2x3 material and the plates tested with 1/8 inch wood-to-wood gaps in S. Pine specimens up (except the 3x4 and 4x4 plates in this series also failed by tooth withdrawal). In addition to these two failure modes, there were several joints that failed due to wood fracture or splitting, mainly in the 4x4, rotated 3x3 and rotated 4x4 plates in the S. Pine series with 2x6 members.

3.3 DISCUSSION

The initial stiffnesses presented in Table 5 show substantial variability due to precision of the instrumentation and the small data increment used to calculate this initial stiffness. The measured deflection over this single load increment was equal to the precision of the LVDTs, thus errors of 100% may result in these initial stiffness values. While these values may be useful if averaged over replicates greater than those used here in some relative manner for comparisons, the variability is too large to consider them useful for most purposes. Plots of the moment-rotation curve in all cases show a constant slope or a reduction in slope as load increases, thus secant stiffnesses from zero to 50% of ultimate should be no greater than the initial stiffness. However, a comparison of the values in Table 5 show that the stiffnesses up to the 50% level often exceeded the initial values. These initial stiffnesses are included in Table 5 only because they are of use in illustrating the very large effect of the 1/8 inch gaps upon stiffness. For the specimens without gaps, the stiffnesses at the higher load level are as low as 56 percent of the initial stiffnesses. However, for the specimens with gaps, the stiffnesses at the higher load level vary as low as 29 percent of the initial stiffness. This is due to yielding of the plates in compression, as illustrated by Figure 6.

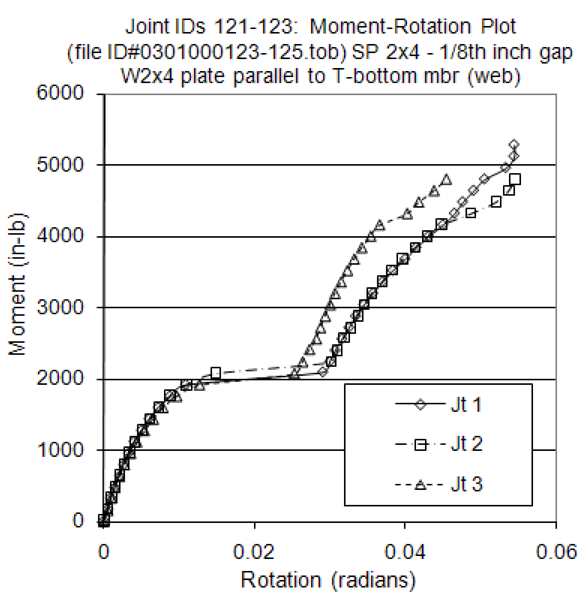

Figure 6: *Moment-rotation plot showing plate yielding in compression.*

This comparison of the stiffness results in Table 5 between the tests with 1/8 inch wood-to-wood joint gaps and the comparable joints with no such gaps show that the gaps permitted between truss webs and chords may result in as much as 70% loss of stiffness of those joints.

The effect of gaps does not have as extreme an effect on ultimate moment capacity. Comparing the ultimate capacities of joint IDs 113-132 (joints with gaps) with joint IDs 82-102 (corresponding joints with no gaps), it is found that average ultimate strength for joints with gaps varies from 80 to 100 percent of that without gaps. This is similar to that reported by Burow [6].

It is also of interest to compare the measured stiffnesses to that suggested by the literature. Cramer [7] provided a method based on the stiffnesses measured in standard axial tests to determine lateral resistance of the metal connector plates. Values for the Wave plate in S. Pine so tested were provided by Gupta [8], permitting use of this method to calculate stiffness corresponding to joint ID's 82 through 101 from the prior tables. Table 6 below provides this comparison.

Table 6: *Comparison of stiffnesses measured from T-joints without gaps in S.Pine 2x4 lumber to calculations*

JOINT ID	PLATE (R=Rotatd)	STIFFNESS (k-in/rad) Measured@50% No Gap	1/8" Gap	Calculated
82-84	1.5x3	243	56	24
85-87	1.5x4	277	58	45
88-90	2x3	322	87	43
91-93	2x4	406	84	75
94-96	3x3	572	329	111
97-99	3x4	603	549	175
100-102	4x4	967	934	306
103-105	3x3(R)	477		124
106-108	3x4(R)	678		228
109-101	4x4(R)	1127		336

As shown by Table 6, in every case the calculated stiffness is lower than the measured stiffness, even for the tests performed with 1/8 inch wood-to-wood gaps. This may be expected, as this calculation depends solely on the lateral resistance of the plates without consideration for any butting between the wood members.

Another comparison that may be made is the effect of the plate orientation. Design methods account for the gross effects of plate size on moment capacity through calculation of an effective moment of inertia that includes consideration for plastic behavior, but there are differences between the lateral resistance stress recognized as the appropriate limiting stress for a metal connector plate subject to moment, with Kevarinmäki [8] reporting use of the highest value for any orientation being appropriate. The results of the 3x3 and 4x4 tests may be compared with the results of the same plate sizes rotated (shown as 3x3(R) and 4x4(R) in the tables). Table 7 shows these comparisons with results varying

somewhat, although not in a consistent manner. Average stiffness for the rotated plate varied from 65 to 129 percent of the stiffness of the nonrotated plate, while average strength for the rotated plate varied from 87 to 118 percent of the strength for the nonrotated plate. Given that this variation is reasonably centered upon 1.00, it is compatible with the prior recommendations.

Table 7: *Comparison of rotated with nonrotated plates*

TEST SET		Ratio of Plate Results (Rotated / Nonrotated)	
Wood Mbrs	Plate	Stiffness@50%	Ultimate
SPF 2x3/2x3	3x3	1.29	0.87
SP 2x4/2x6	3x3	0.65	1.03
	4x4	0.86	1.18
SPF 2x4/2x4	3x3	1.01	1.03
	4x4	0.95	1.13
SP 2x4/2x4	3x3	0.83	1.07
	4x4	1.17	0.87

It is noted that there are several reports providing results of tested splice joint stiffnesses, including that by Gupta [8]. The stiffnesses measured from these tests of in-line members without gaps are much higher than reported from these T-joints with gaps. For example, Gupta reports values of 91,000 and 112,000 kip-in/rad for two orientations of plates measuring 3x4 inches in size and tested in S. Pine, greatly exceeding the 603 and 549 kip-in/rad for joints without and with gaps, respectively, shown in Table 6. Gupta's article provides a calculation method based only upon lateral resistance, which is suitable for use in providing estimates of stiffness that are accurate or conservative, as noted earlier, but this cautionary note is provided due to the potential for misapplication of test results of chord splice-type joints to other joints.

Strength may also be compared to methods specified by design standards. For lateral resistance strength, the TPI 1-2007 design standard [1] provides a design equation to predict ultimate capacity. This equation was evaluated to compare to the S. Pine test specimens and it was found that the tested strength ranged from 3.6 to more than 10 times the normal duration design strength. The design equation makes use of a plastic design methodology recommended by Kevarinmaki [8]. While the lower end of this range is expected based on the TPI 1-2007 design methods, the upper end is about twice that expected. It is believed that this is due primarily to wood-to-wood contact, which is not considered in the lateral resistance design equations.

4 FURTHER COMMENTS

It is also of interest to compare the results of the T-joint tests to the truss tests. The truss tests suggested spring stiffnesses for the 1.5x3 plates of 97 kip-in/rad at the low end (corresponding to the end of the truss with 1/8 inch gaps between the end vertical and the chords) to 213 kip-in/rad at the high end (corresponding to the end of the truss with no gaps between the end vertical and the chords). The T-joint tests gave average rotational stiffnesses of 56 kip-in/rad (from joint IDs 113-114: only 2 replicates were used for this set due to a variation from the intended fabrication) and 243 kip-in/rad (from joint IDs 82-84) for these joints in 2x4 S. Pine lumber like used in the truss tests. These values from the truss tests are 173% and 88% of the stiffnesses from the joint testss. This is considered a reasonable match for several reasons, but the largest is probably that the joint in the truss test had not yielded based on the linear load-deflection correlation in the load-deflection cycles for the truss. The T-joint initial stiffness for this configuration was 117 kip-in/rad and, although this may vary due to the lack of precision in calculating initial stiffness, this is relatively close to the estimate of 97 kip-in/rad from the truss test.

5 CONCLUSIONS

Variability in joint rotational stiffness occurs due to several factors. Stiffness can decrease by orders of magnitude with web-to-chord gaps for small plate sizes, and significant variation can also occur with plate size, plate position and rotation, wood size and wood density. Truss designers must carefully consider this variation in order to assure that joint stiffness is not overestimated, as overestimation of joint rotational stiffness may result in underdesign of the adjacent wood panel. Web-to-chord truss joints should not be presumed to be rotationally rigid.

Tests of various joint configurations using one plate type have been performed and results presented to aid in the assignment of appropriate joint rotational stiffness parameters.

Comparisons made with expected stiffness and strength based on equations using results of axial tests show reasonable agreement, however rotational stiffness variation was large. It is noted that joints such as those tested here simulating web-to-chord joints show much lower stiffness than may be predicted based on some prior research of joints similar to those in chord splices, but can be adequately (conservatively predicted) based solely upon lateral resistance theory.

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REFERENCES

[1] TPI-2007, National design standard for metal plate connected wood truss construction. Truss Plate Institute (www.tpinst.org), Alexandria, VA, 2007.

[2] TPIC-2007, Truss design procedures and specifications for light metal plate connected wood trusses. Truss Plate Institute of Canada (www.tpic.ca), 2007.

[3] EN 1995-1-1, Eurocode 5, Design of timber structures. 2004. European Committee for Standardization (CEN).
[4] ICC ES Report ESR-1118. Alpine Wave, H, S, K, Trulox and Hinge Plate metal connector plates for wood trusses. ICC Evaluation Service (www.icc-es.org), 2009.
[5] NDS-2005, National design specification for wood construction. American Forest & Paper Association (www.awc.org), Washington, DC, 2005.
[6] Burow, J. R.: Effects of joint gap spacing on moment capacity of metal-truss-plate connectors. Thesis, Dept. of Agric. & Biol. Eng., Pennsylvania State Univ. 2003.
[7] Cramer, S.: Computation of Member Forces in Metal Plate Connected Wood Trusses. Structural Engineering Review, Vol. 5, No. 3 pp. 209-217, 1993.
[8] Gupta, R.: Modelling of Chord Break Joints in Metal Plate Connected Wood Trusses. Department of Forest Products, Oregon State University, Corvallis, Oregon. 1998.
[9] Kevarinmäki, A.: Moment capacity and stiffness of punched metal plate fastener joints. In: International Wood Engineering Conference, New Orleans, 1996, Vol. 1 pp. 385-392.