METAL-PLATE CONNECTED TENSION JOINTS UNDER DIFFERENT LOADING CONDITIONS[1]

Rakesh Gupta

Assistant Professor
Department of Forest Products
Oregon State University
Corvallis, OR 97331

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ABSTRACT

Metal-plate connected tension-splice wood truss joints were tested under six different loading conditions: pure axial tension, pure bending, and four different levels of combined (axial tension/bending) loading. All joints were fabricated from 2- by 4-in. nominal No. 2 southern yellow pine lumber and 20-gage metal truss plates. Joints were tested to failure on a newly developed testing apparatus. Combined loading tests showed that the axial load capacity of joints decreased with an increase in applied bending moment. The most common mode of failure was tooth withdrawal, which indicates that tooth-holding capacity governs the strength of the joint.

Keywords: Metal-plate, joints, wood truss, combined loading.

INTRODUCTION

Metal-plate connected (MPC) joints are primarily and extensively used in the fabrication of light-frame wooden trusses. The joints mainly consist of punched metal plates fabricated from 0.035- to 0.080-in. (0.9- to 2.0-mm; 20- to 14-gage) coiled strips of structural steel and 2-in.-thick nominal framing lumber. Metal-plate connected joints are widely used because of their ease of fabrication and low cost. Most MPC joints and trusses are designed, manufactured, and installed in accordance with the recommendations of Truss Plate Institute (1985).

The structural performance of MPC joints has received extensive research attention in the last 20 years (Wolfe 1990). Still, the behavior of MPC joints is by far the least understood aspect of truss behavior (Kirk et al. 1989). Most of the research has focused on the performance of tension joints under axial loads only. Tension joints under combined (axial and bending) loads have received little attention in recent years. A tension joint in the bottom chord of a truss is subjected to both axial and bending loads, but truss design standards do not require moment checks for the tension joint. Sometimes an MPC joint is checked for moment, but the moment check is independent of any axial load (Wolfe 1990). Also, there are no standards to evaluate the axial-bending capacity of tension joints.

This paper describes the testing of MPC tension joints under several different loading conditions. These conditions included pure axial tension, pure bending, and four different levels of combined axial tension and bending. The objectives of this study were to:

1. determine strength and stiffness of tension joints under different loading conditions and
2. describe the failure modes of joints.

LITERATURE REVIEW

In a recent study, Wolfe et al. (1991) developed a test apparatus for simulating interactive loads on MPC tension joints. The test apparatus was specifically designed to test 20-gage (0.91-mm) MPC joints in nominal 2- by 4-in. (standard 38- by 89-mm) lumber. Wolfe (1990)

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used this apparatus to evaluate the load capacity of tension joints under combined bending and axial loads and to show the reduction in connection tensile capacity with an increase in applied moment. On the basis of this finding, he emphasized that MPC tension joints should be checked for their interactive load capacity, not just their axial load capacity.

Several researchers (Felton and Bartlett 1964; Hayashi and Sasaki 1982; Suddarth et al. 1979; McAlister 1989) have tested MPC tension joints under axial loads to determine their strength and stiffness. Gupta and Gebremedhin (1990) developed an apparatus for testing MPC wood truss joints (heel, tension, web) under different loading conditions to determine their strength and stiffness.

Quaile and Keenan (1979) were the first to emphasize the need for testing actual MPC wood truss joints. Since then, only Gupta (1990) has tested three types of MPC wood truss joints. Several joint-testing standards have been developed (American Society for Testing and Materials 1991; Canadian Standards Association 1980; International Organization for Standardization 1990; Truss Plate Institute 1985; European Union of Agrement 1990), but none of these standards provide guidelines for testing truss joints under in-service loading conditions. There is a need for accurate data on strength and stiffness of MPC joints under real loads in order to establish a data base (Gupta 1990; Wolfe et al. 1991).

MATERIALS AND METHODS

Materials

All lumber used for the fabrication of test specimens was 2- × 4-in. southern yellow pine (*Pinus* spp.) No. 2 KD purchased from a local lumber company. The modulus of elasticity (MOE) of each piece of lumber was determined non-destructively. Each 8-ft (2.4-m)-long piece was tested in static flatwise bending with a concentrated dead load at midspan. Moisture content and specific gravity were determined with ASTM D2016 method A and ASTM D2395 method A American Society for Testing and Materials (1990a, b), respectively.

TABLE 1. *Plate specifications.*

Plate description	Specification
Size	3 in. × 4 in.
Thickness (gage number)	0.04 in. (20)
Tooth density	8 teeth per sq. in.
Slot width	⅛ in.
Slot length	½ in.
Tooth length	⅜ in.
Teeth configuration	in-line
Modulus of elasticity	29.5×10^6 psi
Yield strength	36,000 psi

The metal truss plates were supplied by a commercial plate manufacturer (Alpine Engineered Products, Inc.). The specifications for the plates are given in Table 1. The size of the metal plates, teeth density, and placement of the plates were compatible with current design practices recommended by the Truss Plate Institute (1985).

Joint design

The joint was designed for an 28-ft (8.5-m) span, 2 ft (0.6 m) on center and 5 on 12 slope Fink (single W) truss. The truss was designed for 30 lb/ft^2 (psf) (1.44 kPa) of top chord snow load, 7 psf (0.34 kPa) of top chord dead load, and 10 psf (0.48 kPa) of bottom chord dead load. The truss and joints (plate size) were designed by the truss manufacturer who supplied the plates.

Joint fabrication

Each joint was fabricated from a single piece of lumber. A hydraulic press at a commercial truss manufacturing site was used to make the 50 test specimens. Only one plate was pressed at a time. The overall length of a test specimen was about 5.5 ft.

Testing apparatus

In this study, unique and unconventional methods and apparatus were developed for testing truss joints (Figs. 1–3). The testing apparatus consisted of a rigid, horizontal steel frame, which was bolted to the floor to restrict

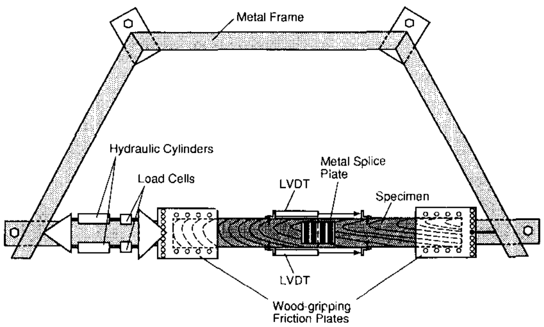

FIG. 1. Test frame with tension splice joint under axial tension loading.

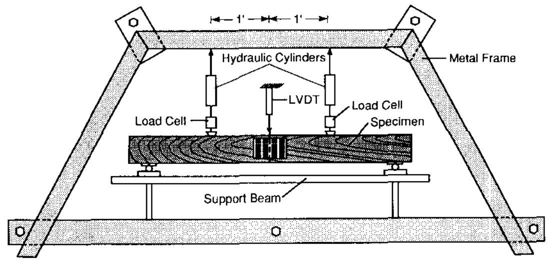

FIG. 2. Test frame with tension splice joint under pure bending loading.

movement. Two parallel hydraulic cylinders were attached to the test frame to exert pressure on the test specimens. In-line pressure to the cylinders was controlled by an electronic hydraulic pressure-control (relief) valve. A force transducer (strain-gage-type load cell) was connected to each cylinder to measure the applied force. An IBM-PC/PS2 and data acquisition system was used to monitor the force so that static equilibrium could be maintained during testing. There were three different types of loading conditions: pure axial tension (Fig. 1), pure bending (Fig. 2), and combined (axial tension/bending) loading (Fig. 3).

For the axial tension and combined loading tests, each end of a specimen was sandwiched between two wood-gripping friction plates (Figs. 1 and 3). The plates were coated with polyurethane (a high-friction material) to create the necessary grip. The friction plates each had nine holes arranged in a column. The center hole allowed concentric axial tension loading (Fig. 1); the other holes allowed eccentric (combined axial tension/bending) loading (Fig. 3).

For the bending test, the cylinders were attached to the top of the test frame, and a support beam was placed beneath the specimen (Fig. 2). During the test, a two-point load on a 48-in. (1.2-m) span was used. The constant moment section (between load points), which contained the joint, was 24 in. (0.6 m) long.

Linear variable differential transformers (LVDTs) were used to monitor test specimen displacement. The LVDTs were clamped onto the specimens; axial displacement of the joint was measured on both sides of the joint (Fig. 1). For eccentric loading, displacement perpendicular to the loading direction also was measured (Fig. 3). Only one LVDT was used to measure displacement in the transverse direction (Fig. 2).

Testing procedure

For all tests, a specimen was loaded such that its deformation was unrestrained in the load plane. The specimen was loaded in increments until a static load could no longer be maintained. Specimens were loaded to failure as follows:

1. System was initialized. This applied a minimum force of 200 lb (890 N) to the test specimen.
2. After 8 sec, force and displacement readings were taken. (The readings stabilized within this period.)
3. Load increment was applied.
4. Volt signals were converted into actual forces and displacements, and the data read in step 2 was printed and stored.
5. Loading was terminated when deflection increased with no detectable increase in load-cell readings, or when a noticeable failure was observed; otherwise, steps 2 to 5 were repeated.

The test period for each specimen was approximately 12 min. This is consistent with the ASTM D 1761 recommendation that failure should occur between 5–20 min (American Society for Testing and Materials 1991). The loading rate for both the axial tension and combined loading tests was approximately 500 lb/min (2.2 kN/min). For the pure bending test, the loading rate was about 60 lb/min (0.3 kN/min) per cylinder. Nine specimens were used to tune-up the testing apparatus and check the three test procedures. The remaining specimens were tested as follows:

1. Pure axial tension—9 joints
2. Pure bending—8 joints
3. Combined loading
   a. 0.5 in. (13 mm) eccentricity—6 joints
   b. 1.0 in. (25 mm) eccentricity—6 joints
   c. 1.5 in. (38 mm) eccentricity—6 joints
   d. 2.0 in. (51 mm) eccentricity—6 joints

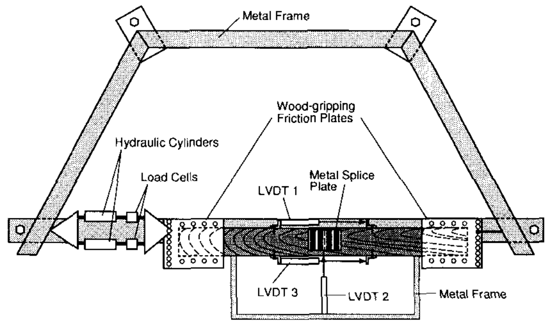

FIG. 3. Test frame with tension splice joint under combined (axial tension/bending) loading.

RESULTS AND DISCUSSION

The average long-span MOE of the lumber used in the fabrication of test specimens ranged from 0.7 to 2.5 × 10^6 psi (4.8 to 17.2 GPa), with an average of 1.4 × 10^6 psi (9.7 GPa) and Coefficient of Variation (CV) of 27.5%. The average MC (weight of water/dry weight) and specific gravity (weight and volume at test) values were 10% and 0.48, respectively.

Pure tension

Table 2 shows descriptive statistics for eight (out of nine) joints tested under pure axial tension. For one joint, the ultimate load was only 2,854 lb (12.7 kN) (not shown in Table 2). A maximum normed residual test (Appendix A) for a single sample (Snedecor and Cochran 1980) showed that under normal conditions, a deviation of this size will occur about 1% of the time. Therefore, this value was considered an outlier and it was not included in further analysis. The CV of ultimate load was small compared to the CV of displacement at failure. This probably indicates that strength rather than deflection governs the failure of the joint because the strength values are more consistent than the deflection values. Therefore, strength is probably a better indicator of failure.

Three different methods were used to determine axial stiffness (Table 2): stiffness at design load, stiffness at critical slip, and stiffness from initial slope. The first two measures of stiffness are secant stiffness; the third is a tangential stiffness. Stiffness at design load was defined as the ratio of the design load and the deflection at the design load (Truss Plate Institute

TABLE 2. *Results of joints tested in pure tension.*

No.	Joint no.	MOE, × 10^6 psi	Ultimate load, lb	Displacement @ failure, in.	Stiffness,[1] lb/in.	Stiffness,[2] lb/in.	Stiffness,[3] lb/in.	Failure mode[4]
1	180	1.9	5,765	0.0597	3.56	2.38	438,619	TW
2	199	0.8	7,493	0.0825	2.66	2.32	300,986	WF/TW
3	204	1.7	7,123	0.0798	3.09	2.54	347,207	WF/TW
4	205	1.2	6,120	0.0780	2.33	2.01	261,852	WF/TW
5	223	2.0	6,227	0.0499	3.26	2.49	368,259	WF/TW
6	232	2.0	5,953	0.0581	3.69	2.49	444,657	TW
7	243	1.4	5,995	0.0724	2.89	2.22	349,649	TW
8	247	1.2	5,761	0.0672	2.32	1.94	252,777	WF/TW
	Mean	1.5	6,305	0.0685	2.97	2.30	345,501	
	(CV)	(28%)	(10%)	(17%)	(18%)	(10%)	(21%)	

[1] Stiffness (× 10^5) at design load = (ultimate load 3)/deflection.
[2] Stiffness (× 10^5) at critical slip = load critical deflection (0.015 in.).
[3] Stiffness calculated from initial slope.
[4] TW = tooth withdrawal; WF = wood failure.

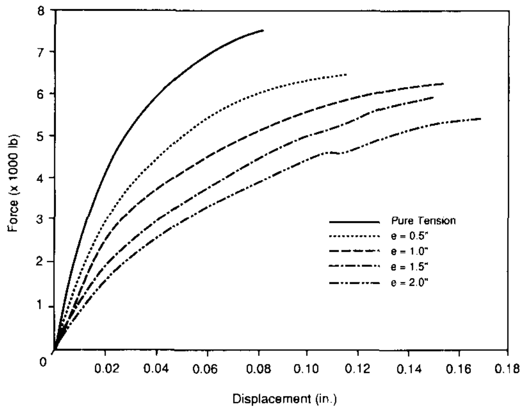

FIG. 4. Load-displacement curves of tension splice joints under different loading conditions.

1985). The design load was taken as the ultimate load divided by a factor of three. Stiffness at critical slip was calculated by dividing the load at critical deflection (0.015 in.) by 0.015 in. (Truss Plate Institute 1985). Stiffness calculated from initial slope was the slope of the straightest portion of the curve below design load (Fig. 4). The straightest portion of the curve was determined by performing several linear regression analyses below design load and looking for the highest R-square values. Stiffness at critical slip was a more consistent estimate of joint stiffness than was stiffness at design load or from initial slope because it had the lowest CV. It also had the lowest value among the three estimates of stiffness. Therefore, the average axial stiffness of the joint was taken as the average stiffness at critical slip, i.e., 2.3×10^5 lb/in. (40.3 kN/mm).

A representative load-deflection curve for a joint under pure axial tension is shown in Fig. 4. As expected, the load-deflection curve is nonlinear. The displacement was taken as the average of two displacements measured on both sides of the joint (Fig. 1). The two displacement measurements were close to each other, but not identical, which shows that some lateral moment was present at the joint. This may have been caused by a lack of alignment between the wood-gripping plates and the specimen, or by metal truss plates that were not centered over the joint. Some of the load deflection curves showed initial signs of failure prior to the catastrophic failure. The initial sign of failure was marked by a small drop in the force as shown in Fig. 4 for 2.0-in. eccentricity.

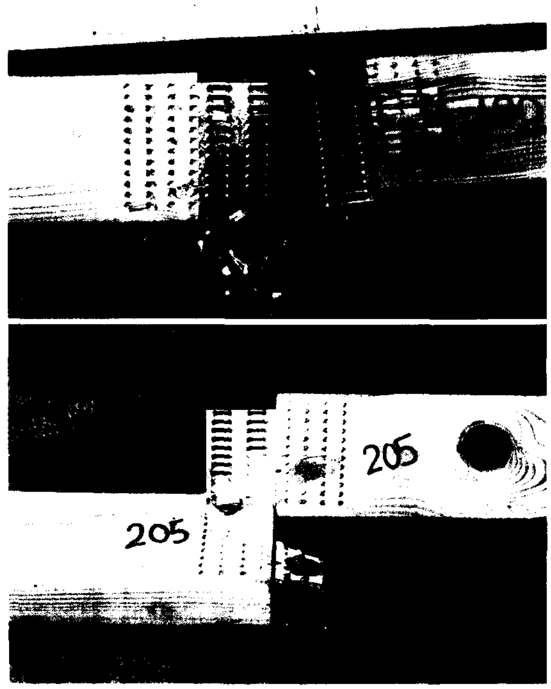

FIG. 5. Failure modes under pure tension (a) tooth withdrawal, (b) wood failure/tooth withdrawal.

Three of the eight specimens failed in tooth withdrawal (TW) (Fig. 5a). The remaining five specimens failed in wood followed by tooth withdrawal (WF/TW) (Fig. 5b). The average load for joints that failed in WF/TW was 11% greater than for joints that failed in TW. In all joints, the first row of teeth close to the gap was critical because at this section the full axial force was transmitted through the effective section of the plate. As loading progressed, the first row of teeth started to bend first and the rest of the teeth withdrew at failure. In some cases, however, a large chunk of wood was picked up by some of the teeth and the rest of the teeth withdrew from the wood, which caused a WF/TW-type failure.

Pure bending

Ultimate moment capacity and other statistics for all eight joints tested in pure bending are given in Table 3. Although the CV of MOE of the tested specimens was high, the low CV of ultimate moment showed that ultimate moment was consistent. Like axial tension, bending failure also was governed by the moment at the joint (as shown by low CV) rather than by the rotation of the joint (high CV).

A representative moment-rotation curve is shown in Fig. 6. Moment was calculated from the loads applied; rotation was calculated directly from the transverse displacement measurements (Fig. 3). Rotational stiffness of the joint (Table 3) was calculated from the moment-rotation curve (Fig. 6). The large variation in rotational stiffness could be caused by variations in the lumber (as indicated by the large CV of MOE) or in the fabrication of the joint. This information is useful in the probabilistic approach to wood structure design.

There were four different types of failure modes. (1) Two specimens failed in wood because of knots in the tension zone. (2) Two specimens failed by partial tooth withdrawal (Fig. 7a); 25% of the plate on both sides of the joints withdrew in the tension zone. (3) In three specimens, one plate failed because of steel yield (Fig. 7b) and the other plate withdrew (not shown). (4) In just one specimen, plates on both sides failed because of steel yield on the tension side.

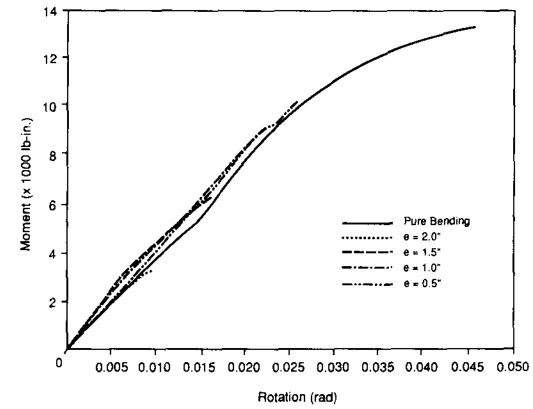

FIG. 6. Moment-rotation curves of tension splice joints under different loading conditions.

Combined loading

The results for joints tested under combined loading are given in Table 4. Maximum load at a given eccentricity was taken as the ultimate load. The ultimate moment was taken as the ultimate load multiplied by the eccentricity. Axial displacement at the joint was taken as the algebraic sum of the displacements at the tension (positive displacement) and compression (negative displacement) sides. The rotation of the joint was calculated from the transverse displacement of the joint, and was used in calculating the rotational stiffness. Two joints (Nos. 188 and 226) showed very large rotational stiffness. The moment-rotation curve of these two joints showed no irregularities and, statistically, they were not outliers. Therefore, they were included in the further analysis.

TABLE 3. *Results of joints tested in pure bending.*

No.	Joint no.	MOE, $\times 10^6$ psi	Ultimate moment, lb/in.	Rotation @ failure, rad	Rotational stiffness,[1] $\times 10^5$ lb-in./rad	Failure mode[2]
1	194	1.2	11,452	0.0496	2.5	PTF/TW
2	207	0.9	10,870	0.0531	2.5	TW
3	224	1.3	8,550	0.0301	3.5	WF
4	225	1.1	12,265	0.0531	3.4	PTF/TW
5	234	0.9	10,193	0.0633	2.1	PTF
6	238	1.3	10,453	0.0452	3.6	TW
7	239	1.5	10,470	0.0408	2.8	WF
8	242	2.1	13,287	0.0457	3.7	PTF/TW
	Mean	1.3	10,943	0.0476	3.0	
	(CV)	(30%)	(13%)	(21%)	(20%)	

[1] Stiffness = (ultimate moment/3)/deflection.
[2] PFT = plate tension failure; TW = tooth withdrawal; WF = wood failure.

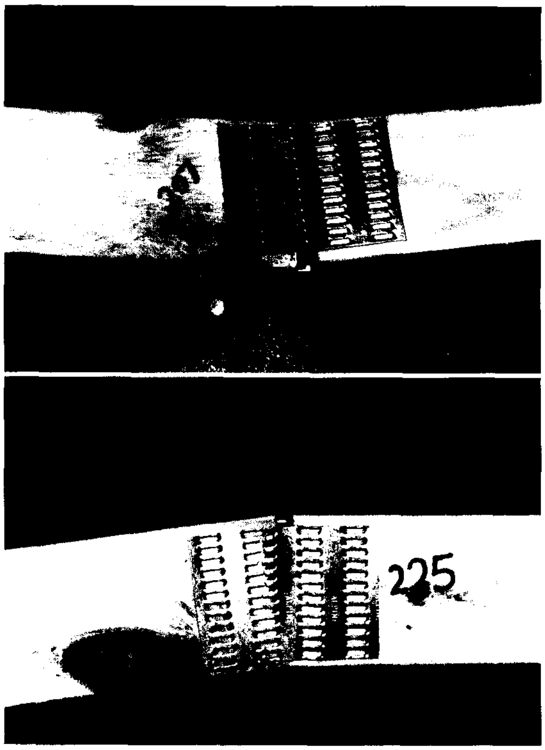

FIG. 7. Failure modes under pure bending (a) tooth withdrawal, (b) plate tension failure/tooth withdrawal.

Typical load-displacement and moment-rotation curves for all four eccentricities are shown in Figs. 4 and 6, respectively. The load-displacement curves (Fig. 4) show that the axial stiffness of the joint decreased as moment at the joint increased (increasing eccentricity); the moment-rotation curves (Fig. 6), however, show that rotational stiffness remained almost the same. In the pure bending tests, the rotational stiffness of the joint was very similar to that of the combined loading tests.

Failure modes in combined loading were a combination of failure modes in pure axial tension and pure bending. Tooth withdrawal was the most common mode of failure in combined loading; 17 of the 24 joints failed in TW. This was similar to the TW failure rate for axial tension tests (Fig. 5a). Four joints had a combination of TW and wood failure, again similar to axial tension (Fig. 5b). One other type of failure included plate failure on the one side and TW on the other side (Fig. 8).

Failure modes

The different modes of joint failure under the six different loading conditions may have been caused by variations in joint fabrication, or in lumber or plate properties. During fabrication, it is difficult to keep the plates both centered over the joint and in vertical alignment with each other. Even when plates are centered and in alignment, they can move during pressing. In addition, plates may be under- or over-pressed. A higher load was required to break test joints that had well-centered plates (e.g., joint No. 204 in Table 2). However, if there was a knot under the plate, the joint usually failed at the knot.

Most of the failures in tooth withdrawal indicate that tooth-holding capacity governs the strength of the joint. The joints should be designed so that failure occurs in the wood. The cause of the TW failures may be the size of the plate. Enlarging the plates would eliminate the TW problem; hence, failure would be in the wood or in the steel.

Tension-bending interaction

The average axial load capacity and average moment capacity for all six loading conditions are given in Table 5 with the failure modes. The axial load capacity was 6,305 lb (28 kN) for pure axial tension; the load capacity was as low as 4,711 lb (21 kN) for maximum eccentricity during combined loading (25% less than for axial tension). A plot of average axial load capacity and average moment capacity (Fig. 9) showed that as moment at the joint increased, the axial load capacity decreased. This decrease was solely due to the eccentricity, but the decrease in the axial load capacity was much slower than the increase in moment. For each 1,000 lb-in. (113 N-m) moment introduced at the joint, the axial load capacity decreased by about 200 lb (0.9 kN). This should be considered when metal-plate connected tension joints are designed. In a similar type of study with a different plate type and much larger eccentricities, Wolfe (1990) also showed that connection tensile capacity decreased with an increase in applied moment.

TABLE 4. *Results of joints tested in combined (axial tension/bending) loading.*

No.	Joint no.	Eccentricity (in.)	MOE, $\times 10^6$ psi	Ultimate load, lb	Deflection at failure, in.	Axial[1] stiffness, lb/in.	Ultimate moment, lb-in.	Rotation at failure, rad	Rotational[2] stiffness, lb-in./rad	Failure[3] mode
1	188	0.5	1.3	5,673	0.1411	122,256	2,837	0.0057	973,818	TW
2	189	0.5	0.7	5,173	0.1257	101,751	2,586	0.0094	231,754	TW/WF
3	196	0.5	1.4	6,453	0.1165	168,700	3,226	0.0095	384,113	TW/WF
4	213	0.5	1.6	6,247	0.1565	177,706	3,124	0.0100	382,583	TW
5	215	0.5	1.7	6,333	0.1336	143,436	3,167	0.0087	385,574	TW
6	226	0.5	2.5	5,764	0.1086	186,442	2,882	0.0032	706,100	TW
Mean (CV)		0.5	1.5 (40%)	5,940 (8%)	0.1303 (13%)	150,049 (22%)	2,970 (8%)	0.0078 (40%)	510,657 (54%)	
7	030	1.0	1.4	6,939	0.1501	147,992	6,939	0.0151	482,234	TW
8	033	1.0	1.5	6,188	0.1464	134,009	6,188	0.0101	654,407	TW
9	177	1.0	0.9	3,893	0.0734	114,462	3,893	0.0123	383,609	TW
10	198	1.0	1.4	5,468	0.1771	120,814	5,468	0.0139	492,915	TW
11	217	1.0	1.5	6,212	0.1546	133,420	6,212	0.0160	485,549	TW
12	231	1.0	1.9	3,992	0.0808	128,695	3,992	0.0111	496,776	TW/WF
Mean (CV)		1.0	1.4 (21%)	5,449 (23%)	0.1304 (33%)	129,899 (9%)	5,449 (23%)	0.0131 (18%)	499,248 (17%)	
13	036	1.5	1.9	5,251	0.1518	106,513	7,876	0.0218	448,914	PTF/TW
14	037	1.5	1.2	5,145	0.1556	101,191	7,718	0.0234	351,024	TW
15	178	1.5	1.3	5,875	0.1497	95,445	8,812	0.0213	450,899	TW
16	186	1.5	1.0	5,282	0.1623	88,475	7,923	0.0271	305,722	TW
17	187	1.5	1.4	5,418	0.1797	113,931	8,127	0.0243	414,235	TW
18	206	1.5	1.9	4,087	0.1030	105,935	6,130	0.0183	469,755	TW
Mean (CV)		1.5	1.5 (25%)	5,176 (11%)	0.1504 (17%)	101,915 (9%)	7,764 (11%)	0.0227 (13%)	406,758 (16%)	
19	203	2.0	1.4	3,576	0.1060	83,210	7,152	0.0224	421,476	TW
20	216	2.0	1.7	4,947	0.1508	88,264	9,895	0.0266	447,373	TW
21	221	2.0	1.0	5,382	0.1694	76,298	10,181	0.0258	399,302	PTF/TW
22	230	2.0	1.1	4,823	0.1372	87,846	9,646	0.0233	401,081	TW
23	246	2.0	1.4	4,773	0.1432	120,987	9,546	0.0262	524,782	PTF/TW
24	248	2.0	1.8	4,765	0.1377	96,707	9,529	0.0269	410,847	PTF/TW
Mean (CV)		2.0	1.4 (23%)	4,711 (13%)	0.1407 (15%)	92,219 (17%)	9,325 (12%)	0.0252 (7%)	434,143 (11%)	

[1] Axial stiffness = (ultimate load/3)/deflection.
[2] Rotational stiffness = (ultimate moment/3)/rotation.
[3] PTF = plate tension failure; TW = tooth withdrawal; WF = wood failure.

Axial force and moment ratios were plotted to show the tension-bending interaction (Fig. 10). Because of the small sample size, it was not possible to get fifth-percentile estimates of joint strength. Therefore, average values were used to fit the curve as shown in Fig. 10. The equation of the curve was:

$$\left(\frac{t}{T}\right)^a + \left(\frac{m}{M}\right)^b \leq 1 \qquad (1)$$

where:

t = axial tension force (lb)

T = axial tension force capacity (6,305 lb) (28 kN)

m = bending moment (lb-in.)

M = bending moment capacity (10,943 lb-in.) (1.2 kN-m)

a = 8.3011 and b = 0.6083.

The large value of exponent a and the small value of exponent b probably occurred because most of the combined loading data was close to the pure tension data (Fig. 10). Data points close to pure bending points would require high eccentricity (5–10 in.) as used by Wolfe (1990).

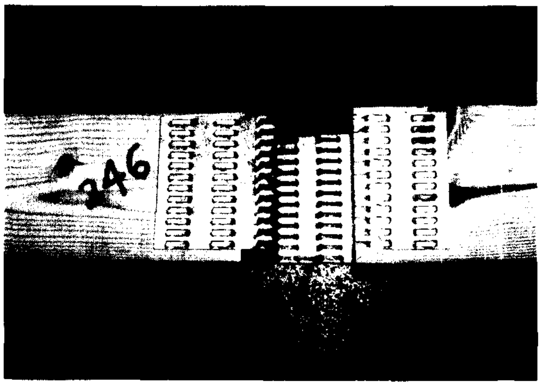

FIG. 8. Failure mode under combined loading; plate tension failure/tooth withdrawal.

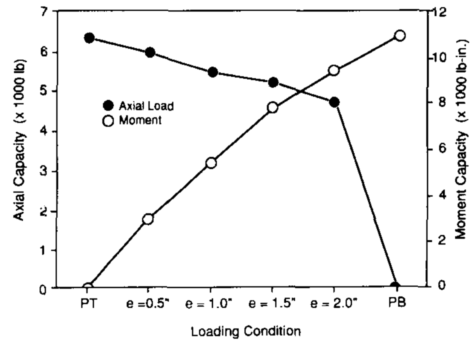

FIG. 9. Moment and axial capacities for different loading conditions.

The tension-bending curve clearly indicates that interaction between tension and bending was not linear. Wolfe (1990) used the same equation to fit the bending-tension interaction and came up with different exponent values. Just as Wolfe (1990) concluded, the significance of this type of study is not in the parameters for the interaction equation, but rather in the point that axial capacity is significantly affected by bending. MPC joints should be tested for their interactive load capacity, not just their axial load capacity.

CONCLUSIONS

The following conclusions may be drawn from this study:

1. The average strength and average stiffness of tension splice joints tested under pure tension were 6,305 lb and 2.3×10^5 lb/in., respectively. The average moment capacity and average rotational stiffness were 10,943 lb-in. and 3.0×10^5 lb-in./rad, respectively.
2. The strength rather than the deflection mainly governs the failure of the joint.
3. The axial load capacity of MPC joints decreased when bending moment was applied in addition to axial tension. The axial capacity of joints decreased by 200 lb for each 1,000 lb-in. bending moment applied in addition to axial load.
4. The most common mode of failure was tooth withdrawal, which indicates that the tooth-holding capacity at the joint governs the strength of the joint.
5. The results presented here apply to the plate type and sizes referred to in this paper.

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The testing apparatus was fabricated at Cornell University (Department of Agricultural

TABLE 5. *Average test results for metal-plate connected tension splice joints under different loading conditions.*

Loading condition	Number of tests	Average axial load capacity (lb)	Average moment capacity (lb-in.)	Coefficient of variation (%)	Failure mode[1]
Axial tension	8	6,305	0	10	TW/WF
Combined loading eccentricity (in.)					
0.5	6	5,940	2,970	8	TW/WF
1.0	6	5,449	5,449	23	TW/WF
1.5	6	5,176	7,764	11	PTF/TW
2.0	6	4,711	9,422	13	PTF/TW
Pure bending	8	0	10,943	13	PTF/TW/WF

[1] TW = tooth withdrawal; WF = wood failure; PTF = plate tension failure.

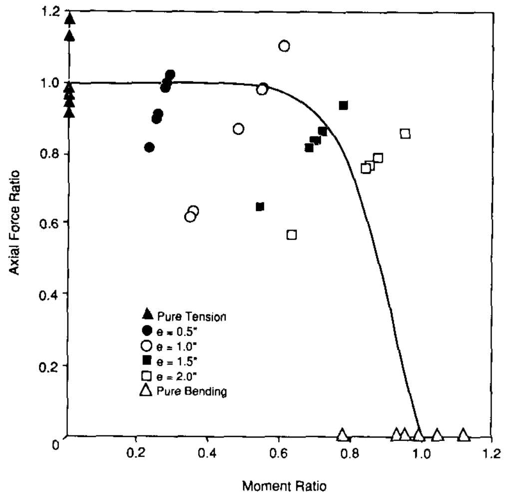

FIG. 10. Tension-bending interaction curve of tension splice joints.

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REFERENCES

AMERICAN SOCIETY FOR TESTING AND MATERIALS. 1990a. Standard test methods for moisture content of wood. D2016-74. Annual book of ASTM standards, vol. 04.09. Philadelphia, PA.

———. 1990b. Standard test methods for specific gravity of wood and wood-based materials. D2395-83. Annual book of ASTM standards, Vol. 04.09. Philadelphia, PA.

———. 1991. Standards methods of testing mechanical fasteners in wood. D1761-77. Annual book of ASTM standards, Vol 04.09. Philadelphia, PA.

CANADIAN STANDARDS ASSOCIATION. 1980. Methods of test for evaluation of truss plate used in lumber joints. CSA Standard S347-M1980. Ottawa, Ontario.

EUROPEAN UNION OF AGREMENT (UEAtc). 1990. Rule for the assessment of punched metal plate timber fasteners. MOAT 16. 4 Avenue du Recteur-Poincare, 75782 Paris Cedex 16.

FELTON, K. E., AND H. D. BARTLETT. 1964. Punched-metal truss plates used in timber joints. Trans. ASAE 7(2):159–163.

GUPTA, R. 1990. Reliability analysis of metal plate connected residential wood trusses. Ph.D. thesis, Cornell University, Ithaca, NY.

———, AND K. G. GEBREMEDHIN. 1990. Destructive testing of metal-plate-connected wood truss joints. ASCE J. Struct. Eng. 116(7):1971–1982.

HAYASHI, T., AND H. SASAKI. 1982. Static tensile strength of wood but joints with metal plate connectors. Pages 22–36 *in* Wood research no. 68, Bulletin of the Wood Research Institute. Kyoto University, Uji, Kyoto, Japan.

INTERNATIONAL ORGANIZATION FOR STANDARDIZATION. 1990. Timber structures—Testing of unilateral punched metal plate fasteners and joints. ISO 8969:1990(E). Case Postales 56, CH-1211 Geneva 20, Switzerland.

KIRK, L. S., T. E. MCLAIN, AND F. E. WOESTE. 1989. Effect of gap size on performance of metal-plated joints in compression. Wood Fiber Sci. 21(3):274–288.

MCALISTER, R. H. 1989. Interaction between truss plate design and type of truss framing. Forest Prod. J. 39(7/8):17–24.

QUAILE, A. T., AND F. J. KEENAN. 1979. Truss plate testing in Canada: Test procedure and factors affecting strength properties. Pages 105–112 *in* Proceedings, Metal Plate Wood Truss Conference. Forest Products Research Society, Madison, WI.

SNEDECOR, G. W., AND W. G. COCHRAN. 1980. Pages 279–280, 489 *in* Statistical methods, 7th ed. The Iowa State University Press, Ames, IA.

SUDDARTH, S. K., D. H. PERCIVAL, AND Q. B. COMUS. 1979. Variability in tension performance of metal plate connections. Pages 98–104 *in* Proceedings, Metal Plate Truss Wood Conference. Forest Products Research Society, Madison, WI.

TRUSS PLATE INSTITUTE. 1985. Design specification for metal plate connected wood trusses: TPI-85. Madison, WI.

WOLFE, R. W. 1990. Metal-plate connections loaded in combined bending and tension. Forest Prod. J. 40(9): 17-23.

———, M. HALL, AND D. LYLES. 1991. Test apparatus for simulating interactive loads on metal plate wood connections. J. Testing Eval. 19(6):421–428.

APPENDIX A—MAXIMUM NORMED RESIDUAL (MNR) TEST FOR OUTLIER

$$MNR = \frac{\max|x - \bar{x}|}{\sqrt{\Sigma(x - \bar{x})^2}} \qquad (A1)$$

where x = any observation
$\bar{x}$ = mean

Tension-splice joint strength values (lb)
7,493, 6,227, 5,765, 6,120, 5,995, 5,953, 5,761, 7,123, 2,854

$$\bar{x} = 5{,}921 \text{ lb}$$

$$\sqrt{\Sigma(x - \bar{x})^2} = 3{,}675 \text{ lb}$$

$$\max|x - \bar{x}| = |2{,}854 - 5{,}921| = 3{,}067 \text{ lb}$$

$$\text{MNR} = \frac{3{,}067}{3{,}675} = 0.835$$

Significance levels of the MNR for a normal sample from Table A16 (i) in Snedecor and Cochran (1980): for sample size = 9,

$$\text{MNR at 5\% level} = 0.783$$
$$\text{MNR at 1\% level} = 0.844.$$

Calculated MNR is slightly less than MNR at 1% level. Therefore, deviation of this size (2,854 lb) will occur less than 5% of the time and slightly more than 1% of the time under normal conditions.