Research paper

Development of Roof Trusses with Metal Plate Connectors Using Domestic Plantation Timber[1)]

Min-Chyuan Yeh,[2,3)] Yu-Li Lin,[2)] Chi-Lung Chiang,[2)] Yuan-Chang Liu[2)]

【Summary】

Metal plate connectors (MPCs) were developed and applied to truss assemblies to evaluate the structural performance of Japanese cedar roof trusses in the study. An MPC with a tooth length of 9 mm, a plate thickness of 0.9 mm, and a tooth density of 1 tooth cm^{-2} was designed and produced using SS33 structural steel sheets. The loading capacities of the designed MPC were 216~265 N per tooth, and the derived allowable lateral resistance was 68.3~86.4 N per tooth for Japanese cedar lumber joints in tension. Ultimate bending capacities and stiffness of the 38×140-mm wood roof truss system were 82.6 and 48.9% higher than those of 38×89-mm wood roof truss, respectively. No significant difference was found between Howe and Fink trusses. The critical failures in Japanese cedar roof trusses were located at the heel joint, and these accounted for 75% due to a combination of tooth withdrawal and plate failure in the 38×140-mm truss, and 87.5% due to the tooth withdrawal in the 38×89-mm truss. Flexural deflections of Japanese cedar roof trusses measured at the design load level were within 5.0~20.5% of the design deflection limitation. Furthermore, the designed roof load was only 9.9~18.9% of the ultimate equivalent distributed loads measured at truss rupture, which assures the safety of Japanese cedar roof trusses in service.

Key words: wood roof truss, metal plate connector, flexural property, Japanese cedar.

Yeh MC, Lin YL, Chiang CL, Liu YC. 2010. Development of roof trusses with metal plate connectors using domestic plantation timber. Taiwan J For Sci 25(2):107-16.

1) This study was supported by a grant (NSC93-2622-B-020-003-CC3) from the National Science Council of Taiwan. 本研究承國科會(NSC93-2622-B-020-003-CC3)補助經費，謹此致謝。

2) Department of Wood Science and Design, National Pingtung University of Science and Technology, 1 Xuehfu Rd., Neipu, Pingtung 91201, Taiwan. 國立屏東科技大學木材科學與設計系，91201屏東縣內埔鄉學府路1號。

3) Corresponding author, e-mail:yehmc@mail.npust.edu.tw 通訊作者。

Received September 2009, Accepted December 2009. 2009年9月送審 2009年12月通過。

研究報告

國產造林木開發刺鐵板組合屋頂木桁架之研究[1)]

葉民權[2,3)] 林玉麗[2)] 江吉龍[2)] 劉原彰[2)]

摘 要

本研究開發三種刺鐵板聯結件，並應用於屋頂桁架之組合，進一步評估所組成之柳杉屋頂桁架之結構性能。所開發之刺鐵板聯結件採用SS33結構用鋼材質，齒長為9 mm，齒密度為每平方公分1齒，鐵板厚為0.9 mm。柳杉材經刺鐵板聯結件組合後，在引張條件下測定，所設計之刺鐵板聯結件的載重能力為每齒216~265 N，同時，經推導後所得之容許抵抗值為每齒68.3~86.4 N。以38×140 mm構材所組成屋頂木桁架系統之最大抗彎載重值及抗彎剛性，分別高於38×89 mm條件者82.6及48.9%，而哈威式與芬克式桁架之間則無顯著之差別。柳杉屋頂桁架在抗彎過程中，其破壞關鍵位置在上下弦桿接合處；在構材尺寸為38×140 mm條件下，桁架有75%是屬於齒引拔及板剪斷之破壞，在38×89 mm條件下，有87.5%是屬於齒引拔破壞。柳杉屋頂桁架在設計載重條件下測定之抗彎變位為設計限制之5.0~20.5%，而屋頂之設計載重亦僅為桁架破壞時之極限等效均布載重之9.9~18.9%，故在實際應用上具有相當之安全性。

關鍵詞：屋頂木桁架、刺鐵板聯結件、抗彎性質、柳杉。

葉民權、林玉麗、江吉龍、劉原彰。2010。國產造林木開發刺鐵板組合屋頂木桁架之研究。台灣林業科學25(2):107-16。

INTRODUCTION

A truss system is generally constructed with slender members in a triangular pattern and becomes a lightweight, stiff, and internally stable load-bearing structure. The truss system can be continuously expanded by adding 2 members for every node, and has benefits based on both strength and economic considerations. It becomes crucial in wood structure applications as large solid-wood members were replaced by small-sized lumber due to scarcities of wood resources and cost considerations. Yeh et al. applied the truss system to flooring joist manufacturing with plantation timber and considered different joist depths (2000), and various nail types, adhesives (2001), and orientations of the metal tube web members (2002). However, only a few studies have examined the structural performance of light-weight wood roof truss systems using domestic plantation timber.

Characteristics of metal plate connectors (MPCs) such as the tooth pattern, size and thickness of the plate, and metal quality differ from each MPC maker, and various products have different strength performances. Yeh and Hsieh (1995) indicated that the wood surfaces might be damaged by using long teeth, and a high-density tooth distribution might also reduce the net area of the plate. A better combination of 8 teeth in^{-2} with a 9-mm tooth length was suggested. Wolfe (1990) found that a maximum 85% reduction in the load resistance for MPC spliced joints might result if an axial tension force were combined with a bending moment. In a similar investigation by Gupta (1994) using a different MPC design, a 26% reduction was also found as the MPC joints were loaded with combined bending and tension forces.

On the other hand, characteristics of the wood should also have a relation to the MPC joint performance. Via et al. (1999) found that teeth withdrew first from the location of low wood density, and the MPC joint withdrawal strength increased as the specific gravity of southern pine lumber increased with a good correlation. McAlister and Faust (1992) developed MPC joints with hardwood lumber and compared the joint strength with softwood. The results indicated that MPC joints assembled with yellow poplar and sweetgum lumber showed better stiffness and higher tooth withdrawal resistance than those with southern pine and laminated veneer lumber. Therefore, it is important to understand the structural performance of MPC joints as applied to domestic plantation timber in Taiwan. In this study, an adequate MPC was designed and applied to a wood roof truss manufacturing technique using small-sized plantation lumber to fabricate full-size trusses. The effects of truss configuration and member size on the flexural performance of the trusses were investigated, and a systematic structural analysis for the design procedure is also proposed.

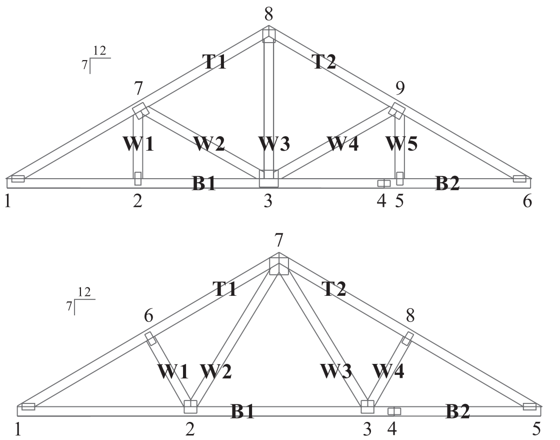
MATERIALS AND METHODS

Materials

The 25~30-yr-old Japanese cedar (*Cryptomeria japonica*) plantation timber was harvested from the no. 7 forest compartment managed by Hsinchu Forest District, Taiwan Forestry Bureau (TFB). Logs were then sawn, kiln-dried, and planed into 100 pieces of 38×89×3600- and 38×140×3600-mm lumber, respectively. Three sizes of MPCs were developed, D6-12 (63×120 mm with 72 teeth), D12-12 (120×123 mm with 144 teeth), and D16-18 (160×183 mm with 188 teeth) using a stamp die. The MPCs were made of SS33 structural steel 0.9 mm thick, with 1 tooth cm^{-2} for the tooth distribution, and a 9-mm tooth length.

Design and assembly of the roof trusses

Two types of wood roof trusses were chosen: Howe (H) and Fink (F) trusses with a slope of 7:12 as shown in Fig. 1. The structural sawn lumber was visually graded based on the rules for bending members as specified



Truss	D6-12	D12-12	D16-18
H-89	1, 2, 4, 5, 6	3, 7, 8, 9	
H-140		1, 2, 4, 5, 6	3(2), 7, 8, 9
F-89	1, 4, 5, 6, 8	2, 3	7
F-140		1, 4, 5	2, 3, 6, 7, 8

Fig. 1. Five-meter Howe and Fink trusses constructed with 2 sizes of Japanese cedar lumber using metal plate connectors.

in the CNS14630 standard. The length and height of the assembled truss specimens were 5000 and 1450 mm, respectively. Three different types of MPCs were then assigned to each adequate node or joint of the truss system for member connections. The assembly procedures of truss members followed both the plate placement method (PPM) and tooth count method (TCM) as specified by ANSI/TPI (2002). Japanese cedar lumber of no. 1 and 2 grades was assigned to the top and bottom chord members and no. 3 lumber was used as web members in the truss system following the suggestion of the TPIC (1996). The pressures applied during the truss member assembly with MPCs were 1.5, 2.1, and 3.3 MPa for the D6-12, D12-12, and D16-18 connectors, respectively. For each experimental condition, 2 truss types×2 member sizes, were tested with 4 replications.

MPC joint strength test

The joint strength of trusses assembled with Japanese cedar and MPCs was evaluated by a tensile test. Two pieces of lumber were butt-joined in parallel (A type) to each other with D6-12 connectors following the suggestion of ASTM D1761 (2002). Two specimen members joined in a perpendicular alignment (E type) were also proposed in the study. All jointed specimens were put into a conditioning room for 14 before testing. The tensile test was evaluated using a universal testing machine, and the maximum tensile resistance, joint stiffness, bearing strength of a tooth, and failure mode were analyzed. Each joint type had 6 replicates. The allowable withdrawal resistance of the PMC was further derived based on ANSI/TPI procedures.

Flexural test of wood roof trusses

The assembled full-size roof trusses were tested using a 4-point bending approach as specified in ASTM D198 (2002). A bending testing machine with 500-kN maximum capacities was employed with 2 loading heads applied to the nodes at the top chords of the truss. The span was 4850 mm, and 4 sets of lateral bracing were applied between supports to prevent truss bulking. The ultimate loads at failure and bending stiffness were calculated, and failure modes of the wood roof trusses were analyzed. The flexural performance of the trusses, including load capacities under the code deflection limitation, maximum flexural deflection under required design loads, and allowable equivalent distributed load estimations, was further evaluated based on the criteria specified in local building codes.

RESULTS AND DISCUSSION

The average moisture content of the Japanese cedar specimens was 14.7±1.0%, and the specific gravity in an air-dried condition was 0.50±0.07. The average annual ring width of sawn lumber was 5.8±2.6 mm. The major factor influencing the grade quality of Japanese cedar structural lumber is the knot size. The grade frequency distributions for the 38×89×3600-mm structural lumber products were 14% for no. 1, 50% for no. 2, and 36% for no. 3 lumber. In the case of 38×140×3600-mm specimens, these were 14% for no. 1, 55% for no. 2, and 30% for no. 3 lumber.

MPC joint strength properties

The A type joint, with both the wood grain and major axis of the PMC parallel to the loading direction, of Japanese cedar members showed 25.8% better tensile resistance than that of the E type joint, with the wood grain perpendicular to the loading direction and the MPC major axis parallel to the loading direction (Table 1). This was due to the

Table 1. Tensile resistance of Japanese cedar metal plate connector joints subjected to a tensile test

Joint type	Ultimate load (kN)	Load per tooth (N)	Displacement at failure (mm)	Stiffness ($kN\ mm^{-1}$) Design load	Critical slip	Initial slope	Failure occurrence (%)
A	15.96±1.18	265±20	4.08±0.44	56.8±21.6	22.5±2.0	87.2±20.6	TW[2]
	(0.07)[1]		(0.11)	(0.39)	(0.07)	(0.23)	100
E	12.73±1.26	216±20	2.90±0.39	22.5±12.7	15.7±3.9	35.3±15.7	TW
	(0.12)		(0.14)	(0.55)	(0.25)	(0.45)	100
HJ-30	14.99	255	--	--	--	--	--

1) Coefficient of variation.

2) TW, tooth withdrawal.

different holding mechanisms of the MPC orientation between the A and E type joints. Each tooth in the A type joint cuts off wood fibers during the MPC embedding process creating better friction, while the tooth in the E type squeezes into wood fibers resulting in weak withdrawal resistance against pulling forces. Similar results were found in the research by McCarthy and Wolf (1987) and Via et al. (1999). The number of effective teeth was 30 on each lumber end at the MPC joint as counted from the original 72 teeth on the D6-12 connector based on ANSI/TPI specifications. Therefore, the loads that could be carried by 1 tooth were 265 and 216 N for the A and E types joints, respectively. In the case of a heel joint (HJ) of the pitched roof truss, the slope formed between top and bottom chord members was 30° (HJ-30) in this study. The MPC joint strength or withdrawal strength at the HJ was calculated using the Hankinson formula revised by Foschi's model based on the report of McCarthy and Wolf (1987). Thus, the estimated value of HJ-30 was between those of the A and E type joints as shown in Table 1. Emerson and Fridley (1996) also reported that the derived withdrawal strength of MPC joints with a 30° orientation from A and E type joints could match the experimental results for southern pine.

The stiffness of MPC joints of Japanese cedar specimens can be estimated through 3 different approaches using the relationship between the load and tensile displacement in the tensile test. Stiffness measured based on both the design load and critical slip is the secant stiffness, while the initial tangent stiffness is the slope of the linear range of the load-displacement curve (Gupta 1994). One-third of the ultimate tensile load at failure was chosen as the design load in the calculation and the critical slip employed a 0.381-mm (0.015 in.) tensile displacement to estimate the secant stiffness of the MPC joint. The initial tangent stiffness was obtained from a linear regression of the linear part in the load-displacement curve below the design load. The results showed that the initial tangent stiffness had the highest estimated values followed by the secant stiffness using the design loads (64.6%), and then the secant stiffness using critical slips (35.1%), which is similar to the trend found by Gupta's studies. Furthermore, the overall stiffness of the E type PMC joint was only 49.9% of the A type joint with Japanese cedar lumber.

The allowable lateral resistance of the MPC joint can also be derived based on the procedure specified in ANSI/TPI (2002). The exclusion value based on the 5% level was 1.65 and the strength ratio was 0.84 when considering the allowable gap between the lumber surface and metal plates after assembly. The duration of the load factor used 1.6 for the case of a short-term test of 10 min. The safety factor was 1.3 for loading cases, stress variation, and environmental influence. The variation coefficient of the MPC ultimate load for Japanese cedar lumber was considered in the evaluation procedure, and the results are given in Table 2. They show that the allowable lateral resistance of the PMC joint was about 32.2~35.7% of the ultimate tensile load capacity. These data assured the safety of the truss system in service as designed with the developed MPC assembly approach for Japanese cedar sawn lumber products. The allowable lateral resistances per tooth were then calculated as 86.4 and 68.3 N for the A and E types of MPC joint, respectively, based on the number of effective teeth for the D6-12 connector.

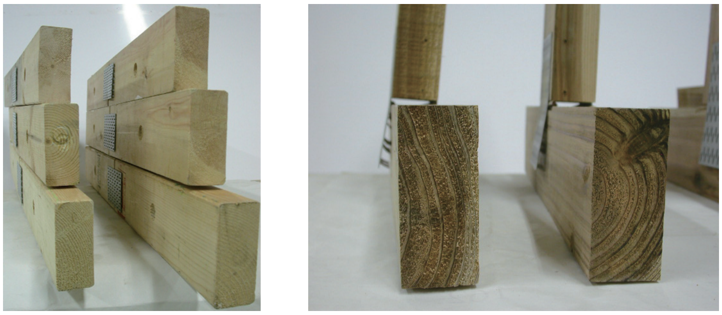
Several studies identified 3 types of major MPC joint failure when subjected to a load: teeth withdrawn (TW) from the wood, wood failure (WF), and MPC failure (PF). All (100%) MPC joints failed due to teeth withdrawn from Japanese cedar lumber in the tensile test. Teeth withdrawal always began from 1 side containing heartwood or pith causing asymmetrical PMC deformation as shown in Fig. 2 and accounted for up to 92% of all failure cases. Most of the teeth experienced fragile breaks and the wood showed plastic deformation at the contact area; the sequence of MPC joint failure was also detailed by Gupta and Gebremedhin (1990).

Flexural properties of wood roof trusses

Four types of wood roof trusses were tested using the 4-point bending test. The average ultimate loading capacity of the truss assembled with 38×140-mm members performed better than that assembled with 38×89-mm members by 82.6% as shown in Table 3. This was due to the larger sizes of the PMC, i.e., D12-12 and D16-18 used 38×140-mm member assembly resulting in twice the effective teeth at each node of the truss. On the other hand, there was no significant difference in the ultimate loading capacity between Howe and Fink truss types. Wolfe

Table 2. Allowable lateral resistance derived for MPC joints subjected to a tensile load

Joint type	MPC allowable lateral resistance (kN)	Allowable lateral resistance per tooth (N)
A	5.70	86.4
E	4.10	68.3



(A) A-type (B) E-type

Fig. 2. Tooth withdrawal failure in A and E types of metal plate connector joints in tension.

Table 3. Location of failure and occurrence of failure types for roof trusses subjected to a four-point bending test

Truss type	Primary location		Occurrence of failure (%)[1)]				Ultimate load (kN)	Stiffness ($\times 10^3$ kN-m^2)
	Heel joint (%)	Splice joint (%)	TW	TW&S	TW&PF	TW&WF		
F-89[2)]	75	25	75	25	--	--	23.0±1.9	2.50±0.26
H-89	75	25	100	--	--	--	21.8±0.8	2.99±0.14
Average	75	25	87.5	12.5	0	0		
F-140	100	--	--	--	100	--	41.7±1.4	3.75±0.02
H-140	50	50	25	25	50	--	40.0±2.0	4.47±0.64
Average	75	25	12.5	12.5	75	0		

[1)] TW, tooth withdrawal; S, shearing; PF, plate failure; WF, wood failure.
[2)] 89, member size 38×89 mm; 140, 38×140 mm.

and LaBissoniere (1991) suggested that the flexural performance of a sloped wood roof depends on the height, span, and configuration of the structure. They reported 20~57% differences in bending capacities among various wood trusses. In this study, the difference in member configurations between Howe and Fink trusses was the vertical web member. Once the vertical web member becomes a zero-force member, it would result in a similar force passing between the 2 truss systems when subjected to a flexural load.

All failures were found at the nodes of trusses with different types of damage to the MPC joint instead of the wood member itself. It was also noted that most (75%) trusses failed at the heel joint, followed by the splice joint at 25%, where a butt joint between 2 bottom chords experienced a tension state during load application. In the case of the F-140 truss combination, all trusses failed at the heel joint. This indicates that the MPC used at the heel joint, i.e., the D6-12 connector for F-89 and H-89, and the D12-12 connector for F140 and H140, is critical for improving the bending capacity of Japanese cedar trusses. Based on the structural truss analysis, the axial tension force of the bottom chord at the heel joint would be 18.72 kN when the H-89 truss combination was subjected to an ultimate bending load. A tension force of 1.33 kN for the bottom chord was estimated based on the design load for the roof truss. On the other hand, the maximum MPC joint strength with a slope of 30° was calculated to be 14.99 kN (Table 1) from the MPC tensile test. This means that the measured heel joint strengths of the H-89 and F-89 trusses were 24.9% higher than that of the derived value, and also 14 times the value under the design load.

Causes of truss failure in bending differed due to the sizes of the wood members used. In the case of the 38×140-mm truss, major damage (75%) was due to tooth withdrawal and plate failure at the heel joint (TW and PF) (Fig. 3). In the case of the 38×89-mm truss, 87.5% of the trusses failed by tooth withdrawal (TW) due to the weak holding ability of the teeth or the small size of the MPC, and another 12.5% failed by tooth withdrawal and tooth shearing (TW and S) (Fig. 4).

The performance of bending stiffness for a roof truss was dependent on the wood member stiffness and joint displacement as a load was applied. The results indicated that the bending stiffness of the truss fabricated with the 38×140-mm Japanese cedar lumber

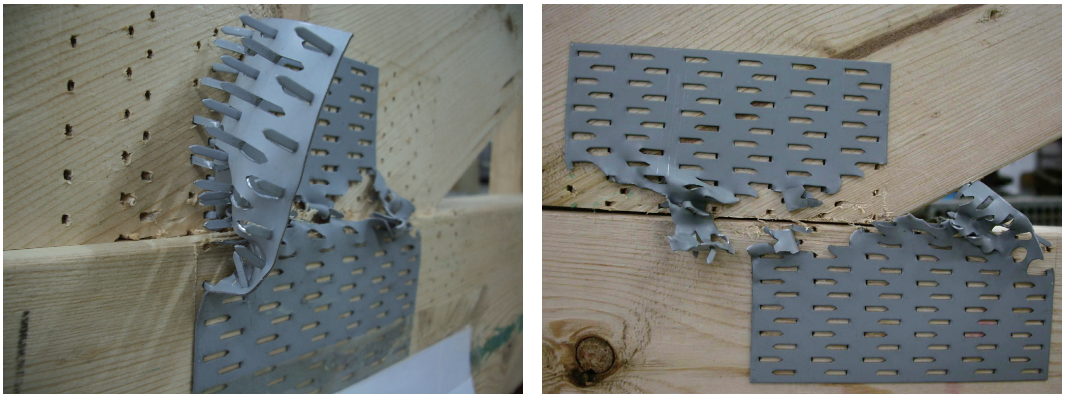
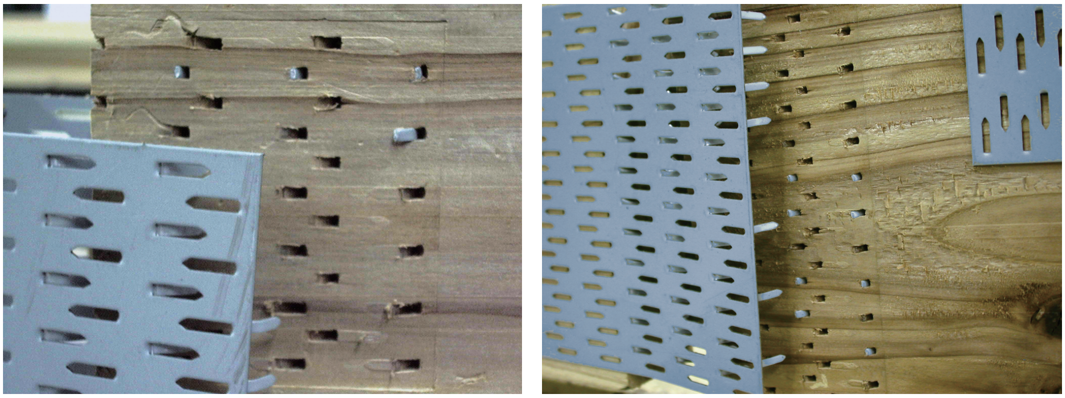


Fig. 3. Heel joint failures resulting from tooth withdrawal and plate failure.



Left: 38×89 mm Right: 38×140 mm

Fig. 4. Splice joint failures resulting from tooth withdrawal and tooth shearing.

was 48.9% higher than that with 38×89-mm members (Table 3). In fact, the moment of inertia of the 38×140-mm truss member was 3.89-times that of the 38×89-mm member, and there were 216 effective teeth at the critical heel joint for the 38×140-mm truss and only 96 effective teeth for the 38×89-mm truss in this study. Wolfe et al. (1986) also indicated that the major contribution to the bending deflection of a truss within the linear range is due to the deformation of the wood member rather than MPC joint displacement. Thus, better stiffness of a truss structure can be obtained by using wood members with a larger size and higher modulus of elasticity (MOE).

Design properties for Japanese cedar roof trusses

There is a limitation on the flexural deflection, for a span length L/300, in the building code for a beam member subjected to a bending load (Ministry of the Interior 2003). In this study, the roof dead load (0.588 kPa) plus live load (0.784 kPa) were considered to be the design load for Japanese cedar trusses constructed with the 600-mm spacing. The measured flexural deflections of wood roof trusses under a design load were then between 0.8 and 3.3 mm, or 5.0~20.5% of the allowable deflection, which showed satisfactory stiffness performance of the 4 truss types in service (Table 4).

Table 4. Flexural properties of Japanese cedar roof truss under design code requirement

Truss type	Displacement at design load (mm)	Load at design displacement: Equivalent distributed load (kPa)	Equivalent distributed load at rupture (kPa)
F-89	3.3±0.8	6.24±0.62	7.91±0.62
H-89	2.3±0.2	6.96±0.24	7.24±0.27
F-140	1.5±0.6	9.17±0.22	13.87±0.47
H-140	0.8±0.4	10.51±0.92	13.31±0.66

The loading capacity of a truss subjected to a 4-point bending test can be converted into an equivalent distributed load for design estimations. It showed that the equivalent distributed loads of Japanese cedar trusses calculated at the upper limitation of the flexural deflection were 4.55~7.66-times the design load and the Howe truss fabricated with 38×140-mm members had the highest loading capacity. Furthermore, the design load for a roof structure was only 9.9~18.9% of the equivalent distributed load at truss rupture, which assures the safety of Japanese cedar roof truss in practice.

CONCLUSIONS

Three types of metal plate connectors were developed to fabricate roof trusses with 2 sizes of Japanese cedar structural lumber in the study. Better MPC joint strength was found with the loading direction parallel to wood grain compared to that in the perpendicular direction. The stiffness of the MPC joint measured based on the design load point or critical slip point were conservative compared to the initial tangent stiffness. After assembly with MPC connectors, the wood roof truss constructed with 38×140-mm members showed better bending load capacity and stiffness than that with 38×89-mm members, but no significant difference was found between the Howe and Fink trusses. All roof trusses showed good bending performance and met the requirement of the flexural deflection limitation and allowable design loads, which assured their safety in service. All MPC-assembled trusses failed at the joints during the bending process instead of the member itself, and the critical location was mostly at the heel joint. This suggests that the loading capacity of a truss can be improved by increasing the size of the MPC at the heel joint.

LITERATURE CITED

American National Standards Institute/Truss Plate Institute. 2002. Commentary and appendices to the national design standard for metal plate connected wood truss construction. Washington, DC: Truss Plate Institute. 116 p.

American Society for Testing and Materials. 2002. Standard test method for mechanical fasteners in wood. ASTM D1761-88. Philadelphia, PA, USA.

American Society for Testing and Materials. 2002. Standard test method of static tests of lumber in structural sizes. ASTM D198-99. Philadelphia, PA, USA.

Emerson RN, Fridley KJ. 1996. Resistance of metal plate connected truss joints to dynamic loading. For Prod J 46(5):83-90.

Gupta R. 1994. Metal plate connected tension joint under different loading. Wood Fiber Sci 26(2):212-22.

Gupta R, Gebremedhin KG. 1990. Destructive testing of metal plate connected wood truss joints. J Struct Eng 116(7):1971-82.

McAlister RH, Faust TD. 1992. Load/deflection parameters for metal plate connectors in yellow-poplar and sweetgum structural lumber. For Prod J 42(3):60-4.

McCarthy M, Wolfe RW. 1987. Assessment of truss plate performance model applied to southern pine truss joints. Madison, WI: US Department of Agriculture. Forest Service. FPL-RP-483. 13 p.

Ministry of the Interior. 2003. Specification of wood-framed structure design and construction techniques. Taipei, Taiwan: Construction Magazine. p 5-1~24. [in Chinese].

Truss Plate Institute of Canada. 1996. Truss design procedures and specifications for light metal plate connected wood trusses. TPIC-1996. Quebec, Canada: Truss Plate Institute of Canada. p 4-5.

Via BK, Zink-Sharp A, Woeste FE, Dolan JD. 1999. Relationship between tooth withdrawal strength and specific gravity for metal plate truss connections. For Prod J 49(7/8):56-63.

Wolfe RW. 1990. Metal plate connections loaded in combined bending and tension. For Prod J 40(9):17-23.

Wolfe RW, LaBissoniere T. 1991. Structural performance of light-frame roof assemblies II. Conventional truss assemblies: conventional truss assemblies. Madison, WI: USDA Forest Service. FPL-RP-499. 28 p.

Wolfe RW, Percival DH, Moody RC. 1986. Strength and stiffness of light-frame sloped trusses. Madison, WI: USDA Forest Service. FPL-RP-471. 16 p.

Yeh MC, Chang LG, Chin SW, Ma CS. 2002. Flexural properties of metal-wood trussed beams. For Prod Ind 21(4):253-62. [in Chinese with English summary].

Yeh MC, Chuang CC, Lin YH, Hung JJ. 2000. A study on trussed beams assembled with domestic plantation timber. Q J Chin For 33(3):407-16. [in Chinese with English summary].

Yeh MC, Hsieh CH. 1995. An evaluation of metal plate connections on domestic *Cryptomeria* sawn lumber. For Prod Ind 14(1):13-31. [in Chinese with English summary].

Yeh MC, Lin YC, Hsu YC, Lin CY. 2001. Effect of nailing on the flexural properties of trussed beams fabricated with China fir. Q J Chin For 34(3):331-42. [in Chinese with English summary].