Research paper

Evaluation of the Structural Performance of Portal Frames Using Japanese Cedar Glulam

Min-Chyuan Yeh,[2,3] Yu-Li Lin,[2] Shu-Yu Deng[2]

【Summary】

In this study, a symmetrical mixed-grade composition glulam was fabricated using 37~39-yr-old Japanese cedar plantation timber. A full-size portal frame was then constructed with two 140×304.8×2200-mm posts and a 140×304.8×3000-mm beam to investigate the resisting performance of the frame system to lateral forces. Each joint between the glulam beam and post members was assembled with a design L- or I-type metal connector with 2 placement types using 8 pin fasteners. The glulam portal frame was subjected to a rack test with 7 stages of a lateral cyclic loading protocol and a monotonic loading application. Results indicated that a 12.7% increase in the maximum lateral load capacity was found for the glulam portal frame using a straight metal connector with pin fasteners arranged in a square compared to fasteners arranged in a circle. Also, the portal frames assembled with fasteners in the square placement for both the L- and I-type connectors showed 20.6~36.4% higher initial stiffness and dissipated energy values than those in the circular arrangement. The reference shear strengths of the Japanese cedar glulam portal frames were 9.6~13.7 kN, which were derived from critical $P_{1/120}$ values among all connector and fastener placement conditions. These reference shear strengths of glulam portal frames were 19.8%, on average, of the maximum lateral load capacity. Resultant multiplier parameters of the shear wall were 2.33~3.35, and better structural performance was found for portal frames assembled with the L-type connector and pins arranged in a square.

Key words: Japanese cedar, portal frame, structural glulam, lateral resisting performance.

Yeh MC, Lin YL, Deng SY. 2015. Evaluation of the structural performance of portal frames using Japanese cedar glulam. Taiwan J For Sci 30(1):31-44.

[1] This study was supported by a grant (MOST 98-2313-B-020-002-MY3) from the Ministry of Science and Technology. 本研究承科技部(MOST 98-2313-B-020-002-MY3)補助經費，謹此致謝。

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Received November 2014, Accepted February 2015. 2014年11月送審 2015年2月通過。

研究報告

柳杉集成材門型剛架之結構性能評估

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摘 要

本研究利用37~39年生柳杉造林木製造對稱異等級結構用集成材，並以尺寸為140×304.8×2200 mm之集成材為柱，以140×304.8×3000 mm之集成材為梁組成實大尺寸之門型剛架，用以評估其結構系統對側向力之抵抗性能。在梁柱接合處分別以L及I型金屬連結件設計並以8支插銷扣件採取不同配置進行組合，所組成之門型剛架以7階段反覆載重及單向載重進行側向載重試驗。結果顯示，在I型連結件組合條件下，插銷扣件以方型排列配置之集成材門型剛架的最大水平破壞強度較以圓型配置條件高出12.7%。在結構剛性及能量散逸方面，以L及I型連結件者在插銷扣件方型排列配置條件下較圓型配置條件均能高出20.6~36.4%。所推導之柳杉集成材門型剛架基準剪斷強度為9.6~13.7 kN，其關鍵載重為該剛架系統之$P_{1/120}$，約為平均最大破壞載重之19.8%。所得之剪力牆乘數約為2.33~3.35，其中又以L型連結件且扣件以方型配置之門型剛架之結構性能為優。

關鍵詞：柳杉、門型剛架、結構用集成材、側向抵抗性能。

葉民權、林玉麗、鄧書宇。2015。柳杉集成材門型剛架之結構性能評估。台灣林業科學30(1):31-44。

INTRODUCTION

Portal frame structures are an effective wood construction type for resisting lateral loads such as seismic and wind loads. They are constructed with a wood beam and column members in rigid joints and feature high bending moment capacities at the joint when subjected to loading applications. The resisting bending moment capacity of a cantilevered wood beam member connected to a column or the column member connected to a sill varies with different bolt or pin fastener alignments when subjected to a bending load (Yeh et al. 2008, 2012, Nakata and Komatsu 2009b). The resulting bending moment capacity of the beam-column connections may further affect the relative rigidity of the assembled portal frame structures. When a boundary condition exists between a fixed-supported and pin-supported connection, it can be recognized as a partial-fixity and results in a different moment distribution on the entire structure, which depends on the relative rigidity of the connection. On the other hand, post and beam structures are a major wooden building construction type in Taiwan, which require proper joint rigidity, such as providing bracing to resist lateral loads. It was suggested that portal frame construction can be an alternative to technical bracing approaches due to its simplicity. However, little research related to evaluating the moment resisting capacity of portal frame structures constructed with glulam members from domestic plantation timber was found.

Noguchi et al. (2006) introduced a multistory frame structure to resolve the problem of the poor lifetime cycle for most Japanese housing due to old-fashioned ideas about house maintenance. He proposed 2 wooden portal frames, one with an improved column to locate low bending moments and the other with an extended panel zone from the column to increase the moment capacity. Both modified frames showed structural advantages, especially in stiffness. Nakata and Komatsu (2009a) developed plates and pins from a compressed laminated veneer lumber (LVL) material as connectors and fasteners for a new portal frame connection. They found that the maximum lateral resistance of the assembled portal frame was determined by the structural performance of the column-to-base joint, and the resulting failure mode was also determined by the characteristic failure of the column-to-base joints. Further, a semi-rigid portal frame was constructed using a new joint system which was composed of hardwood wedges and metal ware (Komatsu and Hosokawa 1998). This timber connection method combined a traditional timber jointing technique with modern timber engineering knowledge, and was shown to be useful for small-scale buildings with a sufficient safety factor and ductility. Therefore, the rigidity of a connection and the structural performance of a portal frame are closely related to the fastener and connector types.

The extent of rigidity of a connection can be developed from a hinge-supported to a fixed-supported situation by increasing the number of fasteners. Yeh et al. (2008) reported that the bending moment capacity of glulam beam-column connections could be improved by 60.2% using steel plate connectors with 6 bolts, compared to values with 4 bolts. However, the failure mechanism for the portal frame structure with multiple connections seemed to differ from a test of a single beam-column joint. On the other hand, the allowable stresses specified in current design codes for wood member connections are limited in either tension or shearing applications which are inadequate for the moment resisting mode (Ministry of the Interior 2011). The resisting stresses incurred at the connection become complex when wood structures are subjected to a lateral load, and therefore investigation of full-sized portal frames was suggested to assess their structural performances.

In this study, structural glulam members were manufactured using timber from local Japanese cedar plantations. Portal frames were constructed using 2 types of metal connectors and 2 types of fastener alignment to investigate the structural adequacy of glulam frames based on the moment resisting performance in relation to the mechanical connection rigidity.

MATERIALS AND METHODS

Materials

Timber from 37~39-yr-old Japanese cedar (*Cryptomeria japonica*) plantations was harvested from a forest located in the no. 7 forest compartment of the Hsinchu Forest District. The timber was sawn, kiln-dried, and planed into a size of 38×140 mm as laminar products. The laminae were then tested with a tap-tone approach using Fast Fourier Vibration Analyzer software (Fakopp Enterprise, Agfalva, Hungary) to determine dynamic modulus of elasticity (MOE) values. A grade based on dynamic MOE values was then assigned to each lamina. The layout process of lamina combinations for the assigned symmetrical mixed-grade composition glulam was performed before the assembly process. During the glulam lamination, resorcinol phenol formaldehyde (RPF) adhesive mixed with a hardener of paraformaldehyde in a ratio of 100: 15 was used. Adhesive at 250 g m^{-2} and a pressure of 0.98 MPa were applied for glulam fabrication. Glulam fabrication was carried out using 8-m-long assembly equipment in a pilot factory on the campus of National

Pingtung Univ. of Science and Technology (Fig. 1). Glulam specimens with a size of 140×304.8×3000 mm were assembled as beam members and 140×304.8×2200 mm as column members following the CNS 11031 procedure for manufacturing symmetrical mixed-grade composition structural glulam products (BSMI 2006). The beam-column connection was designed using 9-mm-thick SS400 steel plates for I- and L-type connectors (Figs. 2, 3). The column-base connection was designed as a π type with a thicker steel plate (Fig. 4). Bolts and pins both had a diameter of 15.88 mm and were used as the moment resisting connection fasteners.

Fig. 1. Structural glulam assembled with Japanese cedar lamina after being glued and pressed.

Methods

For the portal frame assembly, an embedded connection between the beam and column members was designed. The ends of the glulam beam members were first drilled with 16-mm holes to accommodate pin fasteners, and a 10-mm wide slot was ripped to insert the 9-mm thick metal connectors. Two glulam column members were then connected to a beam member on both ends forming a framed specimen. Eight pins were arranged in a square (S) or circular (C) form at both the beam and column member ends at the joints. To evaluate the full-size bending moment

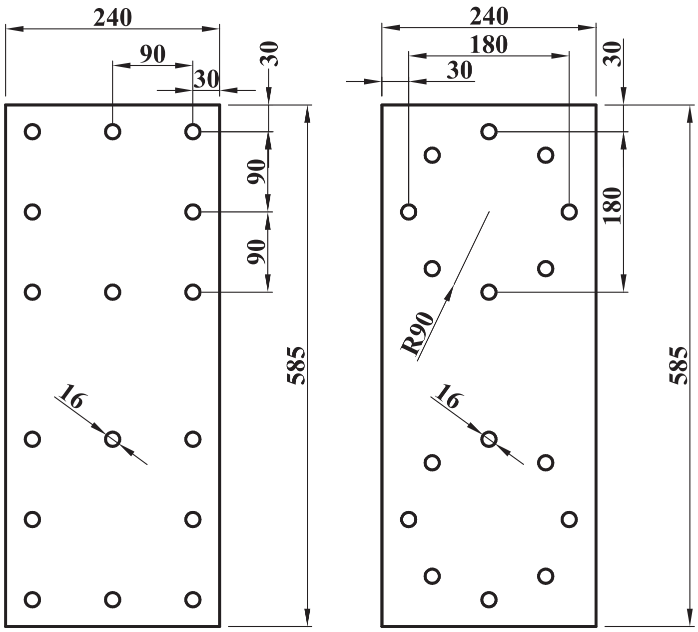

Fig. 2. Design of metal connectors for beam-column member joints in a glulam portal frame (I type) (units: mm).

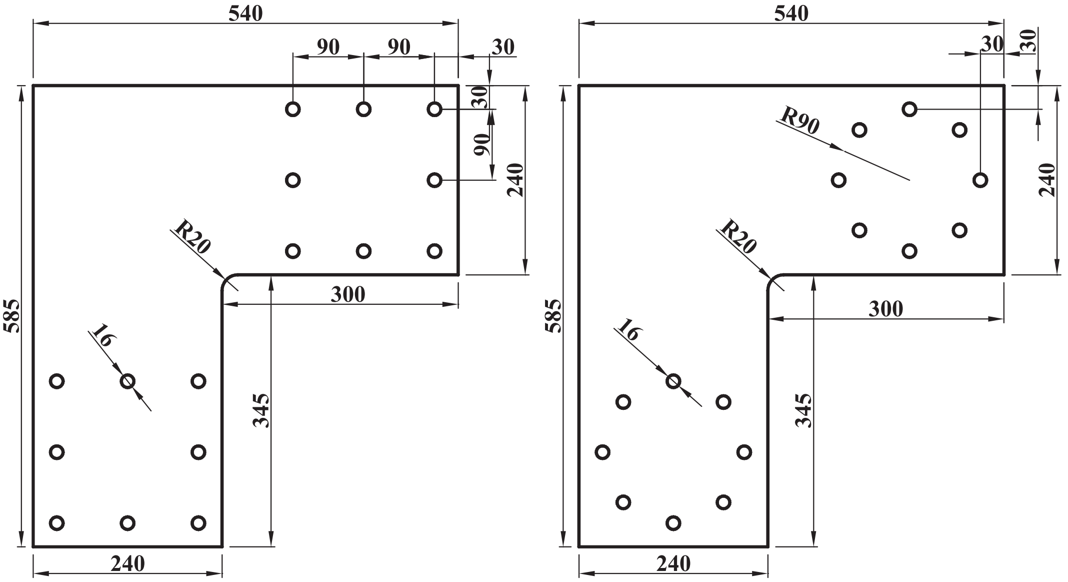

Fig. 3. Design of metal connectors for beam-column member joints in a glulam portal frame (L type) (units: mm).

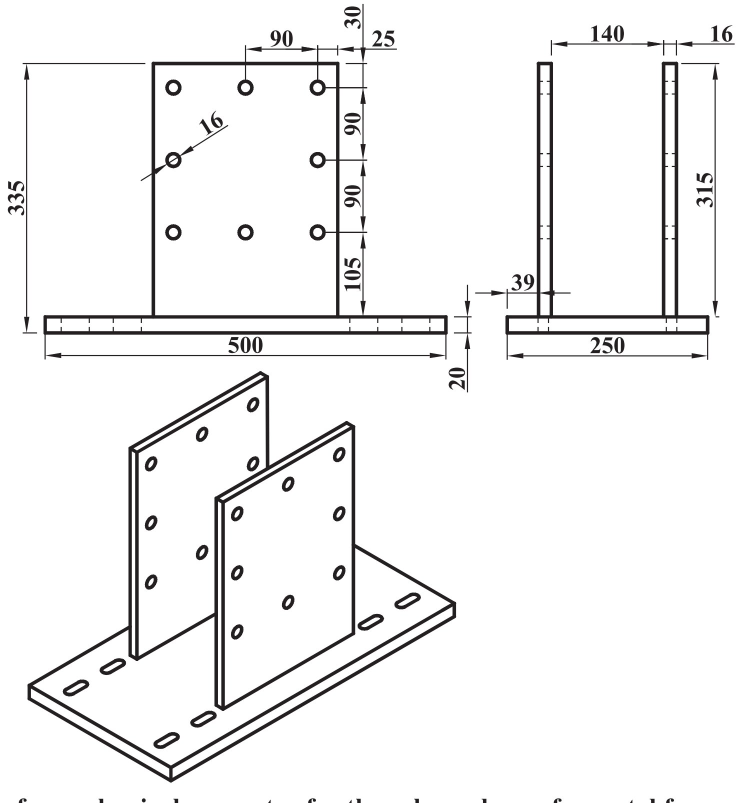

Fig. 4. Design of a mechanical connector for the column-base of a portal frame (πtype) (units: mm).

capacity, the framed glulam structure was installed using the column-base connections which were fixed on the testing steel frame in advance (Fig. 5). Each column-base connection employed a π type of steel connector and was fastened with 8 bolts. All fastener placements followed code recommendations for spacing, edge, and end distance limitations (Ministry of the Interior 2011), and consequently the sizes of the SS400 steel connector could be determined. The fitness of the predrilled holes for pin insertion was 0. A hydraulic loading facility with a capacity of 200 kN was used for the rack test of the portal frames. A cyclic lateral loading protocol with 7 stages ranging 1/300, 1/200, 1/150, 1/100, 1/75, 1/50, and 1/30 radians in lateral deformation angles was applied to the portal frame specimen with 3 repeated loads for each stage as shown in Table 1 (Komatsu and Hosokawa 1998, The Japan Housing and Wood Technology Centre 2001). Strength properties of a glulam portal frame, including the maximum load and deformation, yield load and deformation, stiffness, ductility factor, and structural characteristic factor, were then estimated based on the method proposed by The Japan Housing and Wood Technology Centre (2001). In total, 4 conditions were examined with 3 replicates for each joint configuration.

RESULTS AND DISCUSSION

Failure of glulam portal frames

Four major failure modes of glulam portal frames were identified, i.e., splitting at the bolt holes in the column-base connections, bearing failure of the wood perpendicular to the grain on beam member surfaces, splitting at the pinned holes of column members, and bending of fasteners. Failure at the column-base connections was found for all Japanese cedar glulam portal frame specimens. The column began to crack at the bolt holes when the lateral load was applied up to 0.02~0.03 rad in shear deformation during the 6th~7th stages. This was similar to the results of a portal frame assembled with compressed LVL

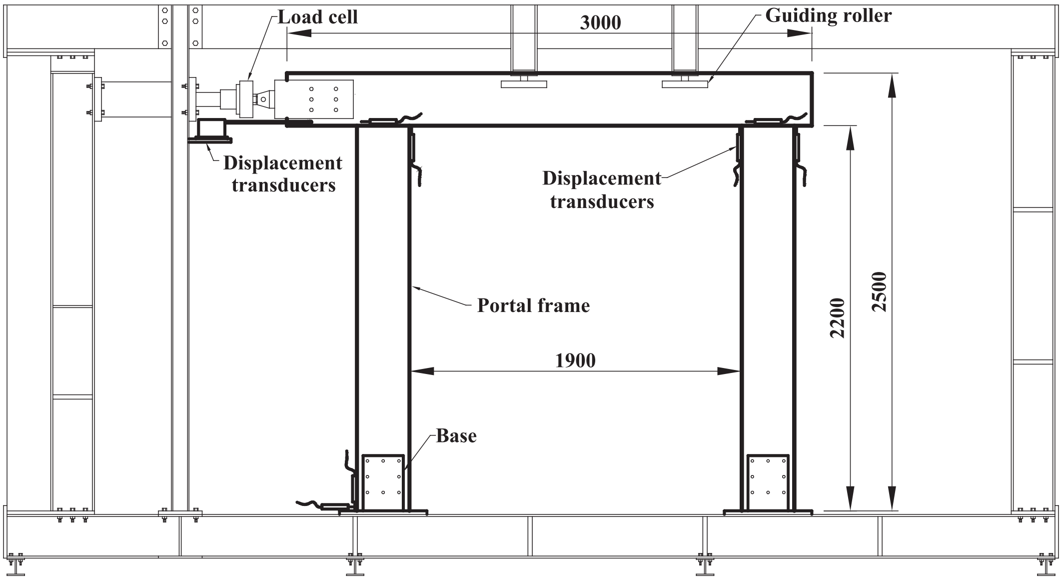

Fig. 5. Cyclic loading test for a portal frame structure assembled with Japanese cedar glulam members (units: mm).

Table 1. Protocol of the lateral cyclic loading test for glulam portal frame structures

Cyclic loading stage	Deformation angle R (rad)	Horizontal displacement δ (R/h)[1)] (mm)	Number of cycles
1	±1/300	7.83	3
2	±1/200	11.75	3
3	±1/150	15.67	3
4	±1/100	23.50	3
5	±1/75	31.33	3
6	±1/50	47.00	3
7	±1/30	78.33	3
Final	1/8 or until failure		

[1)] h, the distance between the bottom of the portal frame column-base joint to the center of the top beam.

pins in which the failure began at 0.025~0.028 rad (Nakata and Komatsu 2009a). Splitting always occurred at the outside bottom bolted hole of the column-base connection, especially on the column member which was near the load application point. In most cases, the split on the column member propagated upward along the bolted row until failure occurred (Fig. 6), and it also split at knots or along a slanted wood grain near the joints. Thus, the spacing between fasteners is important to reduce the tendency to split.

Fig. 6. Splitting that occurred along bolted holes at the column base of a glulam portal frame during the racking test.

In the case of beam-column connections, local compressive failure perpendicular to the wood grain occurred at all glulam beams supported by a glulam column located underneath. Wang (1993) reported that Japanese cedar wood features high compressive strength (21.39 MPa) parallel to the wood grain and low strength (5.69 MPa) perpendicular to the wood grain. The resisting moment was transmitted through the metal connectors and pin fasteners at the joints, and forced the portal frame structure to deform in an integrated manner. Thus, a force concentrated at the corner of the column member top, which was located at the beam-column contact interface, would cause a depression on the beam surface. In most cases of beam-column connections which were assembled with L-type metal connectors using a square placement (S) of fasteners, splitting occurred along the bolt holes at the top end of the column members. The joint might have had a higher internal resisting moment because of a longer distance between 2 fastener placement centers or rotation centers in the L-type connector compared to that in the I-type connector. Splitting was also found at the beam end where a circular placement (C) of fasteners was used with I-type connectors. To resist the resultant moment at each frame joint, some bolts and pins were bent more seriously than others. This suggests that each fastener shares different magnitudes of the applied load due to the orthotropic characteristics of the wood when the connection is subjected to a rotational moment. High stress results when a fastener is loaded parallel to the wood grain, while low stress results when a fastener is loaded perpendicular to the wood grain (The Architecture Association of Japan 1995).

Racking strength of portal frames

The assembled structures of the Japanese cedar glulam portal frame specimens were monotonically loaded to failure after experiencing 7 stages of cyclic loads. In cases with the straight metal connector, the maximum lateral load capacity (P_{max}) of a glulam portal frame was 53.64±4.58 kN for fasteners arranged in the circular form (I-C condition) (Table 2). A 12.7% increase was found for fasteners arranged in the square form (I-S portal frames). Noguchi et al. (2006) reported a maximum lateral load of 16.17 kN for a traditional 2600×2730-mm Japanese cedar frame, which was assembled with 4 bolts at each member connection. In comparison, it was about 25.8~30.1% of the test results for specimens assembled with 8 bolts, which showed reinforcement by increasing the number of fasteners in this study. Further, Nakata and Komatsu (2009b) tested a 2720×2730-mm portal frame assembled with compressed LVL plates and 8 pins at beam-to-column joints and 9 steel bolts at column-to-base joints. A maximum lateral load of 53.4 kN was found, which was close to the results of the I-C condition in our study.

The characteristic parameter of joint strength in a frame structure can be estimated through the load-displacement relationship (Fig. 7).The ultimate yield load (P_u) of a portal frame can be obtained by a bilinear approximation using envelop curves which are based on data of the lateral load-displacement relationship. The bilinear model is usually used to describe the structural behavior of a joint assembled with different fastener types. The resulting P_u was 96.5% of the maximum lateral load on average for all test conditions, which was 27.4% higher than the yield load (P_y) value on average. P_y can be obtained from the cross point of 2 tangent lines estimated between 0.1 and 0.4 P_{max} and between 0.4 and 0.9 P_{max} on load-displacement curves.

Table 2. Strength properties of Japanese cedar glulam portal frames subjected to cyclic lateral loads

Condition	Max. load P_{max} (kN)	Yield load P_y (kN)	Yield deformation δ_y (10^{-3} rad)	Ultimate yield load P_u (kN)	Ultimate yield deformation δ_v (10^{-3} rad)
I-C	53.6±4.6	44.0±10.5 (82.1%)[1)]	36.4±8.0	53.0±7.0 (98.9%)[2)]	45.1±9.2
L-C	57.0±3.9	42.8±4.3 (75.1%)	30.9±4.6	54.4±4.4 (95.4%)	39.4±6.9
I-S	60.5±1.8	40.0±7.1 (66.1%)	22.5±5.8	58.9±1.4 (97.4%)	40.3±3.6
L-S	62.6±0.4	51.5±2.1 (82.3%)	32.9±4.7	59.2±5.0 (94.6%)	44.5±3.0

[1)] Percentage of P_y/P_{max}.
[2)] Percentage of P_u/P_{max}.
Conditions are illustrated in Figs. 2 and 3.

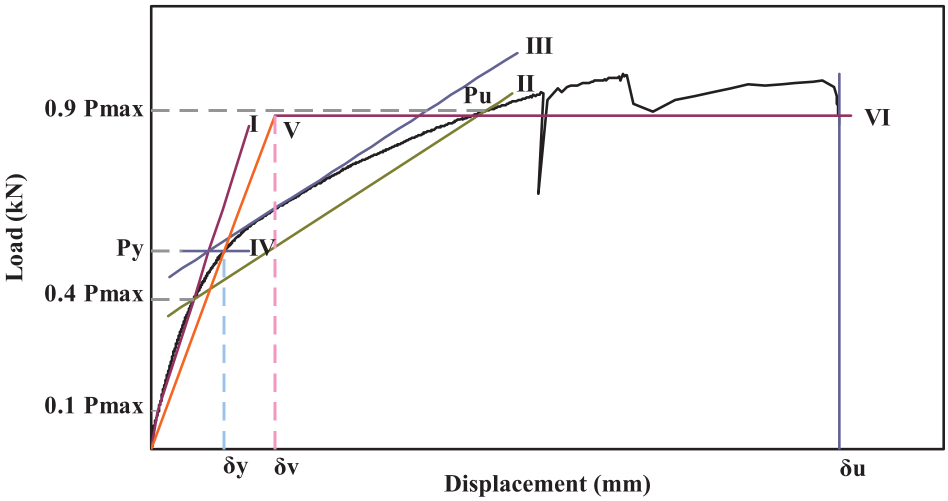

Fig. 7. Demonstration on the evaluation of structural characteristic parameters for Japanese cedar glulam portal frames subjected to lateral loading (Yeh et al. 2012).

Nakata and Komatsu (2009b) reported 3 and 18.1% differences between P_u and P_y for glulam portal frames with 2 types of connector design, respectively, showing different load-resisting performances. A large difference between P_u and P_y indicates a broader transition range from elastic to plastic behavior of a portal frame structure subjected to a lateral load.

In the case of cyclic load application, the load paths for each loading stage basically followed the previous load-displacement curves and showed no obvious damage to the frame connections (Fig. 8). But a frame structure with a lower load resistance resulted in subsequent cycles at each stage than that during the first cyclic load application , which might have been due to the looseness at the connection. However, it was noted that no plastic deformation or hysteresis recovery of portal frame specimens was found after completing each loading cycle. This suggests that the glulam portal frames remained mostly in an elastic state after experiencing lateral loads of up to 1/30 radian shear deformation for 7 cyclic stages.

The initial stiffness (K) of a portal frame was obtained from the slope of a line passing through both the origin point and yield point on the load-displacement curves. In cases of a straight metal connector, the initial stiffness of a Japanese cedar glulam portal frame was $(1.19 \pm 0.12) \times 10^3$ kN rad^{-1} for fasteners arranged in the circular form (I-C condition) (Table 3). A 23.5% increase was found for fasteners arranged in the square form (I-S portal frames) and 32.8% after being further replaced with the L-type metal plate (L-S portal frames). Park et al. (2014) reported that the initial stiffness of a 2400×3600-mm Korean post-beam structure, which was connected with traditional dovetail joints, was 0.55×10^3 kN rad^{-1}. Another reason causing it to be a less-stiff structure might also be attributed to the pinned joint at the post base in that research work. In comparison, the initial stiffness of the dovetail jointed structure in this study was about 34.8~46.2% of the portal frames assembled with bolts and pins.

The dissipated energy of a portal frame was measured over the area of the load-displacement curves with the frame specimen

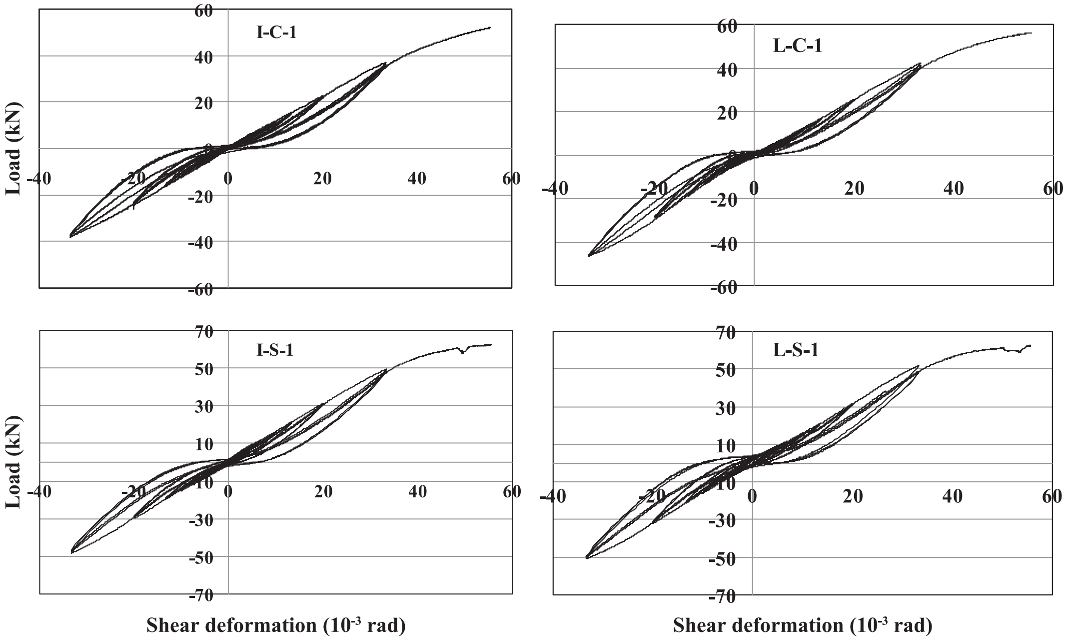

Fig. 8. Relationship between cyclic load and shear deformation of Japanese cedar portal frames jointed with I-C and I-S conditions. Conditions are illustrated in Figs. 2 and 3.

Table 3. Structural characteristics of Japanese cedar glulam portal frames subjected to cyclic lateral loads

Condition	Initial stiffness K (10^3 kN rad^{-1})	Energy dissipation (kN-m)	Ductility factor, μ	Structural characteristic factor, D_s
I-C	1.19±0.12	4.07±0.22	1.27±0.29	0.83±0.14
L-C	1.40±0.18	4.57±0.37	1.44±0.23	0.74±0.10
I-S	1.47±0.10	4.91±0.18	1.39±0.12	0.75±0.05
L-S	1.58±0.19	5.55±0.34	1.33±0.09	0.78±0.04

Conditions are illustrated in Figs. 2 and 3.

loaded to the maximum and then dropped to 0.8 P_{max} during the last monotonic loading. This explains the ability of a portal frame to plastically deform when resisting lateral loads, and hence represents the ability to sustain seismic or wind forces. In cases of the straight metal connector, the dissipated energy of a glulam portal frame was 4.07±0.22 kN-m for fasteners arranged in the circular form (I-C condition). A 20.6% increase was found for fasteners arranged in the square form (I-S portal frames) and a 36.4% increase with the L-S condition. Noguchi et al. (2006) reported dissipated energy of 3.89 kN-m from a traditional Japanese cedar frame which was constructed with 4 bolts at each member connection. In comparison, that was close to the I-C frame results and 42.7% lower than the L-S frame condition in this study. Therefore, designing a joint with a metal connector and multiple bolts or pins can improve both the stiffness and energy dissipation ability of a portal frame.

The ductility factor (μ) of a portal frame specimen is estimated as the ratio of δ_u/δ_v, where the maximum deformation limit (δ_u) is the lateral deformation of the portal frame at 0.8 P_{max} after reaching the maximum load, and δ_v is the ultimate yield deformation. This can be used to measure the deformation capacity of structures, and a lower value of the ductility factor indicates a tendency toward brittle structural performance of a portal frame. Table 3 shows values of 1.27~1.44 for μ of test specimens, and no significant differences between the circular and square placements of fastener parameters or between the straight and L-type plate parameters were found in the study. Nakata and Komatsu (2009b) reported a value of 1.81 for a portal frame which had a similar fastener alignment, i.e., a circular placement at the beam-column connection and a square placement at the column-base connection. However, the ductility factor was improved by up to 2.32-fold when the square placement was replaced with an oval placement in that research work. Pirvu et al. (2000) reported that a value of 4.7 for μ was found by gluing a 540-mm-long metal plate at the beam-column joint compared to 2.5 for a portal frame reinforced with a shorter or 270-mm glued metal plate. Those results suggested better structural behavior of a portal frame glued with long metal plates against an earthquake event than that of frames with shorter metal plates. The reasons for the higher ductility factor of Douglas fir LVL portal frames in Pirvu et al.'s work than of Japanese cedar glulam portal frames in this study might have been due to the stronger wood species used and more-homogeneous LVL material used.

The structural characteristic factor (D_s), or load reduction factor, can be estimated from the ductility factor as $1/(2\mu - 1)^{1/2}$. It indicates the resistance reserve capacity of a structure beyond the elastic behavior and depends on the ductility and type of structure. Values of D_s ranged 0.74~0.83 as shown in Table 3, which was higher than those of portal frames assembled with compressed LVL pins (0.54) from Nakata and Komatsu's work (2009b) and LVL portal frames (0.3~0.6) from Pirvu et al.'s work (2000). It is recommended that values of D_s be between 0.30 and 0.45 for wooden portal frames, where 0.3 indicates excellent ductility of a structure (Housing Bureau, Ministry of Construction 1990). Therefore, results showed that the tendency toward brittle behavior of portal frame structures, which were constructed with Japanese cedar glulam members using bolts and pins, might have been due to the early split failure around the bolt holes at the connections as discussed in the previous section.

Design properties of glulam portal frames

The lateral load measured at 1/120 rad shear deformation ($P_{1/120}$) is commonly used to evaluate the strength of a wooden structure with wall components and is directly related to the allowable load (Komatsu and Hosokawa 1998). The resultant resisting capacities at 1/120 rad shear deformation of the Japanese cedar glulam portal frames measured during both the cyclic load application and final monotonic load application are shown in Table 4. A reduction to 45.0% on average for $P_{1/120}$ during the final monotonic load application after experiencing the previous cyclic load application was found for portal frames assembled with pins in a circular placement at the beam-column connections, while a reduction to 61.9% on average for that in the square placement, i.e., the I-S and L-S conditions, was found. Obviously, the tightness of a connection is significantly influenced by repeated load applications, and more-conservative results were obtained from the final monotonic load application. The allowable shear force (Q_a) was then calculated by multiplying $P_{1/120}$ by 3/4. Results indicated that

Table 4. Allowable structural performance of Japanese cedar glulam portal frames subjected to lateral loads

Condition	Lateral load at 1/120 rad $P_{1/120}$ (kN) (cyclic stage)	Allowable shear force Q_a (kN) (cyclic stage)	Lateral load at 1/120 rad $P_{1/120}$ (final stage)	Allowable shear force Q_a (kN) (final stage)
I-C	9.92±0.70 (18.5%)[1)]	7.44±0.53 (14.4%)	4.40±0.41 (8.2%)	3.30±0.31 (6.2%)
L-C	11.36±0.53 (19.9%)	8.52±0.40 (14.9%)	5.18±0.57 (9.1%)	3.89±0.43 (6.8%)
I-S	12.59±1.10 (20.8%)	9.54±0.75 (15.8%)	7.66±1.27 (12.7%)	5.75±0.95 (9.5%)
L-S	14.25±1.20 (22.8%)	10.69±0.90 (17.1%)	8.96±1.87 (14.3%)	6.72±1.40 (10.7%)

[1)] Values are percentages of P_{max}.
Conditions are illustrated in Figs. 2 and 3.

values of Q_a derived from the cyclic loading process were 14.4~17.1% of P_{max}, while they were only 6.2~10.7% of P_{max} for Q_a derived from the final monotonic loadings.

The reference shear strength (P_a) of a Japanese cedar glulam portal frame can be derived similarly to the process for the wooden wall element. One must first consider the racking test results of P_{max}, P_y, P_u, and $P_{1/120}$ together with their variation coefficients for each assembly condition, and the minimum derived P_r value is then assigned as the final P_a. Results showed that the lowest values for all portal frame conditions were derived from the lateral load resistance measured at 1/120 rad shear deformation, i.e., $P_{1/120}$ (Table 5). The joint efficiency of a wood structure can be expressed as the percentage of the referenced strength to the maximum lateral resisting load. The derived P_r from maximum loads and yield load were 65.2 and 71.5% on average, respectively, while the average critical P_r from $P_{1/120}$ was 19.8%. Large differences among those derived P_r values indicated some improvement on the joint or frame stiffness would be beneficial, potentially to upgrade the structural performance of the portal frames. Nakata and Komatsu (2009a) obtained a value of 19% joint efficiency in box-type glulam portal frame structures, which is close to our results. However, they claimed that a 30% joint efficiency from moment-applying tests of column-to-sill structures might be appropriate for a practical portal frame with a moment-resisting joint.

Portal frame structures can function as a shear wall to resist a lateral load. Normally the structural performance of each shear wall type can be estimated with a multiplier parameter (m). When a wall element experiences a horizontal 1/120 rad shear deformation with a corresponding allowable lateral load of 1.274 kN m^{-1}, the multiplier value is taken as 1. The allowable lateral load is determined as the minimum value of $(3/4)P_{1/120}$ and $(2/3)(3/4)P_{max}$. By considering the width of the glulam portal frame specimens, m_{frame} values for each assembled portal frame types were calculated and are shown in Table 6. Results indicate that double or triple the lateral resisting capacity of a standard shear wall with 15 x 90-mm bracing was obtained from Japanese cedar portal frames in this study. The results can be further compared to a shear wall element with

Table 5. Reference shear strength (P_r) of Japanese cedar glulam portal frames estimated based on P_{max}, P_y, P_u, and $P_{1/120}$ from racking tests

Condition	2/3 P_{max} (kN)	P_y (kN)	$0.2P_u/D_s$ (kN)	$P_{1/120}$ (kN)
I-C	34.3 (64.0%)[1)]	39.1 (72.9%)	12.4 (23.1%)	9.6 (17.9%)
L-C	36.8 (64.6%)	40.8 (71.6%)	14.0 (24.6%)	11.1 (19.5%)
I-S	39.8 (65.8%)	36.7 (60.7%)	15.3 (25.3%)	12.1 (20.0%)
L-S	41.6 (66.5%)	50.5 (80.7%)	14.9 (23.8%)	13.7 (21.9%)

[1)] Values are percentages of P_{max}.
Conditions are illustrated in Figs. 2 and 3.

Table 6. Multiplier (m_{frame}) for shear walls of Japanese cedar glulam portal frames assembled with difference connections

Joint type	m_{frame} (cyclic stage)	m_{frame} (final stage)
I-C	2.33±0.16	1.03±0.10
L-C	2.67±0.12	1.22±0.13
I-S	2.96±0.23	1.80±0.30
I-S	3.35±0.28	2.11±0.44

Joint types are illustrated in Figs. 2 and 3.

a larger size of 45 x 90-mm bracing (m_{wall} = 2) and 90-mm bracing timber (m_{wall} = 3) (The Architecture Society of Japan 2011). In the case of arranging fasteners in a circular placement at the beam-column joint, the average portal frame m_{frame} value was 2.5. A 20.7% increase was found for fasteners arranged in the square form. Pirvu et al. (2000) reported that values of 3.2~5.2 can be further obtained for an LVL portal frame using bolts in addition to epoxy glue application at the connection. Therefore, the structural performance of portal frames can be upgraded for shear wall purposes with adequate stiff connections.

CONCLUSIONS

A full-sized portal frame was constructed with symmetrical mixed-grade composition glulam members using timber from 37~39-yr-old Japanese cedar plantations in this study. The resisting performance of the frame system to lateral forces showed that the glulam portal frames can remain in an elastic state after experiencing lateral loads of up to 1/30 radian shear deformation. The large difference between the ultimate yield load and yield load indicates a broader transition range from elastic to plastic behavior of the portal frame structure. The Japanese cedar glulam portal frames assembled with fasteners in a square placement showed better structural performance of the maximum lateral load capacity, initial stiffness, and dissipated energy, compared to those of frames assembled with fasteners in a circular placement.

The large reduction in the load resisting capacities measured at 1/120 rad shear deformation from the cyclic load application to the final monotonic load application indicated that the tightness of connections was significantly influenced by repeated load applications. The critical reference shear strength (P_a) of the Japanese cedar glulam portal frames was 9.6~13.7 kN and was determined by the lateral load resistance measured at 1/120 rad shear deformation. The frames showed double or triple the lateral resisting capacity of a standard shear wall with 15 x 90-mm bracing, which proves the effectiveness of Japanese cedar glulam portal frames.

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