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SEISMIC PERFORMANCE EVALUATION OF MASS TIMBER FRAMES REINFORCED WITH WOODEN SHEAR WALLS

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ABSTRACT: As wooden buildings gradually become taller, lateral force resistance performance of wooden frames is required. Therefore, this paper presents the results of a cyclic behavior study on mass timber frames reinforced with CLT or SIP shear walls. As a result of the cyclic loading test, the lateral performance of the wooden frame infilled with CLT shear wall was excellent because the strength of CLT was higher. However, when the failure mode of a specimen of wooden frame infilled with SIP shear wall was investigated, the strength of the SIP was sufficiently exhibited because the failure of the SIP occurred without fracture of steel connections. Therefore, if the joint stiffness between GLT frames and CLT shear wall is increased, it is considered that the structural design can fully exhibit the strength of CLT.

KEYWORDS: GLT frame, CLT, Structural insulated panel (SIP), Shear wall, Seismic performance.

1 – INTRODUCTION

Recently, as interest in environmental issues has increased, worldwide, the demand for wood buildings is increasing instead of steel structures and reinforced concrete structures. This is because wood not only emits less carbon but also has excellent thermal performance and natural beauty. Also, it is known that wood structure has light-weight, easy assembly, and high strength. Due to these advantages, middle-to-high rise wooden buildings are continuously being built around the world, and in August 2022, a high-rise wooden building was built in Milwaukee, USA.

Many middle- and high-rise wooden buildings were built around the world because of the development of engineered wood products, such as glued-laminated timber (GLT), cross-laminated timber (CLT), laminated veneer lumber (LVL), oriented strand board (OSB), structural insulated panel (SIP) etc. Representatively, glued-laminated timber (GLT) is defined as an engineered wood made of at least three solid-swan lumber bonded parallel, can be mainly used as a column or beam member. The cross-laminated timber (CLT) is made of at least three orthogonally bonded layers of solid-swan lumber, intended for roof, floor, or wall applications.

In order to utilize the characteristics of engineered wood and apply them to wooden buildings, many researchers have conducted a number of studies to analyse the mechanical properties of engineered wood at the level components. In addition, a lot of research has been conducted on joint technology to connect engineered wood, and connecting metal made of thin steel plates 2~6mm has been developed.

For the construction of middle and high-rise wooden buildings, it is important to secure the stiffness of connection, and it is difficult to ensure the stiffness of connection with existing connection metal of thin steel plate. The reason why the stiffness of the connection is important is to ensure the smooth transfer of force between members when an external force is applied, and the higher the building is, the more the member design must be able to resist lateral load. Recently, a glued-in rod connection using steel rods has been developed to increase the stiffness of joint, and various mechanical properties such as pull-out performance, flexural performance, and shear performance of the glued-in rod connections are being studied.

After evaluating the mechanical properties of engineered wood at the level components, research on engineered wood beam-to-column connection for the application to

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middle-and high-rise buildings is being conducted. At this time, connecting metal with screws or nails is mainly used as connecting components forming the timber beam-to-column connection. In order to secure the stiffness of a beam-to-column connection stronger than the metal of thin steel plates, a glued-in rod connection is applied, or a connection form using thicker steel plates and high-strength bolts are also being studied recently.

It is important to evaluate the performance of the wooden beam-to-column connection for the mid-to-high rise wooden building, but it is also essential to review the structural performance of timer frame system.

The structural design of wooden buildings is carried out according to the performance of the timber shear walls or braces reinforced with timber frames under lateral loads. In addition, the lateral force resistance performance is determined according to the stiffness of the connection between the timber shear wall and the timber frame.

This paper presents the results of the seismic performance of GLT frames infilled with timber shear walls. At this time, CLT and SIP were used for the timber shear walls infilled with the GLT frames, respectively. The structural insulated panel (SIP) that is usually constructed using two sheets with rigid foam insulation is mainly used as a plane member. The study is aimed to investigate the lateral performance of the structural system and the collaborative working mechanism of GLT frames and timber shear walls.

2 – EXPERIMENTAL PROGRAM

2.1 EXPERIMENTAL PLAN

In order to evaluate the seismic performance of wooden frames subjected to cyclic loads, a total of two specimens were manufactured. One specimen installed a CLT wall in the GLT frame, and another specimen installed a SIP panel in the GLT frame.

The width and height of the wooden frames were set to be 4m and 2.55m based on the centreline, and glued-laminated timber was used for all columns and beams. The cross-sectional size of the column and beam are 170x170 and 170x300mm, respectively, and the thickness of the solid-sawn lumber was about 30mm. The glued-laminated timbers were manufactured using larch produced in Korea, and the flexural strength and modulus of elasticity of GLT members were 30MPa and 10,000MPa, respectively.

The size of the cross-laminated timber and the structural insulated panel reinforced in the wooden frame was set to 1,000x2,400mm, and the thickness of the CLT shear wall was about 90mm, and that of SIP was set as much as the width of the beam. The cross-laminated timber was made of five orthogonally bonded layers of solid-sawn lumber using the same material as GLT, the flexural strength and modulus of elasticity were 28.6 MPa and 9,550MPa, respectively. The structural insulated panel (SIP) used in this study was fabricated with 12mm oriented strand boards (OSB) and 2x6 lumbers without insulation.

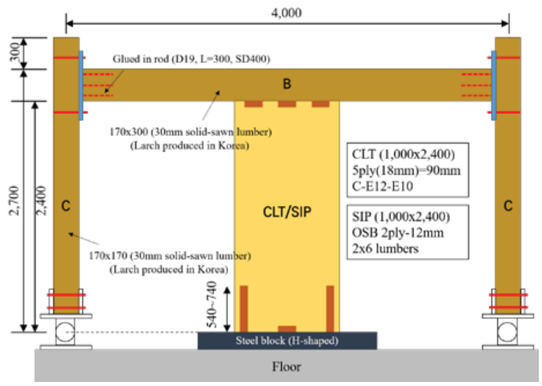

Figure 1: Concept of test specimens

2.2 CONNECTION DESIGN

A glued-in rod connection was applied to the beam-to-column connection when constructing the glued-laminated timber frames. The components used for the glued-in rod joint are liquid adhesive, 19mm rebar (yield strength of 400MPa or more), 15mm steel plate, and M20 high-tensile bolts.

The glued-in rod connection in this study is in the form of connecting a beam and a column based on a 15mm steel plate. First, six 19mm re-bars were welded to the steel plate, holes were drilled in the cross-section of the beam in advance to embed the re-bars, and then attached using liquid adhesive. At this time, the embedding depth of the re-bar is 300mm, and the size of the hole drilled in the cross-section of the beam is 24mm in diameter and 320mm in depth. At this time, the size and embedment depth of the re-bars was designed considering the yield strength of the re-bar and the adhesive strength of the liquid adhesive.

For the connection between the column and the steel plate, four M20-size high-tension bolts were used, and a 22 mm diameter hole for the high-tensile bolt was drilled on the outer part of the steel plate. At this time, the size and thickness of the steel plate were designed to resist the compressive force (or tensile force) caused by the high-

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tensile bolts (T_1) and re-bars (T_2). Figure 2 shows the details of the glued-in rod connection.

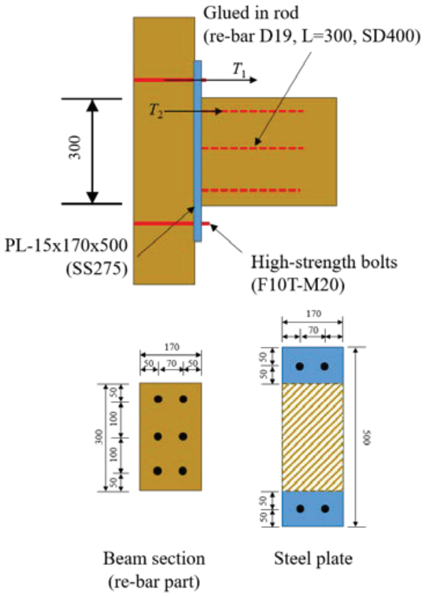

Figure 2: Details of glued-in rod connection

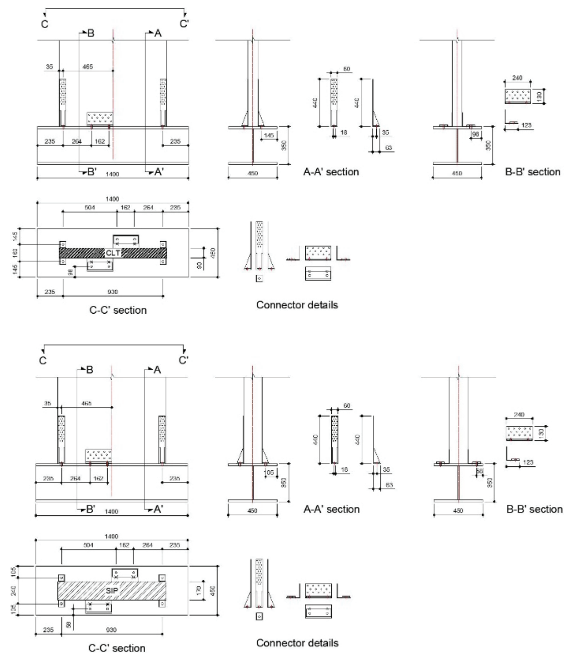

Figure 3: Details of lower connections (Up:CLT, Down:SIP)

An H-beam block was fabricated to fix the CLT and SIP to the floor. The CLT and SIP were fixed to the H-beam block using hold-downs and angle brackets. For the hold-down, Rothoblaas' WHT440 product was used, and for the angle bracket, TITAN S-TCS, a product of the same company, was used.

The angle bracket was attached to the CLT (or SIP) panel using countersunk screws with a diameter of 8mm and a length of 80mm. Due to the length of the screws, the angle brackets were staggered relative to the centerline of the panel. In the case of hold-down, WHT440 products with a thickness of 3 mm and a height of 440 mm were installed at each corner of the CLT (or SIP) panel. In the case of the lower joint, the same joint details were applied to the CLT panel and the SIP panel for complete fixation with the H-beam block.

For the connection between the lower part of the GLT beam and the upper part of the CLT or SIP panel, connecting metal with a thin steel plate thickness was used. Since the thickness of the CLT panel and the SIP panel are 90mm and 170mm, respectively, different connecting metal was applied.

In the case of the CLT panel, because of the difference in beam width, it was joined using an angle bracket, and the angle brackets used were 10 WHO4060 models from Rothoblaas and connected using round-head screws with cylindrical under head.

In the case of the SIP panel, a flat perforated plate was used because it was the same as the width of the beam, and the LBV100200 model with a thickness of 2 mm was used.

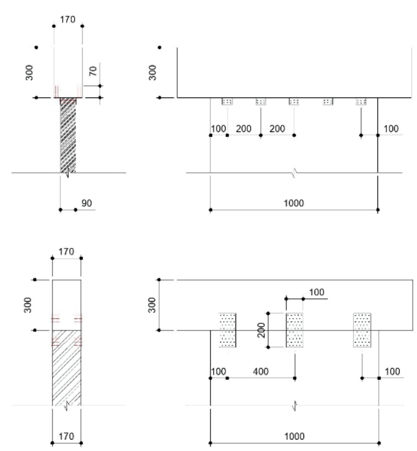

Figure 4: Details of upper connections (Up:CLT, Down:SIP)

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2.3 MATERIALS

The larch produced in Korea was used as the glued-laminated timber for the structural performance evaluation of the cyclic behavior of the wooden frame. For the strength grade of GLT, the 10S-30B glued-laminated timber defined in the Korean Industrial Standard was used, and the material strength was shown in Table 1.

The CLT shear wall was also made of larch produced in Korea, and the CLT panel with a strong-axis allowable flexural stress ($f_{b,CLT}$) of 11MPa, modulus of elasticity (E_{CLT}) of 9,500MPa, allowable shear elastic modulus (G_{CLT})of 91MPa, and allowable shear stress ($f_{s,CLT}$) of 0.23MPa was used in this study.

The mechanical properties of the 2x6 lumber constituting the shear wall of the SIP are the same as those in Table 1, and the structural plywood with 12mm used material with a flexural strength of 22MPa and a flexural modulus of 5.5GPa.

Table 1: Mechanical properties of GLT

Grade	Allowable stress, MPa						
	F_{xx}	E_{xx}	F_{yy}	E_{yy}	F_t	F_c	E
10S-30B	10	9,000	7	8,000	6.5	7.5	8,000

Note : F_{xx} is flexural allowable stress for axis X-X.
E_{xx} is flexural elastic modulus for axis X-X.
F_{yy} is flexural allowable stress for axis Y-Y.
E_{yy} is flexural elastic modulus for axis Y-Y.
F_t is allowable tensile stress in the grain.
F_c is allowable compressive stress in the grain.
E is the elastic modulus.

2.4 TEST SETUP AND LOADING PLAN

An actuator with a capacity of 2,000kN was used to evaluate the cyclic behaviour of a glued-laminated timber frame infilled with a timber shear wall. A hinge was installed on the lower part of the GLT column to realize pin connection, and a guide frame was installed to prevent buckling of the frame in the out-of-plane direction. In addition, strain gauges were attached to observe the deformation of the connecting steel plate and the glued-in rod connection.

Figure 5: Test setup (Left:TF+CLT, Right:TF+SIP)

In order to evaluate the cyclic behaviour of the timber frame, three cycles of loading were repeated for each story drift ratio, and the loading protocol is summarized in Table 2.

Table 2: Loading protocol

Step	Story drift ratio (%)	Target disp. (mm)
1	0.25	6.4
2	0.50	12.8
3	0.75	19.1
4	1.00	25.5
5	1.50	38.3
6	2.00	51.0
7	2.50	63.8
8	3.00	76.5
9	4.00	102.0

3 – BACKGROUND

3.1 FAILURE MODE

In the TF-CLT specimen, deformation occurred in the out-of-plane direction at the CLT panel's lower left and right hold-downs from step 1, and the deformation gradually increased up to step 3. In Step 5, fine cracks occurred at the connection of the lower part of the CLT panel. After that, from the 6th step, the upper left and right connecting metal (WHO40-60) broke, and buckling occurred at the lower left and right hold-downs, and it was broken at the 7th step. In Step 8, a gap occurred between the connecting steel plate and the column at the left beam-column connection, and in Step 9, a crack occurred at the top of the right column, and at the same time, a crack occurred at the beam-column connection, reaching the ultimate state.

(a) 5 step (b) 6 step

(c) 7 step (d) final

Figure 6: TF-CLT specimen' failure mode

In the TF-SIP specimen, damage occurred due to the compressive behaviour of the negative force after the tensile behaviour of the negative strength in the hold-down of the lower left side of the panel in the 3rd step. After that, in 5 steps, bending cracks occurred in the OSB panel around the left and right connecting metal (LBV100-200) on the upper part of the OSB panel, and in the same step, bending cracks also occurred in the lower part of the OSB panel. From step 6, the damaged part of the OSB panel gradually progressed from the outside of the panel to the center of the panel, and at step 9, the bottom crack was connected and reached the ultimate state.

(a) 5 step (b) 6 step

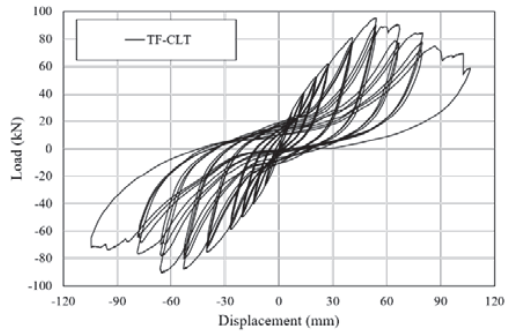
(c) final (upper) (d) final (bottom)

Figure 7: TF-SIP specimen' failure mode

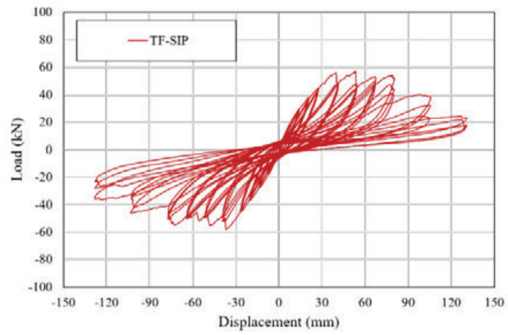
3.2 LATERAL LOAD CAPACITY

The lateral response of the timber frame with infilled timer shear wall system was analysed based on the load-displacement curves shown in Figure 8. In addition, the cyclic load test results obtained through the load-displacement curve are summarized in Table 3.

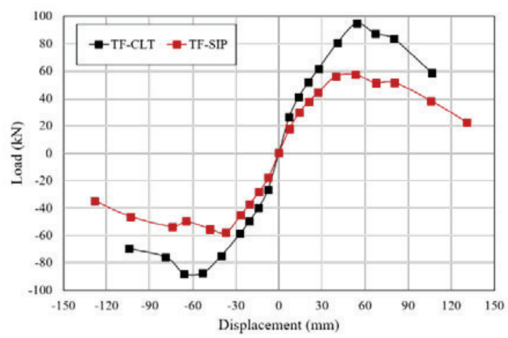
In the case of the specimen with the CLT shear wall inserted into the GLT frame, the maximum strength was 95.09 kN at about 2% of the story drift ratio, and the yield strength was 76.07kN at about 0.8% of the story drift ratio. In the case of the specimen with a GLT timber frame infilled with a SIP panel, the maximum strength was 57.08 kN at about 2% of the story drift ratio, and the yield strength was 46.66 kN at about 0.8% of the story drift ratio. Overall, the TF-CLT specimen showed about 60% better structural performance than the TF-SIP specimen. In addition, when comparing the initial stiffness and ductility ratio, the performance of the GLT timber frame system infilled with CLT was excellent.



(a) TF-CLT specimens



(b) TF-SIP specimen



(c) Envelop curve

Figure 8: Load-displacement curves

Table 3: Test results

	TF-CLT	TF-SIP
Initial stiffness	3.57 kN/mm	2.42 kN/mm
$+P_y$	76.07 kN	46.66 kN
$+\Delta_y$	22.12 mm	21.65 mm
$-P_y$	-72.38 kN	-46.51 kN
$-\Delta_y$	-21.32 mm	-22.27 mm
$+P_u$	95.09 kN	57.08 kN
$+\Delta_u$	53.77 mm	53.50 mm
$-P_u$	-90.48 kN	-58.14 kN
$-\Delta_u$	-64.89 mm	-36.92 mm
μ (=Δ_u/Δ_y)	2.43 / 3.04 (aveg. 2.74)	2.47 / 1.66 (aveg. 2.07)

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3.3 DEFORMATION ANALYSIS

In order to measure the strain generated at the glued-in rod joint and the connecting metal, strain gauges were attached, as shown in Figure 9, and the result is shown in Figure 10.

The steel plate of the glued-in rod connection showed strain within the elastic range for both TF-CLT and TF-SIP specimens. The strain distribution of the SIP panel's upper and lower connecting metal showed a response within the elastic range. On the other hand, the strain distribution of the upper and lower connecting metal of the CLT panel of the TF-CLT specimen was terminated due to fracture in some of the upper connecting metal (WHO4060), and the lower outer hold-downs reached yield from the story drift ratio of 0.25%. For a story drift ratio of 2.5%, it was finally destroyed. On the other hand, in the case of the connecting metal at the lower center of the CLT panel, the response was within the elastic range in all sections.

As a result of strain analysis of each connection metal, in the case of the TF-SIP specimen, the entire connection metal exhibited deformation within the elastic range, which was consistent with the experimental results showing the ultimate state of panel failure after the propagation of a bending crack in the SIP panel. In the case of the TF-CLT specimen, the ultimate state was determined by the damage to the connecting metal due to the rocking behavior, not the damage to the CLT panel, which can be confirmed through the partial fracture of the upper metal and the strain of the lower outer metal.

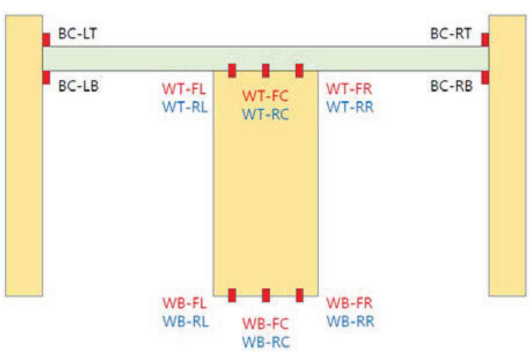
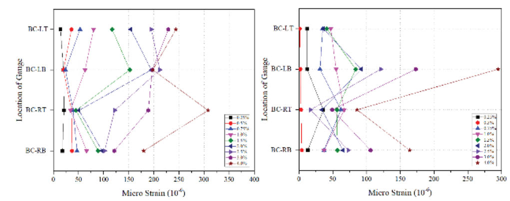
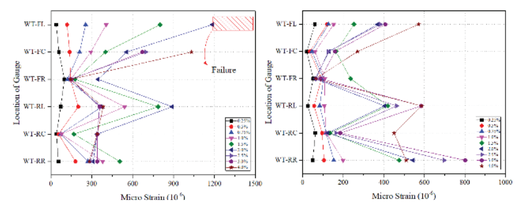


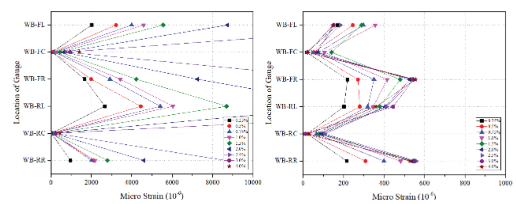
Figure 9: Strain gauge attachment location diagram



(a) Glued-in rod connection (Left:CLT, Right:SIP)



(b) Upper connection (Left:CLT, Right:SIP)



(c) Bottom connection (Left:CLT, Right:SIP)

Figure 10: Deformation distribution

3.4 ENERGY DISSIPATION CAPACITY

Under cyclic loading, the accumulated energy dissipation capacity of two specimen was shown in Figure 11.

Both specimens were loaded up to 4% of the story drift ratio, but in the case of the TF-SIP specimen, up to 5% of the story drift ratio was applied to examine the ductility capacity. When compared with the accumulated energy dissipation capacity up to 4% of the story drift ratio, the energy dissipation capacity of the specimens with CLT shear walls installed on the GLT frame was measured to be about twice as high as that of the GLT timber frame reinforced with a SIP panel.

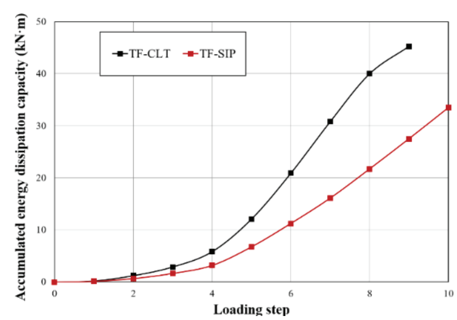


Figure 11: Accumulated energy dissipation capacity

3.5 LOAD DISTRIBUTION ANALYSIS

It can be seen that the strength of the TF-CLT specimen is due to the moment contributed by the glued-in rod joint and the CLT panel's upper and lower connecting metals. The contribution of each component is shown in Table 4, and it is very similar to the maximum load in Table 3.

Also, it can be seen that the load-bearing contribution of the TF-SIP specimen is due to the moment contributed by the beam-column connection and the moment contributed by the SIP panel, and the contribution by each component is shown in Table 5.

Table 4: Contribution of each component for TF-CLT

Glued-in rod	Bottom connection	Upper connection	*V* (kN)
$_jM_u$ (kN·m)	ΣM_{bj} (kN·m)	ΣM_{tj} (kN·m)	
25.5	121.7	61	96.1

$$_jM_u = f_b I / y = 25.5kN \cdot m$$
$$M_j = L \times F_j$$
$$V_u = 2_j M_u / h_0 + \left(\sum M_{bj} + \sum M_{tj}\right) / h_1 = 96.1kN$$

Table 5: Contribution of each component for TF-SIP

Glued-in rod	SIP panel	*V* (kN)
$_jM_u$ (kN·m)	$_wM_u$ (kN·m)	
25.5	40.0	53.3

$$_jM_u = f_b I / y = 25.5kN \cdot m$$
$$_wM_u = f_b I / y = 40.0kN \cdot m$$
$$V_u = 2_j M_u / h_0 + 2_w M_u / h_1 = 53.3kN$$

4 – CONCLUSION

In this study, the lateral force resistance performance of a structural system with a timber shear wall installed on a GLT wooden frame was evaluated. Although the structural performance of the GLT frame system infilled with the CLT shear wall was much better, the load capacity was determined by the connecting metal, and a design method that can induce CLT member fracture by increasing the stiffness of the joint is needed. In the case of the specimen reinforced with the SIP panel, although the lateral resistance performance was low, the overall behaviour showed a stable shape.

In the future, it is expected that the stiffness of the joint connecting the beam and CLT will need to be increased to design the joint so that the CLT member can transmit lateral force.

5 – ACKNOWLEDGEMENT

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