Lateral Loading Behavior of the Poplar LVL Light Wood Shear Wall

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Poplar laminated veneer lumber (poplar LVL) is made of fast-growing poplar veneer and structural adhesive, which can well meet the developing requirement of the modern wood structures. This paper mainly focuses on the lateral loading behavior of the poplar LVL shear wall. For this purpose, six shear wall specimens with different opening types were fabricated and tested under the action of monotonic and cyclic loading. Performances were analyzed on the failure pattern, the load-displacement curve, the shear strength, the ultimate displacement, the elastic lateral stiffness, and the energy dissipation. To strengthen the corner joint, an innovative custom-designed hold-down was adopted, and the mechanical performance was also considered. The results showed that the failure of the specimen was mainly due to the yield of the nails and the separation between the stud and the base plate, while the hold-down can greatly improve the shear strength, the ultimate displacement, and the energy dissipation performance of the poplar LVL shear wall without openings. At last, the evaluation formula of the bearing capacity for the light wood shear wall is proposed so as to promote the theoretical basis for the application of poplar LVL in the light wood frame construction.

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INTRODUCTION

Wood buildings have the advantages of a short construction period, superb seismic performance, and habitation comfort, yet the increasingly strict protection requirements for the natural forest resources largely restrict the development of wood structures in China. In the mid-1970s, Siyang county in northern Jiangsu successfully introduced the fast-growing Italian poplar (the hybridization of *Populus deltoides* and *Populus nigra*) and planted it extensively. This kind of tree grows very fast and takes only 7 to 10 years to reach maturity, yet it is generally used as packing material rather than in construction. As a kind of sustainable modern engineering wood product, the poplar laminated veneer lumber (LVL) is made of the fast-growing poplar log by rotary peeling, drying, gumming, veneer parallel lay-up and hot pressing, which owns the characteristics such as high toughness, durability, accurate specification, and easy processing. Due to the sustainability and availability by mass industrial production, the application of the poplar LVL in light wood structures will greatly promote the development of wood building in China.

In light wood structures, shear wall is an important component for resisting lateral load, which is generally composed by the top plate, the base plate, the studs, and the panel, and these are connected with each other by the nails. For the light wood shear wall, the lateral resistance performance is a key problem in research. To investigate the influence of construction details on the lateral performance, Bagheri and Doudak (2020) completed 26 full-scale model tests. The results showed that the strength and stiffness of the shear wall were directly related to the reciprocal of height to width ratio of the wall. There was little effect on the overall bearing capacity of the shear wall by increasing the number of end bolts or changing the size of the bolt, while the diameter and spacing of the nail could significantly affect the strength of the wall. Guo *et al.* (2020) introduced the Anchor Tie-down System (ATS) into wood shear wall and conducted four medium-thickness wood shear wall specimen tests under the action of cyclic load. It was shown that the installation of ATS increased the lateral bearing capacity, the energy dissipation performance, and the lateral stiffness by 154%, 427%, and 93% respectively, and the application of ATS could effectively avoid the pull-out of the wall nails.

Shadravan *et al.* (2019) studied the effect of a reinforcement belt on the lateral resistant performance by 15 groups of different types of wood shear wall without openings. It was found that the reinforcement belt could greatly improve the lateral bearing capacity of the wall, in which the most significant improvement was offered by the double base plate reinforcement. Besides, a test by Shadravan and Ramseyer (2018) also showed that, for the shear wall constructed by log and oriented strand board (OSB), the lateral performance could be improved by changing the wall length, the connection type, and the quantity and spacing of the nails and anchor bolts. Wang *et al.* (2017) carried out a transverse load test on the light wood shear wall with three types of panels to frame nail connection and two types of panels with different thicknesses. The results showed that increasing the diameter of the nails could significantly increase the bearing capacity of the shear wall, and when the failure mode of the connection between the panel and the frame transferred from edge failure to nail yield, the bearing and deformation capacity could be improved by increasing the panel thickness.

To investigate the effect of double shear nail (DSN), 8 groups of medium-thick wood shear wall were tested by Zheng *et al.* (2015) under the action of monotonic load, and the effects of panel thickness, nail edge distance, and load direction were evaluated. The results showed that the failure pattern of the DSN connection depended mainly on the panel thickness and the nail edge distance, and increasing of the two factors could significantly improve the ultimate strength and ductility, yet had little effect on the initial stiffness. Cassidy *et al.* (2006) compared the shear walls constructed by the ordinary OSB panel and the fiber-reinforced polymer (FRP) reinforced OSB panel and pointed out that the reinforced OSB panel had better energy consumption and bearing capacity. A full-scale light wood house model was tested by Kang *et al.* (2010). The testing results of the single wall were quite different from those of the whole structure. The house as a whole could not only effectively restrict the uplift of the studs, but also it could improve the ultimate bearing capacity of the wall.

He and Zhou (2011) tested 10 pieces of square wood frame shear wall with different thicknesses, which were made of domestic OSB board. It was verified that the shear wall with domestic OSB board achieved the same mechanical properties as the shear wall with imported OSB board. Du *et al.* (2012) investigated the influence of various stud connection patterns on the shear wall mechanical behavior. It was found that the tenon joint could significantly improve the stiffness and deformation controlling performance of the wall. Zheng *et al.* (2014) compared the lateral load resistant behavior of the glulam frame, the wood shear wall, and the glulam frame-shear wall. It was shown that the elastic lateral stiffness of the glulam frame-shear wall could be regarded as the sum of the glulam frame and the wood shear wall, yet its ultimate bearing capacity was much larger than the sum of the latter two. By adding unbonded prestressed steel strands into cross laminated timber (CLT) shear wall, Sun *et al.* (2020) were able to greatly promote the lateral load resistance capacity and the wall specimen was almost intact after loading.

Due to the advantages of the poplar LVL, it would be meaningful to apply this kind of material to the shear wall frame. Yet according to GB50005-2017 (2017), the material of the shear wall frame in light wood construction is defined as dimension lumber, hence the lateral performance of the poplar LVL light wood shear wall needs to be studied. To investigate this problem, on the basis of the previous material experiments, monotonic and cyclic loading tests were carried out on three types of shear wall specimens with different opening forms. The goal of this work was to study the failure pattern, the load-displacement curve, the shear strength, the ultimate displacement, the elastic lateral stiffness, and the energy dissipation, in order to help the application of poplar LVL in wood structures. In addition, considering the fact that the most serious pull-up generally occurred at the edge studs in previous studies, a custom-designed hold-down was adopted in the test to strengthen the corner joint, whose role in lateral performance was compared and discussed.

EXPERIMENTAL

Specimen Design

The design of the poplar LVL shear wall was in accordance with GB50005-2017 (2017) and GB/T50361-2018 (2018). There were 3 types of walls in total, and each type included 2 specimens for different loading patterns, as shown in Table 1. All of the wall frames were made of poplar LVL with a cross section of 40 mm × 90 mm, the spacing of the wall studs were 400 mm, the size of lintel above the openings for Wall-B and Wall-C was 150 mm × 90 mm, and the wall was entirely sheathed with 1.22 m × 2.44 m × 9.5mm thick domestic OSB/2 grade (in accordance with LY/T1580-2010 (2010)) panel, which were laid out vertically.

Table 1. Introduction to the Specimens

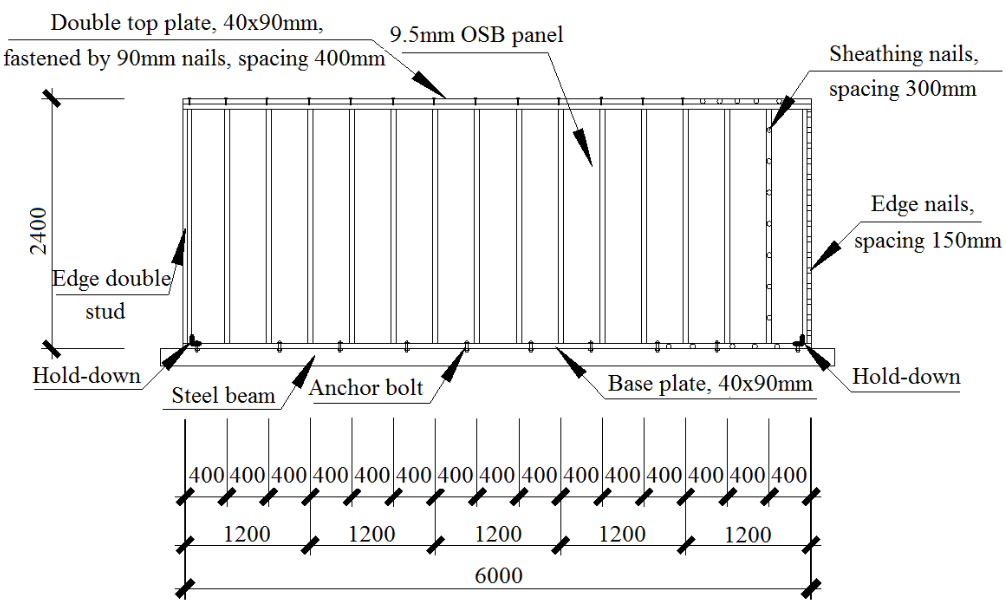
Specimen Type	Loading Pattern	Opening
Wall-A	Monotonic	None
	Cyclic	None
Wall-B	Monotonic	One 1.2 m × 2.1 m portal
	Cyclic	One 1.2 m × 2.1 m portal
Wall-C	Monotonic	Two 1.2 m × 1.2 m window openings
	Cyclic	Two 1.2 m × 1.2 m window openings

The basic physical and mechanical properties of the poplar LVL are shown in Table 2 (Ding 2018). All of the wall frames and the wall panels were fastened with each other by nails, where the nails connecting the studs and the top or base plates were of the type P3.70×90LXL (in accordance with GB27704-2011(2012)), and the nails connecting the wall panel and the wall frame were of the type P2.80×60LXL (in accordance with GB27704-2011(2012)). The scheme of the three specimens is shown in Fig. 1, and all had custom-designed hold-downs at the wall corner, which were made of Q235 grade steel with 5 mm thickness. The details are illustrated in Fig. 2.

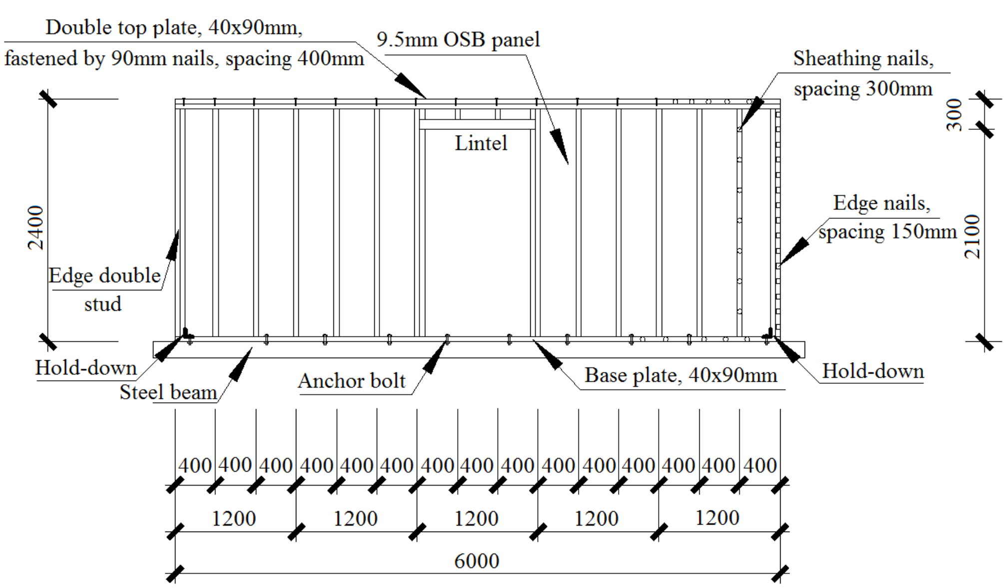
Table 2. Physical and Mechanical Parameters of the Poplar LVL

Moisture Content (%)	12.8
Density (g/cm^3)	0.576
Tensile Strength Parallel to Grain (MPa)	39.4
Compression Strength Parallel to Grain (MPa)	37.03
Compression Strength Perpendicular to Grain (MPa)	6.3
Bending Strength* (MPa)	
-Adhesive Layer Horizontal	61.56
-Adhesive Layer Vertical	64.8
Flexural Elastic Modulus (MPa)	
-Adhesive Layer Horizontal	9877.3
-Adhesive Layer Vertical	10135.4

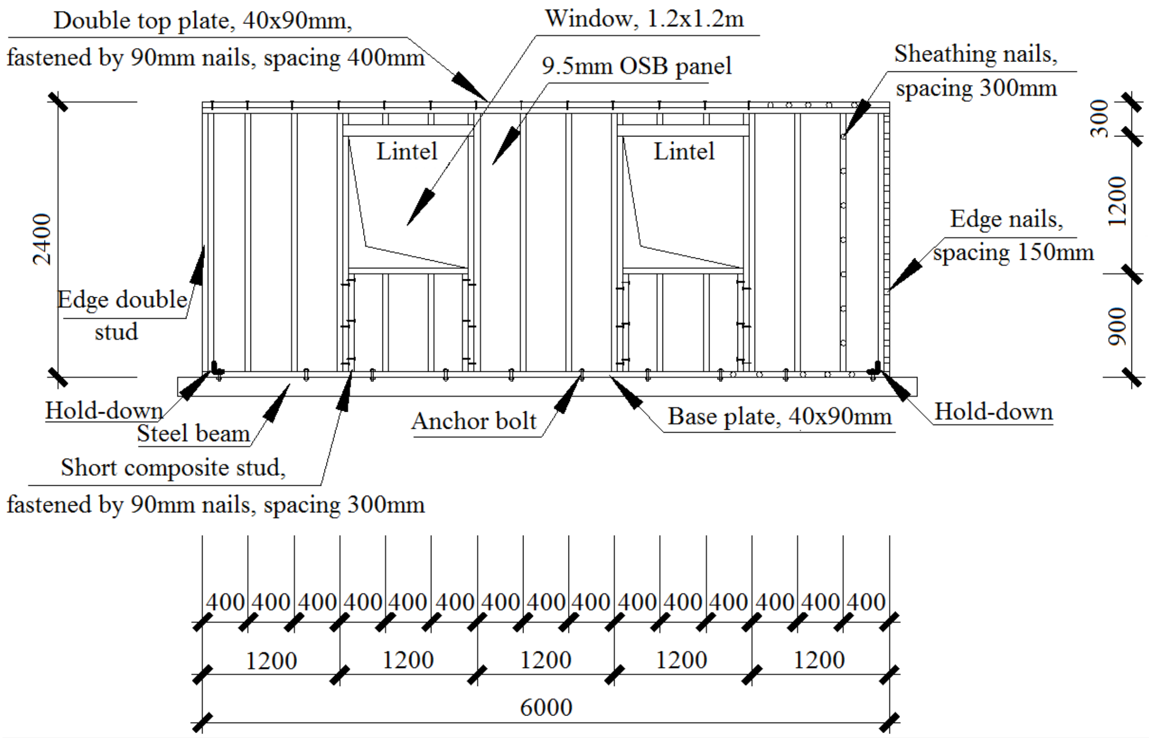
* In accordance with GB/T 50329-2012 (2012)



(a) Wall-A



(b) Wall-B



(c) Wall-C

Fig. 1. Scheme of the poplar LVL shear wall specimens (unit: mm). The OSB panels were laid out vertically.

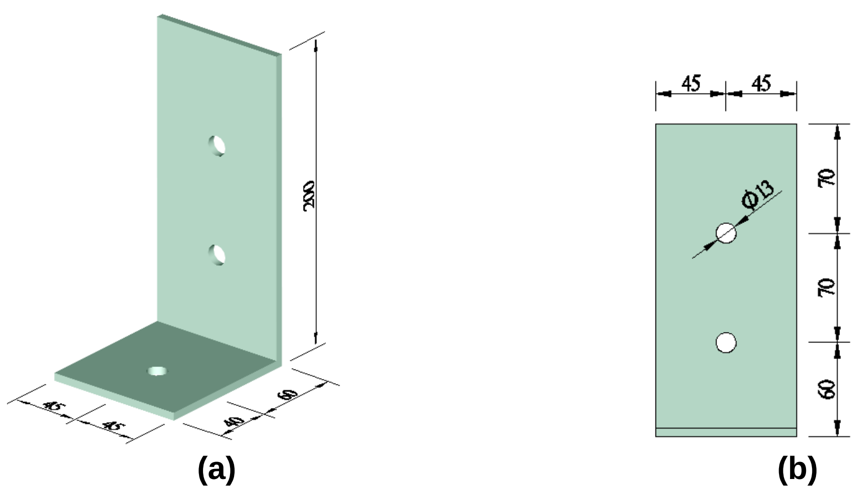
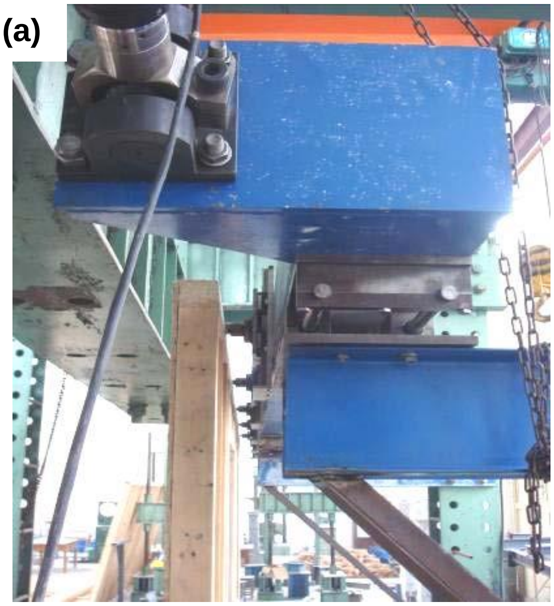


Fig. 2. The details of the custom-designed hold-down (unit: mm). (a) Perspective; (b) side view

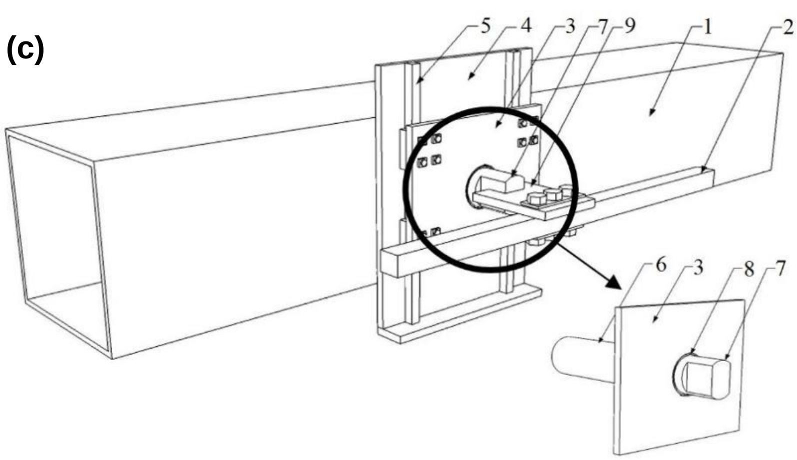
Test Setup and Measuring Points

The shear wall was loaded with a self-designed cantilever load transfer device, as shown in Fig. 3. The load of the FTS hydraulic servo system was transferred to the wall only at 5 points, where the cantilever load transfer device was connected with the top plate, while the two vertical slide rails and the bearing of the device could ensure that the deformation of the top plate was not restricted during loading, so as to truly reflect the bearing capacity, the energy consumption, and the deformation performance of the wood shear wall (Liu *et al*. 2008; Guo 2010).



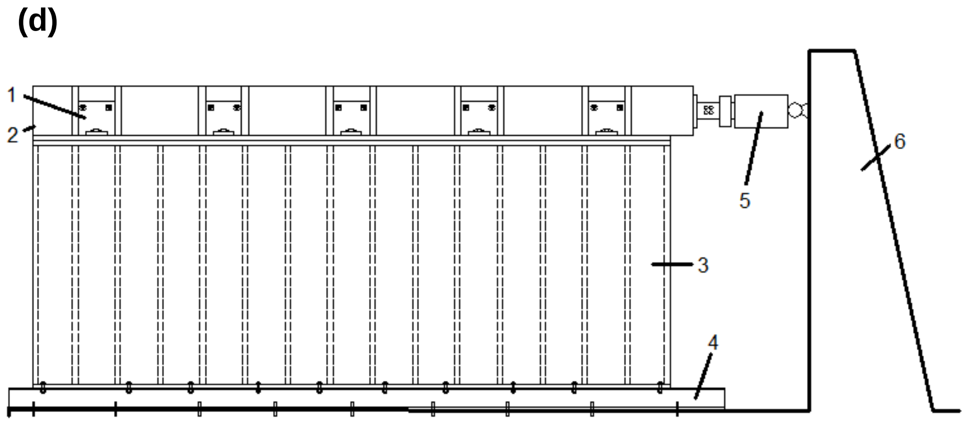
(a) Loading device

(b) Cantilever load transfer device



1. Square steel tube; 2. Top plate; 3. Inner steel plate; 4. Outer steel plate; 5. Slide rail; 6. Steel pipe; 7. Steel rod; 8. Bearing; 9. Center steel plate

(c) Schematic diagram of the cantilever load transfer device



1. Cantilever load transfer device; 2. Load transfer beam; 3. LVL shear wall; 4. Base steel beam; 5. FTS hydraulic servo system; 6. Reaction wall

(d) Schematic diagram of the loading device

Fig. 3. The self-designed cantilever load transfer device

Figure 4 illustrates the layout of the main measuring points of Wall-A, where F1~F6 were bushing type pressure sensors, which were fixed to the base plate by bolts. These pressure sensors were used to measure the uplift force. V1~V6 were displacement meters, which were fixed at the bottom of the stud by a G clip, these sensors were used to measure the vertical displacements of the stud relative to the base plate during the failure of the specimen. Wall-B and Wall-C had a similar measuring point arrangement. Here the horizontal movement and the potential slip between the wall and the base were not measured.

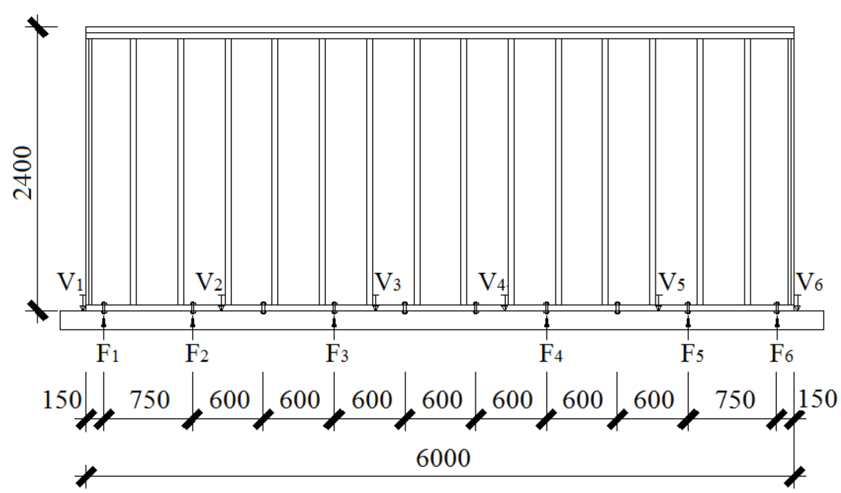


Fig. 4. Layout of the measuring points (unit: mm)

Loading Scheme

The loading of the test is in accordance with ISO-16670 (2003). According to the displacement control protocol: (1) The displacement rate of the monotonic loading was set at 7.5 mm/min, and when the load dropped to 80% of the ultimate load or when the specimen was seriously damaged, the test was terminated; (2) The cyclic loading protocol is shown in Fig. 5. This protocol used the ultimate displacement determined by the monotonic loading of the same specimen (*i.e.*, the displacement when the load dropped to 80% of the ultimate load or when the specimen was seriously damaged) as the control displacement. The displacement loading rate was set at 5 mm/s, with 1 cycle each when the peak displacement was taken as 1.25%, 2.5%, 5%, and 10% of the control displacement, and then with 3 cycles when the peak displacement was taken as 20%, 40%, 60%, 80%, 100%, and 120% of the control displacement before the cyclic test was terminated.

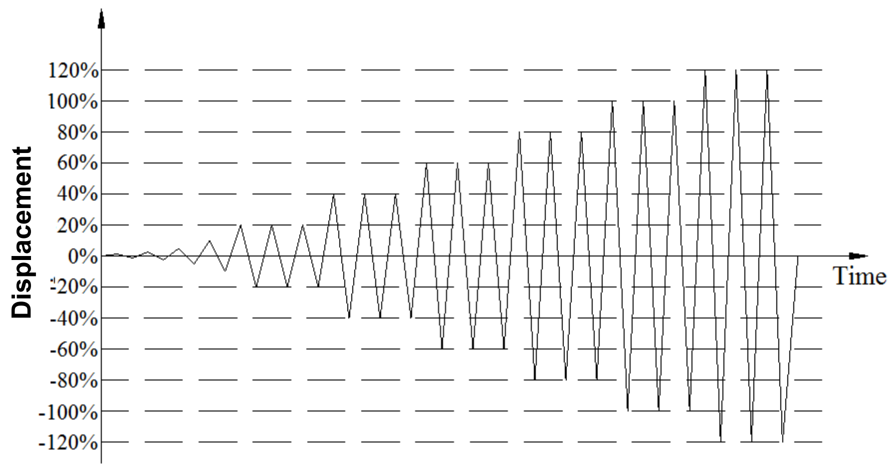


Fig. 5. Cyclic load protocol

RESULTS AND DISCUSSION

Failure Patterns

The predominant failure patterns of the poplar LVL shear wall were the panel nail damage and the separation between the studs and the base plate.

(1) Panel nail damage

In the test, there were mainly four patterns for the failure of the panel nail, as shown in Fig. 6: the nail was pulled out; the nail head penetrated the panel; the panel edge was torn; or the nail was sheared to fracture due to fatigue.

Generally speaking, the nails located at the bottom and the sides near the bottom of the panel were more likely to experience damage, while the nails located at the middle and top of the panel were seldom destroyed. For the four failure patterns, the phenomenon of nail shear fracture due to fatigue only appeared in the cyclic load test, in which the nail was cut near the head.

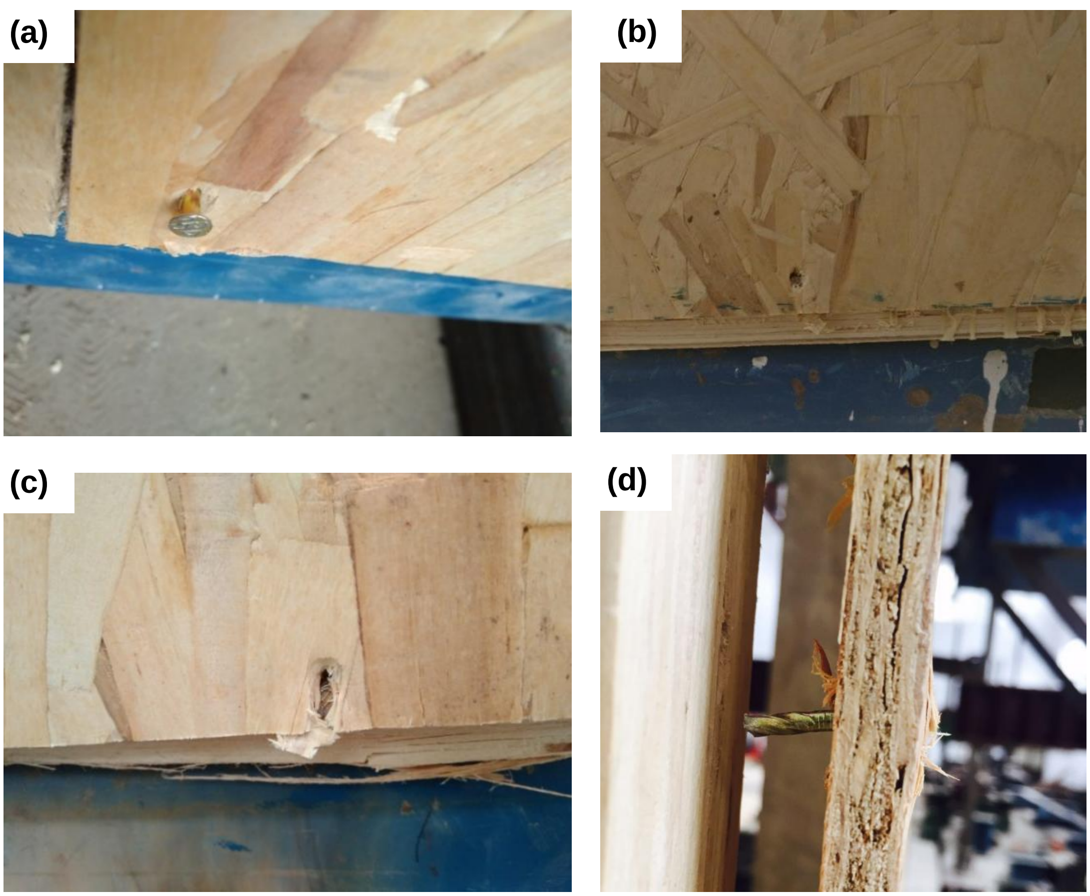
(2) Separation between the studs and the base plate

In the monotonic load test, the studs near the loading end of Wall-A were pulled up, while the studs far from the loading end were compressed. Wall-B and Wall-C exhibited similar experimental phenomena.

Under the action of cyclic load, the maximum pull-out distance at the measuring points of the three specimens are shown in Fig. 7. The figure illustrates that, due to the yield of the hold-down, the end studs of the Wall-A specimen had obvious uplift (Fig. 8a), yet the middle studs were scarcely pulled out. For the Wall-B specimen, the portal jamb studs experienced serious pull-up (Fig. 8b), then the adjacent studs also appeared with uplift, while the pull-out distance of the end studs was very small. For the Wall-C specimen, the middle studs had nearly uniform uplift (Fig. 8c), and just like the Wall-B specimen, the end studs were scarcely pulled out.

In previous work of the current researching group, monotonic and cyclic loading tests were carried out on the shear wall specimens with the same size and construction (Guo 2010), while the wall frame was made of dimension lumber (Spruce-Pine-Fir, S-P-F), and there was no hold-down. The maximum pull-out distances under the action of cyclic load are shown in Fig. 9. A comparison with Fig. 7 illustrates that for the Wall-A type specimen (*i.e.*, without openings), the hold-down was able to control the uplift very well, and the pull-out distance of the studs could be greatly reduced. For the Wall-B and Wall-C type specimens, though hold-down greatly reduced the pull-out distance of the end studs, the uplift of the portal or window jamb studs was much larger than that without hold-downs.

By observing the failure phenomena, it can be found that the difference between the poplar LVL shear wall and the S-P-F dimension lumber shear wall only lies in the effect of the hold-down, while the other failure phenomena are basically consistent with each other, and there is nearly no damage to the wall frame itself. So it can be concluded that, for the light wood shear wall, the dimension lumber wall frame can be well substituted by the poplar LVL wall frame. To investigate the effect of the hold-down on the mechanical behaviors of the wood shear wall, test results will be compared with those of Guo (2010).



(a) Bottom nail was pulled out; (b) Bottom nail head penetrated the panel; (c) Bottom panel edge was torn; (d) Side nail was sheared to fracture due to fatigue

Fig. 6. Failure pattern of the nail joint

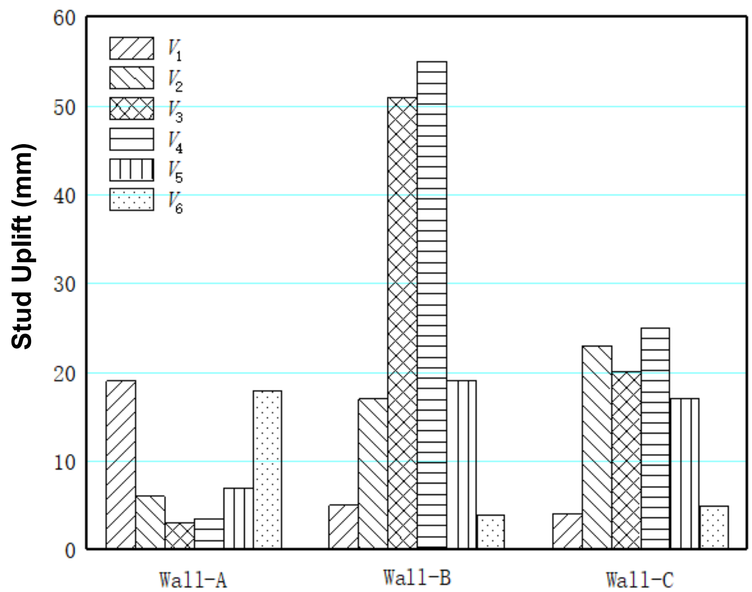


Fig. 7. The maximal separation distance between the studs and the base plate in cyclic test

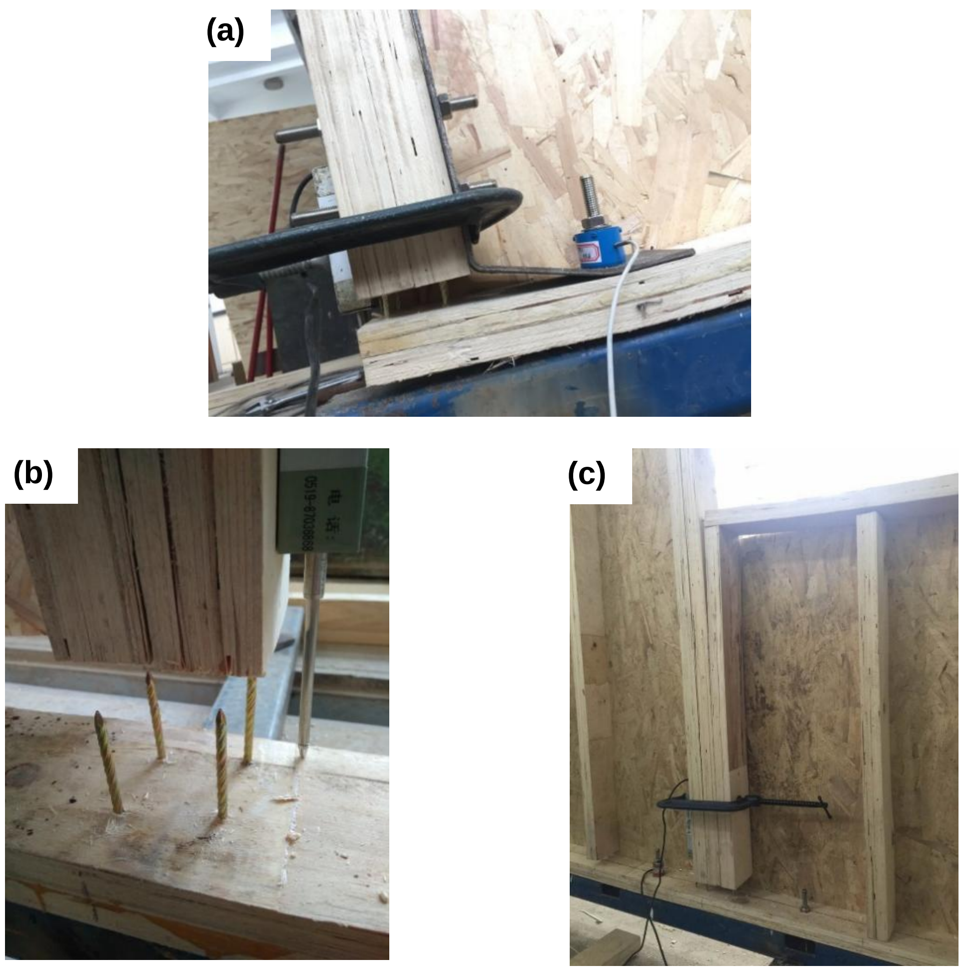


Fig. 8. Separation between the studs and the base plate (a) Wall-A; (b) Wall-B; (c) Wall-C

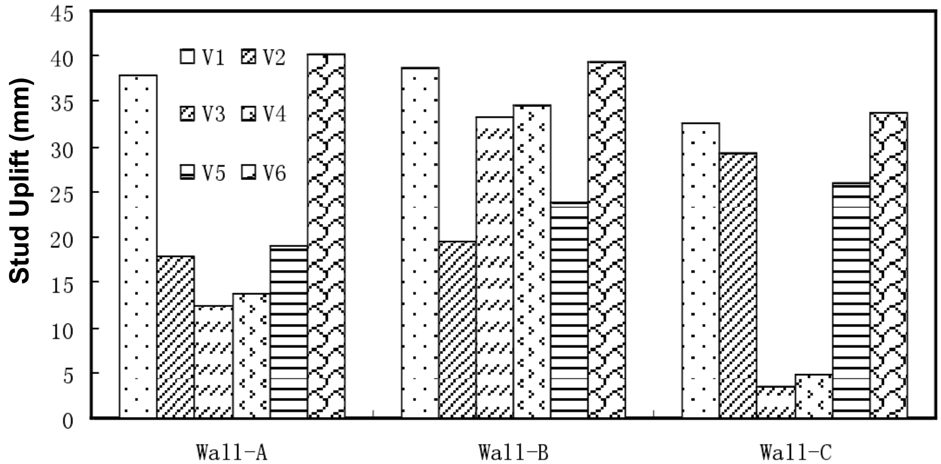


Fig. 9. The maximal separation distance between the studs and the base plate for cyclic test by Guo (2010)

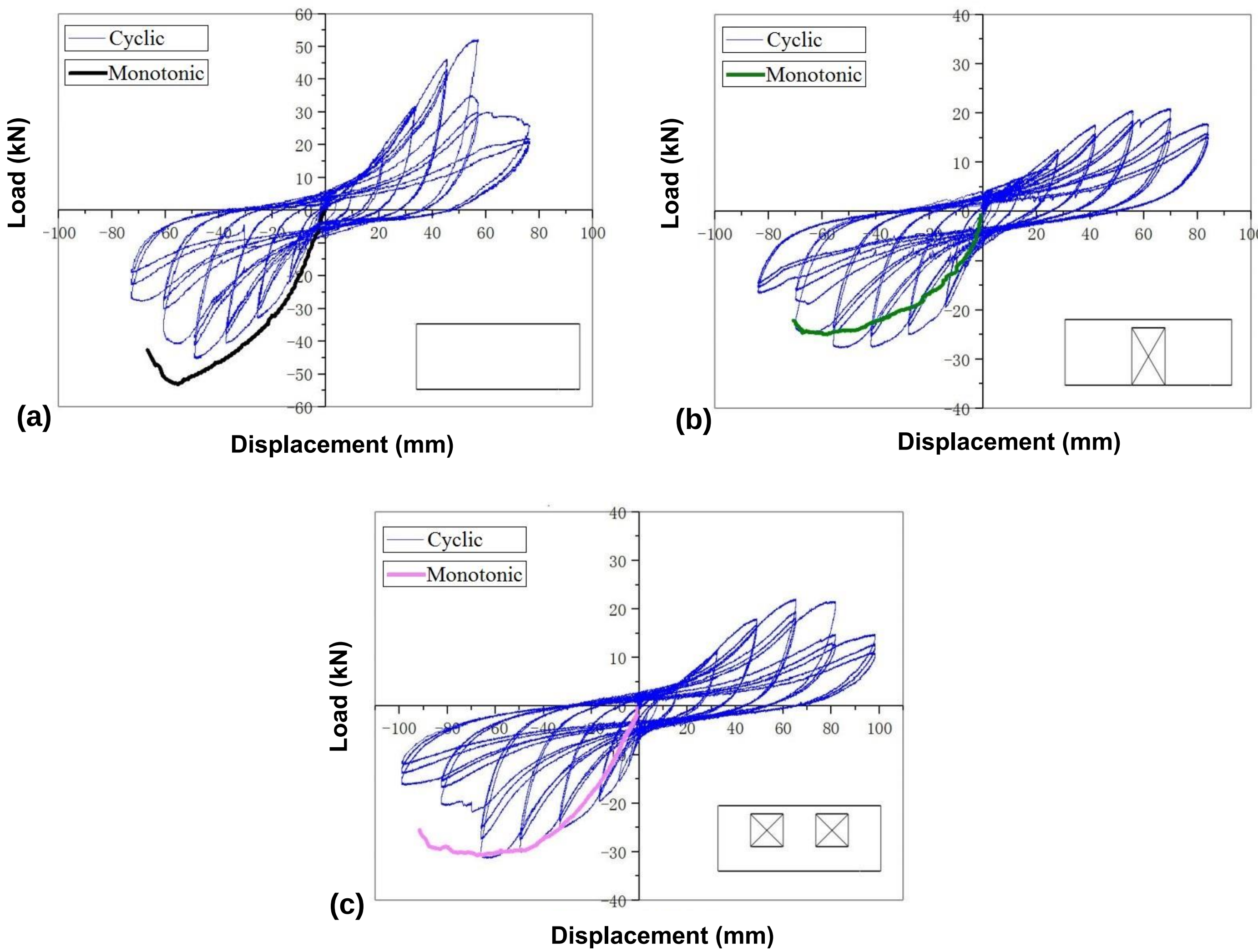


Fig. 10. The load-displacement curve: (a) wall A; (b) wall B; (c) wall C

Load-displacement Curve

The load-displacement curves of the specimens under the action of monotonic and cyclic load are shown in Fig. 10. It can be seen that, in the case of monotonic loading, the bearing capacity of Wall-A was much higher than that of Wall-B and Wall-C, which was directly related to the opening type of the shear wall. In the case of cyclic loading, Fig. 10 indicates that the shapes of the hysteretic curves for all of the shear wall specimens were similar to each other. A notable feature was that the slope of each loading curve increased with the load, while the slope of the same direction loading curve decreased cycle by cycle. After several cycles, the inflection point began to appear on the curve, which reflected the stiffness degeneration of the shear wall specimens. For the unloading phase, the initial part was approximately parallel to the vertical axis, and the deformation recovery was relatively small, yet with further decrease of the load, the curve tended to be flat, and the deformation recovery gradually accelerated. The cyclic load-displacement curve presented typical inverse S-shape with apparent rheostriction, which indicated a significant influence of slip.

Linear Shear Strength

The linear shear strength is defined as $f_{\mathrm{vd}} = F_{\mathrm{max}} / l$, in which F_{max} is the ultimate load of the monotonic loading test, and l is the effective wall limb length, which is 6.0 m, 4.8 m, and 3.6 m for Wall-A, Wall-B, and Wall-C respectively.

Table 3 lists the results of the linear shear strength of this test as well as that of Guo (2010). It can be seen from the table that the portal type opening reduced the strength of the poplar LVL shear wall by about 38%, while the strength was almost unchanged for the window opening type specimen, which indicates the effect of the portal opening on the shear wall strength was much greater than that of the window opening. Furthermore, the comparison with that of Guo (2010) shows that the hold-down could increase the bearing capacity of Wall-A type and Wall-B type shear wall by 43% and 16% respectively, while for Wall-C type shear wall it could hardly increase the bearing capacity. In fact, by observing the failure pattern, it can be found that for the linear shear strength, the wall below the window had nearly the same influence as the hold-down, which can partly explain why the hold-down had little effect on the increase of limb strength.

Table 3. The Linear Shear Strength of the Specimens

Specimen	Current Test (kN/m)	Test by Guo (2010) (kN/m)
Wall-A	9.12	6.36
Wall-B	5.63	4.84
Wall-C	8.98	8.88

Ultimate Displacement

The ultimate displacement is defined as the displacement when the load drops to 80% of the ultimate load in the monotonic loading test, which is listed in Table 4.

Table 4 illustrates that the ultimate displacement of the shear wall with openings was higher than that without openings, in which the ultimate displacement of the Wall-C specimen was even 39% higher than that of the Wall-A specimen. Here the slenderness ratio can account for the phenomenon, in fact with the decreasing of the effective wall limb length, a larger slenderness ratio tends to make the deformation of the wall more like a bending deflection, consequently leading to larger ultimate displacement, which can also be proved by the experimental results of Guo (2010).

The comparison between the current experiment and that of Guo (2010) shows that whether or not an opening exists, the hold-down could greatly improve the ultimate displacement of the wood shear wall, in which the case of window opening increased by 53%.

Table 4. The Ultimate Displacement of the Specimens

Specimen	Current Test (mm)	Test by Guo (2010) (mm)
Wall-A	66	51
Wall-B	70	57
Wall-C	92	60

Elastic Lateral Stiffness

To facilitate comparison, the calculation of elastic lateral stiffness here was kept consistent with that of Guo (2010). In other words, the slope of the line between the origin of the load-displacement curve and the displacement point of $H/250$ in the case of monotonic loading was used, where H is the height of the shear wall specimen. In addition, in order to compare the effect of the opening type on the lateral stiffness, the slope will be divided by the effective wall limb length, *i.e.*, the linear elastic lateral stiffness, which is shown in Table 5.

Table 5. The Linear Elastic Lateral Stiffness of the Specimens

Specimen	Current Test (kN/mm/m)	Test by Guo (2010) (kN/mm/m)
Wall-A	0.35	0.39
Wall-B	0.27	0.31
Wall-C	0.29	0.40

As can be seen from Table 5, the shear wall opening reduced its linear lateral stiffness, in which the decrease for the portal opening was greater than that of the window opening. The reason here could also be explained by the role of the wall below the window in resisting lateral displacement.

Compared with the experimental data of Guo (2010), it can be seen that the hold-down was not able to improve the linear elastic lateral stiffness of the shear wall. Instead, the application of hold-down greatly decreased the linear elastic lateral stiffness of the Wall-C type shear wall. There are two possible reasons for this phenomena: (1) the steel plate of the hold-down was too thin and the distance between the anchor bolt and the wall surface was too big; (2) in the case of monotonic testing, the hold-down promoted the end constraint of the shear wall, while it weakened the role of the wall below the window in lateral stiffness.

Energy Dissipation Performance

The shear wall energy dissipation is defined as the total energy absorbed by the shear wall specimen during the cyclic loading, which can be obtained from the area integral of the hysteretic curve. In addition, in order to reflect the energy dissipation behavior of different opening types of shear wall specimens, the linear energy dissipation is a better indicator, which is defined as the total energy dissipation divided by the effective wall limb length of the specimen, as shown in Table 6.

Table 6 illustrates that the linear energy dissipation capacity of the Wall-C specimen was higher than that of Wall-A and Wall-B specimens by 56.8% and 54.3% respectively. This was mainly because the hold-down made the uplift of Wall-C studs more even during the cyclic test, thus fully utilizing the energy dissipation behavior of the lintel and the wall below the window. As a contrast, for the Wall-C type specimen without hold-down by Guo (2010), the energy dissipation performance of the lintel and the wall below the window could not be fully utilized, which led to unremarkable improvement of Wall-C over Wall-A and Wall-B on the energy dissipation capacity.

Table 6. The Linear Energy Dissipation of the Specimens

Specimen	Current Test (kJ/m)	Test by Guo (2010) (kJ/m)
Wall-A	3.77	2.30
Wall-B	3.83	2.70
Wall-C	5.91	2.81

Compared with the test by Guo (2010), the linear energy dissipation of all specimens with hold-down showed obvious better performance. The reason is that in the cyclic loading test, the hold-down greatly slowed down the stiffness degeneration of the shear wall specimen and it also greatly improved the ultimate displacement, thus resulting in a much larger envelope area of the hysteretic curve.

Anchor Bolt Internal Force

Figure 11 shows the change of internal force of the anchor bolt measured by the bushing type pressure sensors for Wall-A specimen during monotonic loading.

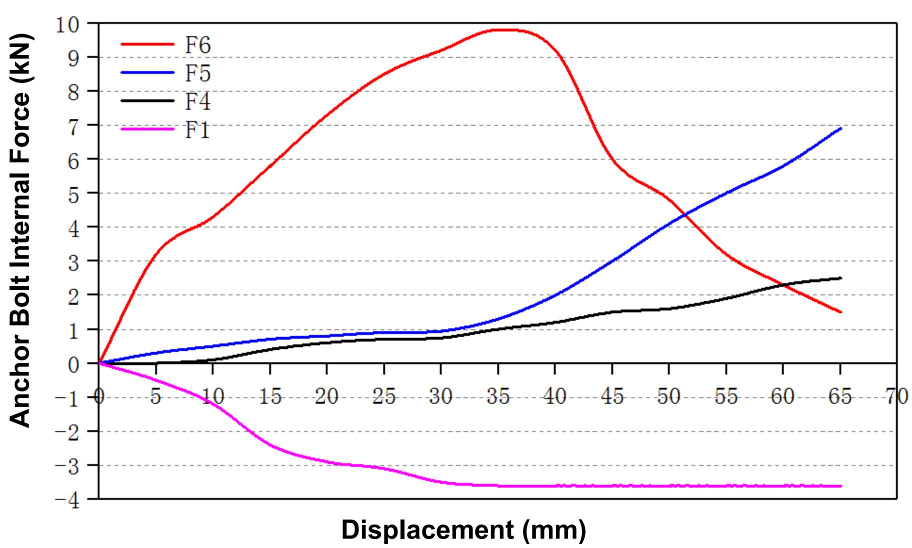


Fig. 11. The internal force of the base plate anchor bolts of the Wall-A specimen

It can be seen from Fig. 11 that, in the initial stage of loading, the internal force of F6 (which was near the loading point) increased linearly with the wall deformation. After the hold-down yielded, the studs at the wall corner began to be pulled up until the maximum bearing capacity of the nail joints was reached, and then its ability of resisting uplift decreased gradually. Accordingly, the internal force that was transferred to the anchor bolt at F6 also decreased gradually. Simultaneously, the pull-up force began to be borne by the adjacent stud nail joint, so that the internal force at F5 and F4 increased gradually. On the other hand, the compression force in F1 (which was far from the loading point) increased with the wall deformation. Thus, the prestressing force of the anchor bolt began to decrease.

Calculation of the Bearing Capacity

With reference to the design method by Ni *et al.* (1999), if the bearing capacity of the shear wall with openings is assumed to be the capacity summation of all the shear wall limbs, as shown in Fig. 12, then the ultimate lateral bearing capacity of the shear wall with hold-down can be determined by the following formula,

$$V = \sum J_{\mathrm{hd}} V_{\mathrm{d}} L_{\mathrm{s}} \tag{1}$$

where V is the ultimate lateral bearing capacity (kN); V_d is the linear shear strength of the wall limb (kN/m), which can be determined by the test results of the shear wall (no openings) with hold-down; L_s is the length of the wall limb (m); and J_{hd} is the influence coefficient for the hold-down or the wall below the window.

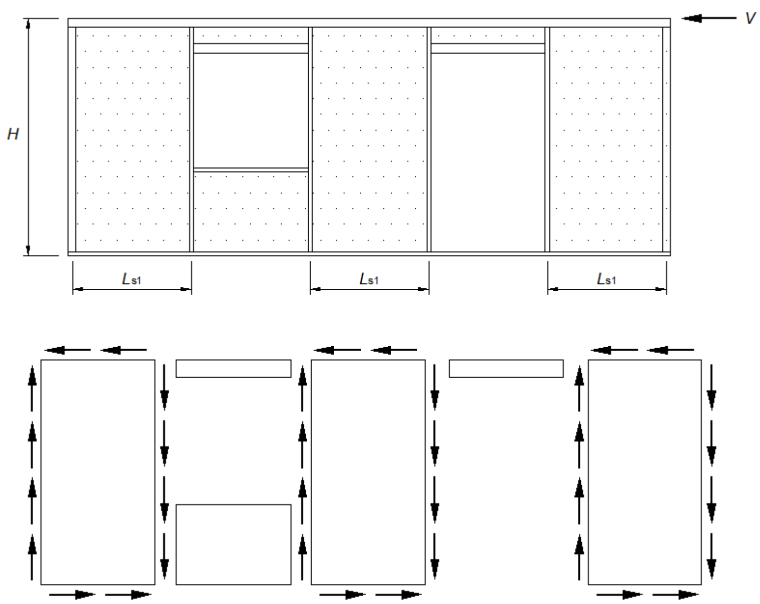


Fig. 12. The schematic diagram for calculation of the shear wall bearing capacity

For J_{hd}, when there is hold-down (or wall below the window) for both of the corners of the wall limb, J_{hd} is taken as 1.0; when there is no hold-down (or wall below the window) for both of the corners, J_{hd} can be calculated as follows

$$J_{\mathrm{hd}} = \frac{1}{1+3\frac{H}{L_{\mathrm{s}}}} \tag{2}$$

where H is the height of the wall limb (m).

Under the case when there is only one corner of the wall limb provided with hold-down (or wall below the window), it can be treated as shown in Fig. 13.

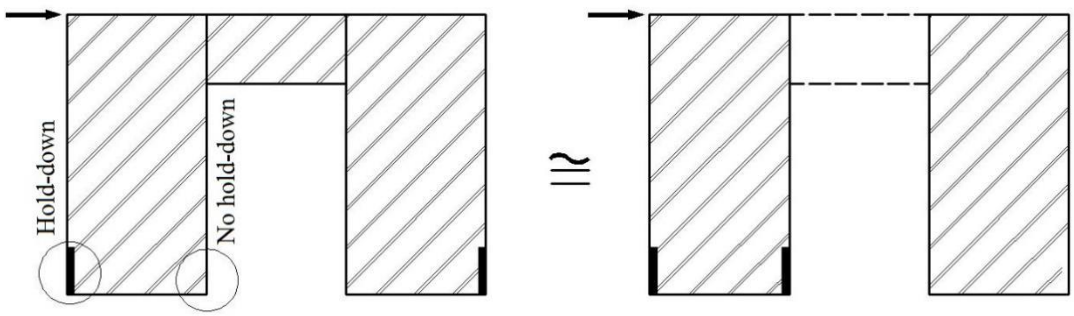


Fig. 13. The calculation guidance for shear-wall limbs with only one hold-down

Based on Table 3, it can be seen that the linear shear strength of the Wall-A specimen was 9.12 kN/m. So, from Eqs. 1 and 2, the lateral bearing capacity of Wall-B and Wall-C can be calculated as 27.36 kN and 32.83 kN respectively, and this result was very close to the test values 27.02 kN and 32.33 kN.

Considering the difference of failure mechanism, when Eq. (1) is applied to the shear wall without hold-down, J_{hd} can be determined by Eq. (3) under the case of no wall below the window at both of the corners

$$J_{\mathrm{hd}} = \frac{1}{1+\dfrac{H}{L_{\mathrm{s}}}} \tag{3}$$

Based on Eqs. 1 and 3, the lateral bearing capacity of Wall-A, Wall-B, and Wall-C type shear wall without hold-down can be determined as 39.08 kN, 21.89 kN, and 25.54 kN respectively, while the experimental values by Guo (2010) were 38.16 kN, 23.23 kN, and 31.97 kN accordingly. So, it can be found that Eqs. 1 and 3 can well evaluate the lateral bearing capacity of the shear wall without hold-down, except that the result for Wall-C type shear wall was slightly conservative.

CONCLUSIONS

1. The failure pattern of the poplar LVL shear wall was similar to that of the S-P-F dimension lumber shear wall, which mainly included the panel nail damage and the separation between the studs and the base plate, while there was hardly any damage to the wall frame itself. So, for light wood shear wall, the dimension lumber wall frame can be well substituted by the poplar LVL wall frame.
2. For the poplar LVL shear wall with different opening types, the weakening of the linear shear strength by the portal type opening was much greater than that of the window type opening, which indicates that the wall below the window has an important role in the lateral bearing capacity of the light wood shear wall. Besides, the test results also show that the effect of the window type opening on the performance of ultimate displacement and energy dissipation is much greater than that of the portal type opening.

3. The corner hold-down can significantly improve the shear strength, the ultimate displacement, and the energy dissipation of the shear wall, and it can effectively reduce the uplift of the shear wall studs without openings. For the poplar LVL shear wall with openings, due to the weakness of the opening side studs, it is suggested that hold-down also be installed at these studs, in order to increase the lateral bearing capacity of the wall.
4. Considering the influence of the hold-down and the wall below the window, the evaluation formula for the bearing capacity based on the linear shear strength of the poplar LVL shear wall without openings is proposed, which can well predict the lateral bearing capacity of the light wood shear wall with and without hold-downs.

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