World Conference on
Timber Engineering
Oslo 2023

EXPERIMENTAL CHARACTERISATION OF GLULAM SHEAR WALLS UNDER LATERAL CYCLIC LOADING

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ABSTRACT: This paper presents the results of an experimental campaign aimed at investigating the lateral response of glulam (glued laminated) shear walls subjected to horizontal cyclic loads. The objective of the study is to explore the potential of this engineered wood product to be used as an alternative lateral load-resisting element in platform timber constructions. The lateral response of massive wooden shear walls is governed by the wall-base connections; therefore, in addition to shear wall tests, cyclic tests were carried out on hold downs and angle brackets attached to glulam elements. Results of the experimental tests of both connections and shear walls showed failure mechanisms characterised by relatively large plastic deformation of the nails and with significant ductility levels. This results in the potential of glulam shear walls being adopted as the lateral load-resisting element of platform timber constructions in seismic prone areas.

KEYWORDS: Glued Laminated Timber, Shear Walls, Experimental Tests, Cyclic Behaviour

1 INTRODUCTION

Mass Timber constructions are gaining in popularity among designer communities as they respond well to today's need for sustainability, permit design with a large architectural freedom and have overcome many limits of traditional wooden structures. The expression "mass timber" refers to the massive, engineered wood products (EWP) which are used for the construction of these structures; glued laminated (glulam) and cross-laminated timber (CLT) are the most widely spread product types. Glulam is used mostly for structural one-dimensional elements, such as beams and columns, whereas CLT is used mostly for planar elements, such as walls and floors. Today's large availability of different EWPs that respond well to different needs of the construction industry results in the possibility of realising mass timber buildings with different structural systems. Among the latter ones, structural systems with shear walls have been largely adopted in the construction practice, CLT panels being the product adopted most.

CLT buildings with shear walls are realized by connecting the CLT panels by means of mechanical anchors and screws. The mechanical anchors are used at the base of the walls, while the screws are used for panel-to-panel connections. The mechanical anchors typically adopted in CLT shear walls are hold downs and angle brackets; the first ones are used at the wall ends to prevent the wall rocking, while the second ones are spread along the wall base and are used to prevent the shear wall sliding.

The lateral behaviour of structural systems made of CLT shear walls has been largely investigated in the last decade, with particular focus on their seismic performance. Shaking table tests on multi-storey CLT buildings [1], racking tests on CLT shear walls [2,3], and cyclic tests on sub-assemblies and connections [4] were conducted, showing the high dissipation capacity and good seismic performance of CLT structures and sub-components. These experimental programmes were instrumental in understanding the seismic behaviour of these structural typologies and in defining the calculation models and specifications needed for the design.

Structural systems with shear walls can also be realised by using different EWPs, such as glulam or laminated veneer lumber (LVL). However, less studies have been conducted on shear wall systems made of glulam and LVL, having the majority of these experimental investigations been conducted on timber walls with dampers and low-damage connection systems. For instance, Wrzesniak et al. [5] investigated the rocking behaviour of glulam shear walls anchored to the foundation with dampers, while Iqbal et al. [6] investigated the seismic resistance of post-tensioned LVL walls coupled with U-Flexural plate dissipators.

If on one hand the use of low-damage systems and dissipators improves the seismic performance of timber walls, on the other hand traditional connection systems are more adopted in practice due to their simplicity of design. However, to authors' knowledge, no studies investigating the seismic performance of glulam and LVL shear walls anchored with traditional hold downs and angle brackets are available in the literature, with the consequence that the relevant information needed for the design are not available to practitioners. This represents a relevant gap of scientific knowledge, which may negatively impact the spread of innovative timber structural systems.

With the aim of filling this gap, this paper presents an experimental study investigating the lateral response of glulam shear walls subjected to lateral loading, and the cyclic response of traditional hold downs and angle brackets attached to glulam elements. In particular, the cyclic behaviour of such structural components is investigated, to explore the potential of glulam shear wall elements to be used in platform timber buildings in seismic prone areas. The evaluation of quantities relevant

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https://doi.org/10.52202/069179-0274

for the seismic design, such as the strength, stiffness, ductility of connections, energy dissipation, and impairment of strength, is carried out, providing valuable insights into the seismic design of structures with glulam shear walls.

2 EXPERIMENTAL TESTS

2.1 MATERIALS

Experimental tests were conducted using glulam specimens with strength class GL24h.

Hold downs type HTT22E and angle brackets type AE116 from Simpson Strong Tie were used for the experimental tests. The geometrical and mechanical properties of both hold downs and angle brackets are described in the ETA-07/0285 [7]. The mechanical anchors are shown in Figure 1 (a) and (b).

Annular ringed nails CNA 4×60 mm from Simpson Strong Tie were used for the connection between the mechanical anchors and the timber elements, see Figure 1 (c). The geometrical and mechanical properties of the nails are described in the ETA-04/0013 [8]. The mechanical anchors were connected at the base with bolts M12 and M16, strength class 8.8.

2.2 METHOD

The lateral behaviour of massive wooden shear walls is governed by the wall base connections. Hold down connections are normally used at the extremities of the wall to prevent rocking, while angle bracket connections are used in the centre of the wall to prevent sliding. Accordingly, the methodology used in this study involved experimental investigations of hold downs and angle brackets subjected to tensile and shear loads, respectively, along with full-scale tests at wall level.

2.3 LOAD PROTOCOLS

The connections were tested in monotonic and cyclic regime according to EN26891 [9] and EN12512 [10], respectively.

The monotonic tests were performed in force control, in order to perform the unloading and reloading path prescribed in the EN26891. The load protocol of the monotonic tests is shown in Figure 3 (a). Two different load protocols were used for the cyclic tests of hold downs and angle brackets due to the different test setups. Hold downs, which were tested for tensile loads, were loaded with an only positive displacement path with imposed displacement ranging from zero up to a specific positive value, see Figure 3 (b). Angle brackets, which were tested under shear loads, were loaded with fully reversed displacement path with positive and negative displacements, see Figure 3 (c). Both hold down and angle bracket cyclic tests were performed in displacement control with a rate varying from 0.1 to 0.5 mm/sec.

2.4 EVALUATION OF MECHANICAL PARAMETERS

Mechanical parameters were evaluated according to EN12512 [10]. The stiffness of the connections, K_{el}, was evaluated according to the method B of the EN12512 as the slope of the line that intersects the load displacement curve in the points of ordinate equal to 10% and 40% of the maximum force. The intersection between the stiffness line and the line with slope equal to one-sixth of the stiffness line and tangent to the load displacement curve defines the yielding point, from which the yielding displacement, V_y, and the yielding load, F_y, are evaluated. The point with the highest ordinate of the load displacement curve defines the maximum force, F_{max}, and the relative displacement, V_{max}. The ultimate force, F_{ult}, is evaluated as the ordinate of the load displacement curve when a 20% reduction of the maximum force is reached. The abscissa of the point of ultimate force defines the ultimate displacement, V_{ult}. Finally, the ductility, D, is evaluated as the ratio between the ultimate displacement and the yielding displacement. Figure 2 shows an example of experimental curve with the relative yielding, maximum and ultimate point.

According to EN12512, the cyclic behaviour of the timber connections is expressed using three parameters: the dissipated energy, E_d, the equivalent viscous damping ν_{eq}, and the impairment of strength, ΔF. The dissipated energy is computed as the area enclosed by each hysteretic loop and is useful for evaluating the total energy dissipated

(a) (b) (c)

Figure 1: *(a) Hold downs, (b) angle brackets and (c) annular ringed nail used for the experimental tests.*

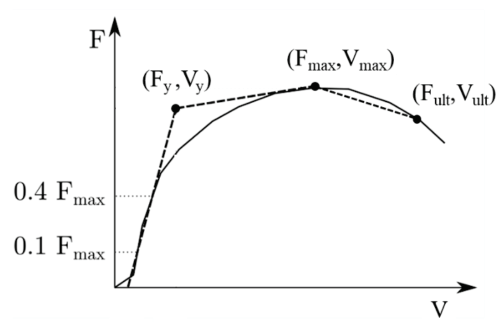

Figure 2: *Representation of the yielding, maximum and ultimate point according to EN12512.*

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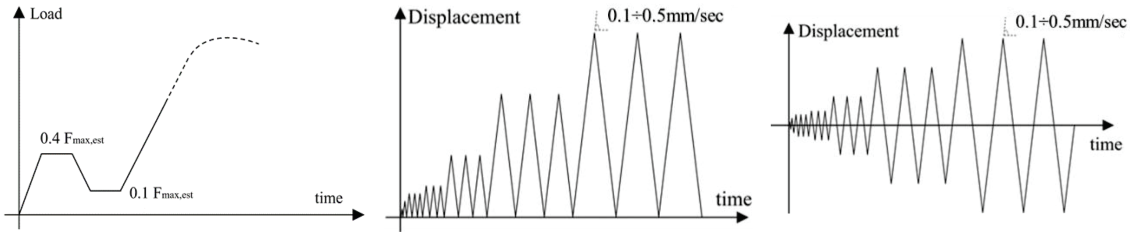
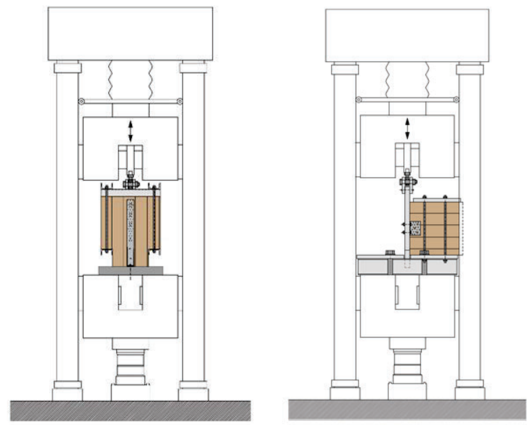


(a) (b) (c)

Figure 3: *Load protocol for (a) monotonic tests, (b) cyclic tests of hold downs, (c) cyclic tests of angle brackets and shear walls.*

during the test when presented cumulatively. The equivalent viscous damping is evaluated at each hysteretic loop and represents the ratio of dissipated energy, E_d, to available potential energy, E_p, multiplied by 2π. The available potential energy E_p is calculated using the maximum force, F_i, in each cycle and the corresponding displacement, V_i, and is expressed as $E_{p,i} = 1/2 \cdot F_i \cdot V_i$. The impairment of strength, ΔF, which quantifies the strength degradation of timber connections, is determined by the difference between the strengths of the first and third envelope curves at the same displacement level. In this study, the normalized form of ΔF, $\Delta F_{1\text{-}3}$, is presented by dividing the impairment of strength by the strength of the first envelope.



(a) (b)

Figure 4: *Experimental test setup of (a) hold down and (b) angle bracket tests.*

2.5 TEST SETUP

2.5.1 Connection Tests

The experimental tests on the mechanical anchors were carried out with the universal machine RBO-2000 with a maximal capacity of 800 kN and a displacement range of ±150 mm. The lamella orientation of the glulam elements was chosen in order to simulate the real condition in which the connections work, when they are anchored to the shear wall. Figure 4 (a) and (b) shows the test setups of the tension and shear tests, for hold downs and angle brackets, respectively. Figure 5 (a) and (b) shows the photos of the hold down and angle bracket test setups. The displacements of the specimens were measured by means of Linear Variable Displacement Transducers (LVDTs) placed on the timber element.

Hold downs were fastened to the glulam elements with 15 annular ringed nails and anchored to the bottom steel plate with one bolt M16. Angle brackets were fastened to the glulam elements with 12 annular ringed nails and anchored to the bottom steel plate with two bolts M12.

One monotonic (n_m) and six cyclic (n_c) tests were performed for each configuration, for a total amount of 14 experimental tests on connections.

(a) (b)

Figure 5: *Photos of (a) tension test setup of hold downs and (b) shear test setup of angle brackets.*

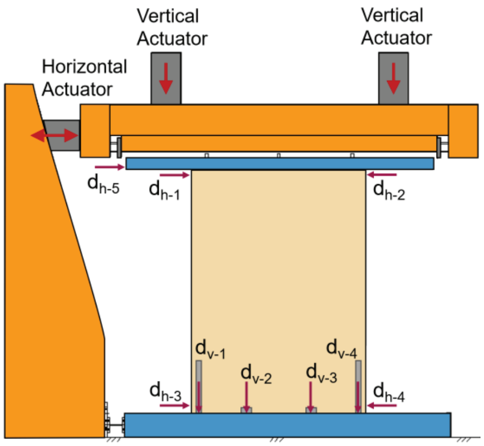


Figure 6: *Shear wall test setup.*

Figure 7: *Photo of the shear wall test setup.*

2.5.2 Shear Wall Tests

Figure 6 and Figure 7 show the test setup used for the shear wall tests. The shear walls had a height of 2500 mm, a length of 1500 mm, and a thickness of 140 mm. All experimental tests were performed with a constant vertical load applied on the top of the shear wall of 30 kN/m. The shear walls were anchored with two hold downs in the shear wall corners, to prevent the shear wall rocking, and with two angle brackets, to prevent the shear wall sliding. Horizontal and vertical loads were applied to the wall specimens through one horizontal and two vertical hydraulic actuators with 400 kN capacity. The vertical actuators were connected to a horizontal steel beam, which was screwed to the shear wall head with self-tapping screws 8×160 mm spaced at 100 mm. The timber walls were anchored through the wall-base connections to a steel beam, which was anchored to a strong floor.

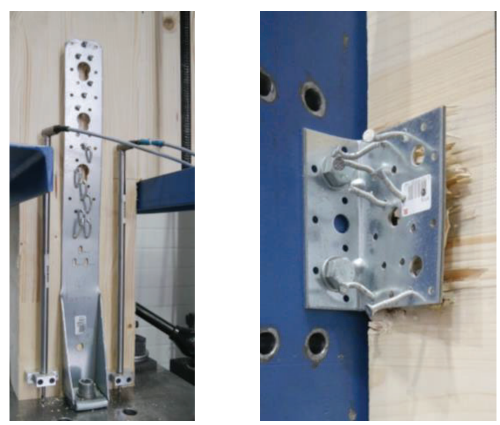
Nine LVDTs were used to measure the wall displacements, see Figure 6. The shear wall lateral displacements were measured with two LVDTs (d_{h-1}, d_{h-2}) placed in the two upper corners of the shear wall. Two horizontal LVDTs (d_{h-3}, d_{h-4}) were used to measure the sliding displacements while four vertical LVDTs (d_{v-1}, d_{v-2}, d_{v-3}, d_{v-4}) were used to measure the rocking of the shear wall as well as the vertical displacements of the two hold downs and the two angle brackets. A horizontal LVDT (d_{h-5}) was used to measure the horizontal displacements of the steel beam connected to the upper side of the shear wall.

The experimental program on shear walls included two cyclic tests in total.

3 RESULTS

3.1 MECHANICAL ANCHORS

The mechanical behaviour of hold down and angle bracket connections was characterised by high ductility and capability to withstand relatively large cyclic displacements. The behaviour of the connection systems was, to a large extent, comparable to that of traditional hold downs and angle brackets attached to CLT elements (see, for instance [4]).



(a) (b)

Figure 8: *Photo of connection specimens at the end of the tests: (a) hold downs, (b) angle brackets.*

Table 1: *Mechanical parameters evaluated from the load displacement curves of monotonic and cyclic connection tests.*

Test type	n_m	n_c	K_{el}	F_y	V_y	F_{max}	V_{max}	F_{ult}	V_{ult}	D
	[-]	[-]	[kN/mm]	[kN]	[mm]	[kN]	[mm]	[kN]	[mm]	[-]
Hold downs	1	6	8.05	43.20	4.96	55.69	15.90	44.55	28.15	5.71
Angle brackets	1	6	3.69	27.90	7.53	32.39	14.70	25.91	21.01	2.99

https://doi.org/10.52202/069179-0274

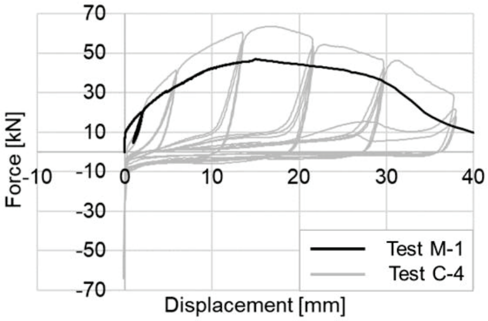


Figure 9: Typical load displacement curve of a hold down connection.

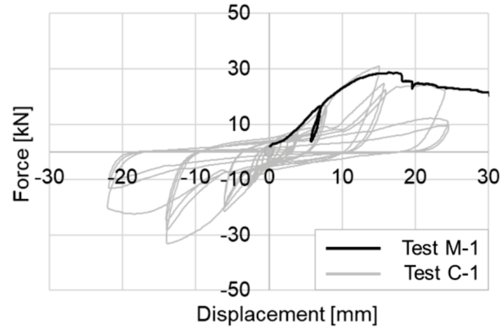


Figure 10: Typical load displacement curve of an angle bracket connection.

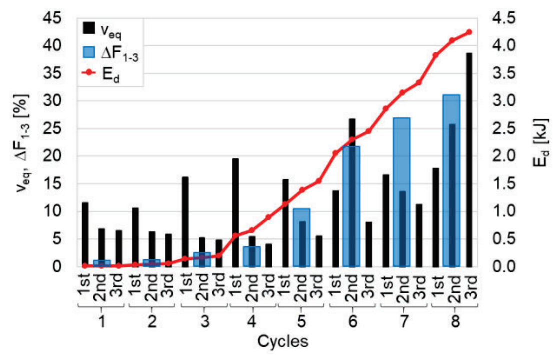


Figure 11: Average cyclic properties of hold down tests.

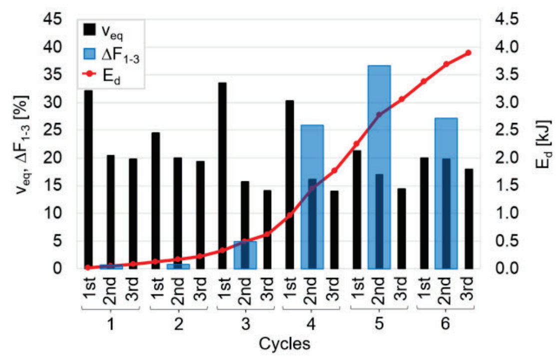


Figure 12: Average cyclic properties of angle bracket tests.

In general, hold down connections showed a failure in the nails of the steel-to-timber connection with one or two plastic hinges, see Figure 8 (a). The hold down steel brackets showed small deformation, while the overall displacement of the tests was mainly provided from the nails of the steel-to-timber connection.
Angle brackets connections showed a failure in the nails of the steel-to-timber connection with one or two plastic hinges combined with a failure due to the opening of the wooden lamellas composing the glulam element. In all tests, the metal brackets showed small deformation, while the overall displacement of the tests was mainly provided from the nails of the steel-to-timber connection, see Figure 8 (b).
Typical load displacement curves of hold downs under tension and angle brackets under shear are shown in Figure 9 and Figure 10, respectively. Table 1 summarizes the average mechanical parameters of the connection tests evaluated according to EN12512.
The results show that the hold down connections have a higher stiffness of 8.05 kN/mm as compared to the angle brackets, which have a stiffness of 3.69 kN/mm. The maximum loads the connections can withstand before failure show variation between the two connection types, with the hold down connections having a higher maximum load capacity than the angle bracket connections. For the hold down connections, the maximum load value is 55.69 kN, while for the angle bracket connections, the maximum load value is 32.39 kN. The ductility was higher for the hold down connections with a value of 5.71, compared to the angle bracket connections with a value of 2.99. The higher performances of the hold downs compared with the angle brackets are due to the larger number of nails used in the connection with the timber panel.
Figure 11 and Figure 12 shows the cyclic properties of hold downs and angle brackets, respectively. The graphs report the number of cycles in the horizontal axes, while two vertical axes are used to plot the dissipated energy (E_d), the equivalent viscous damping (ν_{eq}), and the normalised impairment of strength ($\Delta F_{1\text{-}3}$).
Averagely, 4.24 kJ and 3.89 kJ were dissipated at the end of the hold down and angle bracket tests, respectively. The average equivalent viscous damping was 12.7 % and 20.6 % for hold down and angle bracket tests, respectively. The normalised impairment of strength showed an increasing trend up to the failure of the connections with maximum values of 31.5 % and 36.5 % for hold downs and angle brackets, respectively.

3.2 SHEAR WALLS

The mechanical behaviour of the shear walls was governed by the connections and characterised by large ductility and capability to withstand large cyclic displacements. The behaviour of the wall was, to a large extent, comparable to that of traditional CLT shear walls (see, for instance, [2]).

__Table 2:__ Mechanical parameters evaluated from the load displacement curves of cyclic shear wall tests.

Test type	n_c	K_{el}	F_y	V_y	F_{max}	V_{max}	F_{ult}	V_{ult}	D
	[-]	[kN/mm]	[kN]	[mm]	[kN]	[mm]	[kN]	[mm]	[-]
Shear walls	2	3.44	33.70	9.90	55.29	55.60	44.23	71.10	7.55

In general, the deformation mechanism was governed by the wall-base connections, while the panel deformation was negligible. The shear walls failed with a prevalent rocking mechanism with failure in the nails of the hold downs, see Figure 13 and Figure 14. The nails of the angle brackets showed also plastic deformations, due to the bidirectional tensile-shear loads they were subjected to. At the end of the tests, the hold down steel brackets showed small deformation whereas larger deformations were reached in the angle brackets' steel.
Figure 15 shows the load displacement curves of the shear wall tests, while Table 2 summarizes the average mechanical parameters of the shear wall tests evaluated according to EN12512.

The results show that the glulam walls have an average stiffness of 3.44 kN/mm, a maximum load of 55.29 kN and an ultimate lateral displacement of 71.10 mm. The average ductility of the shear walls is 7.55, indicating that the shear walls have a significant amount of deformation capacity beyond the yield point.
Figure 16 shows the cyclic properties of wall tests. Averagely, 27.1 kJ were dissipated at the end of the shear wall tests, with an average equivalent viscous damping of 13.6 %. The normalised impairment of strength showed an increasing trend up to the failure of the connections with maximum value of 10.8 %.

__Figure 13:__ Photo of a shear wall specimen at the end of the test.

(a) (b)

__Figure 14:__ Photo of wall-base connections at the end of a shear wall test: (a) hold downs, (b) angle brackets.

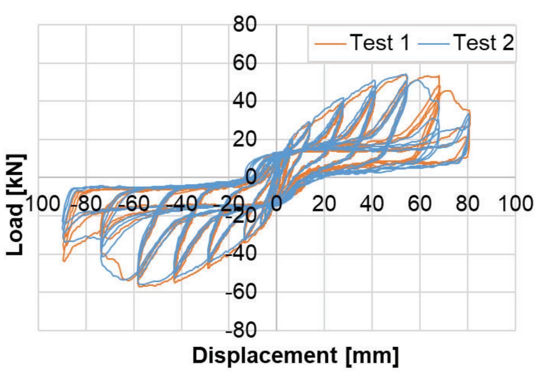


__Figure 15:__ Load displacement curves of the shear walls.

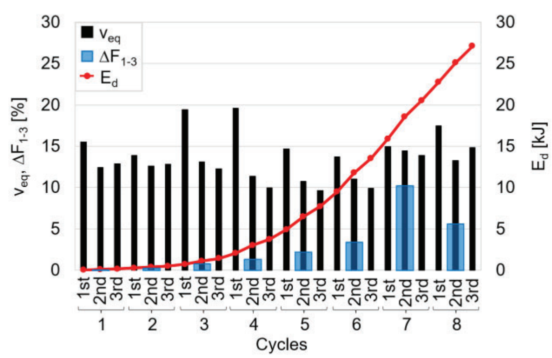


__Figure 16:__ Average cyclic properties of the shear walls.

https://doi.org/10.52202/069179-0274

4 CONCLUSIONS

This paper presented an experimental study on the cyclic behaviour of glulam shear walls and typical hold down and angle bracket connection systems fastened to glulam elements with annular ringed nails.

The EN12512 procedure was used to calculate relevant design parameters for connections, including stiffness, strength, ductility, impairment of strength, and equivalent damping ratios. This data is significant when evaluating the seismic performance of timber structures since the majority of seismic forces and energy dissipation occur at the wall-base connections.

The mechanical behaviour of the connections was characterized by local deformation of the nails embedded into the timber and low deformation of the mechanical anchors. Hold down connections showed higher performance in term stiffness, strength, and ductility than angle bracket connections due to the higher number of nails used in the steel-to-timber joint. The mechanical behaviour of the shear walls was governed by the connections and characterised by large ductility and capability to withstand large cyclic displacements.

Results of this experimental study showed that typical hold down and angle bracket connection systems attached to glulam elements and glulam shear walls exhibit a ductile behaviour with mechanical performance similar to that of hold downs and angle brackets attached to CLT elements and CLT shear walls.

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