New Glulam-Framed Shear Wall Concept with Enhanced Behaviour Characteristics for Tall Timber Buildings in Seismic Areas.

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Abstract

This investigation covers the lateral performance of a new concept of hybrid timber shear wall, and includes individual connection testing, full-scale shear wall testing and numerical modeling. The main objective of this study is to develop enhanced timber structural solutions that facilitates the design and construction of mid-rise timber buildings in areas with high seismic demand. The new shear wall concept involves structural configurations similar to those found in conventional light-framed timber shear wall, but using glulam members as framing elements, connected to OSB sheathing panels using conventional nails. More specifically, the OSB sheathing is embedded within grooved glulam members in order to enhance the lateral capacity and stiffness of the wall. Connections and full-scale monotonic and cyclic shear wall testing were performed, and the results indicated that it is possible to obtain a high performing timber shear wall with the proposed concept. In particular, the obtained values are promising with strength and stiffness levels that are 3 times that of a conventional CLT shear wall. Furthermore, the proposed prototype comprises 0,89 m³ of wood volume, which represents less than one-fourth of the amount of wood in a CLT shear wall of equivalent lateral capacity. In addition, the ductility obtained for the proposed concept can be classified as high ductility class (HDC) according to the Eurocode 8. It is expected that the ductility may be further improved by limiting potential brittle failure observed in one of the framing members at high displacement levels. Finally, it was found that available modelling tools with hysteretic models, such as the MSTEW, is capable of predicting the lateral strength and stiffness of the proposed concept, since modeling errors below 10% were obtained.

1. Introduction

Considerations for sustainable design in recent years have highlighted the need to involve more wood material in construction. Whether in North America where timber design is well advanced and timber structures represent about 90% of low-rise construction (Hafeez et al., 2018), or in countries like Chile, where only around 21% of the building stock consist of timber as main structural elements (Banco Mundial, 2020), there is significant interest in the design and research communities in how to advance the structural performance of timber assemblies. Recent developments in material and connection manufacturing allowed the construction industry to go beyond light-frame and low-rise structures, especially in an effort to help resist lateral loads, such as wind and earthquake.

The introduction of cross laminated timber (CLT) strongly influenced the increasing effort to target taller buildings using timber, because of its superior strength and stiffness in comparison to conventional light-frame timber systems. Although concrete and steel walls and assemblies can be used as primary lateral force resisting systems in CLT buildings, there has been an important focus on understanding and improving the potential of using CLT as primary lateral force resisting system for mid- and high-rise timber buildings. Some examples of important findings in this field include establishing the behaviour of CLT shear wall, including characteristics such as strength, stiffness, as well as energy dissipation through engagement of the boundary connections such as vertical joints. [Gavric et al., (2013); Casagrande et al., (2016); Estrella et al., (2021a), Estrella et al., (2021b), Estrella et al., (2020), Popovski et al., (2011); Mestar et al., (2020); Mestar et al., (2021); Nolet et al. (2019)]. Capacity-based design methodologies have also been developed to ensure appropriate hierarchy of failure between ductile and less ductile or brittle components [Masroor et al., (2022) and Casagrande et al., (2019)] Although shear walls with solid wood panels are generally suitable for taller buildings (Conolly et al., 2018), the limitation of using them in buildings constructed in countries with high seismicity, like for instance Chile, is the relatively low ductility they provide (Ugalde et al., 2019), as well as the relatively higher cost of the structure.

Light frame wood shear walls have been considered suitable and efficient lateral load resisting systems with great energy dissipative capabilities, especially when wood sheathing panels like OSB or plywood are used together with nails [e.g., Bagheri & Doudak (2020); Bagheri & Doudak (2021)]. The use of gypsum wallboard is also common as a sheathing material, whether structural or simply used as finishing material, however, this type of system is usually accompanied by a reduction in the overall energy dissipation in the system (Lafontaine and Doudak, 2017). The performance of light-frame wood shear walls has been demonstrated in laboratory testing as well as in reconnaissance missions post-earthquake events, and the limited failures and low fatalities were attributed to the light weight and flexibility of the system, as well as redundancies and multiple load paths, resulting in a reduction in the inertia forces [e.g., Dolan and Madsen (1992); Karacabeyli and Ceccotti (1996)]. Despite this performance, the use of such structural systems remains limited to low rise structure due to limitation in capacity and stiffness.

Research investigating how light-frame wood shear walls systems may be scaled to provide efficient solutions for taller buildings has been very scarce. Some examples of such attempt include the introduction of the mid-ply shear wall (Ranger L., and Dagenais C., 2021) and a new hybrid shear wall solutions using Frame Panel Shear Walls (FPSW) (Carrero et al., 2021), involving different materials such as steel and CLT. Other attempts in developing hybrid platform-frame shear walls have demonstrated excellent lateral performance while maintaining simplicity and predictability of the structural behavior [e.g., Quintana et al., (2021); Kho et al. (2018); Iqbal et al. (2015); Priestley et al. (1999)].

The review of the available literature highlights the needs for innovation beyond traditional shear wall systems such as CLT and LFTW while using the simple and workable concepts found in these systems. The current research study aims at investigating a new timber hybrid shear wall concept that would provide a cost-competitive alternative that is capable of being easily industrialized (Guindos et al. 2019). The hypothesis underpinning this research was that embedding OSB sheathing in grooved glulam framing would increase the contact between

framing and sheathing and thus enhance capacity and stiffness. In addition, because the new concept is built upon the light-frame structural principle, it was assumed that the energy dissipation would primarily be provided by the connections (panel-to-framing nails). The development of the proposed concept is presented in the paper and compared to other timber systems, which allows for analyzing the relative performance of the obtained results. Finally, the work also covers modelling, which includes the hysteretic parameters optimization that allow for further refinement of the proposed concept.

2. Proposed Wood Panel Shear Wall (Wpsw)

The proposed shear wall concept comprises a glulam framing, connected with four OSB sheathing panels. The standard proposed wall measures 2255 mm in length and 2680 mm in height. Sheathing consists of 18 mm OSB boards with dimensions of 890x2400 mm (LP Building Products, 2019). The wood species used for manufacturing both the sheathing and the framing was radiata pine, and it had moisture content equal to 12%, classified according to NCh819 (Instituto Nacional de Normalización, 2012b). The glulam elements were code-compliant with the Chilean standard NCh2148 (Instituto Nacional de Normalización, 2013). The adhesive used for manufacturing the glulam complies with AS / NZS 4364 (2010) standard and DIN1052 (2008) for the glue lines and BS 1204 (1993) for the finger-joints. Fig. 1 presents a graphical illustration of the proposed shear wall.

The framing consisted of two horizontal beam elements with cross-sectional dimensions of 230 mm x 250 mm and a length of 2255 mm, and three vertical frame elements (posts). The end posts have cross-sectional dimensions of 230 mm x 250 mm and length of 2180 mm, and the interior post has a cross-sectional dimension of 230 mm x 500 mm and a length of 2180 mm, see Fig. 1. The panel-to-framing connections consisted of two rows of 3 mm diameter and 70 mm long nails spaced at 50 mm along the perimeter of the OSB panels, as shown in Fig. 1, resulting in 1200 nails per wall. The OSB panels are embedded in, and nailed to, the glulam frames as shown in Fig. 2.

At the framing-to-framing connections, fifteen TX 30-160mm long [Rothoblaas SRL] screws were used as perpendicular-to-the-grain reinforcement for the framing beams. Furthermore, three additional 340 mm long TX 30 screws were used to join the framing elements at the post-to-beam connections, see Fig. 3 and Fig. 4.

The fabrication process, including the embedment of the OSB boards within the framing using a sling is shown in Fig. 4 and Fig. 5. Also presented is one of the reinforcement and beam-to-post connection, see Fig. 4a. Finally, the wall assembly is shown in Fig. 5.

3. Test Program

The main objective of the experimental testing was to investigate and characterize the seismic response of the proposed WPSW Wall. The methodology followed in the experimental program involved first establishing the behavior of the panel-to-framing connection, due to the critical role the connections play in the overall seismic response of the shear wall. Four glulam-frame connections were tested, one under monotonic load and three under cyclic loads. The main goal of this part of the testing program was to ensure the feasibility of using the glulam and OSB materials with the proposed nail joint to construct the WPSW wall system. The joint connection specimens consisted of a 200mm x 200mm x 600mm glulam frame and four OSB panels with dimensions 400mm x 250mm x 18mm. The connection between the panels and the frame consisted of 3 mm diameter and 70mm-long nails. Fig. 6 shows the joint configuration.

The connection test set-up is shown in Fig. 7. A hydraulic actuator, connected to the glulam element through two bolted steel plates, was used to apply the load on the connections between the OSB panel and the glulam framing. The OSB panels were supported against the strong floor. Four Linear Variable Displacement Transducers (LVDT) were placed around the connection to measure the slip between the OSB panels and the glulam framing element. The use of four LVDTs allowed averaging the displacements as well as revealing any potential overturning effects during testing.

One full-scale WPSW shear wall was tested under cyclic lateral load in order to establish the feasibility of the proposed system. The testing protocol followed the EN12512 procedure according to the European Committee for Standardization (2013), using a displacement-controlled regime, as shown in Fig. 8. The monotonic procedure applied a constant displacement at the specimen top of 0.2mm/s. The hydraulic actuator used had a capacity of 500 kN and a travel range of ± 200 mm. The test set-up is shown in Fig. 9 and 10.

The instrumentation of the test specimen included displacement transducers to capture the global displacements of the wall, deformation in the diagonal direction of the panels, and key relative displacements to allow for the characterization of the wall behavior. The locations of the relevant instruments used are shown in Fig. 11.

The boundary conditions in the experimental testing of the wall are shown in Fig. 12, including the wall's attachment to the foundation beam the loading beam, which prevented in-plane rotation and out-of-plane displacement.

The ductility (D) of the wall was calculated as the ratio of the ultimate and yield displacements (Δ_u / Δ_y), as provided in EN12512. Failure displacement (Δ_u) was calculated as the displacement reached when load-carrying capacity decreased to 80% of the maximum value. The calculation of ductility was also undertaken using the ASTM E2126 standard (ASTM International 2019) to facilitate the comparison between test results. Other important parameters obtained from testing include strength and stiffness reduction for each set of three cycles, and equivalent viscous damping ratio at each tested ductility level.

4. Test Results

4.1. Connections and wall test results

Table 1 summarizes the main test results of the connections measured in the monotonic and cyclic tests, including initial secant stiffness (K), yield load (F_y) and yield displacement (Δ_y), maximum measured load (F_{max}) and ultimate measured displacement (Δ_u). Fig. 13a shows the force-displacement response of the monotonic test, as well as the envelopes of the corresponding responses obtained from the cyclic tests. The envelope curves shown in Fig. 13a were obtained as the average of the absolute values of the load and the displacement of the corresponding positive and negative envelope points for each cycle (ASTM International, 2019). From Table 1 and Fig. 13a, it can be observed that the joint tests subjected to cyclic protocol experienced more degradation of the strength and stiffness compared to the case of monotonic test, which can be attributed to the premature fatigue failure associated to the nail joint when subjected to cyclic loading. In addition, Table 1 contains three types of cyclic test using results from Carrero et al., (2020). It can be seen that all connections exhibited high ductility under the reverse cyclic loading, which was also consistent with the monotonic response. As shown in Table 1 and Fig. 13b, the WPSC connections provide greater strength and inelastic deformation capacity than other hybrid connections previously tested (Carrero et al., 2020). The final failure behavior of a connection after cyclical testing is shown in Fig. 14.

Table 1
Summary of test results of connection specimens

Specimen	K (kN/mm)	F_y (kN)	Δ_y (mm)	F_{max} (kN)	Δ_u (mm)	D (unitless)
CYC-Test-01-Joint	4.39	2.50	2.00	2.51	12.50	6.25
CYC-Test-02-Joint	5.49	2.58	2.50	3.02	14.83	5.93
CYC-Test-03-Joint	6.65	2.50	2.50	2.84	15.34	6.14
MON-Test-04-Joint	6.80	3.00	1.07	3.05	15.52	14.50
CLT-Steel envelope (*)	2.31	5.30	2.98	6.7	16.10	8.89
CLT-RCwo envelope (*)	5.30	7.10	3.30	7.27	17.00	5.15
CLT-LSLn envelope (*)	0.86	3.00	3.48	6.22	30.00	8.62

Note (*): hysteresis test results from Carrero et al., (2020) on hybrid connections.

In the case of the shear wall test, it was anticipated that the tightly spaced nailing along with engaging the nails in double shear would result in a wall with very high rigidity. Table 2 summarizes the main test results from the cyclic, including maximum measured lateral load (F_{max}), lateral displacement at maximum load (Δ_{max}), and ultimate displacement (Δ_u). Table 2 also shows parameters calculated from the test results according to both the EN12512 (European Committee for Standardization, 2013) and ASTM E2126 (ASTM International, 2019), such as yield strength (F_y) and displacement (Δ_y), secant initial stiffness (K_{sec}) and ductility. The equivalent damping was calculated using Eq. (1).

$$\varepsilon_{eq} = \frac{E_H}{2 * \pi * K_{sec} * \Delta_{max}^2} \quad \text{Eq. (1)}$$

Where:

ε_{eq} : Equivalent viscous damping.

E_H : Hysteretic energy associated with the considered cycle. It corresponds to the area enclosed by cycle.

K_{sec} : Secant initial stiffness.

Δ_{max} : Maximum displacement.

The cyclic response of the wall is presented in Fig. 15. The measured response indicates that the WPSW load-displacement curve exhibited a very high capacity under reverse cyclic loading. This is consistent with the behaviour exhibited by the connections (see Table 2) except that the

deformation capacity was smaller. This could, in part, be attributed to the brittle failure of the top framing beam. It was also observed that the stiffness degradation increased for load cycles up to 4, see Fig. 18, whereas the viscous damping increased from cycle 8.

Table 2. WPSW shear wall test results

Wall	F_{max} (kN)	Δ_{max} (mm)	Δ_u (mm)	Δ_y (mm)	F_y (kN)	K_0 (kN/mm)	Dissipation energy	Ductility	ξ_{eq}^M (%)
According to EN 12512									
WPSW	266.5	33.00	50.10	8.50	200.00	23.53	Per cycles	5.89	12.23
According to ASTM E2126									
	266.5	33.00	48.17	8.39	235.91	28.11	Per cycles	5.74	12.23

The full-scale wall test result was compared to published data obtained from the literature, as shown in Fig. 15. These include a strong hybrid hollow steel framing and CLT sheathing shear wall, FPSW (Carrero et al., 2021), a common CLT wall (Popovski et al., 2011), and a strong light-frame timber wall, LFTW wall (Guíñez et al., 2019). It can be noted that the curve representing the WPSW from the current study has higher lateral force capacity than all other shear walls in this comparison. It can also be noted that the lateral load capacity and stiffness of the proposed concept are significantly higher (about three times) than those obtained from conventional CLT shear walls of similar lengths (see Fig. 15). Furthermore, the proposed wall concept uses only 0.89 m³ of wood, which is about one-fourth of the wood used in a CLT shear wall with equivalent capacity. Fig. 16 shows that the cumulative energy dissipation from WPSW cyclical behavior is higher than CLT and LFTW walls, even though the wall had an abrupt and brittle failure shown in Fig. 15. It can also be observed in Fig. 17 that the deformation in the wall mainly occurred in fastener slip at the top beam.

A comparison between the hysteresis envelopes, equivalent viscous damping (EVD), energy dissipation (ED), stiffness degradation and strength degradation from conventional LFTW wall (Guíñez et al., 2019), CLT Wall (Popovski et al., 2011), FPSW (Carrero et al., 2021) and the proposed WPSW concept is presented in Fig. 18. As shown in Fig. 18, WPSW exhibits an excellent lateral force capacity, stiffness, energy dissipation, and equivalent viscous damping relative to the other walls. The envelope curve of WPSW clearly shows the superior capacity and stiffness of the proposed system.

4.2. Crack pattern detection using Digital Image Correlation (DIC)

A digital image correlation (DIC) technique was used to evaluate the deformation of the wall, as presented in Fig. 19. DIC is an optical technique based on the measurement of displacements in two and three dimensions (2D-DIC and 3D-DIC). It is suitable to compare the deformation state of material when subjected to given stresses compared to their original unloaded condition (Sutton et al., 2009). On the backside of the wall, a layer of white background paint was used, on which a mottled pattern (black dots) was applied over the region of interest using black spray paint. Two lamps were placed at 2050 mm away from the wall surface and a spacing of 5000 mm was provided between the lamps. A 24.2 Megapixel camera (Canon DSLR) was used to capture the images.

At the time of failure, the additional slip of the top beam was clearly identified, as shown in Fig. 20. In this case the failure mode was evident, however this technique makes it possible to see deformations not visible to the human eye.

5. Failure Modes

As expected, the failure modes of the connection specimens comprised ductile hinges at the nail joints, followed by nail withdrawal as shown in Fig. 21. This is considered to be a reliable failure mode, because the fastener maintain the connection between the OSB board and glulam frames. It can be noted that the minimum required wood thickness for ductile failure in conventional timber connections was sufficient to ensure ductile failure in this connection. At the full-scale shear wall test, however, there was a brittle failure at the top beam, which was attributed to closely-spaced nails (50 mm) and the lack of sufficient end edge distance in the beam member, see Fig. 22. This type of failure mode has also been observed in previous studies with closely spaced nail patterns (Bagheri & Doudak 2020). It is recommended that future designs consider and prevent this type of failure mode.

6. Model Validation

In order to model such connection test behavior, a non-linear model was carried out in the MCASHEW software (Pang et al. 2013) to predict the experimental results, which allowed for an evaluation of the observed response with numerical modeling.

The hysteretic behavior of each joint test and shear wall was modeled using the Modified-Stewart (MSTEW) model proposed by Folz & Filiatrault (2001), which is able to capture phenomena associated with large damaged states, such as stiffness degradation, force degradation, and pinching. The MSTEW model system consists of 10 modeling parameters that capture the crushing of the wood (framing and sheathing) and yielding the nails (Pang et al., (2007)).

The cyclic connection tests were used to calibrate the MSTEW hysteretic model (Pang et al., 2007). Table 3 presents the corresponding MSTEW modeling parameters for each of the tested joint specimens. The hysteresis modeling parameters are scaled in Table 3 to represent the properties of a single nail. The comparison between the cyclical test results and the parameters obtained from MSTEFIT (Pang et al., 2013) seem to match reasonably well, as shown in Fig. 23.

Table 3. MSTEW connections hysteretic modeling parameters.

Parameter	CYC-Test-01-Joint	CYC-Test-02-Joint	CYC-Test-03-Joint
K_0 (kN/mm)	4.39	5.49	6.65
r_1 (unitsless)	0.032	0.026	0.017
r_2 (unitsless)	-0.015	-0.013	-0.011
r_3 (unitsless)	2.226	3.669	7.399
r_4 (unitsless)	3.26E-03	4.95E-04	3.08E-04
F_0 (kN)	2.135	2.203	2.220
F_1 (kN)	0.85	0.58	0.57
d_u (mm)	4.20	4.47	4.32
a (unitsless)	0.85	0.95	0.95
b (unitsless)	1.25	1.15	1.15
CEE (error fit, %)	14.10	16.75	14.10

Subsequently, the MCASHEW software (Pang et al., 2013) was used to model the entire shear wall. MCASHEW was originally developed for modelling light-frame wood shear walls consisting of framing members (interior and end studs and top-bottom plates), framing-to-framing connectors, sheathing panels, sheathing-to-framing connectors, anchorage devices, and shear bolts. An Euler-Bernoulli approach is used for modelling the framing members. The model utilizes a corotational formulation and three degrees of freedom per node.

In the current study, the shear modulus, G , used for OSB panels was 1960 MPa (Segura, 2017), and the mean value of modulus of elasticity for the wood framing was set as $E = 8500$ MPa according to the Chilean standard (Instituto Nacional de Normalización, 2014). Framing to framing (screw) and panel to framing (nail) connectors were modeled as pin-ended connections using two-node 3 degrees of freedom link elements with rigid springs for translation and zero stiffness for rotation. OSB panels were modeled using rectangular shear panel elements with 5 degrees of freedom (two rigid body translations, one rigid body rotation and two in-plane shear angles). It was assumed that the bottom beam-to-foundation connection was very large, linear and elastic using a 3-DOF link element. The pantograph was used to restrict rotation in the experiment and thus a high compression stiffness was implemented in the model to simulate the contact between the bottom plate and the foundation.

A comparison between the load-displacement behavior obtained from the test results and those predicted numerically by the MCASHEW model is shown in Fig. 24. The resulting stiffness was very similar between the tested and modeled wall with an error of only 2%. Similarly, the maximum capacity was obtained with an error of only 9%. This indicates that the MCASHEW model, which was created for modelling light-frame timber structures is also able to predict the behavior of the proposed concept.

Once it is established that the behavior of the WPSW can be predicted with the MSTEW hysteretic model and MCASHEW software, the next step was setting up a simplified 1-DOF model of the entire wall using the hysteresis' parameter calibration of the MSTEFIT tool (Pang et al., 2007). The result of the calibration is presented in Table 4 and Fig. 25. As can be noted from the figure, MSTEW can accurately mimic the test results, properly capturing the nonlinear behavior of the specimen in terms of lateral force capacity, stiffness, strength, and deformation capacity.

Table 4. MSTEW wall hysteretic modeling parameters.

Parameter	Wall Test
K_0 (kN/mm)	37.374
r_1 (unitsless)	0.059
r_2 (unitsless)	-0.088
r_3 (unitsless)	1.01
r_4 (unitsless)	0.008
F_0 (kN)	191.353
F_1 (kN)	31.623
d_u (mm)	36.692
a (unitsless)	1
b (unitsless)	1.131
CEE (error fit, %)	14.79

Table 5 presents the comparison between the experimental and numerical values of both the detailed MSTEW model based on the connection results and the simplified (one-DOF) MSTEW model. As it can be observed, the models tend to overestimate the force, displacement, and drift values. The experimental test has a lateral load capacity of 266.46 KN and a maximum displacement in the LVDT (top beam) of 64 mm, while the numerical model yielded a lateral load capacity of 271.39 KN and a maximum displacement of 66 mm. The error observed in the drift and displacement values is around 3.14% for both numerical cases relative to the experimental results.

Table 5. Analytical and experimental wall results.

Force (kN)			Error (%)	
Numerical (MCASHEW model)		Experimental		
271.39		266.46	8.65	
Displacement (mm)			Error (%)	
Numerical Simplified 1-DOF Model (1)	Numerical based on connection results (2)	Experimental	(1)	(2)
62	66	64	3.14	3.13
Drift (%)			Error (%)	
Numerical Simplified 1-DOF Model (1)	Numerical based on connection results (2)	Experimental	(1)	(2)
2.75	2.93	2.84	3.14	3.13

7. Conclusions

A new shear wall concept has been introduced to optimize the walls' strength and stiffness while maintaining high levels of ductility. The proposed system builds on the simplicity and established performance of light-frame timber shear walls while using larger glulam members as framing elements to introduce more panel-to-framing nail joints. Testing and numerical modeling at the joint and full-scale wall levels have been undertaken. The main conclusions that can be drawn from the current study are as follows:

- The study has demonstrated the feasibility of introducing more shear planes in the shear wall system, and hence significantly enhancing the strength and stiffness of the wall, while maintaining reasonably high ductility ratio to utilize the shear wall in regions with high seismic demands.
- The stiffness and strength values obtained from the proposed shear wall concept are significantly higher (about three times) than those obtained from conventional CLT shear walls of similar lengths.
- The proposed shear wall prototype contains only 0.89 m³ of wood, and it compares to or exceeds the performance of a CLT shear wall containing approximately 4 times more wood. This is anticipated to lead to a solution that does not only provide adequate structural performance but also be cost-competitive in comparison to current timber lateral systems.
- Maintaining high ductility class (HDC) was an essential and desired requirement in the proposed shear wall. This means that it could be used in regions with high seismic demand. The ultimate failure mode observed in the top beam, consisting of splitting, could be improved in future designs simply by increasing the nail spacing or the edge distance in the beam.
- Modelling tools with hysteretic behaviour, such as the MSTEW, was found to be capable of predicting the lateral strength and stiffness of the proposed concept, with reasonable discrepancy.

Declarations

The authors declared that there is no conflict of interest.

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Figures

Figure 1

Mounting scheme of the glulam-OSB wall (WPSW): (a) components (b) dimensions.

Figure 2

Plan view WPSW wall posts: (a) mid-section post (b) end post

Figure 3

Framing to framing connections: (a) reinforcement implemented (b) compression reinforcement with blue square (c) post-beam connections in the red square.

Figure 4

Shear wall assembly procedure (a) screw placement (b) OSB panel installation (c) use of sling to assemble panels

Figure 5

Photograph of shear wall assembly.

Figure 6

Connection specimen configuration.

Figure 7

Full detail of the assembly of the glulam-OSB connections.

Figure 8

Experimental protocol according to EN 12512 (a) cyclic protocol; (b) determination of strength degradation between the first and third repetition

Figure 9

Wall test set-up.

Figure 10

Photograph of the anchoring and Instrumented wall test set-up of the WPSW.

Figure 11

Elevation front view of the WPSW wall with measuring sensors.

Figure 12

Shear wall corners anchoring: (a) Top beam (b) Bottom beam.

Figure 13

Monotonic and cyclic envelopes: (a) responses of the tested connections (b) comparison of envelope tested curves to others type of connections by Carrero et al., (2020).

Figure 14

Panel-to-framing failure of WPSW connection (CYC-Test-01-Joint).

Figure 15

Comparison of cyclic test curves from different wall tests.

Figure 16

Comparison of the cyclic performance for shear walls cumulative energy dissipation including WPSW from our investigation.

Figure 17

Photography of WPSW shear wall post-test.

Figure 18

Comparison of the cyclic performance for existing shear walls compared to WPSW shearwall. (a) hysteresis envelopes; (b) energy dissipation (ED) per cycle; (c) equivalent viscous damping (EVD) per cycle; (d) stiffness degradation (%) per cycle; (e) strength degradation per cycle.

Figure 19

Configuration of WPSW shearwall for Digital Image Correlation (DIC).

Figure 20

(a) Actual failure from the test (b) Digital Image Correlation results from WPSW shearwall in deformation.

Figure 21

Failure modes WPSC connections.

Figure 22

Failure mode of top beam from sheathing to framing connection at WPSW wall test.

Figure 23

Test connection data and model MSTEW (WPSC).

Figure 24

Comparison between WPSW test result and MCASHEW hysteresis load-displacement model.

Figure 25

Test wall data and model (WPSW)