World Conference on
Timber Engineering
Oslo 2023

TESTING OF STRONG MULTI-LAYERED WOOD FRAME SHEAR WALLS WITH NON-STRUCTURAL LAYERS

Diego Valdivieso[1], Diego Lopez-Garcia[2], Jairo Montaño[3], Pablo Guindos[4]

ABSTRACT: In areas of high seismic activity it is important to provide Light Frame Timber Buildings (LFTBs) with enhanced levels of lateral stiffness and strength, as well as to prevent excessive levels of non-structural (NSC) damage. Chilean wood-frame shear walls are usually sheathed at both sides with OSB and covered by one/two-ply type X gypsum wallboard (GWB) fastened to the frame with narrow patterns of nails or screws. The result is a multi-layered strong shear wall (MLSSW), which is not considered as such by design codes and mechanical models. The objective of this paper is to report an experimental evaluation of typical Chilean MLSSWs, with emphasis on the influence of NSCs. Connection-level and assembly-level of 1:1 aspect ratio shear walls were evaluated through experimental tests. Results showed increments of 53% and 160% in elastic stiffness and maximum capacity, respectively, while keeping virtually the same deformation capacity and energy dissipation of equivalent bare (non-GWB finished) shear walls. It is postulated that such increases may arise from the high embedment strength of the GWB, and that the deeply screwed GWB may prevent nails from pulling out during hysteresis cycles. It is concluded that GWBs have a significant structural influence on MLSSWs, and such influence should be taken into account in structural design.

KEYWORDS: Wood frame construction, light-frame shear walls, multi-layered, non-structural, gypsum wallboard.

1 INTRODUCTION

Light Frame Timber Building (LFTB) is one of the structural systems currently evaluated by the Chilean construction industry, public authorities, and academia to enhance the sustainability of the Chilean building inventory. Since Chile is subjected to strong earthquakes, it is essential to provide LFTBs with enhanced levels of lateral stiffness and strength. Equally important is the prevention of excessive levels of non-structural damage, as significant costs of damage repairs (i.e.,gypsum wallboard replacement) after earthquake events have been reported [1]. In Chile, wood-frame shear walls usually have a strong structural configuration, consisting of 41 mm × 185 mm (2×8) framing members, sturdy end studs (typically comprising 4 or more members), conventional or continuous holdown devices, wood structural panels (WSPs) -typically OSB on both sides- and closely spaced nails for attachment of sheathing to wood-frame members [2]. On the other hand, the non-structural sheathing customarily consists of one or two layers of Type X gypsum wallboard (GWB) at both sides, fastened to the framing with screws or staples through the OSB. These features cast multi-layered strong shear walls (MLSSWs), as exemplified in Figure 1, whose characteristics have neither been thoroughly investigated nor explicitly considered by design codes or mechanical models. More precisely, although previous investigations have reported a distinct behavior for these types of strong walls (i.e., more prevalence of the rocking effect [2-4]), the influence of the non-structural finishes is rather unknown, and no adequate modeling procedures are currently available. The structural effect of the GWB has been mainly studied in conventional light-frame shear walls (the term conventional was introduced by Estrella et al. [2]), hence a brief summary of the experimental testing of conventional walls with non-structural finishes is presented next.

1.1 EXPERIMENTAL EVALUATION OF THE EFFECT OF NON-STRUCTURAL GWB FINISH LAYERS

GWB is the most common interior wall sheathing material for fire protection used in residential construction [5]. Due to the brittle nature of its core material and its supposedly low stiffness and strength relative to that of wood-based panel materials, the structural contribution of GWB to the lateral response of light-frame buildings is rarely recognized [5]. For this reason, manufacturers have focused on the characterization of GWB for acoustic and fire protection purposes rather than on the mechanical properties that influence the lateral response of a shear wall, such as the shear modulus [6-8]. However, previous research has evaluated the contribution of GWB finish layers to the lateral response of different configurations of

[1] Diego Valdivieso, Pontificia Universidad Católica de Chile, Centro Nacional de Excelencia para la Industria de la Madera (CENAMAD) & Centro de Innovación en Madera (CIM UC-CORMA, Chile, dnvaldivieso@uc.cl

[2] Diego Lopez-Garcia, Pontificia Universidad Catolica de Chile & Research Center for Integrated Disaster Risk Management (CIGIDEN) ANID FONDAP 1522A0005, Chile, dlg@ing.puc.cl

[3] Jairo Montaño, Centro Nacional de Excelencia para la Industria de la Madera (CENAMAD) & Centro de Innovación en Madera (CIM UC-CORMA), Pontificia Universidad Católica de Chile, Chile, jamontano@uc.cl

[4] Pablo Guindos, Pontificia Universidad Católica de Chile, Centro Nacional de Excelencia para la Industria de la Madera (CENAMAD) & Centro de Innovación en Madera (CIM UC-CORMA, Chile, pguindos@ing.puc.cl

 https://doi.org/10.52202/069179-0282

conventional shear walls where the OSB and GWB sheathing layers are installed at opposites sides of the frame. For example, the effect of the GWB finish layer on the lateral response of wood-frame shear walls was experimentally evaluated in [9]. A 12% and 60% increase in lateral strength and stiffness, respectively, was found, but also a 31% reduction in deformation capacity due to significant strength degradation. Moreover, GWB impacted the failure mode of the shear walls by limiting the twisting in the stud caused by the eccentricity due to sheathing placed at only one side. In [10] the effect of GWB on a full-scale two-story wood-frame townhouse was evaluated. A reduction of up to 9% of the fundamental period was found because of a 21% increase in the lateral stiffness of shear walls, which was attributed to the incorporation of GWB in the interior side. Also, contrary to other findings [9], when the finish layer was incorporated the lateral stiffness degradation was smaller than that of bare shear walls. The study highlighted the need to develop a seismic design method that takes into account the effect of wall finishes materials.

Even though the aforementioned research results were promising, the experimental evaluation of the MLSSW configuration typically used for mid-rise buildings in highly seismic-prone areas was not considered. In this context, a first approach was given in [11]. Improvements in the performance of GWB in wood-frame shear walls were investigated, motivated by the fact that the configuration typically used in houses (i.e., OSB and GWB panels installed on opposite sides of the frame) tended to trigger substantial damage to the GWB, mainly because of the different lateral stiffness of OSB and GWB. A promising solution was to install the GWB at the top of a shear wall sheathed on both sides with OSB, which resulted in improvement of the GWB performance (i.e., reduction of earthquake damage) due to minimization of the difference in lateral stiffness between both sides of the wall. However, the effect of the finish layer on the lateral behavior of the shear wall (which is of great interest for MLSSWs and mid-rise LFTBs) was not quantified. Recently, in [12] the contribution of Type X GWB to the racking strength of wood-frame shear walls with representative multi-story details (i.e., the racking restraint system was a continuous rod system [13]) was evaluated experimentally. GWB and OSB were installed on opposite sides of the frame, as in previous research [9, 10]. Results showed that Type X GWB increased the peak strength by 3% and the initial stiffness by 11% when shear walls were tested cyclically and monotonically, respectively, compared to bare shear walls. Contrary to previous research, the study reinforced the traditional practice that ignores the contribution of GWB in the seismic design of LFTBs.

In summary, even though several studies have demonstrated the benefits of non-structural finish GWB layers, the experimental evaluations have been limited to wall assemblies that are different from those typically used in mid-rise LFTBs located in highly seismic-prone areas (i.e., MLSSWs). Hence, it becomes important to quantify experimentally and/or numerically the effect of GWB layers in strong shear wall assemblies, particularly in the context of development of tall timber buildings in seismic areas.

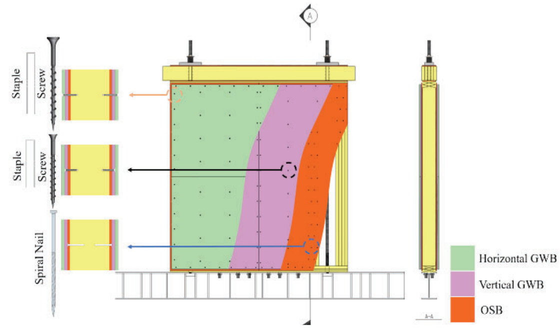

Figure 1: *Typical Chilean MLSSW configuration*

2 SCOPE

In this paper, the contribution of Type X GWB finish layers to the lateral response of multi-layered strong shear walls (MLSSW) is evaluated through reverse cyclic tests on 2.44 m x 2.44 m full-scale MLSSWs and comparisons with previous findings on bare shear walls [3,4].

3 METHODOLOGY

An experimental program was developed to characterize the behavior of multi-layered sheathing-to-frame connections and full-scale shear walls under monotonic and/or cyclic loading.

3.1 CONNECTION-LEVEL TESTS

Three different configurations of multi-layered sheathing-to-frame connections were assembled considering the typical fasteners used for attaching OSB and Type X GWB to wood frames. As shown in Figure 2, the connection-level specimen consisted of a frame of 41 mm x 185 mm (2 x 8) dimensional Chilean radiata pine lumber mechanically graded as C16 according to NCh1198 [14] and attached to different sheathing materials and fastener types (see Table 1). For each configuration, four specimens were considered (i.e., one and three specimens for monotonic and cyclic test, respectively).

Table 1: *Connection-level specimens (all dimensions in millimetres)*

ID	Wood Structural Panel (OSB)		Type X Gypsum Wallboard (GWB)		
	Thick	Nail	Thick.	Type	Fast.
A	11.1	2.9x80	-	-	-
B	11.1	-	15	Screw	4.0x63.5
C	11.1	-	(2) 15	Screw	4.0x76.2

Notes:

a) OSB sheathing layer attached to frame with pneumatically driven wire coil spiral nails (80 x 2.9 x 6.5 mm) according to EN14592:2008+A1:2012 [29].

b) Type X GWB sheathing first layer attached to frame through the OSB with type "W" screws (63.5 x 4.0 x 8.0 mm)
c) Type X GWB sheathing second layer attached to frame through the 1st Type X GWB and OSB with type "W" screws (76.2 x 4.0 x 8.0 mm)

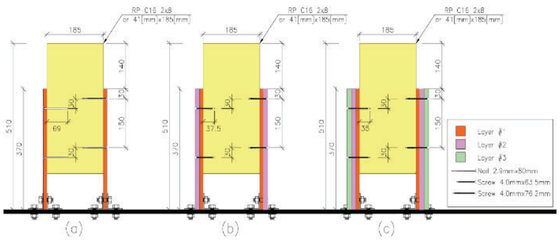

Figure 2: Multi-layered connection specimen for: (a) Test Group A, (b) Test Group B, and, (c) Test Group C. All dimensions in millimeters

The test setup is shown in Figure 3. The reaction steel frame is anchored to a concrete floor. A heavy-duty steel beam is installed on the frame at a suitable location to accommodate the specimen, which is attached to the heavy-steel beam through bolted L-shape elements (Figure 3). The load was applied by a double-action cylinder of +/- 86 kN and +/- 75 mm of force and displacement capacity, respectively, which transfers the vertical load to the specimen through a load-transfer system. All specimens were instrumented with two displacement transducers (LVDTs) and one double-effect load cell to capture the slip and shear force between the frame and sheathing multi-layers.

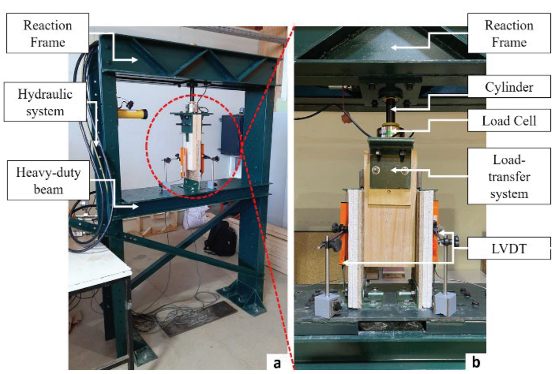

***Figure 3:** Connection-level test set-up: (a) general view of the reaction steel frame and (b) detailed view of the specimen set-up*

The loading protocol was established according to ASTM E564-06 [15] and ASTM E2126-19 [16] for the monotonic and cyclic tests, respectively. The ultimate displacement observed in the monotonic test (i.e., the maximum displacement at which the strength has not yet dropped below 80% of the peak strength) was used to compute the reference displacement for the simplified CUREE-Caltech cyclic testing protocol [17] according to method C of ASTM E2126-19 [16]. The loading protocol was displacement-controlled and applied until failure of the specimen.

3.2 ASSEMBLY-LEVEL TESTS

Specimens are representative of typical ground-level walls of a 7-story building designed per the Chilean seismic design code NCh433 [18] (see Figure 4). Four of them are MLSSWs, whereas the remaining one (i.e., the control wall) is a bare strong shear wall. As shown in Figure 4, the walls had a 1:1 aspect ratio (i.e., 2481 mm in height and 2440 mm in length). Wood-frame consisted of eight studs distributed along the length every 400 mm center-to-center distance. Due to the high levels of overturning moments, each sturdy end-stud consisted of four members mechanically joined and located symmetrically with respect to the continuous rod system. Double plates at the top and bottom of the wall were nailed to the studs with ϕ3.0 mm x 80 mm nails. All framing elements were 41 mm x 185 mm (2 x 8) C16 Chilean RP dimensional lumber, with a nominal modulus of elasticity E = 7900 MPa according to NCh1198 [14]. The walls were sheathed on both sides with 11.1 mm thick APA-rated OSB panels [19] with G = 1307.5 MPa (measured in previous studies [3]), and pneumatically driven to the frame with ϕ2.9 mm x 80 mm helical nails. The edge and field nails were installed at 100 mm and 200 mm center-to-center, respectively. According to SDPWS prescriptions [20], edge-nailing at the end studs should be uniformly distributed among the four framing members and spaced at a maximum of 300 mm. The walls were sheathed on both sides with two-ply 15 mm thick Type X GWB panels [6] with a measured G = 1177.9 MPa according to the ASTM D3044-16 [21] prescriptions. The first Type X GWB layer was vertically oriented and attached to the frame through the OSB with ϕ4.0 mm x 63.5 mm (i.e., Nº 8 x 2-½") drywall screws, whereas the second Type X GWB layer was horizontally oriented and attached to the frame through the first GWB layer and OSB with ϕ4.0 mm x 76.2 mm (i.e., Nº 8 x 3") drywall screws. The edge screws and field screws were installed at 200 mm and 300 mm center-to-center, respectively, according to the NCh1198 [14] fire prescription draft. In order to transfer the lateral load to the wall, a built-up collector beam of 205 mm x 207 mm (i.e., five members of 41 mm x 185 mm C16 RP plus an 11.1 mm thick OSB layer on top and bottom) was mechanically attached to the top sole plate through 38 Simpson Strong-Tie's SCDP221100 screws. The MLSSW racking restraint system consisted of a continuous rod system (i.e., Strong-Rod® system [13]) fabricated in Pleasanton, CA, USA. High-Strength ASTM A193 Grade B7 (i.e., ultimate strength equal to 125 ksi) fully threaded steel rods of ϕ38.1 mm were installed on both sides of the specimen. The rods were attached to the walls by a reaction system over the collector beam that consists of (from bottom to top) a 31.75 mm thick bearing plate (i.e., Simpson Strong-Tie PL16-5x12), a take-up device (i.e. Simpson Strong-Tie ATUD14), a 9.5 mm thick bearing plate (i.e. Simpson Strong-Tie BP 1-1/2), and a finger tightened double heavy hexagonal nut ASTM A563 Grade DH. To avoid sliding between the specimen and the reaction beam, 14 ϕ32 mm x 220 mm ASTM A193 Grade B7 anchor bolts were used to attach the bottom double sole plate to the top flange of the reaction steel beam.

https://doi.org/10.52202/069179-0282

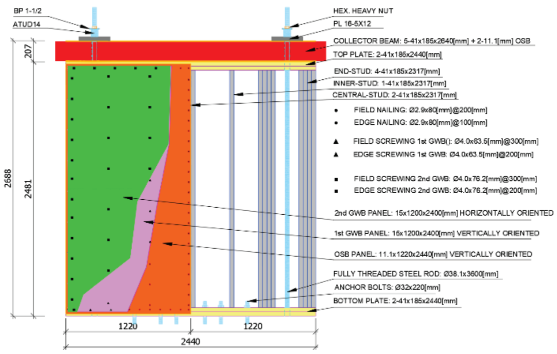

Figure 4: *Monotonic response of connection-level groups*

A cantilever reaction wall, a strong floor, and a reaction steel beam were used to perform the assembly-level tests. As shown in Figure 5, the reaction steel beam was attached to the strong floor by two transversal steel beams attached to the strong floor by four high-strength fully threatened steel rods with a post-tensioned force of 120 kN each, avoiding possible sliding of the reaction beam. The specimens were attached to the reaction beam through a rod-to-steel beam connector (i.e. Simpson Strong-Tie ATS-SBC10H connector) and 14 ϕ32 mm x 220 mm ASTM A193 Grade B7 anchor bolts to prevent overturning and sliding, respectively. The lateral load was applied by a hydraulic bidirectional actuator of +/- 245 kN and +/- 250 mm of force and displacement capacity, respectively, which transfers the lateral load to the specimen through the collector beam. To prevent out-of-plane displacements, the specimens were laterally braced by two steel A-frames which allowed in-plane displacements. To capture the lateral displacement and shear force along the axis of the collector beam, the slip of the wall with respect to the steel reaction beam, the diagonal (shear) deformation, uplift in the exterior edge of the wall, the relative displacement between the multiple layers of the wall, the relative displacement of the steel reaction beam with respect to the strong floor, and the compressive deformation under the bearing plate of the strong-rod system, all specimens were instrumented with thirteen displacement transducers (LVDTs), one laser displacement transducer, and one load cell and displacement transducers (LVDT) incorporated into the actuator. To measure the tension in the rods of the continuous holdown, two unidirectional strain-gauges were attached to the rods.

In order to characterize the in-plane cyclic behavior of specimens, the CUREE-Caltech cyclic testing protocol proposed by Krawinkler et al. [17] was applied. The reference displacement was based either on: a) previous monotonic tests conducted by Guiñez et al. [4] on bare shear walls of comparable specimens features for the case of discrete hold-downs; or b) the shear walls investigated by Estrella et al. [3] for the case of continuous rod systems. The maximum limit for the reference displacement Δ = 61 mm was set for the specimens (i.e. 0.0025 times the wall height) as established in method C (i.e. the simplified CUREE-Caltech protocol) of ASTM E2126-19 [16]. The loading protocol was displacement-controlled and applied until the specimens reached a safe minimum capacity after the peak strength.

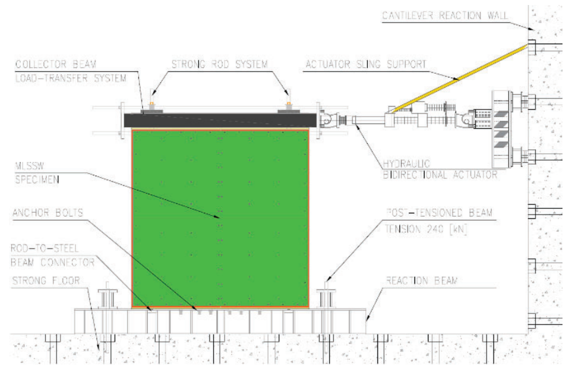

Figure 5: *Front view of the test set-up for assembly-level specimens.*

4 RESULTS

In this section, test results and a discussion of the findings are presented. Failure mode, hysteresis shape, and six engineering parameters were established for connection-level and assembly-level test results: (1) elastic stiffness (K_e); (2) yield displacement (Δ_y); (3) yield force (F_y); (4) ultimate displacement (Δ_u); (5) ultimate force (F_u); and, (6) ductility (μ). Moreover, the lateral behavior of MLSSWs is compared with that of bare SSWs used in this study as reference.

4.1 CONNECTION-LEVEL TESTS

The specimens were inspected after each cyclic test in order to evaluate typical failure modes. On the nailed OSB-to-frame connection (i.e., test group A) two failure modes were identified: (i) excessive bend in the nail leading to shearing-off of the fastener; and (ii) pull out or pull-through of the nail from the OSB-to-frame joint, leading to detachment of the OSB. In both cases, crushing in the wood and OSB panel and fiber tear in the OSB panel were observed. On the screwed 1-ply type X GWB+OSB-to-frame connection (i.e., test group B) two failure modes were identified: (i) excessive bend in the screw leading to shearing-off of the fastener; and (ii) pull-through of the screw from the 1-ply type X GWB+OSB-to-frame joint, leading to detachment of the GWB and OSB sheathing. In both cases, crushing in the wood and panels and tearing in OSB and GWB panels were likewise observed. Finally, on the screwed 2-ply type X GWB+OSB-to-frame connection (i.e., test group C), one failure mode was identified: (i) excessive bend in the screw leading to shearing-off of the fastener. Wood crushing and tearing in OSB and GWB panels were observed.

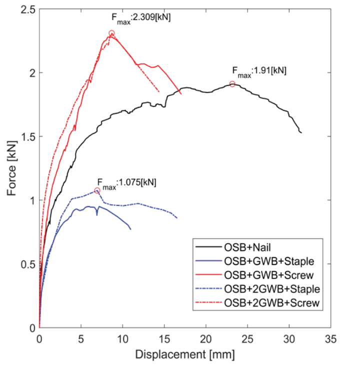

Figure 6: *Monotonic response of connection-level groups*

Table 2: *Engineering parameters from monotonic connection-level test results*

Test Group	K_e kN/mm	Δ_y mm	F_y kN	Δ_u mm	F_u kN	μ
A	0.91	1.87	1.69	31.51	1.53	16.9
B	0.91	2.15	1.94	17.05	1.83	7.9
C	1.89	1.01	1.91	14.39	1.85	14.3

The monotonic force-displacement test response for all the tested groups is presented in Figure 6, in which the reported displacement is the differential slip between the wood frame and the multi-layer sheathing, and the force is that taken by only one fastener along a single shear plane. In specimens with two Type X GWB, two screws are needed (one for each GWB), but results reported in this section refer to either one nail (for bare connections) or one screw, regardless of the number of Type X GWB. This makes possible a direct evaluation of the use of only one fastener in different configurations. Six engineering parameters are summarized in Table 2, where the Equivalent Energy Elastic-Plastic (EEEP) [22] approach was used to estimate the parameters according to ASTM E2126-19 [16]. Monotonic test results indicate that connections A (OSB+nail) and B (OSB+Type X GWB+screw) exhibited almost the same elastic stiffness, even though connection B has multiple layers of sheathing. In contrast, connection C (OSB+(2)Type X GWB+screw) was about twice stiffer than connections A and B. Results for connections A and C were consistent with the analytical stiffness expressions reported in [23]. However, connection B exhibited smaller stiffness than the analytical prediction, attributable to installation defects that tend to leave a gap between the sheathing layer and the wood-frame due to difficulties in screwing throughout the finish layers. Regarding capacity (strength), the screwed connections B and C exhibited stronger capacity than the nailed connection A. The screwed connection takes advantage of the axial capacity of the fastener, whereas the nailed connection is easily pulled out. From a ductility point of view, connection A performs better than all other GWB-sheathed connections. Ductilities of connections B and C were expected because screws are typically less ductile than nails and the reinforcing effect of the Type X GWB produced a more prominent strength degradation and a reduction of the inelastic ultimate displacements. However, the behavior of connection C is similar to the one reported in concrete-to-wood hybrid connections [24] in terms of elastic stiffness (i.e., elastic stiffness is almost twice that of connection B), but the peak strength and the ultimate displacement are similar to the ones of connection B. That is why the ductility of connection C is 80% higher than that of connection B. Likewise, it was found that even when all connections showed comparable yielding displacements of about 1-2 mm, the ultimate displacement of the bare connection A was about twice larger than that of the other connections, indicating that GWB-sheathed connections can clearly undergo lesser inelastic displacements.

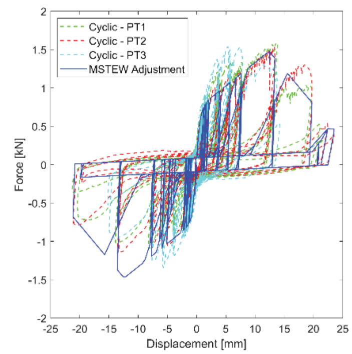

Figure 7: *Force-displacement response of test group A (i.e., nailed OSB-to-frame)*

The cyclic force-displacement test response for all the tested connections is presented in Figures 7 to 9. Again, results are expressed in terms of differential slip per fastener/shear plane. Cyclic test results for all tested connections depict a strong pinching effect due to wood frame and OSB crushing at the shear planes. Moreover, an abrupt strength degradation after repeated cycles at the same target displacement was found in all tests. A markedly asymmetric hysteretic response was found for bare OSB connections (group A), which was not consistent with previous findings [2, 25-27]. This asymmetric response could be attributed to: (1) the threaded portion of the nail is located at its end rather than distributed along the whole length (as in previously reported tests [2]), which would affect the rope effect in the connection [25]; and/or (2) an installation effect (i.e., the use of a pneumatic nail gun). This asymmetric response behavior of full-scale MLSSWs should be analyzed with more detail in the future. Apart from the response asymmetry, cyclic test results were consistent with monotonic test results: larger strength and stiffness of screwed connections, and a significantly larger ultimate displacement capacity of nailed connections.

https://doi.org/10.52202/069179-0282

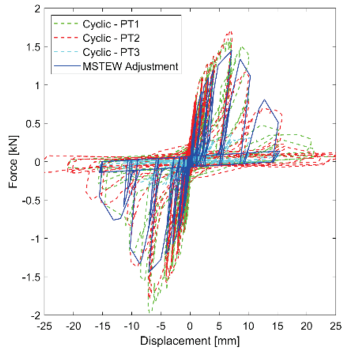

Figure 8: *Force-displacement response of test group B (i.e., screwed one layer Type X GWB+OSB-to-frame).*

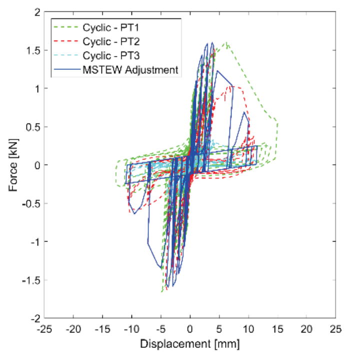

Figure 9: *Force-displacement response of test group C (i.e., screwed two layers Type X GWB+OSB-to-frame).*

4.2 ASSEMBLY-LEVEL TESTS

The wall specimens were inspected after each test in order to evaluate typical failure modes. In all four specimens (all of them had a 2-Type X GWB screwed configuration) five failure modes were identified: (i) pulling out of the nails and screws; (ii) pulling out of the nail and screw heads through the OSB or Type X GWB panels; (iii) shear-off of nails and screws due to excessive fastener bending; (iv) local embedding failure (crushing) in the OSB and Type X GWB panels attributable to an excessive stress concentration around the fastener; and, (v) detachment (out-of-plane unsheathing) of the OSB and GWB panels from the wood-frame because of failure of the fasteners (i.e. nails and screws). In all cases, excessive and moderate crushing in the wood and in the OSB and Type X GWB panels were observed, respectively. The fasteners failure was initiated at the center studs of the walls and propagated to the edge of the walls at the final stages of the loading protocol. This phenomenon is consistent with findings of previous researchers for continuous rod hold-downs [3], and can be explained by the concentration of fasteners in end studs around the continuous rod, which typically initiates failure at interior sheathing edges.

It is remarkable that double shear failure, pull-out, and pull-through of fasteners, along with local embedding of sheathing, were found as failure modes. However, there was no evidence of shear Type X GWB failure, and apparently, there was no reduction of the shear wall racking deformation capacity. Typically, the failure of non-structural finishes has been a cornerstone in restraining the design inter-story drift limit because it is commonly thought that it has much less deformation capacity and is more brittle than OSB. In MLSSWs, however, there was neither evidence of GWB failure nor shortening of the deformation capacity. This behavior is attributed to the fact that OSB sheathing offers protection to GWB [11], preventing brittle failure modes if OSB and GWB are located on both sides of the frame (i.e., the GWB always has a protective OSB layer beneath. Moreover, failure of the nails and detachment of the OSB occurred only at the ultimate stages of the loading protocol because of the minimal reinforcing effect of the GWB after the general failure of the GWB-screwed connections.

Figure 10: *Main failure modes observed in MLSSW specimens: (1) nail and screw failure pattern; (2) and (3) failure of the screwed* Type X GWB*+OSB-to-wood frame connection (in orange) for the 2nd layer and 1st layer, respectively; (4) and (6) pulling out of screws and nails; (5) and (9) local failure of the* Type X GWB *panels and OSB, respectively; (7) pulling through of nails and screws;(8) sheathing layers detachment from the wood-frames; and, (10) double shear failure of the screws.*

The wood frame showed moderate to low damage in all cases. Damage was concentrated mainly on the double central (interior) stud of the specimens and its connection to the top and bottom double plate, and was not as excessive as observed in previous tests on specimens with denser nailing patterns of 50 mm [3]. At the final stages of the loading protocol, detachment between the end studs located at the edge of the specimen and the bottom double plate was observed due to failure of the nailed OSB-to-bottom plate connection.

As expected, the rocking restraint system showed no damage in all the wall tests (it was designed to behave elastically even at the peak strength of the MLSSW specimens). The top-bearing steel plates were not damaged, as no crushing into the OSB of the collector beam was observed. This was also attributable to the overstrength factor used to design the specimens, which led to predominant nail and screw ductile failures.

The backbone hysteretic curves of the four MLSSW specimens (CT-MLSSW-0i, all comprising 2 screwed Type X GWB) and the control wall (CT-100-38, only with bare OSB without any GWB sheathing) are shown in Figure 10. The reported displacement is the effective displacement of the wall measured at the collector axis where the actuator was located. The effective displacement is the measured lateral displacement at the collector of the wall minus the displacement measured at the specimen-to-reaction beam relative to the reaction beam-to-strong floor. The overall shape of the hysteresis loops was consistent with that reported in previous research [3, 4, 28]. The MLSSW specimens showed elastic response up to a drift of about 1.0%, and then a nonlinear response was observed, attributable to the multilayer sheathing-to-wood frame connection. After the specimens reached the peak strength, progressive and smooth strength and stiffness degradation was found. As expected, high redundancy was evident in the specimens because of the multiple screwed and nailed connections at multiple layers, resulting in high drift levels with no brittle failures. Hence, as the lateral behavior of the MLSSWs was governed by the connection-level response, the MLSSW hysteresis was markedly pinched because of the non-reversible crushing effect of the fasteners (i.e., nails and screws) over the wood-frame components, which leads to a gap between the wood and the fasteners.

The engineering parameters of the backbone curves were estimated according to the Equivalent Energy Elastic-Plastic (EEEP) approach [22] per ASTM E2126-19 [16]. The resulting mean values are summarized in Table 3. All the shear walls tested in this research had a panel edge nail spacing of 100 mm and an anchoring rod of 38 mm in diameter.

Table 3. *Engineering parameters from cyclic MLSSW and control wall test results*

Test Group	K_e [kN/mm]	Δ_y [mm]	F_y [kN]	Δ_u [mm]	F_u [kN]	F_{peak} [kN]	μ
MLSSW	5.9	28.6	157.5	93.6	139.6	174.5	3.5
CT-100-38*	3.8	15.4	58.6	90.2	54.0	67.5	5.9

* CT -NP-AD: CT = cyclic test; NP = nail spacing pattern (i.e., 100 = 100[mm]); AD = anchorage diameter (i.e., 38 = 38[mm]).

The MLSSW specimens showed a mean peak strength of 174.5 kN, which is up to 160% higher (i.e., almost 3 times) than the peak strength (67.5 kN) observed in the equivalent bare strong shear wall specimen (CT-100-38), see Table 3. These surprising results confirm that double sheathing and screwing Type X GWB in timber shear walls can make an enormous structural difference, as it may have an even stronger influence than denser nailing patterns or stronger anchorages (these two parameters are currently among the most important design parameters to increase the capacity of timber shear walls). This enormous increase is thought to be generated not only by the "parallel spring" action of the screws but also because of the axial strength of the screws and their axial sheathing fixing may also reinforce and benefit the nailed OSB-to-wood frame connection, which is believed to control the strength of bare shear walls. This is related to the fact that in MLSSWs the evident pulling-out of OSB-to-wood frame connections took place at the final stage of the testing protocol, whereas in the CT-100-38 specimen the same phenomenon started right after the peak strength was reached.

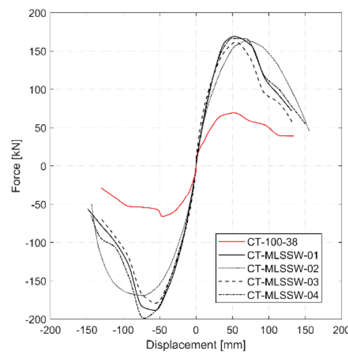

Figure 10: *Comparison between backbone curves of all tested specimens.*

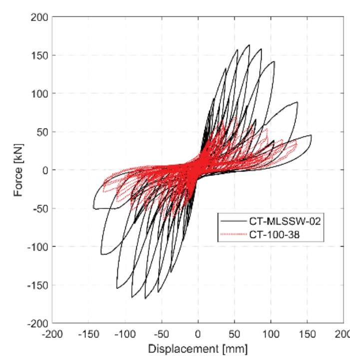

Figure 10: *Comparison between force-displacement hysteretic response of MLSSW sample CT-MLSSW-02 and control wall (i.e., bare strong shear wall) CT-100-38.*

The elastic stiffness of the MLSSWs, CT-100-38 and other research specimens are reported in Table 5. MLSSW specimens show a mean elastic stiffness of 5.854 kN/mm, which is up to 53% higher than the elastic stiffness observed in the control specimen CT-100-38 (3.821 kN/mm). All specimens show a clear stiffness degradation as the lateral drift increases, presenting a residual stiffness between 0.5 kN/mm to 1.5 kN/mm. At 0.1% to 4% lateral drift the MLSSWs exhibit the highest levels of secant stiffness, and the degradation is linear rather than quadratic as observed in bare shear walls. Finally, at lateral drifts smaller than 0.1%, specimen CT-100-38 present a secant stiffness that is 99% smaller than that of the MLSSWs. These results confirm that the actual

https://doi.org/10.52202/069179-0282

elastic and secant stiffness of MLSSWs are greater than those based on the assumption of bare shear wall.

In bare wood-frame strong shear walls the ductility is governed mainly by the nailed sheathing-to-frame connection. MLSSWs, on the other hand, have multiple sheathing-to-frame connections and each of these has a different ductility level, hence the overall ductility of MLSSWs depends on the combined effect of all connections. Values of ductility μ are reported in Table 3. They were computed according to ASTM E2126-19 [16] as the ratio of ultimate to yield displacement: $\mu = \Delta_u/\Delta_y$. MLSSW specimens show a mean ductility of 3.5, which is 42% smaller than the observed ductility in the control specimen CT-100-38 (5.929). This reduction in ductility could be attributed to the screwed $1^{st}/2^{nd}$ layer GWB+OSB-to-frame connection, which contributes mainly to stiffness and strength rather than to deformation capacity because screws tend to fail first as the lateral drift of the MLSSW increases. In other words, the screwed connections themselves are less ductile than the nailed connections. However, once the screwed connections fail in MLSSWs, a rapid failure of the nails is expected as they are unable to take all the load previously taken by the screws. Therefore, the ductility of the screws (rather than that of the nails) is thought to govern the MLSSW ductility as was reported by previous researchers [3, 4], the nail spacing in OSB-to-frame connection controls the ductility of bare shear wall.

5 CONCLUSIONS

Findings of this investigation reinforce the idea of taking advantage of the effect of the finishes layer on the lateral response of MLSSWs to achieve a cost-effective structural design. Such approach would cast a paramount criterion in the context of targeting earthquake-prone tall timber buildings, and traditional design practices that ignore the contribution of GWB finish layers should be discouraged. Further research is needed to elucidate the potential contributions of the finishes layer on seismic performance factor of light-frame timber buildings.

ACKNOWLEDGEMENTS

Financial support was provided by CONICYT (Doctorado Nacional 2018–21180074), by VRI-UC, by ANID BASAL FB210015 (CENAMAD), and by the Research Center for Integrated Disaster Risk Management (CIGIDEN) ANID FONDAP 1522A0005.

REFERENCES

[1] Kircher, C. A., Reitherman, R. K., Whitman, R. V., & Arnold, C. (1997). Estimation of earthquake losses to buildings. *Earthquake Spectra*, 13(4), 703-720.

[2] Estrella, X., Guindos, P., Almazán, J. L., & Malek, S. (2020). Efficient nonlinear modeling of strong wood frame shear walls for mid-rise buildings. *Engineering Structures*, 215, 110670.

[3] Estrella, X., Malek, S., Almazán, J. L., Guindos, P., & Santa María, H. (2021). Experimental study of the effects of continuous rod hold-down anchorages on the cyclic response of wood frame shear walls. *Engineering Structures*, 230, 111641.

[4] Guíñez, F., Santa María, H., & Almazán, J. L. (2019). Monotonic and cyclic behaviour of wood frame shear walls for mid-height timber buildings. *Engineering Structures*, 189, 100-110.

[5] Wolfe, R. W. (1983). *Contribution of gypsum wall board to racking resistance of light-frame walls* (Vol. 439). US Department of Agriculture, Forest Service, Forest Products Laboratory, Madison, USA.

[6] Etex Group (2022). *Technical Guidelines for Type X Gypsum Wallboard*. Gyplac. Accessed on: May 26, 2022. [Online]. Available: www.gyplac.com/es-cl/productos/gyplac-portafolio/placas-de-yeso/yeso-carton-resistente-fuego/

[7] Rahmanian, I. (2011). Thermal and mechanical properties of gypsum boards and their influences on fire resistance of gypsum board based systems. *Ph.D. Thesis*, The University of Manchester, Manchester, UK.

[8] Cramer, S. M., Friday, O. M., White, R. H., & Sriprutkiat, G. (2003). Mechanical properties of gypsum board at elevated temperatures. *Fire and Materials 2003 Conference*, San Francisco, USA.

[9] Uang, C. M., & Gatto, K. (2003). Effects of finish materials and dynamic loading on the cyclic response of woodframe shearwalls. *Journal of Structural Engineering*, 129(10), 1394-1402.

[10] Filiatrault, A., Christovasilis, I. P., Wanitkorkul, A., & van de Lindt, J. W. (2010). Experimental seismic response of a full-scale light-frame wood building. *Journal of structural engineering*, 136(3), 246-254.

[11] Goodall, S. I., & Gupta, R. (2011). Improving the performance of gypsum wallboard in wood frame shear walls. *Journal of Performance of Constructed Facilities*, 25(4), 287-298.

[12] Line, P., Hohbach, D., & Waltz, N. (2021). Gypsum wallboard finish contribution to the racking strength of wood-frame shear walls with representative multi-story details. *World Conference on Timber Engineering*, Santiago, Chile.

[13] Tyrell, T. (2007). *Full-Scale Three Dimensional Wood Structure Test Project*, Gilb Research Laboratory, Simpson Strong- Tie Company. Detailed information available at: http://www.strongtie.com/news/industry/wall-bracing/3Dtest/index.html. Accessed 15 October 2008.

[14] INN (2014). *Madera - Construcciones en Madera - Cálculo NCh 1198*. Instituto Nacional de Normalización, Santiago, Chile. (in Spanish)

[15] ASTM (2006). *Standard Practice for Static Load Test for Shear Resistance of Framed Walls for Buildings ASTM E564-06*. ASTM International, West Conshohocken, USA.

[16] ASTM (2019). *Standard Test Methods for Cyclic (Reversed) Load Test for Shear Resistance of Vertical Elements of the Lateral Force Resisting Systems for Buildings ASTM E2126-19*. ASTM International, West Conshohocken, USA.

[17] Krawinkler, H., Parisi, F., Ibarra, L., Ayoub, A., & Medina, R. (2001). Development of a testing protocol for woodframe structures. *CUREE Publication No. W-02*, Consortium of Universities for Research in Earthquake Engineering, Richmond, USA.

[18] INN (2009). *Diseño Sísmico de Edificios NCh 433 Of1996 Mod. 2009*. Instituto Nacional de Normalización, Santiago, Chile. (in Spanish)

[19] APA (2012). *Panel Design Specification*. APA - The Engineering Wood Association, Tacoma, USA.

[20] AWC (2021). *Special Design Provisions for Wind and Seismic ANSI/AWC SDPWS-2021*. American Wood Council, Leesburg, USA.

[21] ASTM (2016). *Standard Test Methods for Shear Modulus of Wood-Based Structural Panels ASTM E3044-16*. ASTM International, West Conshohocken, USA.

[22] Foliente, G. C. (1996). Issues in seismic performance testing and evaluation of timber structural systems. *International Wood Engineering Conference*, New Orleans, USA.

[23] CEN (2008). *EN 1995-1-1:2004+AC:2006+A1:2008 Eurocode 5 - Design of Timber Structures - Part 1-1: General - Common Rules and Rules for Buildings*. European Committee for Standardization, Brussels, Belgium.

[24] Carrero, T., Montaño, J., Santa-María, H., & Guindos, P. (2020). Static and dynamic performance of direct hybrid connections of cross-laminated timber with steel, concrete and laminated strand lumber composites. *Latin American Journal of Solids and Structures*, 17(4), e279.

[25] Fonseca, F. S., Rose, S. K., & Campbell, S. H. (2002). Nail, wood screw, and staple fastener connections. *CUREE Publication No. W-16*, Consortium of Universities for Research in Earthquake Engineering, Richmond, USA.

[26] Sartori, T., & Tomasi, R. (2013). Experimental investigation on sheathing-to-framing connections in wood shear walls. *Engineering Structures*, 56, 2197-2205.

[27] Dolan, J. D., & Madsen, B. (1992). Monotonic and cyclic nail connection tests. *Canadian Journal of Civil Engineering*, 19(1), 97-104.

[28] Pei, S., & Van de Lindt, J. W. (2009). Coupled shear-bending formulation for seismic analysis of stacked wood shear wall systems. *Earthquake Engineering & Structural Dynamics*, 38(14), 1631-1647.

[29] BSI (2008). *BS EN 14592:2008+A1:2012 Timber Structures - Dowel-type Fasteners - Requirements*. British Standard Institution, London, UK.

https://doi.org/10.52202/069179-0282