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# Seismic Performance of High-capacity Light Wood Frame Shear Walls with Multiple Rows of Nails

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## ABSTRACT

With the permission of up to 6-storey light wood-frame construction in National Building Code of Canada, stronger shear wall systems have been facing higher demands, especially for mid-rise wood-frame buildings located in high seismic zones. In collaboration with FPInnovations, a new high-capacity shear wall system with two or three rows of nails was developed. This paper presents the results of shear wall test programs that were conducted in 2020, 2021 and 2022. The tested shear walls, 30 walls in total, cover a range of configurations including different sheathing thicknesses, nail sizes and spacings, stud dimensions and numbers, in which standard shear walls were also tested as a reference. Test results showed that shear walls with two and three rows of nails have approximately two- and three-times lateral load resistance of a standard shear wall with the same sheathing thickness, nail diameter and nail spacing, respectively. The initial stiffness and ultimate displacement of the high-capacity shear wall are also greater than the comparable standard shear wall. Seismic equivalency of high-capacity shear walls to standard shear walls was evaluated in accordance with ASTM D7989. It was found that the design resistance of shear walls with two and three rows of nails is less than two and three times the resistance of a standard shear wall in order to meet the ductility criteria in ASTM D7989. Further study should be carried out to improve the ductility of shear walls with two and three rows of nails.

Keywords: High-capacity shear walls, two and three rows of nails, reversed cyclic load, light wood frame construction, seismic performance.

## INTRODUCTION

Light wood frame shear wall systems have been widely used in the construction of multi-story residential and commercial wood buildings in North America. Consisting of wood framing members, wood-based sheathing materials such as plywood or oriented strand board (OSB), and fasteners such as nails and bolts, wood frame shear walls are the main component in resisting lateral loads caused by wind or seismic loads. The lateral stiffness of shear walls is mostly provided by the sheathing panels, while the nails connecting sheathing to framing members contribute to the lateral resistance and energy dissipation of shear walls. Due to the high strength-to-weight ratio of wood and the ductility of fasteners, light wood frame buildings generally performed well in earthquakes [1]. The height limit of wood frame buildings has been increased from 4 to 6 stories in the provincial and national building codes of Canada (NBCC) [2,3], and the seismic design spectra in the 2015 and 2020 NBCC [2,3] have been increased substantially for all site classes. The combination of the effect has brought an increasing demand for higher lateral load resisting systems in mid-rise wood frame buildings, especially in high seismic zones.

Significant amount of experimental and numerical studies have been conducted on wood frame shear walls [4,5]. Efforts have also been made to improve the lateral resistance of shear walls, such as the use of adhesive between the wood frame and sheathing panels [6], shear walls with oversized sheathing panels and larger aspect ratios [7,8]. New shear wall systems have also been developed, such as midply shear wall system which sandwiches sheathing panels with light wood frames with the studs and plates rotated by 90˚ relative to those in a standard shear wall [9–11], and "mid-ply truss wall" in which nail-connected wood frames were replaced with metal-plate-connected wood trusses [12].

To respond to the demand of higher strength shear wall system, a new high-capacity shear wall system with multiple rows of nails along sheathing edges has been jointly developed by FPInnovations and the University of Victoria. Shear walls with two and three rows of nails along sheathing edges were designed and tested in a three-year period from 2020 to 2022, a total of 30 shear walls have been tested, including reference shear walls with one row of nails.

## MATERIAL AND METHODS

### Details of high-capacity shear walls

There have been 30 high-capacity shear walls with two or three rows of nails along sheathing edges and reference walls tested in the past three years [13,14]. Shear walls with two rows of nails were tested in 2020, and shear walls with three rows of nails were tested in 2021. Failure modes that are not common in regular shear walls were observed in high-capacity shear wall tests, such as separation of end studs from bottom plates, which led to reduction of ultimate displacement and ductility. Additional connections linking end studs and bottom plates were used in 2022 to prevent these failure modes. Table 1 summarizes the details of shear wall specimens that have been tested in the last three years.

*Table 1. Tested shear wall configurations*

| Year of testing | Wall # | Lumber dimension (mm) | OSB thickness (mm) | Nail size (mm) | Row of nails | Nail spacing along panel edges (mm) | Replicates |
|---|---|---|---|---|---|---|---|
| 2020 | 1 | 38 × 89 | 9.5 | 63.5 × 3.33 | 2 | 75 | 3 |
| | 2 | | 9.5 | 63.5 × 3.33 | 2 | 100 | 2 |
| | 3 | | 15 | 76 × 3.76 | 2 | 75 | 2 |
| | 4 | | 15 | 76 × 3.76 | 2 | 100 | 2 |
| | 3r | | 15 | 76 × 3.76 | 1 | 75 | 1 |
| 2021 | 1 | 38 × 140 | 11 | 63.5 × 3.33 | 3 | 100 | 2 |
| | 2 | | 11 | 63.5 × 3.33 | 3 | 75 | 3 |
| | 2r | | 11 | 63.5 × 3.33 | 1 | 75 | 1 |
| | 3 | | 15 | 76 × 3.76 | 3 | 100 | 3 |
| | 4 | | 15 | 76 × 3.76 | 3 | 75 | 2 |
| | 4r | | 15 | 76 × 3.76 | 1 | 75 | 1 |
| 2022 | 1 | 38 × 140 | 12 | 63.5 × 3.33 | 1 | 75 | 2 |
| | 2 | | 12 | 63.5 × 3.33 | 2 | 75 | 2 |
| | 3 | | 12 | 63.5 × 3.33 | 3 | 75 | 4 |

*Note: r = regular shear wall with only one row of nails along sheathing panel edges.*

The tested shear wall specimens were designed based on CSA O86 [15] with consideration of following design aspects: the compression capacity of the end studs, bearing capacity of top and bottom plates, shear strength of anchor bolts, tensile strength of tie-rods (hold-downs), and shear strength of the sheathing panels. The shear wall specimens are 2.4 m × 2.4 m in dimension, constructed with 2 × 4 (38 mm × 89 mm) or 2 × 6 (38 mm × 140 mm) No.2 and better grade Douglas Fir dimension lumber, and sheathed by 1.2 m × 2.4 m vertically placed performance rated OSB panels. Figure 1 shows the shear wall configuration. Lumbers were pre-sorted with densities within the range of 0.5 ± 0.03 (± 0.04 for 2021's tests). The pre-sorting aims to minimize variation between shear wall specimens. Relative density and moisture content were measured and calculated according to ASTM D2395 [16] and ASTM D7438 [17], respectively.

Sheathing panels were connected to framing members with either F1667 NLCMMS36 (63.5 mm in length × 3.33 mm in shank diameter × 7.14 mm in head diameter) or F1667 NLCMMS72 (76.2 mm in length × 3.76 mm in shank diameter × 7.92 mm in head diameter) power driven nails. Nails were spaced at 100 mm and 75 mm on center along panel perimeter, with a minimum edge distance of the outmost row of nails at 12.7 mm (1/2 in.) for 2020's tests and 9.5 mm (3/8 in.) for 2021 and 2022's tests. Minimum distance of 50 mm to the end of top and bottom plates and studs was used in 2020's tests, while a minimum distance of 75 mm was used for 2021 and 2022's tests. The detailed nailing pattern of shear wall with two and three rows of nails (tested in 2020 and 2021) is given in Figure 2. The framing members were connected using F1667 NLCMMS69 (76.2 mm in length × 3.05 mm in shank diameter × 7.18 mm in head diameter) power driven common nails, with two rows of nails spaced at 200 mm on center for built-up end studs, built-up interior end studs, and top and bottom plates. For built-up center studs, two rows of F1667 NLCMMS69 common nails spaced at 75 mm on center were used in 2020's tests, three rows of nails spaced at 100 mm on center was used in 2021 and 2022's tests to prevent the separation of center studs [14]. To prevent loading beam from restraining sheathing movement, one additional lumber plate was added on top of the wall. The number of end studs used in each specimen was determined based on engineering calculation in accordance with CSA O86 [15].

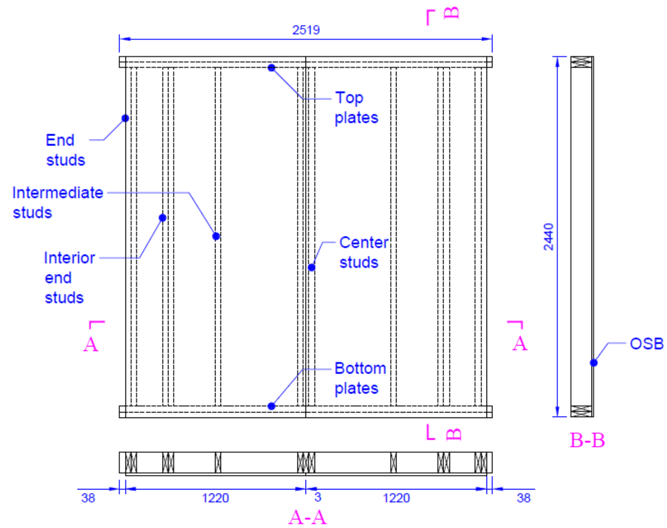
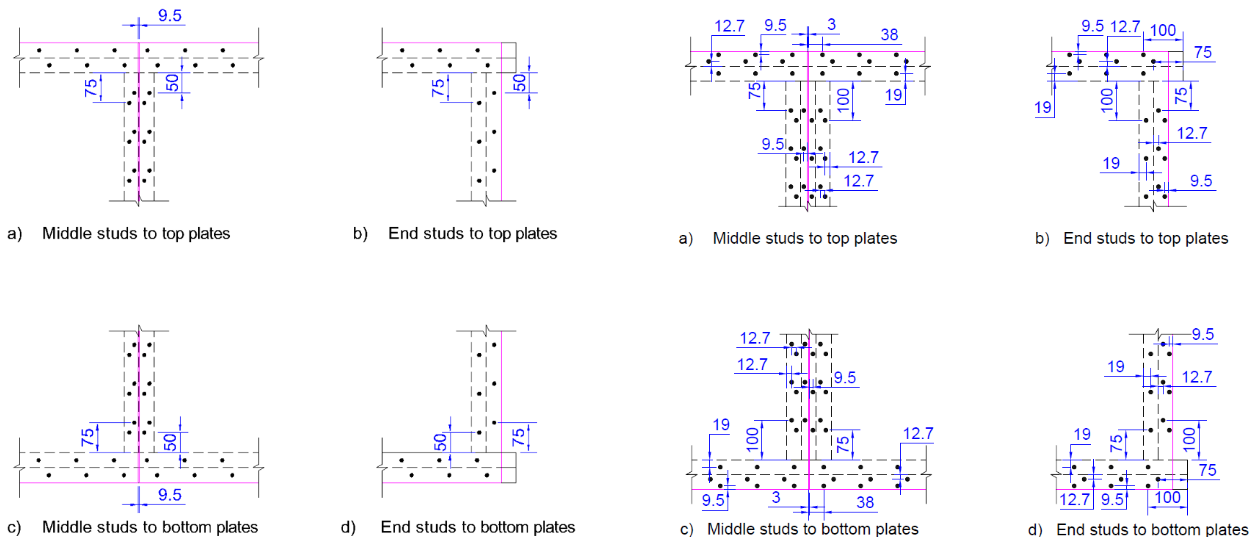



*Figure 1. Schematic of shear wall configuration (mm) [14].*




*(a)* *(b)*

*Figure 2. Nailing patterns (mm): (a) Nailing pattern of two rows of nails (2020) [13], (b) Nailing pattern of three rows of nails (2021) [14].*

To prevent the separation between end studs from bottom plates, two construction details were proposed in 2022's tests. The first type of detail was used in shear walls with two and three rows of nails (Table 1). L102 × 102 × 6.4 steel angles were installed at locations of hold-downs, where the vertical flange of the steel angle was connected to end studs with four SD9212 (63.5 mm × 3.33 mm) screws, as shown in Figure 3 (a). Vertical slots were made on the vertical flange to allow uplift of end studs. The horizontal flange of the steel angle functions as bearing plate to prevent bottom plate splitting. The second type of detail was only used in two shear walls with three rows of nails (Table 1). Three CS16 steel straps were wrapped around the bottom plates to tie the plates to the end studs. Two rows of F1667 NLCMMS36 (63.5 mm × 3.33 mm) nails were used to connect each strap to the front and back side of the end studs, as shown in Figure 3 (b)(c). In one of the specimens, three additional steel straps were also used to wrap the end studs to top plates. For shear walls with steel straps, 89 mm × 127 mm × 19 mm steel bearing plates were used at locations of hold-downs to prevent bottom plate splitting. In shear walls with two and three rows of nails, two A34 framing angles were used to connect the center studs to top and bottom plates with SD9112 (38.1 mm × 3.33 mm) screws to prevent separation of center studs from top and bottom plates.

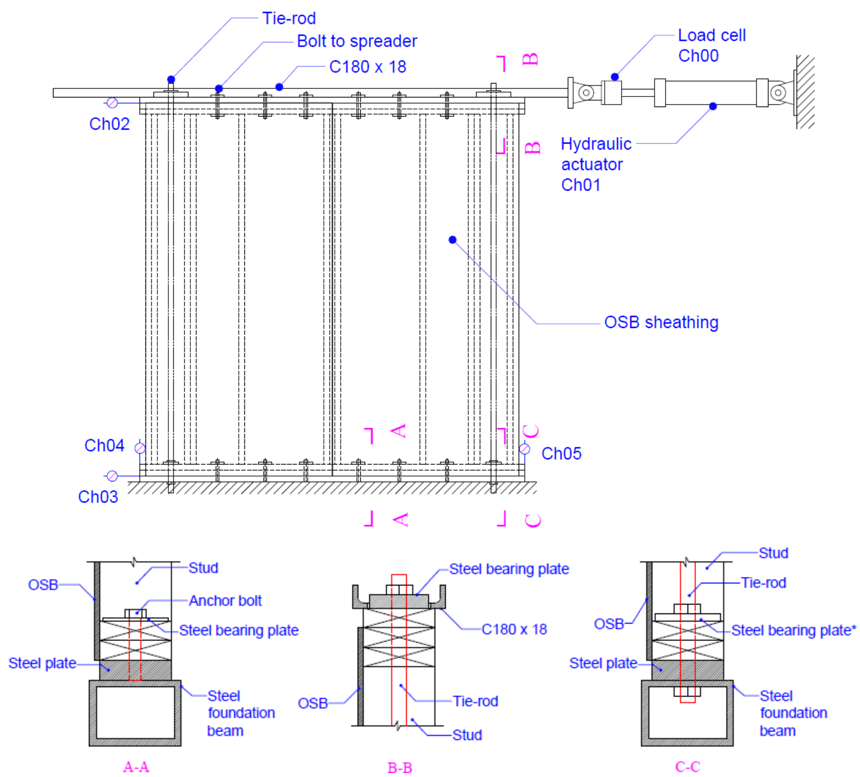
*(a)* *(b)* *(c)*

*Figure 3. New shear wall details: (a) Steel angle at locations of hold-down, (b) Steel straps at the front of the end studs, (c) Steel straps at the back of the end studs.*

**Testing set up and loading protocol**

The shear wall specimens were fabricated and installed according to ASTM E2126 [18]. Figure 4 shows the schematic test set up. For all shear wall specimens, continuous tie-rods (hold-downs) were used to resist overturning moment: in 2020, tie-rods with 25.4 mm (1 in.) diameter were used; in 2021, tie-rods with 28.6 mm (1-1/8 in.) diameter was used; in 2022, tie-rods with diameter of 15.9 mm (5/8 in.), 22.2 mm (7/8 in.) and 25.4 mm (1 in.) were used for shear walls with one, two and three rows of nails, respectively. Six anchor bolts with steel bearing plates were used to connect both top and bottom plates to load spreader beam (C180 × 18) and foundation steel beam, respectively. Anchor bolts with 16 mm diameter and 75 mm × 75 mm × 6 mm steel plates were used in 2020's tests, and 22.2 mm diameter anchor bolts with 127 mm × 127 mm × 6 mm steel plates were used in 2021 and 2022's tests. Steel bearing plates (165 mm in length × 114 mm in width × 25 mm in thickness) were used to hold the tie-rod against the top plates (Figure 4 B-B). The lateral load was applied through the steel load spreader beam to the top of the shear wall, as shown in Figure 4. No vertical load was applied on the wall specimens.

Reversed cyclic loading was applied on tested specimens, following the CUREE protocol (method C) in ASTM E2126 [18]. Figure 5 shows the loading protocol that was used for the test program. The reference displacement was taken as 63.5mm (2.5 in.). Each subsequent phase of the CUREE protocol consisted of a primary cycle with an increase in amplitude of α (= 0.5) over the previous primary cycle. A displacement rate of 7.6 mm per second was used for the reversed cyclic loading. The test was terminated when the load dropped by more than 20% of the maximum load.




**Steel bearing plates at locations of tie-rods on bottom plates were only installed in 2021 and 2022's tests.*

*Figure 4. Schematic test set up [14].*

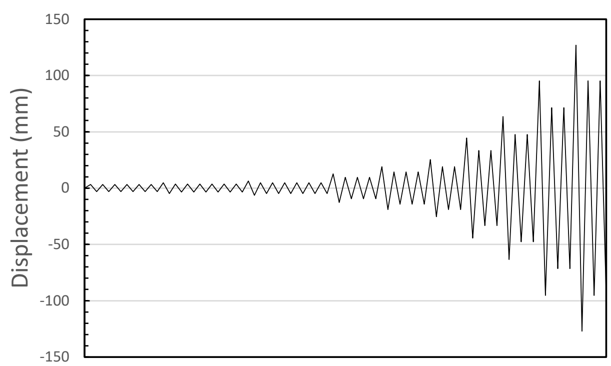



*Figure 5. Load protocol for shear wall tests [14].*

## RESULTS AND DISCUSSION

### Load displacement and mechanical properties

In general, shear walls with the same configuration (sheathing thickness, number of rows of nails, nail size and spacing) achieved similar lateral load capacity, while the ultimate displacement varied due to different failure modes. Figure 6 compared the load-displacement results of each year's high-capacity shear walls with their corresponding reference shear walls. In the legend of the chart, the first number indicates the wall configuration group shown in Table 1, the second number denotes the number of replicate.

The mechanical properties of the tested shear walls have been derived based on the equivalent-energy-elastic-plastic (EEEP) method described in ASTM E2126 [18]. Average of absolute positive and negative envelope curves were used to derive the EEEP curves. Table 2 summarizes the stiffness, yield load and yield displacement, peak load, ultimate displacement and energy dissipation capacity of shear wall specimens, where the secant stiffness $K_e$ is obtained between the origin and the point with 40% of maximum load on the ascending phase; $P_{yield}$ is the yield force based on EEEP method and $\Delta_{yield}$ is the corresponding displacement; the ultimate displacement $\Delta_u$ is in the post peak region where the load drops to 80% of the maximum load ($P_{peak}$)

or failure of the specimen happens; μ is the ductility ratio, defined as the ratio between ultimate displacement over the yield displacement; E is the total energy dissipated in the hysteresis loops. The average values of the mechanical properties of the same shear wall configuration with same sheathing thickness and nail size and spacing were shown in Table 2.

It can be seen that the lateral load resistance, stiffness, ultimate displacement and energy dissipation capacity of shear walls with multiple rows of nails are generally larger than that of standard shear wall with one row of nails, the peak load of the shear walls are proportional to the number of row of nails (Table 2). It is also found that, the highest lateral resistance for shear wall with two and three rows of nails are obtained in 2020 and 2021's tests with 15 mm sheathing thickness and 3.76 mm nail diameter. The ductility ratio of shear walls with multiple rows of nails are lower than the corresponding reference shear wall. This is due to the significantly increased yield displacement of the high-capacity walls (almost two- and three-times for shear walls with two and three rows of nails, respectively, in 2021 and 2022's tests) while the ultimate displacements were only increased by around 50% compared to the reference wall.

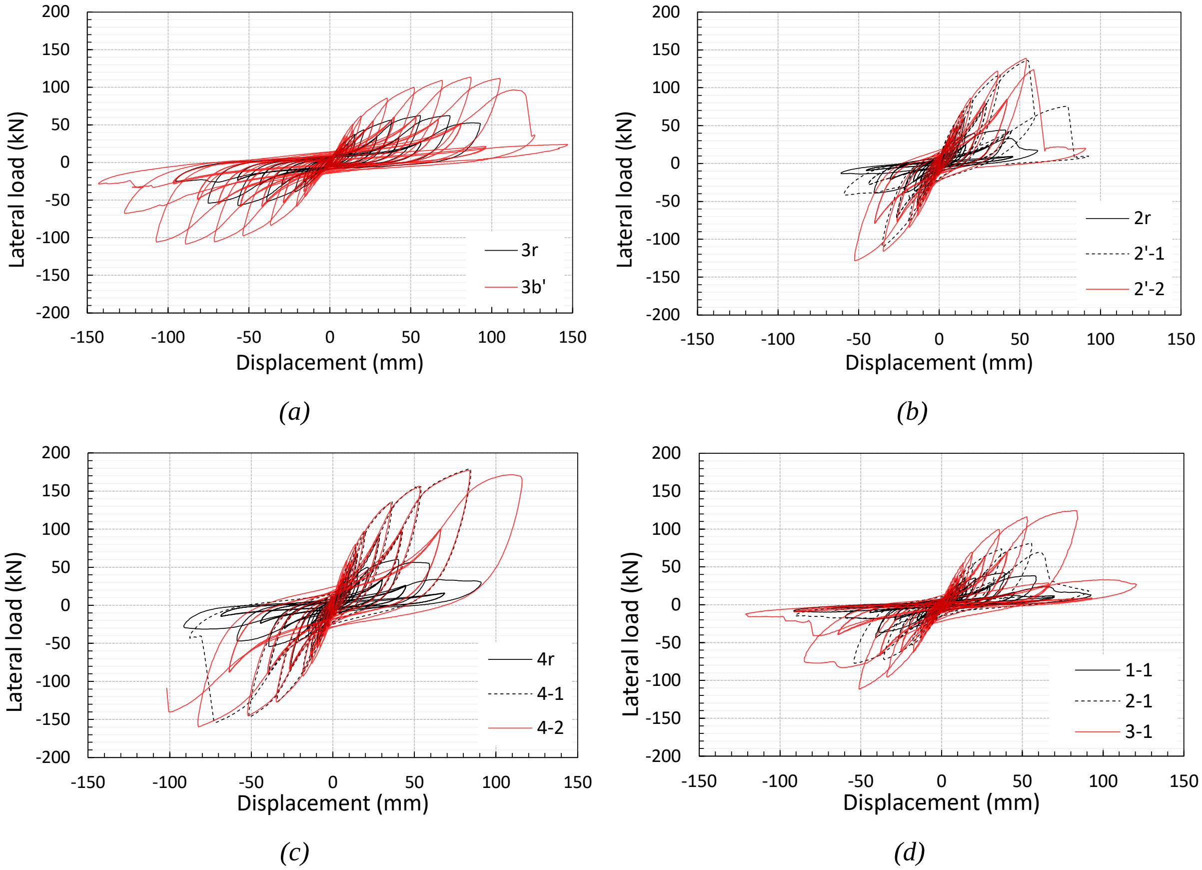



*Figure 6. Comparison of hysteresis loops of high-capacity shear walls and reference shear walls: (a) Wall 3 with two rows of nails and Wall 3r reference wall (2020) [13], (b) Wall 2 with three rows of nails and Wall 2r reference wall (2021) [14], (c) Wall 4 with three rows of nails and Wall 4r reference wall (2021) [14], (d) Wall 1, Wall 2, Wall 3 with one, two and three rows of nails, respectively (2022).*

*Table 2. Mechanical properties of the tested shear walls based on ASTM E2126 [18]*

| Year of testing | Wall # | $K_e$ (kN/mm) | $P_{yield}$ (kN) | $\Delta_{yield}$ (mm) | $P_{peak}$ (kN) | $\Delta_u$ (mm) | $\mu$ ($\Delta_u/\Delta_{yield}$) | E[1] (kN·m) |
|---|---|---|---|---|---|---|---|---|
| 2020 | 1 | 2.92 | 86.2 | 29.7 | 96.0 | 93.0 | 3.1 | 37.9 |
| | 2 | 2.64 | 67.0 | 26.0 | 74.1 | 83.5 | 3.3 | 26.2 |
| | 3 | 3.25 | 100.0 | 30.5 | 111.4 | 122.5 | 4.0 | 78.0 |
| | 4 | 2.82 | 76.2 | 26.5 | 85.6 | 101.5 | 3.8 | 39.0 |
| | 3r | 2.67 | 53.3 | 20.0 | 59.8 | 84.0 | 4.2 | 23.1 |
| 2021[2] | 1[3] | 3.50 | 89.7 | 25.6 | 99.0 | 96.6 | 3.8 | 40.5 |
| | 2[3] | 5.41 | 105.7 | 19.6 | 123.9 | 54.0 | 2.8 | 24.3 |
| | 2r | 5.29 | 37.4 | 7.1 | 41.7 | 46.9 | 6.6 | 8.7 |
| | 3 | 4.75 | 117.0 | 24.8 | 133.6 | 83.7 | 3.4 | 46.6 |
| | 4 | 5.90 | 146.1 | 24.9 | 167.6 | 93.0 | 3.8 | 51.0 |
| | 4r | 4.92 | 49.8 | 10.1 | 57.2 | 62.9 | 6.2 | 17.0 |
| 2022 | 1 | 3.88 | 31.5 | 8.2 | 35.7 | 51.5 | 6.5 | 7.7 |
| | 2 | 3.51 | 68.0 | 19.6 | 77.8 | 65.8 | 3.4 | 18.3 |
| | 3[4] | 4.52 | 98.7 | 21.9 | 113.5 | 72.9 | 3.3 | 27.7 |

*Note:*

*1. The total energy dissipation is calculated to the end of the first primary cycle that causes drop of the peak resistance below 80% of the maximum load. The two subsequent cycles with 75% amplitude are not included.*

*2. In 2021, for shear walls failed on the primary cycle, the test was stopped before primary cycle was completed. As a result, the dissipated energy is calculated to the last point of that cycle.*

*3. One of the specimens in Wall 1 and Wall 2 failed due to bottom plate splitting, one of the replicates of Wall 2 was loaded twice before it was tested to failure in the third time. The properties of these two specimens were not included in the table.*

*4. One of the specimens in Wall 3 was rebuilt during construction, thus the capacity was compromised, the properties of that specimen were not included in the table.*

**Failure modes**

The most common failure modes observed in standard shear walls are sheathing-to-framing nail joint, such as nail withdrawal from studs, nail head pull through sheathing, nail chip-out at sheathing edge, and nail fracture. For high-capacity shear walls, besides these failure modes, failure related to other components such as sheathing panels, studs and plates were also observed, which affected the ultimate displacement and ductility of the shear walls.

In 2020's tests, panel buckling was observed in several shear wall specimens with 9.5 mm thick sheathing (Figure 7 (a)), in which the bottom plate splitting, or stud separation from plates were also observed [13]. These failure modes occurred in the post peak load region, which affected the ultimate displacement of the shear walls. In shear walls with 15 mm thick sheathing, there was no buckling, however, center studs separation was observed, due to the nails connecting center studs having less resistance compared to the shear force induced by two rows sheathing-to-framing nails joints [13].

In 2021's tests, bottom plate splitting was observed in two of the tested specimens (Figure 7 (b)), which was due to the uplift force from sheathing-to-bottom plate nail joints while the bottom plates were not restrained from uplifting. This failure mode was prevented by installing bearing plates at locations of tie-rods in the rest of the shear walls. After that, the common failure mode in shear walls with three rows of nails was the separation of end studs from bottom plates, as shown in Figure 7 (c), which was caused by the out-of-plane moment developed at end studs due to nailing on one side of framing [14]. This type of failure is brittle and caused tests to stop early. Separation of end studs also led to sheathing rupture. Sheathing rupture caused by buckling and tension in the panel was observed in one of the shear walls with 11 mm thick sheathing. The separation of center studs was observed and prevented by adding additional row of nails and decreasing nail spacing for the built-up center studs.

In 2022, separation of end studs from bottom plates or separation of center studs was prevented with the additional connection installed between studs and bottom plates. However, sheathing buckling was observed in shear walls after the sheathing-to-framing nail joints along edges of the panel failed. Sheathing rupture under tension and compression occurred in one of the shear walls, as shown in Figure 7 (e). End and center stud splitting was also observed (Figure 7 (d)), caused by the failure of sheathing-to-framing nail joints along top or bottom plates, which changed the relative movement between sheathing and framing.

*(a)* *(b)* *(c)*

*(d)* *(e)*

*Figure 7. Failure modes: (a) Sheathing buckling [13], (b) Bottom plate splitting [14], (c) End stud separation from bottom plate [14], (d) End stud splitting, (e) Sheathing rupture under tension and compression.*

**Comparison of factored design value and analysis of seismic equivalency**

According to the CSA O86-19 [15], the factored shear resistance of a shear wall is governed by either the sheathing-to-framing nail joints or sheathing panel buckling, whichever is smaller. The factored shear resistance of the shear walls determined in accordance with O86 was compared with the test results. It was found that, although according to O86 design value, the resistance of shear walls with two and three rows of nails is governed by sheathing buckling, the resistance of tested walls was controlled by sheathing-to-framing nail joint resistance. Buckling only occurred after the peak resistance based on nail joints was reached, which indicates that the capacity of shear wall with multiple rows of nails is still governed by nail joint capacity. The design strength of shear wall due to sheathing buckling in CSA O86 [15] seems too conservative and need to be reviewed.

In CSA O86 [15], panel-through-thickness strengths usually do not need to be checked for regular shear walls, however, sheathing rupture under tension and compression was observed in shear walls with three rows of nails. It is suggested that for high-capacity shear walls, the shear and tensile strength of sheathing panels should also be checked [14].

The ASTM standard D7989 [19] establishes a method to demonstrate the equivalent seismic performance of an alternative shear wall system to standard shear walls. If the alternative shear wall meets the seismic equivalency parameters (SEPs) specified in D7989, as summarized in Table 3, then the seismic force modification factors for standard shear walls can be used.

*Table 3. SEPs for equivalency to light wood frame shear walls*

| Parameter | SEP requirement |
|---|---|
| Component overstrength | $2.5 \leq \frac{P_{peak}}{P_{ASD}} \leq 5.0$ |
| Drift capacity | $\Delta_u \geq 0.028h$ |
| Ductility | $\frac{\Delta_u}{\Delta_{ASD}} \geq 11$ |

*Note:*

$P_{peak}$ *peak load of the wall configuration*

$P_{ASD}$ *allowable design load of the wall configuration*

$\Delta_u$ *ultimate displacement*

$h$ *height of the shear wall*

$\Delta_{ASD}$ *displacement corresponding to the allowable design load of the wall*

The SEPs of the tested walls were analyzed, in which the allowable design strength of shear wall with two and three rows of nails, $P_{ASD}$, was assumed to be two and three times the allowable design strength of comparable standard shear wall according to AWC's Special Design Provisions for Wind and Seismic [20], multiplied by an adjustment factor.

As the component overstrength requirement and ductility requirement are dependent on the ASD design force and corresponding ASD displacement (Table 3), a lower adjusted $P_{ASD}$ value will lead to higher overstrength factor and higher ductility. For 2020's tests, adjustment factors of 0.9 can be used in the design of shear walls with two rows of nails [13]. For 2021 and 2022's tests, much lower adjustment factors have to be used for the design of high-capacity walls [14]. This was mainly caused by the brittle failure modes such as end studs separation, sheathing buckling etc. which led to lower ultimate displacement. Correspondingly, the drift capacity requirement was not met in several shear walls. Although the new design details in 2022's tests prevented separation of end studs from bottom plates, other brittle failure modes such as sheathing rupture and sheathing edge tear out have caused smaller ultimate displacement. It is believed that thicker panels can be used to prevent these panel related failure modes and therefore to improve the ductility.

## CONCLUSIONS

In this study, the three-year test program of high-capacity shear walls with two and three rows of nails along sheathing perimeters was summarized. A total of 30 wall specimens, including standard shear walls with one row of nails, were tested under reversed cyclic loading according to Method C of ASTM E2126 [18]. Seismic equivalency of shear walls with two and three rows of nails was evaluated according to ASTM D7989 [19]. The main findings from the test results are summarized as follows:

- The lateral load resistance of light wood frame shear walls are proportional to the number of rows of nails along the panel edges. The initial stiffness and ultimate displacement of shear walls with two and three rows of nails are generally larger than the comparable regular shear walls with one row of nails.
- New brittle failure modes such as bottom plate splitting, separation of studs from bottom plates were observed for high-capacity shear walls. These failure modes can be prevented through strengthening the stud to plate connections. However, other brittle failure modes were triggered which requires further research.
- Sheathing buckling and sheathing rupture were observed in several tested shear walls with thinner sheathing panels, which reduced the ultimate displacement, consequently resulting in lower ductility of shear walls. To prevent such failure modes, thicker panel is recommended.
- According to CSA O86-19 [15], the resistance of shear walls with two and three rows of nails is governed by sheathing buckling. However, test results showed that the lateral load resistance of high-capacity shear walls is still governed by nail joint resistance.
- Although shear walls with two and three rows of nails have two and three times lateral load capacity of a standard shear wall, the design resistance of these shear walls are smaller in order to meet the ductility requirements in ASTM D7989 [19].

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