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EARTHQUAKE PERFORMANCE OF TALL WOOD-FRAME WALLS

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ABSTRACT

A series of 13 quasi-static tests were performed to determine the behaviour of tall wood-frame shearwalls subjected to seismic loading. Tall walls tested were 4.9 m x 4.9 m in size and included a variety of sheathing-to-stud nailed connections, spruce-pine-fir (SPF) dimensional lumber and laminated strand lumber (LSL) studs and blocking, various stud spacing, various stud-to-plate connections, and sheathing material and thickness. The research results showed that with efficient stud spacing, nailing pattern, stud to plate connection details, and appropriate sheathing thickness, both SPF and LSL studs are viable material options for tall walls. Walls that used LSL studs spaced 1220 mm on centre were able to withstand large lateral forces and dissipate high amounts of hysteretic energy. Tall walls with dimensional SPF lumber studs spaced 610 mm on centre, aside from being able to withstand large lateral forces, showed increased ability to sustain large deformations provided that a close nail spacing is used. Commercially available stud-to-plate connectors were found to be adequate in resisting the shear and uplift forces induced by earthquake loads.

Introduction

In North America, wood-frame construction utilizing dimension lumber has been in use since the early 19th century. There are many examples of houses built with this system that are more than 100 years old and still continue to perform their original function. Although the system has evolved and changed over time, wood-frame construction still remains simple in concept and well within the scope of the average builder. The investments in research on the performance of platform wood-frame construction under various loading conditions have paid off. Today more than ninety percent of North American homes are being constructed using this building method. Lately, efforts have been made to extend the use of this method to non-residential applications. Developments such as hotels, motels, low-rise commercial properties, and community centres are all benefiting from the advantages that wood-frame construction has to offer. The proportion of non-residential buildings constructed with wood, however, still remains relatively low compared to other construction materials such as steel, concrete or masonry. For above-mentioned applications, the wood-frame construction concept can be used with little modification from its residential version. That is not the case, however, for most industrial or commercial buildings. These buildings usually require larger open spaces and greater heights than other non-residential buildings and need walls that are usually referred to as tall walls. Tall wood-frame walls can be a viable alternative to the steel systems and concrete tilt-up construction currently being used in box-type non-residential construction in North America. This is especially true for big box buildings in seismic prone areas, where

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there are some concerns regarding the seismic performance of concrete tilt-up buildings.

Previous research in the field of wood-frame walls has focused on regular residential walls. Numerous experimental and analytical studies have been performed to quantify the response of residential walls under various loading conditions. Consequently, design guidelines for these walls have been developed and introduced in North American codes and standards. On the other hand, small amount of research has been undertaken to investigate the structural performance of tall walls. Changes in design philosophy, supported by experimental and analytical research results will help this wood-frame system to become a more attractive alternative. For these reasons, Forintek Canada Corp. in collaboration with the Department of Civil Engineering at the University of British Columbia has undertaken a research project on the structural performance of tall wood-frame walls. This paper deals with one aspect of this project: Experimental testing to determine the seismic behaviour of tall wood-frame walls.

Specification of the Tall Walls Tested

The objective of the research presented in this paper was to quantify the influence of different connection detailing, anchorage, sheathing thickness, nail length, framing material, framing configuration and blocking on the response of tall wood frame walls subjected to lateral loading. Tall shear walls 4.9 m x 4.9 m (16' x 16') in size were tested in the study with two distinctive wall series. The first series, referred to as the 600 series, consisted of five walls with spruce-pine-fir (SPF) lumber studs, while the second 700 series included eight walls with laminated strand lumber (LSL) studs (Table 1). Prior to testing, material properties for framing members were determined, including the modulus of elasticity (MOE) determined by vibration methodology according to ASTM standard D 6874 (ASTM 2003).

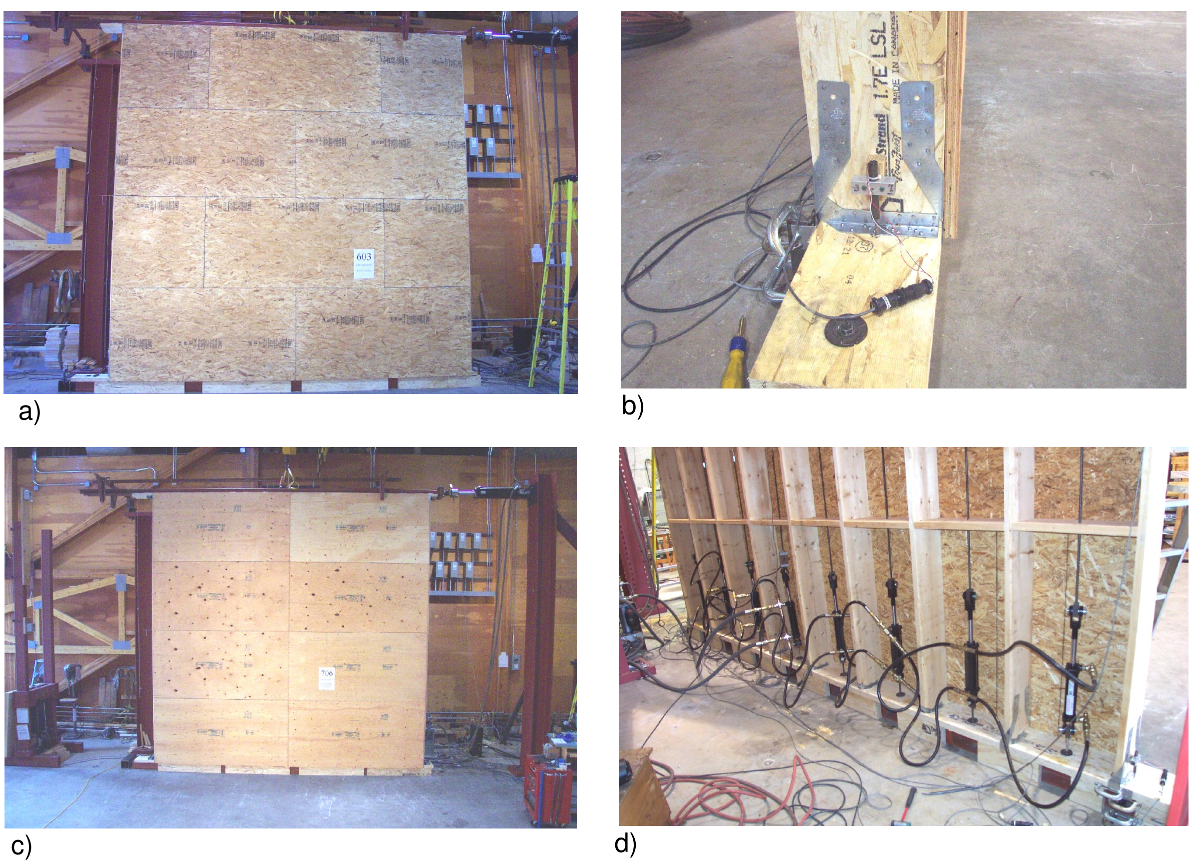


Figure 1. Tall wall with: a) Typical staggered panel orientation used; b) Dual H6 tie and hanger stud to plate connection; c) Stacked panel orientation (wall 706); d) Actuator assembly used for applying vertical load for walls 604 and 703.

Walls 601 and 701 were subjected to monotonic lateral loading, while all other walls were subjected to reversed cyclic loading. Walls 604 and 703 were subjected to vertical loading in addition to the lateral cyclic loading. Eight 13.3 kN hydraulic actuators spaced at 610 mm were placed inside the wall specimens to deliver the 20 kN/m of vertical loading (Figure 1d). Slots were cut through the blocking to allow for passage of the threaded rods connecting the actuators to the top and bottom plates.

The 600 wall series had 38 mm x 235 mm (2x10) No. 2 or better SPF studs and 1220 mm x 2440 mm OSB sheathing with thickness of 9.5 mm and 15.1 mm placed in staggered formation (Figure 1a). The studs were spaced at 610 mm on centre. The top and bottom plates consisted of 44 mm x 242 mm 1.7E LSL. The plate to stud connections consisted of two main components, Simpson Strong Tie LU28L joist hangers placed at every stud, and different configurations of Simpson H6 hurricane ties, which were the primary connector to resist the uplift forces. For wall 601, the H6 ties were placed on the top and bottom of all studs on the opposite side of the sheathing. Wall 602 used the same arrangement of ties on all studs except the last two studs on either end of the wall that had ties on both, the sheathed and non-sheathed side of the stud (Figure 1b). Walls 603 to 605 had the dual tie formation only on the end two studs on either side of the wall and the middle stud, while the remaining studs were connected without any H6 ties. The hangers and the ties were connected to the studs and plates by conventional 3.75 mm diameter, 38 mm long common nails. Walls 601 and 602 were sheathed with 9.5 mm OSB, while walls 603 to 605 were sheathed with 15.1 mm OSB. All 600 series walls used 2.5 mm diameter, 65 mm long spiral nails to connect the sheathing to the studs. SPF blocking was used in all 600 series walls spaced at 1220 mm on center. The blocking was attached to the studs by either three toenails, or end nailed with three 3.3 mm diameter, 83 mm long common nails. The test matrix for all walls included in the testing program is presented in Table 1.

Table 1. Test matrix used in the study.

Wall	Stud		Sheathing Thickness [mm] & Type	Nail Spacing and Properties				Load Protocol[4]
	Type	Spacing [mm]		Perimeter [mm]	Interior [mm]	Diameter [mm]	Length [mm]	
601	SPF	610	9.5 OSB	152	305	2.5	65	→
602	SPF	610	9.5 OSB	152	305	2.5	65	↔
603	SPF	610	15.1 OSB	152	305	2.5	65	↔
604	SPF	610	15.1 OSB	152	152	2.5	65	↔ **+** ↓
605	SPF	610	15.1 OSB	102	152	2.5	65	↔
701	1.5E LSL	1220	15.1 OSB	152	305	2.5	65	→
702	1.5E LSL	1220	15.1 OSB	152	305	2.5	65	↔
703	1.5E LSL	1220	15.1 OSB	152	305	2.5	65	↔ **+** ↓
704	1.7E LSL	1220	25.4 DFP	152	305	3.0	76	↔
705	1.7E LSL	1220	25.4 DFP	152	305	3.75	76	↔
706 [1]	1.7E LSL	2440	25.4 DFP	102	n/a	3.0	76	↔
707 [2]	1.7E LSL	1220	25.4 DFP	152	152	3.0	76	↔
708 [3]	1.5E LSL	1220	15.1 OSB	152	305	2.5	65	↔

[1] Sheathing placed in non-alternating pattern i.e. stacked formation; [2] Unblocked; [3] 15.5 mm thick gypsum wallboard added on the non-sheathed side. Screw spacing 203 mm c/c; [4] symbol → indicates monotonic loading, ↔ cyclic loading, ↔ + ↓ is cyclic plus vertical loading.

Both 1.5E and 1.7E LSL studs and blocking with cross section of 44 mm x 242 mm were used for the 700 series walls. Walls 701 to 703, and wall 708 used 15.1 mm thick OSB, while walls 704 to 707 used 25.4 mm thick Douglas fir plywood (DFP). As a result of the 25.4 mm thick sheathing, walls 704, 706 and 707 utilized 3.0 mm diameter, 76 mm long spiral nails for sheathing application, while wall 705 used 3.75 mm diameter, 76 mm long common nails. The stud to plate connections were consistent for all 700 series walls, with two H6 ties top and bottom being used on all studs, as well as Simpson HU9 hangers on all studs (Figure 1b). The hangers were screwed into the plates using 38 mm long No. 8 wood screws, and nailed into the studs using 3.75 mm diameter, 38 mm long common nails. Same nails were used for the H6 ties. Blocking was used in all walls except wall 707. The blocking material matched the stud material in all cases. Wall 701 used the same type of nailing scheme for the blocking as in the 600 series. Walls 702 to 705 and wall 708, however, used 6 nails per blocking connection, while wall 706 used 9 nails. All blocking used 3.3 mm diameter, 83 mm long common nails. Wall 708 also had 15.5 mm thick gypsum wallboard applied to the side opposite of that of the sheathing. The gypsum wallboards were attached using 41 mm long coarse thread drywall screws and then mudded. The configuration of the 700 wall series is also given in Table 1. All 700 series walls had scattered sheathing pattern (Figure 1a), except for the wall 706 that used stacked sheathing pattern (Figure 1c).

Test Setup and Loading Protocols

A reinforced hollow steel beam provided a foundation to which the specimens were bolted down, while a hollow steel bar bolted to the top plate was used as load spreader bar. The spreader bar had attachments that allowed for lateral guides to be used to ensure a steady and consistent unidirectional movement of the walls. For most of the walls, the bottom plate was bolted to the steel beam using 12.7 mm grade five bolts. Walls 706 to 708 used same diameter grade 8 bolts. In all cases, bolts were located on the centre line of the LSL plates. Bolted connections located outside the both ends of the wall were subjected to some changes during the testing, as more knowledge about wall behaviour and force levels that they attracted became available. The walls were subjected to monotonic or cyclic lateral loading using a 110 kN hydraulic actuator (Figure 1c).

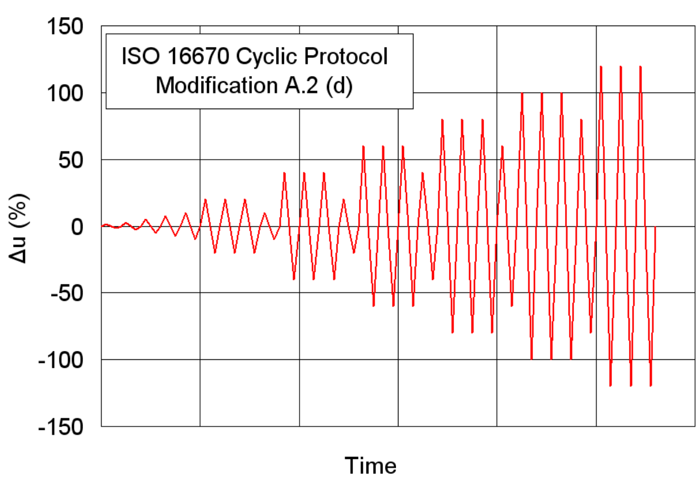


Figure 2. Modified ISO cyclic protocol 16670 used for the testing.

String displacement transducers were placed at the top and at mid-height of the walls to measure lateral deflection. Six other displacement transducers were used at the both end studs and the middle stud to measure plate-to-stud uplift. In addition, two displacement transducers were used to measure bottom plate slip and its uplift from the foundation. A modified form of the loading protocol according to the ISO 16670 standard was used during the testing (ISO, 2003). Such modifications are permitted under section A.2d of the standard. The protocol used during the testing is shown in Figure 2. Walls 601 and 701 were subjected to monotonic load pattern of 15.2 mm/min and were used to obtain the necessary parameters such as yield and ultimate displacements, later used for defining the cyclic protocol. The cycle pattern used was as follows: 1 cycle at 1.25%, 2.5%, 5%, 7.5%, and 10% of the ultimate displacement from either wall 601 or 701, and then a series of 3 cycles at 20%, 40%, 60%, 80%, 100%, and 120% of the ultimate displacements. After each set of 3 cycles, one cycle at the previous displacement level was applied.

Results and Discussion

Most important wall properties obtained from the experimental testing are presented in Table 2. The symbols used in the table are the following: P_{max} is the maximum load attained by the wall, Δ_{max} is the wall displacement at maximum load, Δ_{ult} is the wall displacement at 80 % of the maximum load after the peak load was reached, E is the hysteretic energy dissipated by the wall, and μ is the ductility of the wall ($\mu = \Delta_u / \Delta_y$). Ductility related parameters were determined according to the European Standard EN 12522 (EN 2001) using the first envelope curve of the wall response in the initial cyclic direction.

During the first test (wall 601), torsional failure of some of the studs was observed. The failure was attributed to the eccentricity that occurred between the plane of the sheathing and that of the single ties placed on the studs on the side opposite to the sheathing. Although this is not a problem in regular shearwalls, it should be taken into account when designing walls with increased stud depth. To mitigate this type of stud failure, certain studs on the remaining walls were connected with dual H6 ties, one on the back and one at the front of the stud. Wall 602 that did not utilize dual ties on the middle stud of the wall, also experienced a torsional stud failure at that particular stud. No torsional stud failures were observed in the rest of the walls tested. Longitudinal cracking in some non-tied studs in wall 605 was observed but just after the wall had completely failed.

Table 2. Tall wall properties obtained from quasi-static tests.

Wall	Δ_{max} [mm]	P_{max} [kN]	Δ_u [mm]	μ	E [kJ]
601	152.0	38.5	161.0	5.0	n/a
602	93.7	38.2	102.9	6.6	25.0
603	92.5	39.6	121.7	7.2	32.3
604	93.1	42.1	127.7	8.3	34.9
605	124.8	62.6	136.5	7.2	51.6
701	105	49.5	149	8.8	n/a
702	89.8	53.4	98.7	8.7	29.6
703	90.1	48.1	98.9	6.8	29.0
704	91.1	64.6	102.2	6.9	45.8
705	90.5	77.8	94.5	6.4	58.4
706	91.3	83.2	95.1	6.5	51.6
707	91.7	26.2	104.7	6.4	23.0
708*	59.3	64.1	76.7	7.7	32.5

* 15.5mm thick gypsum wallboard placed on non-sheathed side.

Walls 603 to 605, which had 15.1 mm thick sheathing, had much higher maximum load, ductility, maximum displacement, and energy dissipation than the walls with 9.5 mm sheathing (Table 2, Figure 3). In the walls with 15.1 mm thick sheathing, the connecting nails failed due to shear and fatigue, after being exposed to a number of alternating cycles. This failure mode was different than that experienced in case of the walls with thinner 9.5 mm sheathing, where nails typically pulled through the sheathing. It was found that, except for energy dissipation, the strength and ductility properties of all walls were proportionally related to the number of nails and the total cross-section area of the nails around the perimeter of the sheathing panels. In walls with SPF studs, ductility and energy dissipation were the properties most influenced by changes in nail spacing. Load-deformation hysteretic curves for walls 602, 603 and 605 are shown in Figures 3a, 3b, and 3c, respectively.

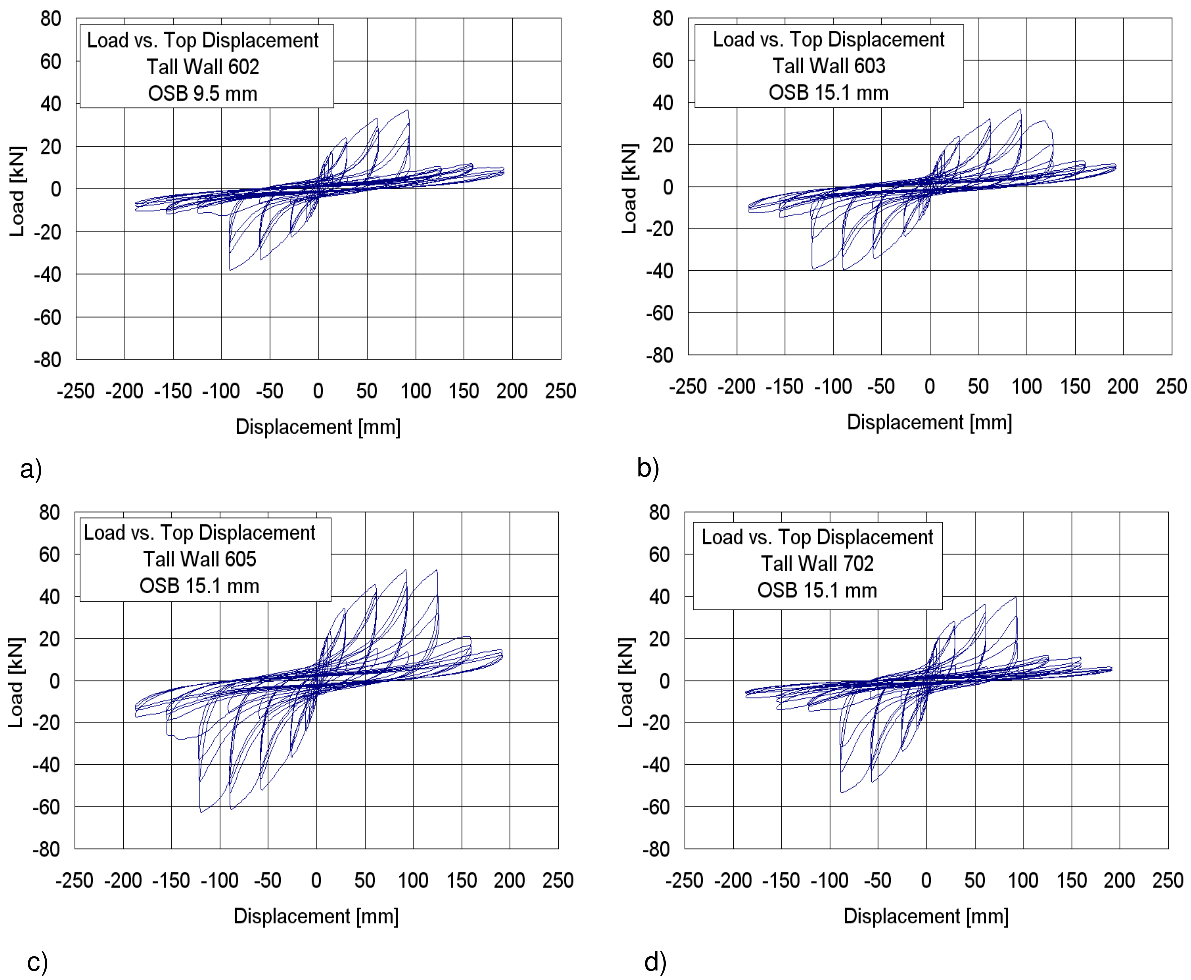
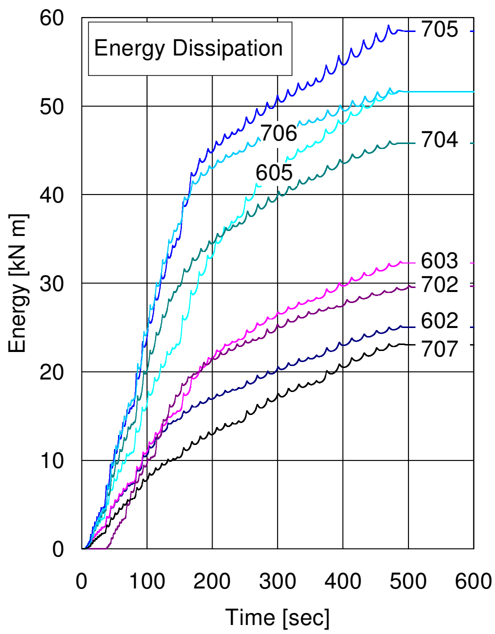


Figure 3. Hysteretic behaviour of walls: a) wall 602; b) wall 603; c) wall 605; d) wall 702.

The deformed shape of the SPF walls in elevation consisted of bending (bowing) of the studs in the lower half of the wall, while the upper half of the studs remained relatively straight during the testing (Figure 4b). There was total failure of almost all plate-to-sheathing fasteners along the bottom of the wall, and up the wall sides to a height of 1.2 m. The presence of vertical load in wall 604 resulted in a switch of the failure location to that along the horizontal blocking at the mid-height of the wall. Ductility and energy dissipation of this wall were also higher than those of the non-vertically loaded wall 603, which had similar properties (Figure 4a). This is in agreement with results reported from other experimental studies on regular shear walls (Dean and Shenton, 2005). In general, blocked tall walls with SPF studs spaced at 610 mm on centre were found to be effective lateral load-resisting systems. They were able to dissipate large amounts of hysteretic energy, especially when the nail spacing was reduced.

Although not an issue with the 600 series walls, the first LSL wall tested (701) experienced a bottom plate failure on the tension side of the wall due to very high uplift forces and use of an anchor bolt not close enough to the last stud (figure 4b). It was also found that the end-nailing pattern used for the blocking of that wall was not sufficient, as nail pull-out of the studs was observed. Consequently, alternative nail patterns were introduced for the blocking in the rest of the walls. This included paired end-nailing and toe-nailing of the blocking to the studs. Similarly, changes to the anchor bolt pattern were made for all remaining walls of the 700 series.



(a)

(b)

Figure 4. a) Energy dissipation of various walls obtained during the testing; b) Typical deformed shape of the wall studs in the lower half of the walls.

Figure 5. Typical nail failure mode exhibited in walls 704 to 706 with 25.4 mm thick plywood.

Wall 702, which had 15.1 mm OSB sheathing, had lower maximum load and energy dissipation compared to wall 704, which had 25.4 mm DFP sheathing (Table 2, Figure 4a). The higher energy dissipation was attributed to the different failure mode of the nails. In walls with thinner OSB sheathing, the nails tended to develop only one plastic hinge along the shank length. In case of the walls with 25.4 mm thick DFP, the nails were able to develop three or more plastic hinges along the shank (Figure 5). Both values of sheathing thickness (15.1 mm and 25.4 mm) were able to prevent nail pull-through failures, thus allowing the nails to develop higher load capacity. Wall 705 that utilized common nails had the highest energy dissipation of all walls (Figure 4a). The common nails used in this wall, however, experienced many withdrawals, which was not the case for other walls where spiral nails were used.

The use of larger stud spacing (1220 mm) used for the 700 series walls did not appear to cause any detrimental effects on the tall wall behaviour under cyclic loads. Moreover, wall 706, which had the largest stud spacing of all walls (2440 mm instead of 1220 mm), was able to carry the highest load of all walls and exhibited a ductility level similar to that of other walls. This was attributed to the facts that this wall used 25.4 mm plywood sheathing, larger diameter nails, and smaller nail spacing. It should be noted, however, that in case of tall walls with large (2440 mm) stud spacing some issues related to load transfer from the roof have to be addressed. Due to larger spans of the top plate, an increase in its deflection should be expected from the joist or truss loads. Use of double top plate is therefore recommended. The only one unblocked wall tested (wall 707), experienced lower maximum load and hysteretic energy dissipation than other walls (Table 2, Figure 4). Its behaviour was in correlation with previous findings from tests on 2.44 m tall unblocked shear walls (Ni and Karacabeyli, 2002). The use of engineered wood products should not be encouraged in unblocked tall walls. The effect of using a higher strength (and relatively more expensive) product is offset by the lower wall strength and overall performance due to lack of blocking.

The deformed shape of the LSL walls in elevation was similar to that of the SPF walls and included bending of the studs in the lower half of the wall (Figure 6). The upper half of the wall remained relatively straight during the testing. The wall failure mode included connection failures of plate-to-sheathing fasteners along the bottom of the wall, and up the sides. Unlike the SPF walls, where the wall failure mode and failure location were affected by the addition of a vertical load, the same load level did not appear to have any significant effect on the 700 wall series. It seems that larger vertical loads, that unfortunately exceed the capacity of the test setup, were needed in order for LSL studded walls to have any positive effect of the vertical load.

(a) (b)

Figure 6. a) Typical deformed shape of the wall during the testing; b) Stud-to-sheathing failure along the bottom plate and along the sides of the wall.

When 15.5 mm gypsum wallboard was applied to the opposite side of the wall (wall 708), an increase in initial stiffness was observed. An increase in maximum load, ductility and energy dissipation was also observed (Table 2). The maximum load, however, occurred at a lower displacement when compared to the equivalent non-gypsum wall (wall 702). The gypsum wallboard failure mode was screw tear-out at lower displacement levels than those for the OSB sheathing. After the gypsum wallboard failure, the OSB sheathing was able to withstand another load cycle.

The capacity and overall performance of the inexpensive off-the-shelf connectors used in the testing program was satisfactory. The dual H6 tie configuration wrapped under the bottom plate along with the joist hangers and the dense nailing pattern used, provided an effective stud-to-plate connection. No significant uplift of the studs occurred in any of the tests when such anchorage was provided. High uplift forces did cause minor transverse bowing of the bottom plate in some instances. Wider washers for the anchor bolts were used to mitigate this issue.

A simple material cost analysis showed that LSL based tall walls are more expensive than SPF based walls, even before construction time and labour factors are included. This is significant to mention since cost issues related to the material used for the walls are often of a bigger concern than the labour related to driving few more nails in the wall. This cost and weight advantage of SPF-based walls can therefore be utilized in applications where the larger vertical and lateral load-carrying capacity of the LSL-based walls is not required. It should be noted, however, that while the SPF-based walls are easier to construct and maneuver, larger sizes of dimension lumber studs are susceptible to warping, which may lead to poor fitting and delays in construction. To mitigate some of these issues, these walls can be prefabricated in an assembly plant setting and then transported and tilted up on the construction site. On the other hand, when only stud material and spacing differed, the LSL-based walls exhibited higher lateral loads and higher energy dissipation.

Conclusion and Design Recommendations

In this paper, results are presented from a series of quasi-static tests aimed to determine the seismic behaviour of tall wood-frame walls. Tall walls 4.9 m x 4.9 m in size were tested with a variety of sheathing-to-stud connections, stud material, stud spacing, stud-to-plate connections, as well as sheathing material and thickness.

Tall walls built with SPF studs spaced at 610 mm and 15.1 mm thick OSB sheathing showed comparable performance to tall walls with LSL stud walls with the same sheathing thickness but larger (1.2 m) stud spacing. The larger stud spacing was not detrimental to the wall performance when LSL studs were used. For lumber stud walls, use of engineered wood products in the top and bottom plates may be needed to resist the increased uplift loads. Thinner (9.5 mm) sheathing should not be used in combination with studs made of denser wood products such as LSL, due to the nail pull-through the sheathing failure mechanism. Such mechanism doesn’t allow for full potential of the connection to be developed. Use of thicker (e.g. 15.1 mm) sheathing provides higher resistance as the failure mode changes to nail failure with formation of one plastic hinge. Nails in walls with 25.4 mm thick plywood were able to develop three or more plastic hinges during the testing, thus providing high lateral load capacity and energy dissipation of the wall.

SPF based tall walls should be designed with a stud spacing of 610 mm or less. On the other hand, LSL should be used with larger stud spacing (1.2 m to 2.4m) and thicker sheathing (25.4 mm) to be cost-effective. Although tested walls with 2.4 m stud spacing showed comparable strength characteristics, top plate deformation between studs may become an issue in transferring large gravity loads. Blocked tall walls performed far better in all comparable aspects of the wall behaviour. Use of engineered wood products such as LSL in unblocked walls is not effective.

Stud-to-plate connection geometry was found to be important for tall walls. Off the shelf stud-to-plate connections consisting of dual hurricane ties and a hanger were found to be very efficient and can allow for cost-effective modular construction. Use of asymmetrical stud to plate connections (such as single tie on the non-sheathed stud side only), should be avoided due to susceptibility of the studs to torsional failure.

Application of gypsum wallboard increases the initial stiffness and the lateral load capacity of tall walls. The maximum load, however, occurs at displacement level that is lower than that of an equivalent tall wall without gypsum board.

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