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Alternative Bracing Systems in Light-Wood Frame Buildings

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Abstract: Wood portal frame systems have been identified by engineers and builders as a viable option to meet the increasing demand for larger structures, wider openings, and open concept design. In order to develop construction details for portal frames so that they can be integrated into a light wood frame structure containing wood shear walls, there is a need to better understand the behavior of such systems. This study has two main goals: to investigate the behavior of light wood based portal frames and improve on their behavior so that they can be used as a substitute to light frame shearwalls with wood sheathing panels; and to develop high capacity portal frame bracing systems for use in mid-rise wood construction.
Based on the preliminary test results and numerical analysis, it was found that the wall height has a significant impact on the performance of portal frame walls; for walls with different types and locations of metal strap, it is found that the tensile strength of metal straps has the highest impact on the lateral load capacities of portal frame walls; and that the walls with sheathing attached on both sides of the framing have approximately 30% higher lateral load capacities and stiffness than the walls with sheathing attached on one side of the framing. The next phase of the research program will be focused on developing other alternative high-capacity bracing systems that can be used in mid-rise timber construction.

1. Introduction

Light-frame wood shearwall assemblies have been successfully used to resist lateral loads such as earthquake and wind loads for many decades. With the increasing demand for larger structures, wider openings, and open concept design, a need has been identified to consider other bracing systems that would fulfill this need and provide designers with more flexibility without compromising the integrity of the structural system. Wood portal frame systems have been identified by engineers and builders as a viable option to meet this need. Such systems have been developed APA - The Engineered Wood Association in early 2000s (APA 2003, 2004, 2005, 2008) and has been implemented in International Residential Code (IRC) since 2006. In order to develop construction details for portal frames so that they can be integrated into a light wood frame structure containing wood shear walls, there is a need to better understand the behaviour of such systems.

This study has two main goals: to investigate the behaviour of light frame wood based portal framed and improve on their behaviour so that they can be used as a substitute to light frame shearwalls with wood sheathing panels; and to develop high capacity bracing systems for use in mid-rise construction. Only selected results from the first phase of the study will be reported in this current paper.

Available test data on shear walls and connections, and data from the FPInnovations portal frame testing program has been used as input into the 3-d building model as reported in Andi et. al. (2011). The joint effort will provide design guidelines and a better understanding of the behaviour of portal frame bracing systems.

2. Experimental Method

2.1. Component Level Tests

Component tests were conducted for the purpose of using as input data in the numerical modelling of the portal frame walls. Nail connection tests were conducted to obtain the basic mechanical properties of sheathing-to-framing and framing-to-framing connections used in the portal frame walls. Six types of nail connection tests were conducted in this study, with four types of sheathing-to-framing nail connections, and two types of framing-to-framing metal strap connections according to ASTM D1761-06 (ASTM 2009). Screw withdrawal tests were conducted on different framing materials and orientations to evaluate the withdrawal resistance from a framing member.

Shear through-thickness and tension tests were conducted to obtain the mechanical properties of sheathing used in the portal frame walls (ASTM 2009a and 2009b). The same OSB panels that were used in portal frame tests were used in the panel property tests. Also, the tension properties parallel and perpendicular to the surface strand orientation of OSB panel were evaluated. Six specimens were tested for each group. The specimen dimensions were 305 mm wide by 1220 mm long.

Tension test of the metal straps was not conducted. As metal straps are used to connect wall and header, framing-to-framing metal strap connection tests were conducted.

2.2. Portal Frame Tests

Nine portal frame walls were tested monotonically as well as cyclically. The displacement rate was 10.2 mm (0.4 inch) per minute and 20.3 mm (0.8 inch) per second for the monotonic and the reversed cyclic tests respectively. The displacement schedule for the reversed cyclic tests followed the ISO 16670 Standard (ISO, 2003). Based on the monotonic tests of portal frame walls 1, 2 and 3, the reference ultimate displacement was taken as 55.8 mm (2.198 inch) for portal frame walls without hold-downs and 88.9 mm (3.5 inch) for portal frames walls with hold-downs.

All the portal frame walls were 3.66 m in length and 2.44 m in height, with 406 mm wall segment at each end of the portal frame. The wall segment framing was constructed with 38 mm x 89 mm NLGA No.2 and better Spruce-Pine-Fir lumber. The average moisture content of the lumber was 13% at the time of fabrication and testing and the average specific gravity of the lumber was 0.43. The header was built up with either 45 mm x 302 mm (1.75 inch x 11.875 inch) 1.5E laminated strand lumber (LSL) or 38 mm x 286 mm No.2 and better Spruce-Pine-Fir lumber. Oriented strand board (OSB), with a thickness of 12.5 mm and a span rating of 2R32, was used as sheathing panels. 8d common nails (3.3 mm in diameter and 63.5 mm in length) were used to attach sheathings to framing members. Two rows of 10d common nails (3.8 mm in diameter and 76 mm in length) were used to connect the double end studs, spaced at 300 mm on centre.

For wall segments without hold-downs, 12.7 mm diameter anchor bolts were used to fasten the bottom plate to the test frame. Two types of hold-down device were used: a) Simpson Strong Tie HTT16's which are attached to end studs with 16-16d sinker nails, b) 12.7 mm diameter continuous steel rods. The use of hold-downs was intended to simulate the upper bound of end restraint due to fully sheathed return wall, header and dead weight from above. A Simpson Strong Tie LSTA 21 (1000 lb capacity) was used to connect the header and end studs to provide vertical continuity and moment resistance at the corner of the portal frame. Details of test portal frame assemblies are provided in Table 1.

Table 1: Test Matrix of Wood Portal Frame

Wall No.	Sheathing	Hold-down	Load protocol
1†	One side	No	Ramp & cyclic
2†	One side	HTT 16#	Ramp
3†	One side	HTT 16*	Ramp & cyclic
4†	One side	HTT 16#	Cyclic
5‡	One side	HTT 16#	Cyclic
6‡	One side	HIT 16*	Cyclic
7‡	Both sides	No	Cyclic
8‡	Both sides	Steel rods*	Cyclic
9‡	Both sides	HIT 16#	Cyclic

Note:

Hold-downs are installed at ends of portal frame (no hold-downs at the opening)

* Hold-downs are installed at ends of portal frame and opening

† The top plate and the header were connected with two rows of 16d nails (4.2 mm in diameter and 89 mm in length) at 75 mm on center. The end stud and the end of header were connected with two rows of the same nails at 75 mm on center.

‡ The top plate and the header were connected with two rows of screws (5 mm in diameter and 90 mm in length) at 75 mm on center. The end stud and the end of header were connected with two rows of the same screws at 75 mm on center.

A photo of the test setup is shown in Figure 1. The lateral load was applied through a steel spreader bar at the top of the wall. The spreader bar had lateral guides to ensure a steady and consistent unidirectional movement of the wall.

Figure 1: Test setup of the portal frame wall assembly

A displacement transducer was placed at the top of the wall to measure the lateral deflection of the wall. Two displacement transducers were placed at the bottom of the header around the wall corners to measure the relative vertical movements between end studs and the header. Four transducers were used at the end-studs to measure the uplift of studs to the foundation.

3. Results of Experimental Tests

The results were analyzed in accordance with ASTM Standard E2126 (2009) requirements based on the Equivalent Energy Elastic-Plastic (EEEP) curve (Figure 2).

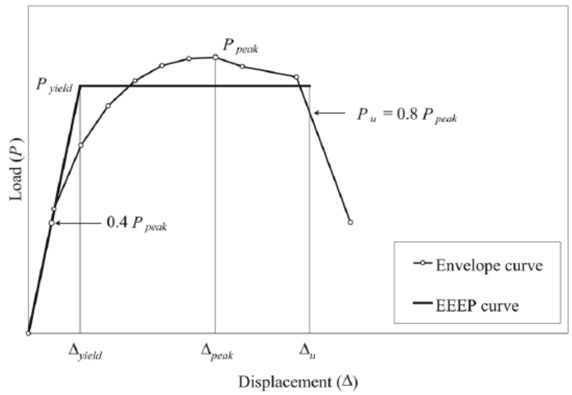


Figure 2: Determining the response properties based on the EEEP curve as per ASTM 2126

A summary of the test results is provided in Tables 2 and 3. The notations in Tables 2 and 3 for a specific envelope curve are defined as follows: Ky is the initial (yield) stiffness; Fy is the yield load; Δy is the yield deflection; Fmax is the maximum load; ΔFmax is the deflection at which the maximum load was reached; Δu is the ultimate deflection (deflection in post maximum load region where the load dropped to 80% of the maximum load); Δu/Δy is the ductility of the portal frame for that particular envelope curve.

Table 2: Test results of portal frame walls under monotonic loading

Wall	Specimen	Ky	Fy	Δy	Fmax	ΔFmax	Δu	Ductility	Energy
		[kN/mm]	[kN]	[mm]	[kN]	[mm]	[mm]	(Δu/Δy)	[kJ]
1	PF-01	0.45	9.01	20.0	10.93	55.4	55.8	2.80	0.41
2	PF-03	0.38	12.88	33.8	14.77	67.5	80.2	2.37	0.82
3	PF-04	0.62	14.91	24.0	16.37	158.7	163.5	6.81	2.26

Table 3: Test results of portal frame walls under reversed cyclic loading (average of positive and negative envelope curves)

Wall	Specimen	Ky [kN/mm]	Fy [kN]	Δy [mm]	Fmax [kN]	ΔFmax [mm]	Δu [mm]	Ductility (Δu/Δy)	Energy [kJ]
1	PF-02	0.42	9.00	21.5	10.27	52.4	71.8	3.39	4.18
4	PF-05	0.56	14.00	24.9	15.74	87.6	124.7	5.00	11.20
3	PF-06	0.54	14.31	26.5	16.54	64.2	90.2	3.41	6.59
5	PF-07	0.63	18.67	29.6	21.37	80.2	140.0	4.75	19.35
6	PF-08	0.68	18.78	27.7	21.21	63.1	121.2	4.38	15.40
7	PF-09	0.61	15.76	25.9	18.04	65.0	82.9	3.19	5.79
8	PF-10	0.51	20.46	39.8	23.24	136.0	153.4	3.86	16.45
9	PF-11	0.99	23.07	23.4	25.84	61.2	71.8	3.07	6.61

3.1. General Observations of Failure

Observations showed that the failures started with OSB tearing at corners of portal frame walls, followed by metal straps fracture (Fig. 3).

For portal frame walls without hold-down (Wall 1 and Wall 7), the lateral load carrying capacities were approximately 75% the capacities of identical walls with hold-down (Wall 2 and 8). It was also observed that the lateral load carrying capacities were similar for portal frame walls with hold-downs installed at ends of portal frame (no hold-downs at the opening) or hold-downs installed at ends of portal frame and opening (Wall 3 versus Wall 4). The strength of the wall with metal straps installed over sheathing (Wall 9) were slightly better than that of the wall without (Wall 8).

a) panel ripped (left pier)

b) panel ripped (right pier)

c) metal strap broke (left pier)

d) metal strap broke (right pier)

Figure 3: Failure modes of portal frame No. 3

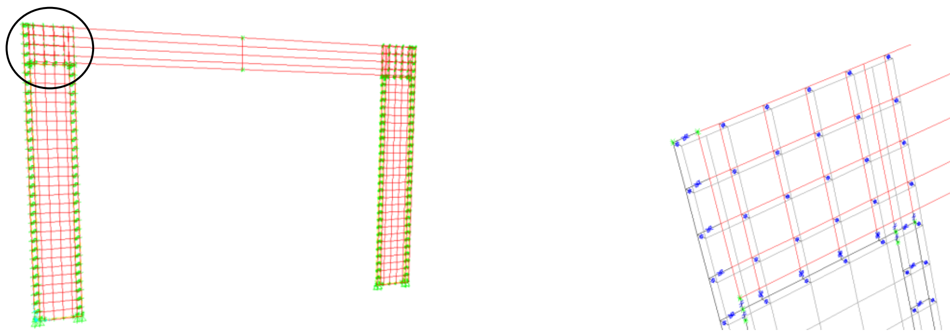
4. Numerical Modelling

4.1. Description of the Finite Element Modelling

Using SAP2000, a non-linear finite element model was developed to study the racking performance of portal frame walls. In the model, the framing members of the wall segments were represented by beam elements, whilst the header and the OSB sheathing were represented by shell elements. The sheathing-to-framing and framing-to-framing nail joints, metal strap joints and hold-downs were modeled with link elements. For framing-to-framing joint in compression, a contact element was introduced. For the framing members and header, linear elastic behaviour was assumed and the average modulus of elasticity from Canadian timber design code CSA O86 (CSA, 2009) was used. For sheathing-to-framing nail joints and metal strap joints, the actual average load-slip properties of the joints from the tests were used. More details on input data for the finite element model can be found in Chen and Ni (2010).

Each sheathing-to-framing and framing-to-framing joint was modeled by a 3D link element that can represent behaviour of the joint under withdrawal, loading parallel to the grain of the framing member and, loading perpendicular to the grain of the framing member.

A one-dimensional link element is used to represent the metal strap joint. Because of the different behaviour of metal strap joints before and after sheathing rupture, the numerical modelling was carried out in two steps. The first step is to predict the load-displacement responses of the portal frame walls before sheathing rupture. The second step is to predict the ultimate lateral load-carrying capacity of the portal frame walls after sheathing rupture. The finite element model of portal frame wall is shown in Figure 4. The lateral load is applied at the middle of the header. In the analysis, the out-of-plane deformations in wall framing and header, and the torsional deformation of wall frames were restricted.



a) 3D view of FE model of portal frame b) Details of corner connection

Figure 4: FE model of portal frame

4.2. Verification of Model Prediction

In order to validate the finite element model, comparisons were made between the model predictions and the test results of selected full-scale portal frame walls, which cover some key variables in the numerical modelling such as walls with or without hold-downs and metal straps installed directly over framing or sheathing.

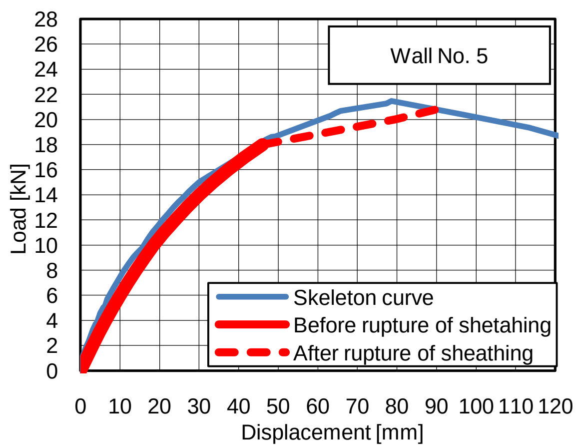


Figure 5: Comparison of the load-displacement responses between tests and modelling

An example of the load-displacement responses of the tests and the numerical modelling is provided in Figure 5. As can be seen from the figure, the predicted load-displacement curve is generally in reasonable agreement with the test results up to the ultimate capacity of the wall. However, the model failed to predict the wall performance after peak load. This aspect requires further study. Because of the difficulty to model the behaviour of portal frame wall after fracture of a component e.g. OSB sheathing, in this study only the ascending portion of the load-displacement response was studied.

4.3. Parametric Studies

Four basic portal frame walls were selected as the reference walls in this study. The walls are 3.66 m in length and 2.44 m in height, with 406 mm wall segment at each end of the portal frame. Walls Types 1 and 2 have the panels sheathed only on one side of the framing. Walls Types 3 and 4 have the panels sheathed on both sides of the framing. For Walls Types 1 and 3, hold-downs were installed at the ends of portal frame walls. No hold-downs were used in Walls Types 2 and 4.

Five parameters were studied for each wall type. The first parameter is the height of portal frame wall. In this study 3.05 m (10 ft) high portal frame walls were studied. The second parameter is the metal strip types and locations. Table 4 and lists the detailed types and locations of metal strips. For metal strip in configurations C1 and C2, Simpson Strong Tie LSTA 21 metal strap is used. Although the locations of metal strips in configuration C2 and C3 are the same, the tensile strength of the metal strips in C3 is twice the tensile strength of Simpson Strong Tie LSTA 21. Figure 6 shows the locations of the metal strips in Configurations C1, C2 and C3

The third parameter is double bottom plates. In this case, two rows of nails spaced at 75 mm on centres were used in bottom plates. The forth parameter is unblocked sheathings that were joined at the middle of the wall height. The last parameter is different nailing pattern, in which a single row of nails is used in end studs.

In order to compare the effect of these parameters on the performance of portal frame walls, each individual parameter was studied with the reference walls without change of other parameters.

Table 4: Types and locations of metal strips

Walls Types 1 and 2			Wall Types 3 and 4		
Configuration	Side 1	Side 2[a]	Configuration	Side 1	Side 2
1	None	C2			
2	None	C3			
3	C1[b]	C1	1	C1[b]	C1[b]
4	C2[b]	C2	2	C2[b]	C2[b]
5	C3[b]	C3	3	C3[b]	C3[b]
6	C1[c]	C1	4	C1[c]	C1[c]
7	C2[c]	C2	5	C2[c]	C2[c]
8	C3[c]	C3	6	C3[c]	C3[c]

Note: [a] Metal strip is placed on framing members.
[b] Metal strip is placed over wall sheathing.
[c] Metal strip is placed underneath wall sheathing.

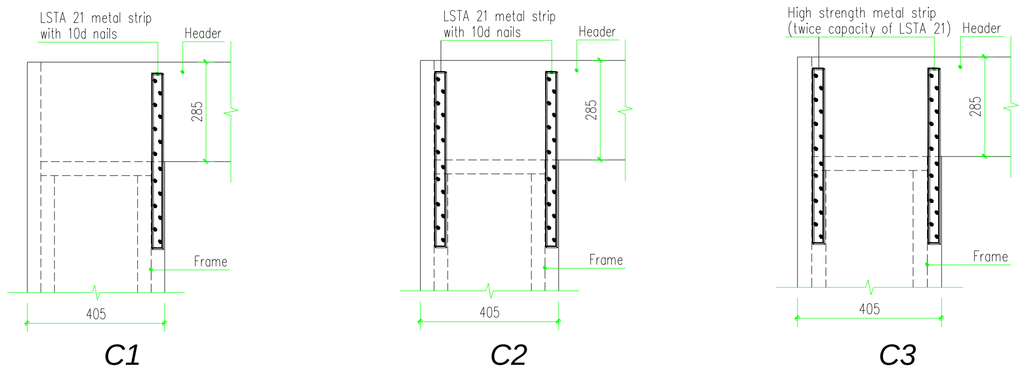


Figure 6: Locations of metal strips in Configurations C1, C2 and C3

5. Discussion and Analysis

5.1. Height Effects

The stiffness and ultimate lateral load of reference walls and 3.05 m high walls are summarized in Table 5.

As expected, results show that wall height has significant impact on the performance of portal frame walls. For 3.05 m high walls, the ultimate lateral loads are approximately 20% and 25% lower respectively than those of the corresponding 2.44 m high walls. The stiffness of the 3.05 m high walls is about 20% to 40% lower than that of the corresponding 2.44 m high walls.

Table 5: Summary of results of 2.44 m and 3.05 m high portal frame walls

	Wall Type 1		Wall Type 2		Wall Type 3		Wall Type 4	
	2.44 m	3.05 m	2.44 m	3.05 m	2.44 m	3.05 m	2.44 m	3.05 m
k (N/mm) *	526	313	442	357	710	421	618	360
Fmax (kN) †	15	12	11	9	19	15	15	12

* k is the secant stiffness between 0 and 40% of the ultimate lateral load.

† Fmax is the ultimate lateral load.

5.2. Types and Locations of Metal Straps

The stiffness and ultimate lateral load of portal frame walls with different types and locations of metal straps are listed in Tables 6 and 7.

For wall configurations 5 and 8 of Wall Type 2, the lateral load capacities were achieved when sheathing-to-framing joints failed at the bottom plate. For walls with metal straps placed over sheathing (except wall configuration C3 of Wall Type 1), the lateral load capacity continued to increase until metal straps broke after sheathing rupture. For the rest of the walls, the lateral load capacities were achieved at the time of sheathing rupture.

The results showed that portal frame walls with sheathing attached on both sides of the framing have approximately 30% higher lateral load capacities and stiffness than the walls with sheathings attached on one side of the framing. Also, the use of high strength metal straps (twice the tensile strength of Simpson Strong Tie LSAT 21) can greatly increase the lateral load capacities and stiffness of portal frame walls. It was also observed that portal frame walls with metal straps installed on both ends of the studs (Configuration C2) have higher lateral load capacities and stiffness than the walls with metal straps installed at the inner corner of the walls (Configuration C1).

Table 6: Summary of results of Wall Types 1 and 2 with different types and locations of metal straps

Configuration	Wall Type 1		Wall Type 2	
	k (N/mm)	Fmax (kN)	k (N/mm)	Fmax (kN)
Reference	526.7	15	441.3	11
1	550.7	16	464.6	12
2	594.7	20	509.5	15
3	530.6	17	455.5	11
4	575.8	19	495.0	13
5	622.6	27	556.6	16
6	547.8	19	463.8	14
7	594.7	20	509.5	15
8	634.1	27	573.5	16

Table 7: Summary of results of Wall Types 3 and 4 with different types and locations of metal straps

Configuration	Model 3		Model 4	
	k (N/mm)	Fmax (kN)	k (N/mm)	Fmax (kN)
Reference	710.5	19	617.7	15
1	746.9	18	654.0	14
2	764.2	23	683.7	16
3	783.3	30	688.6	22
4	750.6	26	661.9	20
5	800.3	27	708.6	21
6	834.8	35	743.9	28

5.3. Other Parameters

Portal frame walls with double bottom plates, unblocked sheathing at the middle of wall height and different nailing pattern parameters were also studied. Based on the results, it is noticed that for portal frame walls with double bottom plates, both stiffness and lateral load capacities are slightly higher than those of the corresponding reference walls. For walls where unblocked sheathings were joined at the middle of the wall height, though the lateral load capacity is the same as those of the corresponding reference walls, the stiffness is slightly lower than those of the corresponding reference walls. For walls where a single row of nails is used in end studs, while the lateral load capacity is slightly higher, the stiffness is slightly lower than those of the corresponding reference walls.

6. Conclusions and Future Work

There is a need to find alternative bracing systems to light-frame wood shearwalls that would allow for the increasing demand for more open structures. The first phase of the research project dealt with understanding the behaviour of light portal frames as an alternative to braced shear walls designed based on prescriptive requirements. Based on the preliminary test results and results from numerical modelling, the following observations are made:

- The wall height has a significant impact on the performance of portal frame walls. With the same wall configurations, the lateral load capacities and stiffness of 3.05 m high walls are approximately 20% and 20 - 40% lower respectively than that of the corresponding 2.44 m high walls.
- For walls with different types and locations of metal straps, it is found that the tensile strength of metal straps has the highest impact on the lateral load capacities of portal frame walls.
- The walls with sheathings attached on both sides of the framing have approximately 30% higher lateral load capacities and stiffness than the walls with sheathings attached on one side of the framing.

Test data will be used as input into analytical models that are under development. The next phase of the research program will identify other alternative high-capacity bracing systems that can be used in mid-rise timber construction.

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